No. 73

FEASIBILITY STUDY
FOR
SEWERAGE AND DRAINAGE PROJECT
BUTTERWORTH/BUKIT MERTAJAM METROPOLITAN AREA
MALAYSIA

VOLUME III

DRAINAGE SYSTEM

FEBRUARY 1979

JAPAN INTERNATIONAL COOPERATION AGENCY

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ORDER OF PRESENTATION

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LIST OF ABBREVIATIONS

EHEU - Environmental Health and Engineering Unit

DE - Department of Environment

EPU - Economic Planning Unit, Penang Stage Government

EU - Engineering Department

PWD - Public Works Department

DID - Drainage and Irrigation Department

PDC - Penang Development Corporation

PWA - Penang Water Authority

TCP - Town and Country Planning

MPSP - Municipal Council Province Wellesley

MPPP - Municipal Council Penang Island

WHO - World Health Organization

IBRD - International Bank for Reconstruction and Development

ADB - Asian Development Bank

CONVERSION TABLES

Length (1)

m	cm	yd	ft	in.
1	100	1.0936	3.2808	39.370
0.01	1	0.0109	0.0328	0.3937
0.9144	91.440	1	3	36
0.3048	30.480	0.3333	1	12
0.0254	2.540	0.0278	0.0833	1

Length (2)

km	yd	mi
1	1,093.61	0.62137
0.00091	1	-
1.60934	1,760	1

Area

ha	km^2	acre	sq mi	m^2	sq ft
1	0.0100	2.471	0.00386	10,000	107.640
100	1	247.10	0.3861	-	-
0.4047	0.004047	1	0.00156	-	-
259	2.590	640	1	-	-
-	-	-	-	1	10.764
-	-	-	-	0.09290	1

1 sq ft = 144 sq in.   1 sq in. = 0.006946 sq ft

CONVERSION TABLES (Continued)

Volume

l	m	cu ft	Imp.gal
1	0.001	0.03531	0.220
1,000	1	35.31	220
28.317	0.02832	1	6.231
4.546	0.004546	0.1605	1

Weight

Kg	t	ounce	lb
1	0.001	35.27	2.2046
1,000	1	3.257×10^4	2,204.6
0.02835	2.835×10^{-5}	1	0.06250
0.4536	4.536×10^{-3}	16	1

Velocity

m/sec	km/hr	ft/sec	mile/hr
1	3.600	3.2808	2.237
0.2778	1	0.9113	0.6214
0.3048	1.0973	1	0.6818
0.4470	1.6093	1.4667	1

CONVERSION TABLES (Continued)

Rate of Flow (1)

l/sec	m^3/hr	m^3/sec	Imp.gal/min
1	3.6	0.001	13.198
0.2778	1	2.778×10^{-4}	3.666
1,000	3,600	1	1.3198×10^{4}
0.07578	0.2728	7.577×10^{-5}	1
7.866×10^{-3}	0.02832	7.866×10^{-6}	0.10381
28.32	101.94	0.02832	373.7
52.61	189.41	0.05261	694.4
0.01157	$4,167 \times 10^{-2}$	0.1157×10^{-4}	0.1528

Rate of Flow (2)

cu ft/hr	cu ft/sec	Imp.MGD	m^3/day
127.13	0.03531	0.01901	86.4
35.31	9.810×10^{-3}	5.279×10^{-3}	24
1.2713×10^{5}	35.31	19.01	86,400
9.632	0.002676	1.440×10^{-3}	6.547
1	2.778×10^{-4}	1.495×10^{4}	0.6796
3,600	1	0.5383	2,447
6,688.2	1.858	1	4,546
1.471	4.087×10^{-4}	2.200×10^{-4}	1

CHAPTER 1

INTRODUCTION

Drainage system feasibility study has been made in order to prepare a comprehensive drainage programme to meet both immediate and future requirements for solving the flood problems in the Study Area. The principle for the planning and design is to ensure that there is no inconvenience of flooding from the Initial Storm (2 or 5-year return period) and to check that there is no significant damage by the Major Storm (100-year return period). All the work under this study have been established in accordance with DID's Planning and Design Procedure No. 1 "Urban Drainage Design Standards and Procedures for Peninsular Malaysia."

To identify the problems and requirements as to the drainage system, field surveys and investigations have been carried out, and then the possible alternative countermeasures have been prepared and studied in depth, including development of design criteria for the system and preliminary engineering design for the entire Study Area, as described in Chapters 2, 3 and 4.

On the basis of the results of preliminary engineering design, the implementation schedule of the system has been made for the first stage programme considering the order of priority for construction. The first stage programme includes open channels, bridges, culverts, tide gates and outfalls, covering the entire Study Area of 3,480 ha (8,600 acres) with an estimated construction cost of M$6,403,000 at 1977 price level. The technical, administrative and financial considerations are discussed and recommendations are presented in Chapter 5.

Various types of the benefits will be derived from the implementation of the recommended drainage programme. The anticipated benefits, although not fully quantifiable, are described in Chapter 6.

CHAPTER 2

STUDY AREA, LAND USE AND POPULATION

2.1 Study Area

The drainage study area has been determined mainly in accordance with the recommendations in the Master Plan Report and also in consultation with various agencies concerned. The Study Area covers totally 3,478 ha (8,591 acres) comprising a part of Drainage Basin II and the entire Drainage Basin IV, each having tributary area of 1,932 ha (4,772 acres) and 1,546 ha (3,819 acres) respectively (see Figure 2.1). Due to the topographic conditions of the area, approximately 1,160 ha (2,865 acres) of tributary outside of the Study Area is considered for calculating the drain capacities because the tributary contributes its storm runoff to the Stury Area.

For the preliminary engineering purpose, the entire Study Area is further divided into small watersheds. Naming and numering of the drains that are to be designed under the present study are basically the same as those given in the Master Plan Report. The population and drainage area of each drainage system are shown in Table 2.1. The total area of the proposed Drainage Basin IV, as shown in Table 2.1, is 30 ha less than that indicated in the Master Plan Report. The difference is that the zone near the Ferry Port is curtailed from the originally planned area because the drainage construction is now underway in the area by Penang Port Commission Administration.

2.2 Land Use

The land use pattern of the area affects significantly to the characteristics of the stormwater runoff. Presently, agricultural and residential areas are predominant in the Study Area, while the builtup urban area which is a mixture of residential and commercial zones occupies only a small portion of the area. The future land use pattern envisaged for 2000 is assumed that the major portion of the area would be of the residential area and a small portion of commercial and industrial areas. For the design of the drainage system, the land use pattern for the year 2000 has been taken into consideration. The land uses both at present and in the future are presented in Figures SD-8, SD-9, SD-12 and SD-13 of Volume V.

2.3 Population Estimates

The staged populations of the Study Area are estimated at approximately 152,800, 184,800 and 247,400 for the years 1976, 1985 and 2000 respectively for the design purpose.

The average population densities for the years 1976 and 1985 are estimated to be 44 and 53 persons per hectare respectively. These indicate that by the year 1985 a considerable part of the Study Area will yet to be developed. In other word, the provision of the drainage system must meet the requirements for the development of the area. The estimated populations in different years by individual drainage system are shown in Table 2.1.

Table 2.1 Study Area and Population

(1) Butterworth

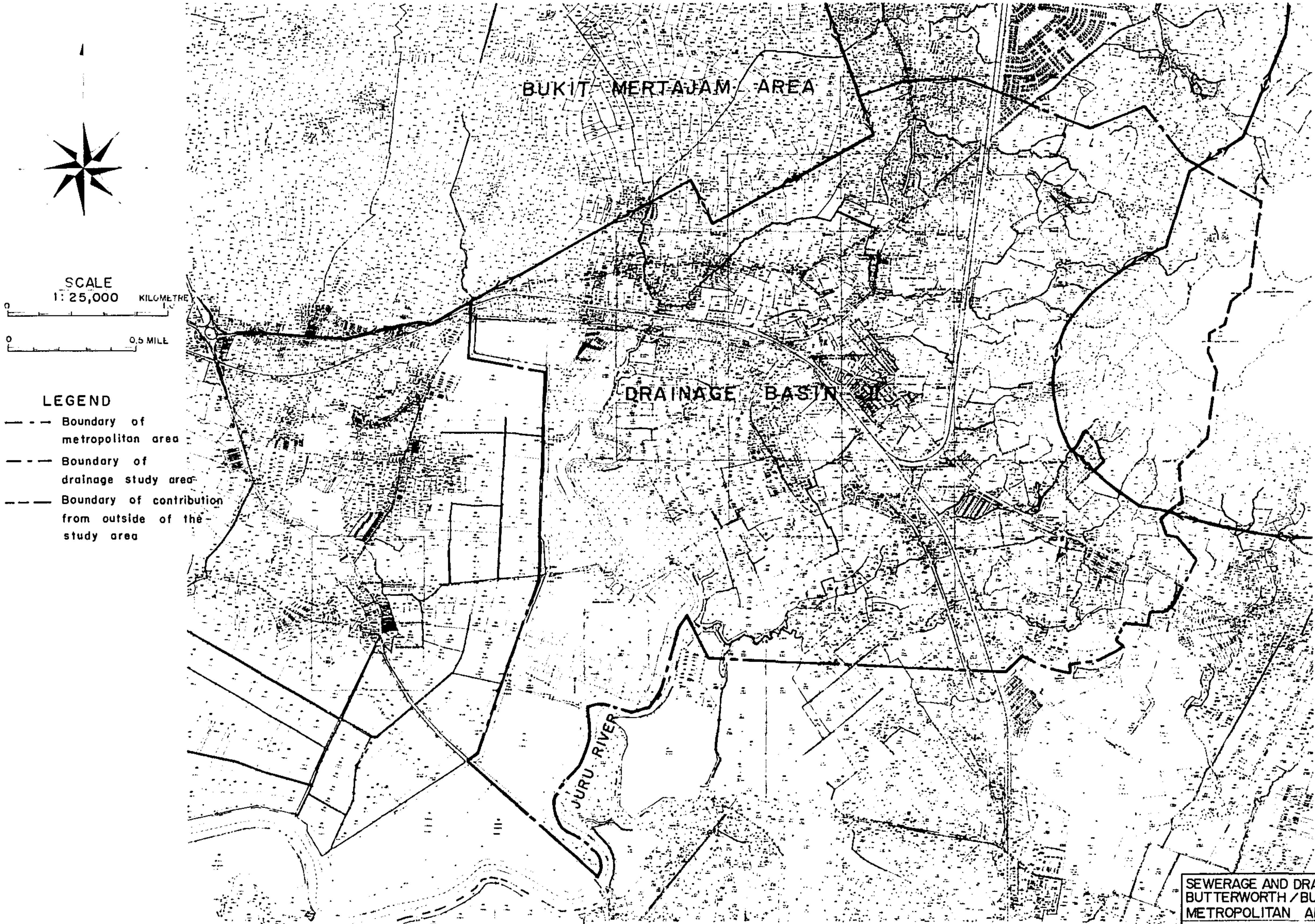
Name of Drainage System	Area		Served Population by Drainage System		
	Served area by drainage system (ha)	Contributing area (ha)	1976 (person)	1985 (person)	2000 (person)
BUTTERWORTH DRAIN A-A	101.5	0	7,280	7,829	8,743
BUTTERWORTH DRAIN A-B	153.7	0	16,900	17,421	18,288
BUTTERWORTH DRAIN A-C	201.6	0	13,800	20,642	32,044
BUTTERWORTH DRAIN B	222.6	32.8	12,555	17,277	25,148
BUTTERWORTH DRAIN C-A	181.8	0	9,400	11,402	14,739
BUTTERWORTH DRAIN C-B	229.1	0	4,515	6,001	8,478
BUTTERWORTH DRAIN D	28.7	0	2,025	2,571	3,480
BUTTERWORTH DRAIN E	81.3	0	13,345	11,947	11,108
SEA DRAIN-A	30.1	0	2,835	3,144	3,660
SEA DRAIN-B	15.5	0	815	1,207	1,860
SEA DRAIN-C	11.3	0	2,010	2,017	2,028
SEA DRAIN-D	18.9	0	3,380	3,123	2,968
SEA DRAIN-E	33.4	0	3,785	4,074	4,556
PRR-A	23.7	0	1,080	1,050	1,032
PRR-B	12.6	0	440	568	780
Direct Discharge to Sea or River	200.2	0	6,630	9,536	14,493
Total	1,546.0 ha (3,819 acres)	32.8 ha (81 acres)	100,795	117,792	153,405

(to be continued)

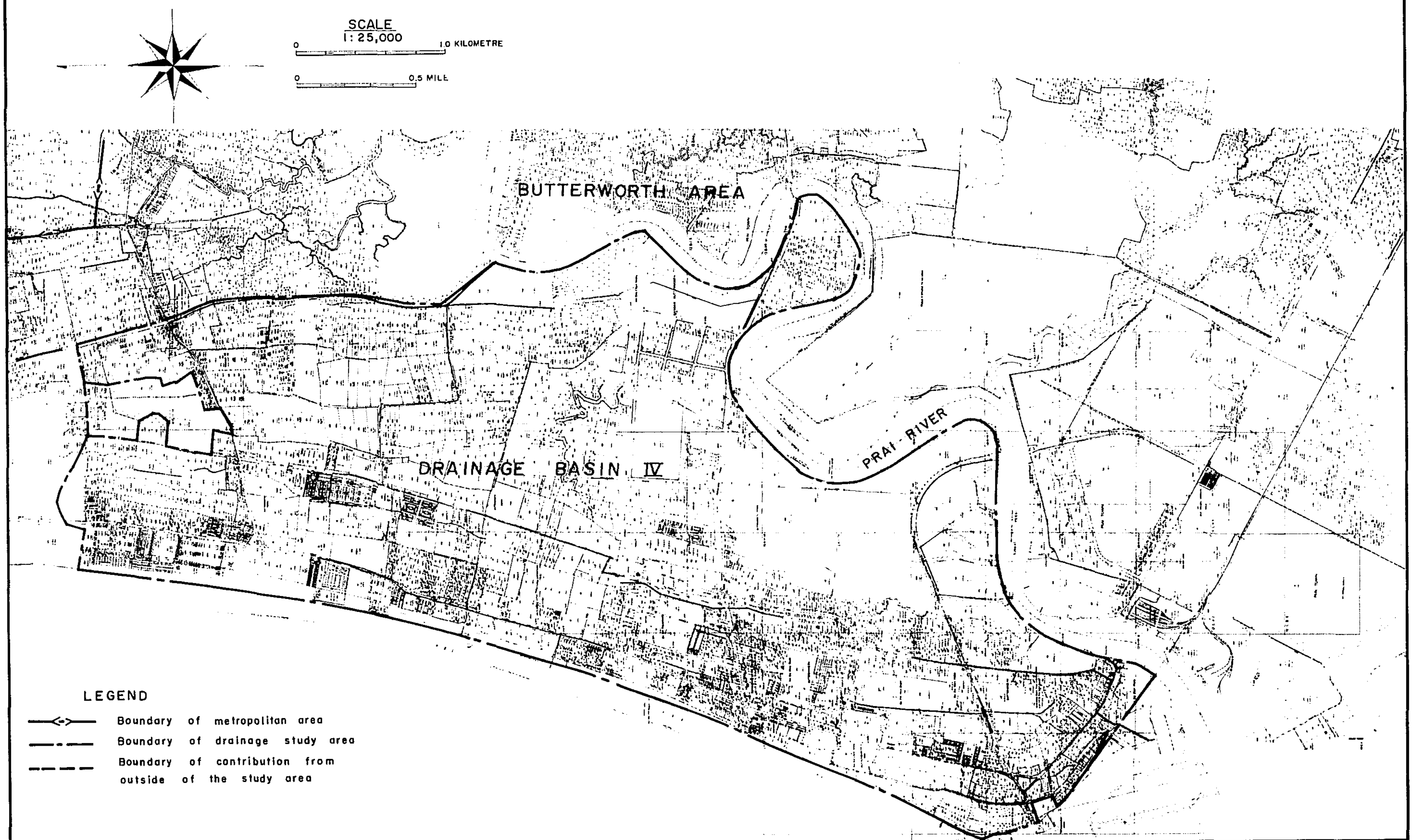
(continued)

(2) Bukit Mertajam

Name of Drainage System	Area		Served Population by Drainage System		
	Served area by drainage system (ha)	Contributing area (ha)	1976 (person)	1985 (person)	2000 (person)
TANAN DRAIN	234.5	41.3	-	951	2,536
SUNGAI ARA	402.4	16.7	6,401	8,100	10,932
PAYA DRAIN	78.4	16.3	2,297	2,297	4,100
BUKIT MERTA-JAM DRAIN	122.4	4.8	11,135	11,941	13,284
SUNGAI RAMBAI	99.4	499.0	1,748	2,120	2,740
SUNGAI PASIR	399.6	29.5	10,735	12,354	15,052
SUNGAI PEKAN BHARU	168.0	49.7	9,197	11,223	14,600
BUKIT KECHIL DRAIN (A)	74.9	0	4,504	5,308	6,648
BUKIT KECHIL DRAIN (B)	77.1	0	4,853	4,998	5,240
PMTG KEBUN SIREN DRAIN	42.9	503.0	1,036	1,245	1,592
BUKIT TENGAH DRAIN	176.2	0	48	5,673	15,048
STP AREA	56.0	0	-	840	2,240
Total	1,931.8	1,160.3	51,954	67,050	94,012
Grand Total	3,477.8 ha (8,590 acres)	1,193.1 ha (2,947 acres)	152,749	184,842	247,417



SEWERAGE AND DRAINAGE SYSTEM PROJECT BUTTERWORTH / BUKIT MERTAJAM METROPOLITAN AREA, MALAYSIA	
STUDY AREA	FIGURE 2·1·(1)



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

STUDY AREA

FIGURE 2·1·(2)

CHAPTER 3

DESIGN CRITERIA

Design criteria presented herein are basically in accordance with those recommended in DID's "Planning and Design Procedure No. 1, Urban Drainage Design Standards and Procedures for Peninsular Malaysia." Following is the brief description on the design criteria adopted for the first stage programme:

3.1 Calculation of Runoff

3.1.1 Rational Formula

The modified Rational Method as recommended in the DID's Procedure No. 1 is applied for estimating the stormwater runoffs, in the form:

$$Q = \frac{1}{360} CsCIA \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.1)$$

where

Q : peak discharge, m^3/sec

I : average intensity of rainfall, mm/hr

A : catchment area, ha

C : runoff coefficient

Cs: storage coefficient which is expressed as:

$$Cs = \frac{2tc}{2tc + td}$$

tc: time of concentration, min.

td: time of flow in the drain, min.

Practical definition of variables of the formula resulted from long-time experience is given as follows:

3.1.2 Rainfall Intensity-Duration-Frequency Curves

Rainfall intensity is expressed in the form of intensity-duration-frequency curves developed for Georgetown as follows:

2-year frequency	$I_2 = \dfrac{6,270}{t + 32}$
5-year frequency	$I_5 = \dfrac{8,070}{t + 30}$
100-year frequency	$I_{100} = \dfrac{13,940}{t + 33}$

3.1.3 Rainfall Frequency for Design

The average frequencies of rainfall occurrence used for the drainage design are set for the respective land use patterns as follows:

Residential area	2-year
Commercial area	5-year
Industrial area	5-year

The above figures are applicable to local drains usually serving for small watershed, but for main drains serving wider tributary area the 5-year frequency is recommended because they generally flow through areas comprising various types of land use pattern.

3.1.4 Runoff Coefficient

The recommended coefficients for the first stage programme area are as follows (for detail see Appendix I, Master Plan Report Volume II):

Land Use Pattern	Runoff Coefficient
Residential	
Densely inhabited (120 p/ha or more)	0.65
Sparsely inhabited (less than 120 p/ha)	0.35
Commercial	0.85
Industrial	0.50
Mountain	0.50

3.2 Hydraulic Design for Storm Drains and Reservoirs

3.2.1 Storm Drain

(a) Flow Friction Formula

For the hydraulic design of open channels, the Manning's Formula is applied and expressed as follows:

$$V = \frac{1}{n} R^{2/3} \cdot I^{1/2} \quad \text{.............................} \quad (3.2)$$

where

v : velocity, m/sec

n : roughness coefficient

R : hydraulic radius, m

I : gradient

The values of 'n' for different materials are defined as follows:

Concrete drain	
cast-in-place	= 0.015
pre-cast	= 0.013
Wet masonry drain	= 0.025
Earth drain	= 0.030

(b) Velocity of Flow

To prevent deposition of grit and sand in storm drains, the velocity of flow shall not be lower than 0.6 metre per second (2 ft/sec) in any type of drain. Care should also be given to maximum velocity of flow to prevent erosion of drains. The recommended minimum and maximum velocities for various types of drain are summarized below:

Type of Drain	Design Velocity (m/sec)		
	Minimum	Maximum	
Concrete Drain	0.6	3.0	
Stone Drain	0.6	2.5	
Grass Lined Drain	0.6	2.2	(1)
Earth Drain	0.6	1.0	(2)

Data source (1) DID's Procedure

(2) Portier & Sioby

3.2.2 Reservoir

For computing the required capacity of the reservoir, the following steps are taken:

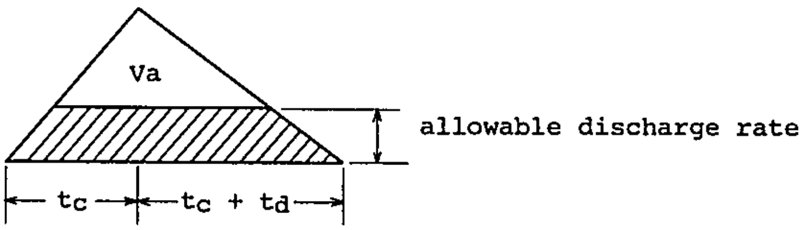
Develop inflow hydrograph,

Set allowable discharge rate, and

Calculate required storage capacity of the reservoir.

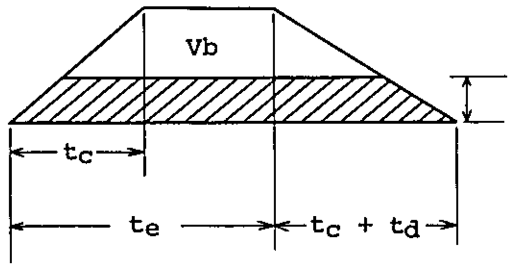
In general, two types of inflow hydrograph are considered; one is for the case of $t_c > t_e$ and the other for $t_c < t_e$. The required capacity is then determined by comparing these two cases in the manner as illustrated in the following:

(a) $t_c > t_e$



Va: required storage volume

(b) $t_c < t_e$



Between Va and Vb, the larger one should be adopted as the required storage volume.

3.3 Survey Datum and Sea Water Level

3.3.1 Survey Datum

The data used in this study are those established in the Malaysian Survey Ordnance Datum which sets the mean sea water level (1912) as zero level.

The ground elevations used in this study are expressed in a reduced level (RL) which is identical to the zero level of the Survey Ordnance Datum. Sea water levels, used for drainage design of this project is determined based on the record from 1952 through 1967. The applied figures, as described below, are also used is the "Project Report on Drainage and Reclamation of Sungai Prai Basin." This report has various relation with the drainage plan in the Study Area.

HHWL (highest recorded level)	SOD +1.68 m (+5.5 ft)
MHWL (spring tide)	+1.10 m (+3.6 ft)
Mean sea level	+0.15 m (+0.5 ft)
KLW (spring tide)	-0.79 m (-2.6 ft)

SOD: Survey Ordnance Datum which is the height above mean sea level at Port Swettenham in 1912.
It is expressed as,
mean sea level (in 1912) .... SOD + 0.00

3.3.2 Sea Water Level Used for Design

As the Study Area is generally low-lying and flat in nature, the major part of the drains is influenced by the tide. The selected levels for this study are shown below:

(a) For checking the drainage system for the Major Storm, the tailwater is determined by adopting "Mean Sea Level (+0.15 m or +0.5 ft)" tide conditions.

(b) For designing the drainage system for the Initial Storm, the tailwater is determined by adopting "Mean High Water Level (+1.1 m or +3.6 ft)" tide conditions.

(c) For designing storage systems subject to tidal influence, the following is assumed:

- Tide is diurnal of approximately 12-hour duration.
- Rise and fall of tide is sinusoidal.
- High level is +1.1 m (+3.6 ft) and low level is -0.15 m (-0.5 ft).

(d) For land filling, ground elevation to be raised is determined by adopting "Highest Recorded Level (+1.68 m or 5.5 ft)".

3.4 Types of Drainage Facility

3.4.1 Drain

Facilities recommended for the system include open channel, box culvert, pipe and bridge, as described in the following paragraphs:

(a) Open Channel

Open channels designed in the first stage programme include; (1) trapezoidal earthen channels, (2) trapezoidal grass lined channels, (3) trapezoidal rubble wall channels, (4) rectangular reinforced concrete channels with cast-in-place retaining walls and U-shaped channels either of precast or cast-in-place, and (5) V-shaped concrete channels of either precast or cast-in-place.

Side slopes of trapezoidal drains have been determined conforming to the standards as illustrated in Figure 3.1.

Increase of the ratio of depth to width will increase the direct construction cost but cut down the cost for land acquisition. The optimum ratio of the two has been therefore selected in the preliminary engineering design taking specific local conditions into account so that the drains are designed most economically.

Lining of the surface of drains has advantages of; (1) increased capacity with smooth surface, (2) reduced land requirements for the right-of-way by steeper side slope, and (3) easy maintenance. As a principle, lining is required for side walls of all open channels in the area; however, the lining of inverts has not been considered most of the drains except those in the hilly land of Bukit Mertajam because the ground urface in the Study Area is generally flat and the erosion of invert is assumed to be not so significant.

In order to give smooth flow in the drains during the dry weather, it is genbrally preferable to provide trickle channel especially in large channels. In Butterworth, trunk drains generally have a large amount of flow even during the dry weather due mainly to the backwater from the sea. The backwater raises the water surface in the drains resulting in greater water depth. In Bukit Mertajam, most of the drains are flowing full under the dry weather condition because of the inflow from the paddy and mountainous areas. Thus the trickle channel is not required for such drains. Standard cross sections of various types of channel are shown in Figure 3.1 and the related design criteria are summarized in Table 3.1.

(b) Box Culvert

At a road crossing, box culvert is generally used. Where traffic in heavy, it is preferable to use precast box culvert available in Malaysia. Currently the available market size of the precast box culverts is limited to small ones, hence multiple numbers of box culvert may be laid in parallel to flow the stormwater from large capacity drains. If the conditions allow for longer period of drain construction, cast-in-place box culvert may be used.

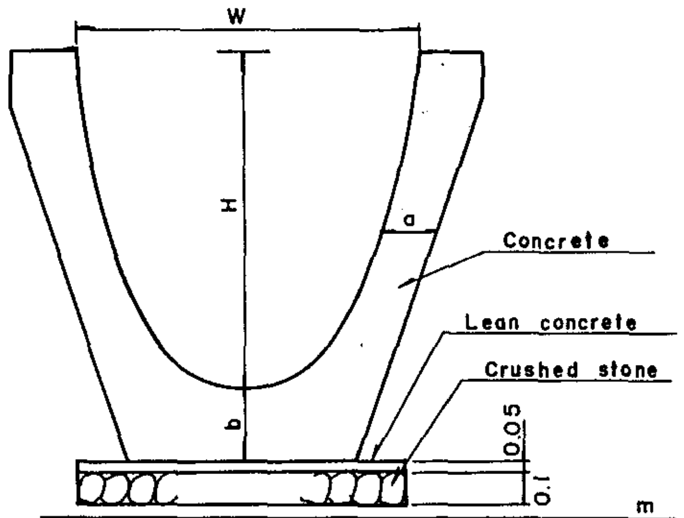
(c) Pipe

Pipes are also used for road crossing of small drains. The pipes should generally be of centrifugally cast reinforced concrete with sufficient strength to sustain the heavy traffic loads expected.

3.4.2 Bridge

Where a large drain crosses a road, a bridge over the drainage channel should be provided to maintain the smooth traffic. An adequate clearance should be maintained between the water surface and the bridge so that accumulation of debris can be avoided. The application of bridges to various types of crossing is described in detail in Section 4.3 and also illustrated in Figure 4.8. For detail of the procedures of hydraulic analysis of bridge waterway, refer to DID's "Design Standards and Procedures."

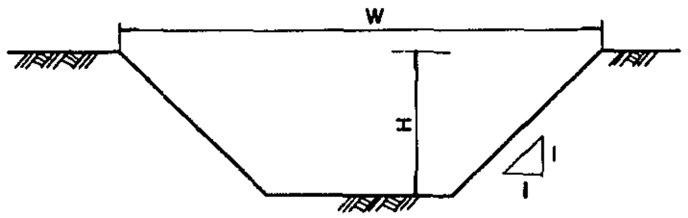
V - Shape Drain



m

W	0.23 ~ 0.61	a	0.05 ~ 0.09
H	0.23 ~ 0.61	b	0.06 ~ 0.10

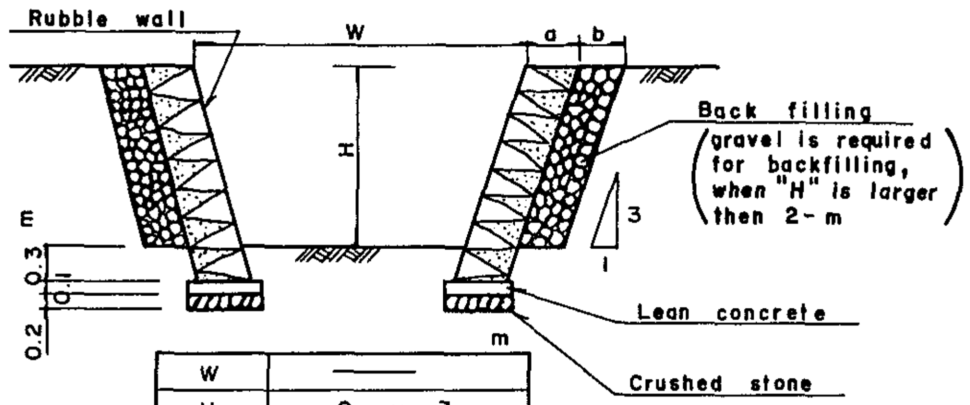
Earth Channel



m

W	——
H	0 ~ 3

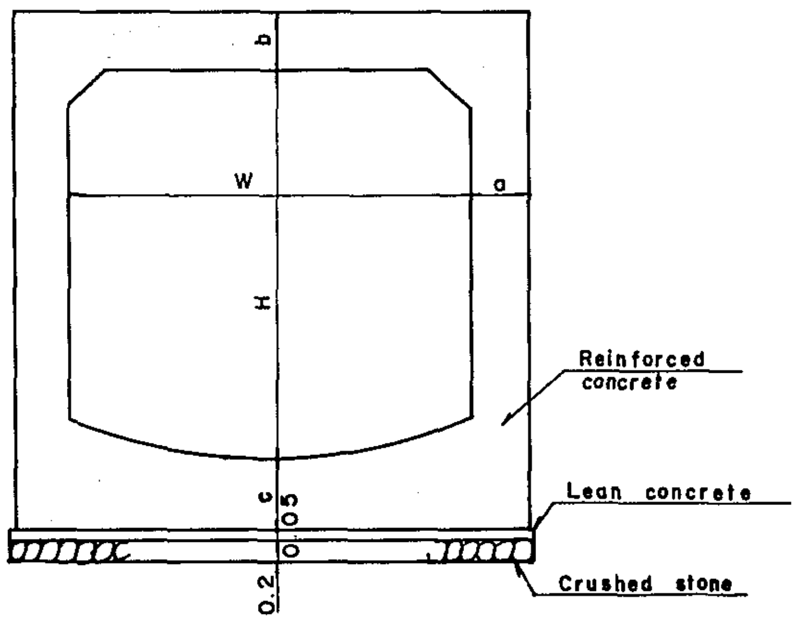
Rubble Wall Channel (With & Without Mortar Linning)



m

W	——
H	0 ~ 3
a	0.15 ~ 0.30
b	0 ~ 0.25

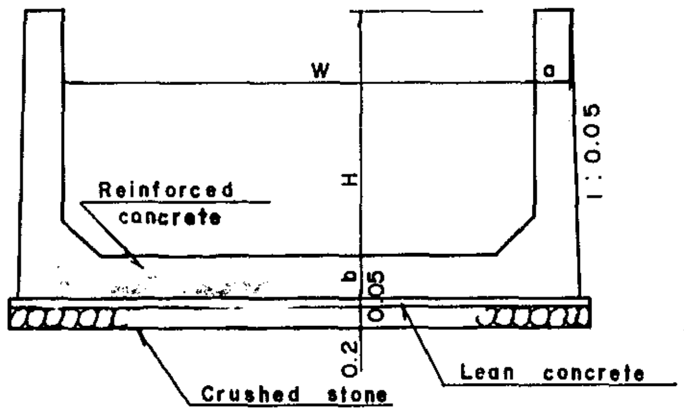
Cast in place R.C Box Culvert



m

W	1.0 ~ 4.2	a	0.15 ~ 0.47
H	1.0 ~ 4.2	b	0.20 ~ 0.50
		c	0.25 ~ 0.58

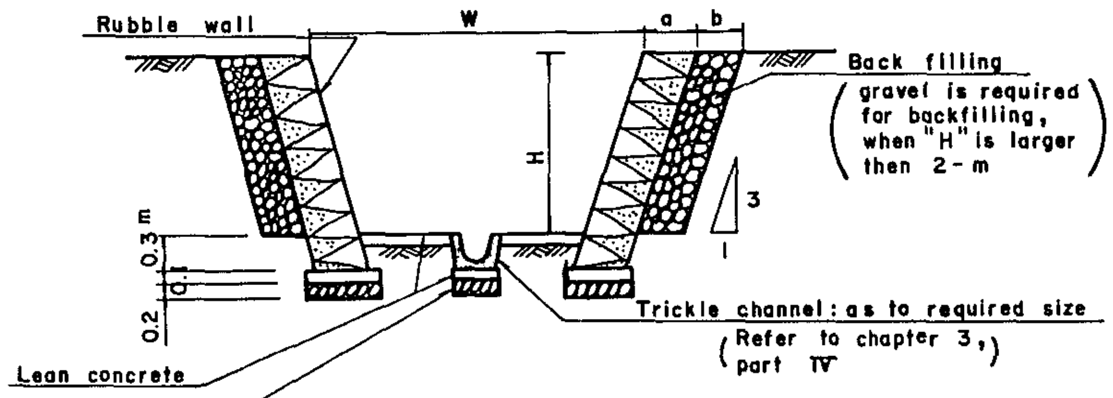
Cast in place R.C Open Channel



m

W	0.5 ~ 11.0
H	0.4 ~ 4.6
a	0.15 ~ 0.2
b	0.15 ~ 0.43

Rubble Wall Channel with Trickle Channel (With & Without Mortar Linning)



m

W	——
H	0 ~ 3
a	0.15 ~ 0.30
b	0 ~ 0.25

SEWERAGE AND DRAINAGE SYSTEM PROJECT BUTTERWORTH/ BUKIT MERTAJAM METROPOLITAN AREA, MALAYSIA	
TYPICAL CROSS SECTION OF DRAINS	FIGURE 3·1

CHAPTER 4

PRELIMINARY ENGINEERING DESIGN AND RECOMMENDATIONS

4.1 Present Conditions and Requirements for Improvement

The present conditions of the existing drainage system in the Study Area have been evaluated through field surveys. The requirements for drainage improvement are then identified on the basis of the results of the surveys and also in consultation with various government agencies concerned. The present conditions and evaluation of the system are described briefly in the following paragraphs:

4.1.1 Evaluation of Existing System

Information and engineering data have been gathered through reconnaissance and survey and also from the government agencies. The survey includes measurement of cross sections and levelling of the existing drainage channels. The results of survey and evaluation on the system in each drainage area are described in the following paragraphs and also presented in Figures DD-21 through DD-24 of Volume V.

(a) Butterworth Area (Drainage Basin IV)

Existing drainage system in this basin consists of various types of facilities and elements provided to relieve low-lying area from floodings caused by either stormwater or backwater due to high tide.

There exist four major outlets for the drainage system as shown in Figures DD-1 and DD-2 of Volume V. They are basically natural streams which have been improved partly to meet the increased stormwater runoff due to the gradual urbanization in the area.

Under the present ground surface conditions which have low runoff coefficients, the major streams generally have sufficient capacity to flow the surface runoff for the storm of 2-year frequency. In addition to the effect of the low runoff coefficients, the existing swamps have contributed significantly to alleviate the burden of flooding. Location and capacity of the existing drainage system are indicated in Figures DD-1, DD-2 and

DD-21 through DD-24 of Volume V.

At the individual outlet of the drain, a tide gate either sluice or flap type is provided and operated daily by hand. Some of the gates are not functioning properly due mainly to inadequate maintenance. These gates will be efficient for protecting the lowest part of the area from the backing up of the high tide if appropriately operated. In some cases, however, the gates cause flooding because of their capacity not sufficient to pass the increased water or clogging by accumulated debris.

Throughout the builtup urban area, open channel system of either V-shape or rectangular section is provided. These channels are frequently cleaned by the Municipality and have been maintained in good condition. In kampung area, however, there are no lined channels but earth excavation with insufficient capacity and density to cater for the stormwater.

There are several outfalls along the seashore of Butterworth to discharge the stormwater runoff. They are of rectangular open channel and reinforced concrete pipe without having much problems of sand deposit. From this experience, concrete pipe is preferred for the outfall system.

(b) Bukit Mertajam Area (Drainage Basin II)

The entire Bukit Mertajam area is tributary to the Juru River as shown in Figure DD-2 of Volume V. In the hilly zone in the area, there are now seven stormwater outlets which have been provided by the piecemeal improvement of natural waterways. Bukit Mertajam drain is the only exception because it has been improved entirely.

In view of the present low runoff coefficients and the steep slopes of the drain, it is evident that the existing drains have sufficient capacity to cater for the runoff from the area for the time being.

The existing drains in urbanized area are of concrete or rubble wall with V-shaped or rectangular cross section. In most cases, they are provided with trickle channel. These lined drains are generally in good condition because of frequent cleaning, while in kampung cleaning is not done properly and local pondings are frequently observed.

4.1.2 Flood Problems and Drainage Requirements

In determining the priority of drainage implementation, the flood problems and drainage requirements have been evaluated. The data and information needed for the study are collected from DID and MPSP and also through the house-to-house visit during the course of the study.

For the assessment of the drainage requirements in the immediate future, the information on new housing schemes, both under implementation and in the process of application, are collected from TCP and MPSP. The findings and evaluation of the present situation of the drainage system are briefly explained in the following:

(a) Butterworth Area (Drainage Basin IV)

On the basis of the preliminary discussions with the government agencies, inquiries are made to the residents together with investigations on conditions of drains and flooding.

The causes of flooding in the area are due to low ground elevations, inadequate capacity of drains, poor maintenance and the lack of sufficient drainage reticulation system. The location of the present flood prone areas is shown in Figures DD-1 and DD-2 of Volume V and the conditions in each of drainage system are presented in Table 4.1.

Generally, the existing drainage system has been well managed so far to control stormwater runoff; however, the rapid urbanization in recent years has resulted in the increase of surface water runoff, which cannot be handled by the existing system unless necessary improvement measures are taken. It is also found that the present development in the upstream of the area has contributed to the flood in this area to some extent.

In view of the above findings and evaluation of the area, it is apparent that if no countermeasures are taken the situation will be further deteriorated. For the improvement of the drainage situation in the area, the following measures should be taken immediately:

- Allevialation of the present flood problems at the earliest possible date by implementing an immediate scheme, which in turn would be integrated into the overall drainage system, and

- establishment of a drainage programme to meet the expected requirements considering the future urbanization in the area.

(b) Bukit Mertajam Area (Drainage Basin II)

In the hilly zone of the area, no significant floods have been caused; however, because of the recent urban development spreading farther toward lower portion of the area, the flood problems have gradually been increasing. The flood prone areas in this basin are shown in Figures DD-2 of Volume V and the causes and countermeasures for the flood are also presented in Table 4.1.

Currently a housing development programme is underway along the Juru River and swamps are being reclaimed. The swamp was functioning originally as stormwater reservoir for the areas upstream of the existing Juru tide gate. Consequently, as the programme further proceeds the river water level will rise and the flood problems will become more serious. This situation has to be prevented by an appropriate drainage programme.

4.2 Alternative Study of Drainage System

On the basis of the previous studies possible alternative countermeasures for flood control have been analyzed including both engineering and economic aspects.

4.2.1 Alternative Study on Type of Channel

Types of channel presently used in the Study Area are V-shaped concrete channel with semi-circular invert, rectangular section with rubble wall and trickle channel invert, trapezoidal section lined with concrete slab or rubble, and earthern channel (see Figure 3.1). Taking into consideration of the advantages on hydraulic, economic and esthetic points, the following types of drains are proposed for the drainage system design:

- V-shaped concrete channel with semicircle invert,

- Trapezoidal section of rubble wall with or without trickle channel,

- Rectangular section either reinforced concrete or retaining wall, and

Table 4.1 Flood Prone Area and Cause

Area Code*	Name of Outlet	Cause of Flood
A	BWD	• Shortage of infrastructual drains • Low ground elevation
B	BWE	• Inadequate maintenance and capacity
C	SEA•A	• Inadequate capacity of culvert • Low ground elevation
D	BWA	• Inadequate capacity of channel and culvert • Low ground elevation
E	BWC	• Inadequate capacity of channel • Low ground elevation
F	BWB	• Low ground elevation
G	ARA	• Inadequate capacity of culvert • Shortage of infrastructual drains

* Area Code is shown in Figures DD-1 and DD-2 of Volume V.

- Earthern or grass lined channel.

Economic analysis has been made on trunk and large drains, comparing rubble wall trapezoidal channel with rectangular reinforced concrete retaining wall channel, as described in Annex 1. The analysis indicates that the rubble wall trapezoidal drain is superior to the rectangular channel in terms of total cost if land cost is M$160/m^2 or less, however, at the area where land cost is M$160/m^2 or higher, reinforced concrete retaining wall channel is more feasible.

4.2.2 Alternative Routes of Main Drains

To select the most relevant routes of main drains for the Study Area, possible alternative routes of the drains are selected and studied on their advantages and disadvantages. The study is briefly described in the following:

(a) BWA Drain System

As shown in Figures 4.1 and 4.2, the tributary of this drain system lies along the coast of Butterworth and covers the different land use areas, i.e., congested areas mixed with residential and commercial at lower reach and areas at upper reach wherein rapid urbanization is now underway. Under the circumstances, the drainage improvement is urgent in the upper reach but relatively moderate in the lower reach.

The ground elevation in the area is generally high, ranging between +3.0 and +3.5 metres (+9.8 to +11.5 feet) in the upper reach and +1.5 and +2.0 metres (+4.9 to 6.6 feet) in the lower reach. In view of the topographic conditions of the area, the tributary can be divided into two basins, upper and lower reaches, for the drainage provision.

Taking the above mentioned situations into account, two possible alternative means are studied, including (1) diversion of the stormwater of upper reach, and (2) separation of BWE-1 and BWE-2 drains from BWA drainage system.

Diversion of Stormwater of Upper Reach to the Sea:

On the basis of detailed field survey and discussions with the government agencies concerned, two possible routes (A-3 and A-6) for diversion have been selected and analyzed in their advantages and disadvantages including cost estimates as summarized in Figure 4.2.

In this drainage area, the improvement of the drains A-4, A-5 and A-6 is imminent, while the drains A-1, A-2 and A-3 can cater for the present runoff from the tributaries. Case A (see Figure 4.2) plans to separate the system into two groups, one is combination of A-1, A-2 and A-3, and the other A-4, A-5 and A-6. By doing so, it will be possible to relieve the area A-1 from flooding in a relatively short time at a minimum construction cost, which otherwise will require more time and cost for the construction of drains downstream.

There are at present 12 outfall systems along the seashore of Butterworth discharging the stormwater directly to the sea. A survey on the outfalls indicates that 11 outfalls are of reinforced concrete pipe functioning properly without having sand deposit and clogging problem; but one system of open channel has been broken by the wave. It is also found that the sea bed of Butterworth beach is covered by the cohesive mud layer and the outfall systems provided on the mud layer have been functioning successfully without much sand accumulation. Thus, it is concluded that the outfall system be of concrete pipe extending up to the portion of the mud layer.

Construction costs for the two cases of outfall system are estimated and compared each other, as shown in Figure 4.2. From the figure, it may be found that Case A is superior to Case B in terms of the construction cost. In view of its lower construction cost and cost effectiveness, Case A is recommended for drainage outfall system in this area.

Separation of BWE-1 and BWE-2 from BWA Drainage System:

In the lower reach of BWA, the tributaries of E-1 and E-2 can be excluded from the planning because the right-of-way of the drain A-4 is not available. Tributaries of E-1 and E-2 will be drained directly to the Prai River through E-3 and E-4. If this is the case, the additional construction of E-3 is required. There is presently no effective drain network in the tributaries of E-2 and E-3, but only small size earthern drains, channels lined by wood sheets, or both, provided by either inhabitants or the Department of Health, are serving to drain sullage water. The provision of the effective drains is therefore urgently needed in these areas to bring to a minimum sanitation level. Thus the construction of E-3 drain will benefit not only its own tributary but also BWA drain as a whole.

Currently, the flows from the tributaries of A-12 and A-13 are to be emptied to BWA drain system through A-14. From the viewpoint of the limited available right-of-way in A-14, it is preferable to connect A-13 to E-1. However, the connection will require immediate improvement of E-1 and E-2 to effect the system. These conditions will make the improvement and construction of such system unwarranted at this stage because of its significant requirements for advanced investment.

Construction costs for the above two cases are estimated, as shown in Figure 4.3, however, there is no significant cost difference between the two cases.

(b) BWB Drain System

BWB drain system collects the stormwater from the central portion of Butterworth town area and empties it to the Prai River (see Figures 4.1 and 4.4), but at several points of the downstream the stormwater is presently dispersed into swamps.

The downstream (B-5 and B-6) of the drain should be improved in the near future with the new housing development scheme. At the upstream of the drain, there are also several housing development schemes either under planning or in progress.

Since there exist swamps in the drainage area, a full scale improvement of the drainage system is considered not feasible at this time, although the existing drain capacity is not sufficiently enough to cope with the expected increase of runoff by these development schemes. Under the above mentioned conditions of the area, the possible alternative improvement plans for the area are as follows:

Connection of Upper Reach to BWC Drain System:

As a means to reduce the discharge at the downstream of the drain, the possibility to direct the upper reach of BWB to BWC is studied with its advantages and disadvantages. The construction cost to be required for the diversion (Case A), as summarized in Figure 4.4, indicates that the connection system is a little more expense. Although the required cost for the connection is not so significant, the additional cost for C-2, C-3 and C-5 will also be required for the increased capacity inflowing from BWB drains (B-1 and B-2). Since the drains of C-2, C-3 and C-5 have to be constructed at an earlier stage than that of B-1 and B-2 due to the expected housing development schemes in the respective area, this pre-investment, which will not effectuate for the time being, is not warranted from the view point of effective investment. Therefore, the connection system (Case A) is not recommended.

Relief of Downstream of BWB Drain System:

In order to relieve the downstream of BWB drain, provision of a reservoir in the middle of the drain is considered feasible. As a site for the reservoir, existing swampy area can be used. The distance from the site to the point of discharge to the Prai River is about 2.5 km. As mentioned previously, the portion of about 1.1 km from the outfall point should be improved in accordance with the new housing development schemes in the near future, hence the portion which will be improved by the reservoir is reduced to about 1.4 km. Areas at both sides of this part are presently not fully developed yet, but when these areas are developed, contribution for the construction of the system and drain reserve by private developers of the schemes can be expected.

For the time being, it is not necessary to construct the reservoir to reduce the cross sectional area of the drain. The existing swamps will contribute to relieve the area from flooding. Besides, the improvements of the down reach and road crossings of the area will make the conditions much better than it is now.

Improvement of Gates:

The ground surface elevations of the area vary from +3.5 metres (+11.5 feet) to lower than +1.0 metre (+3.3 feet). The low-lying area lies in the down reach of the BWB drain system and is protected presently by a tide gate from the high tide of +1.1 metres (+3.6 feet) or higher and from the mean high water level of spring tide of +1.68 metres (+5.5 feet) which is the highest one ever recorded.

It is apparent that the area which lies at such lower ground elevation liable to be attacked by the flood due to high tide occupies only a small portion of the entire drainage area. As the area is being urbanized the low-lying area will be further reduced in the future.

Thus, it is recommended that either enlargement or reconstruction in accordance with the implementation of the over-all new drainage system be not carried out, instead, the existing lower area be protected separately with various other remedial measures, including bunding and pumping. With this regard, more suggestions are described in detail in Section 4.3.

(c) BWC Drain System

This drain system consists of natural waterways. The only exception is the drainage system in Mak Mandin industrial estate.

This industrial area is situated in the extreme down reach of BWC drain and is presently protected by the swamp and tide gate at the outfall of the drain from flooding caused by high water level.

The key factor to be considered for the area is to protect the industrial area from the excepted increase of flow from the upstream portion due to the urbanization and also by the high tide.

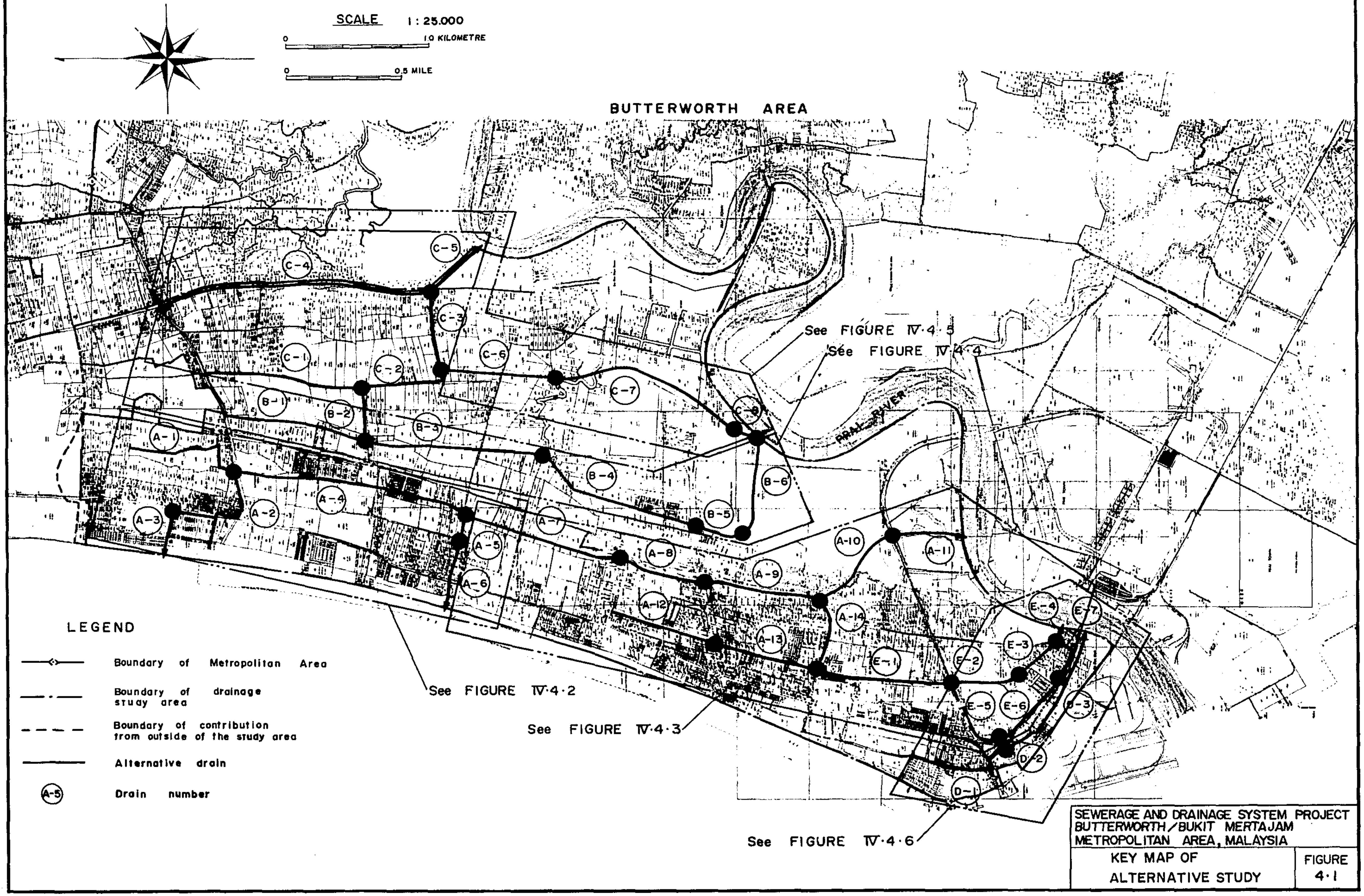
As the possible alternatives two cases (Case A and Case B) are studied. According to the new housing development scheme, the urbanization in upstream area is expected to start earlier than other area but leaving middle part of the area (C-6 in Figure 4.5) untouched. The capacities of C-7 and C-8 are large enough to flow the runoff under the present conditions of the tributary.

In Case B, C-6 and lower drains have to be improved to meet the increased discharge resulted from the urbanization in upper reach. The cost for this improvement will include large amount of pre-investment in addition to the cost required for the reconstruction of the existing tide gate when C-7 drain is improved. Since the existing gates are generally in good condition and functioning satisfactorily, it is not recommended to replace them by new ones at this stage.

Most desirable tide gate system for the area should be to serve only lower areas, and for this reason only the gates at lower part of the drain need to be modified. However, the modification and provision of the new tide gates will not be warranted until the bulk of the drainage area is urbanized.

On the other hand, an improvement and modification of the existing system are required in Case A. Further, the system has an outlet at the upstream serving only the residential area in which the 2-year rainfall frequency is applicable.

Disadvantage expected in Case A is the possible water pollution problem in the Prai River. Since the outfall of the upper reach of the system is located at the upstream of the tide gate near the Pontoon bridge, the continuous discharge of the dry weather flow might cause water pollution in the river. To by-pass the dry weather flow, a low weir is to be provided in C-5, and C-3 drain will be connected to C-6 drain with a flow control device. Under the by-pass system, the dry weather flow will be discharged to the Prai River at a portion downstream of the tide gate. In case of storm, the runoff flows over the low weir to the river and the portion not overflowed is led to C-6 through the control device provided at the connecting point.



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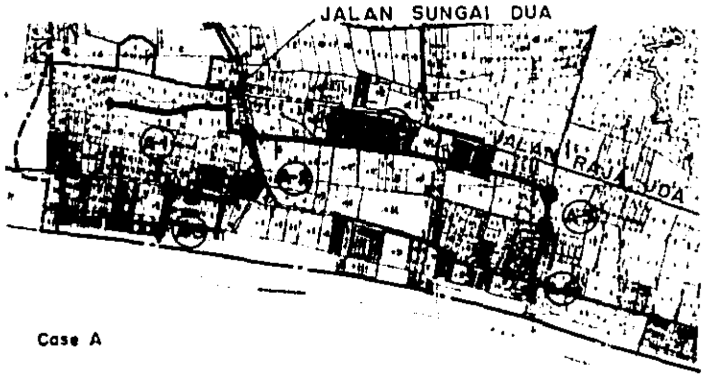
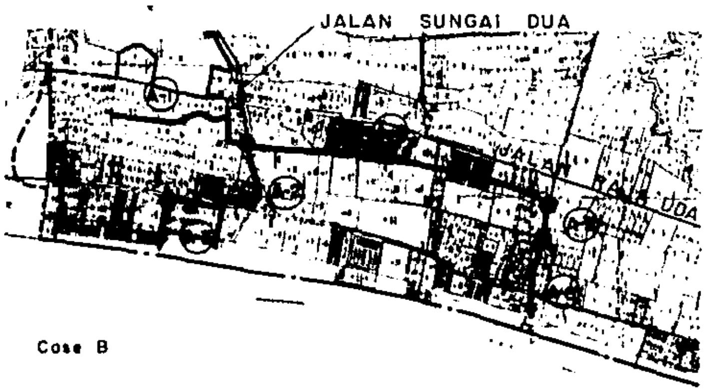
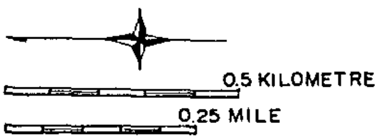
KEY MAP OF ALTERNATIVE STUDY	FIGURE 4·1



Case A



Case B



LEGEND

- Boundary of drainage study area
- Alternative drain
- Drain number

Case	A				B			
LINE NO	SIZE (mm)	CONSTRUCT-ION COST	LAND ACQ COST	M$1000 TOTAL	SIZE (mm)	CONSTRUCT-ION COST	LAND ACQ COST	M$1.000 TOTAL
A-4	MU 5 500 4.000 x 2 200	709	220	929	MU 6 500 4 800 x 2 600	848	281	1 129
A-5	MU 5500 4 000 x 2 200	88	33	121	MU 6 500 4 800 x 2 600	105	39	144
A-6	MU 6 500 4 800 x 2 600	264	99	363	MU 7 500 5 500 x 3 000	301	115	416
A-2	MU 6 000 4 400 x 2 400	412	153	565	MU 4 500 3 300 x 1 800	324	112	436
A-3	MU 6 000 4 400 x 2400	159	59	218	MU 5 000 3 700 x 2 000	136	49	185
TOTAL				2 196				2 310

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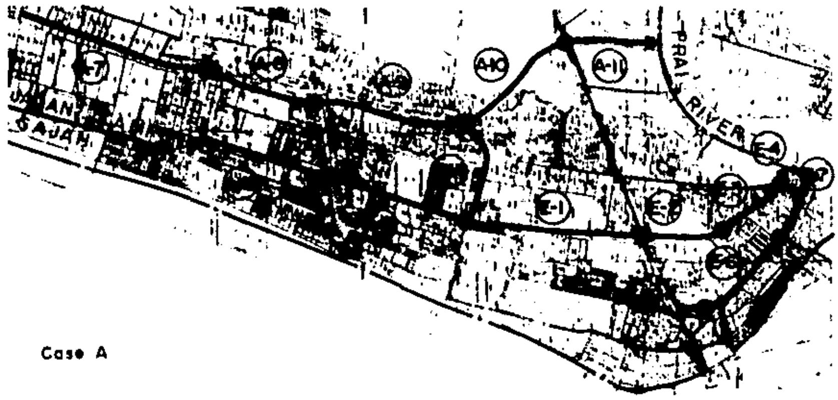
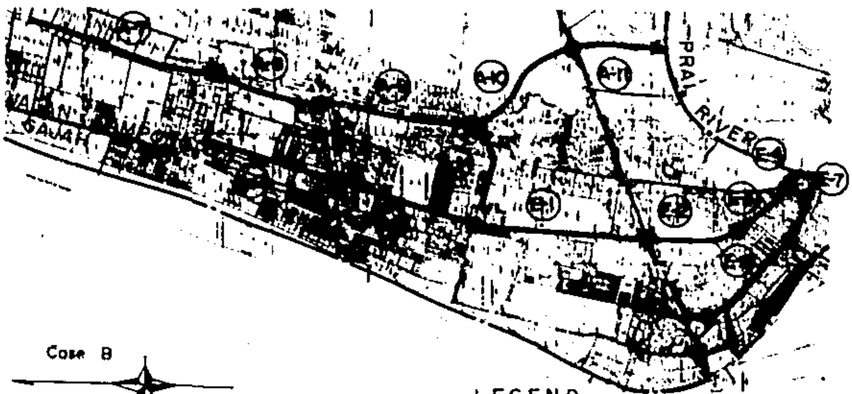
ALTERNATIVE ROUTES OF BWA DRAIN SYSTEM

FIGURE 4·2



Case A



Case B

0.5 KILOMETRE

0.25 MILE

LEGEND

Boundary of drainage study area

Alternative drain

Drain number

Case	A				B			
LINE NO	SIZE (mm)	CONSTRUCT-ION COST	LAND ACQ COST	M$1.000 TOTAL	SIZE (mm)	CONSTRUCT-ION COST	LAND ACQ COST	M$1000 TOTAL
A-10	MU 8 500 6 500 × 3 000	390	6	396	MU 7 000 5 100 × 2 800	340	0	340
A-11	MU 9 000 7 000 × 3 000	340	0	340	MU 7 500 5 500 × 3 000	298	0	298
A-14	MU 5 000 3 000 × 3 000	270	778	1 048	MU 4 000 2 900 × 1 600	191	542	733
E-2	MU 4 800 2 900 × 2 900	227	102	329	MU 5 500 3 500 × 3 000	251	123	374
E-1	MU 4 400 2 600 × 2 600	385	1 263	1 648	MU 5 000 3 000 × 3 000	438	1 468	1 906
E-3	MU 5 000 3 100 × 2 900	162	40	202	MU 5 500 3 500 × 3 000	174	43	217
E-4	MU 6 500 4 800 × 2 600	49	0	49	MU 7 500 5 500 × 3 000	56	0	56
TOTAL				4 012				3 924

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ALTERNATIVE ROUTES OF BWA AND BWE DRAIN SYSTEM

FIGURE 4·3

0.5 KILOMETRE

0.25 MILE

LEGEND

Boundary of drainage study area

Alternative drain

(B 2) Drain number

Case	A				B			
LINE NO	SIZE (mm)	CONSTRUCT-ION COST	LAND ACQ COST	M$1,000 TOTAL	SIZE (mm)	CONSTRUCT-ION COST	LAND ACQ COST	M$1,000 TOTAL
C-8	MU 12 000 10 000 x 3 000	166	60	226	MU 12 000 10 000 x 3 000	166	60	226
C-6	MU 5 500 4 000 x 2 200	331	31	362	MU 7 500 5 500 x 3 000	450	53	503
C-7	MU 12 000 10 000 x 3 000	1 012	296	1 308	MU 12 000 10 000 x 3 000	1 012	296	1 308
C-3	MU 5 500 4 000 x 2 200	260	53	313	MU 6 000 4 400 x 2 400	284	58	342
C-5	MU 4 000 2400 x 2400	182	32	214				
TOTAL				2 423				2 379

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ALTERNATIVE ROUTES OF BWB AND BWC DRAIN SYSTEM

FIGURE 4·4

0.5 KILOMETRE

0.25 MILE

LEGEND

Boundary of drainage study area

Alternative drain

Drain number

Case	A				B			
LINE NO	SIZE (mm)	CONSTRUCT-ION COST	LAND ACQ COST	M$1,000 TOTAL	SIZE (mm)	CONTRUCT-ION COST	LAND ACQ COST	M$1,000 TOTAL
B-3	MU 5 500 4 000 x 2 200	545	117	662	MU 6 500 4 800 x 2 600	650	147	797
B-4	MU 7 000 5 100 x 2 800	859	202	1 061	MU 8 000 6 000 x 3 000	756	245	1 001
B-6	MU 8 500 6 500 x 3 000	502	0	502	MU 10 000 8 000 x 3 000	549	0	549
B-5	MU 7 000 5 100 x 2 800	223	63	286	MU 8 000 6 000 x 3 000	197	71	268
C-2	MU 7 000 5 100 x 2 800	351	69	420	MU 5 500 4 000 x 2 200	273	56	329
C-3	MU 7 000 5 100 x 2 800	334	66	400	MU 5 500 4 000 x 2 200	260	53	313
C-5	MU 4 800 2 900 x 2 900	210	37	247	MU 4 000 2 400 x 2 400	182	32	214
B-2	MU 6 000 4 400 x 2 400	194	32	226	MU 2 000 1 200 x 1 200	65	7	72
TOTAL				3 804				3 543

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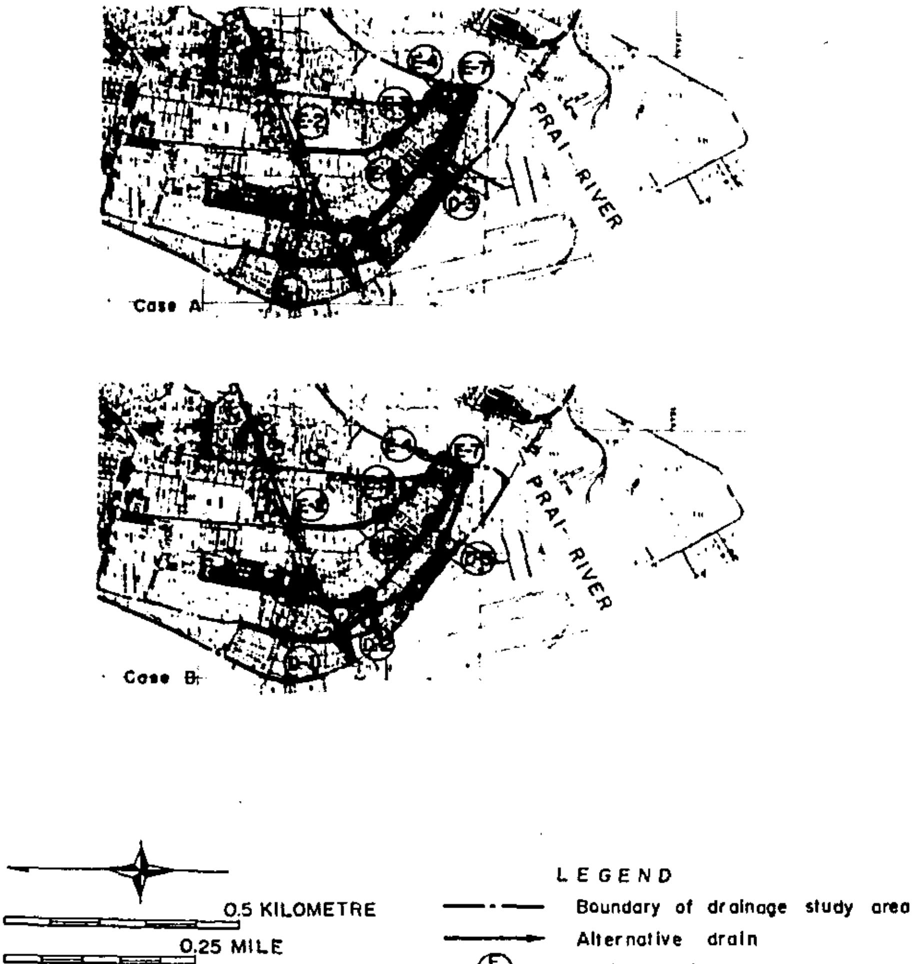
ALTERNATIVE ROUTES OF BWC DRAIN SYSTEM

FIGURE 4·5



LEGEND

Boundary of drainage study area

Alternative drain

Drain number

Case	A				B			
LINE NO	SIZE (mm)	CONSTRUCT-ION COST	LAND ACQ COST	M$1000 TOTAL	SIZE (mm)	CONSTRUCT-ION COST	LAND ACQ COST	M$1.000 TOTAL
D-3	MU 4500 3300 x 1800	394	62	456	MU 5000 3700 x 2000	428	72	500
E-6	MU 3000 1 400 x 2400	189	28	217	MU 2400 1 100 x 1 900	136	22	158
E-4	MU 4 000 2 400 x 2.400	35	0	35	MU 3 000 1 400 x 2.400	29	0	29
E-7	MU 3 000 1 400 x 2 400	122	22	144	MU 2 700 1 300 x 2 200	108	20	128
TOTAL				852				815

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ALTERNATIVE ROUTES OF BWD AND BWE DRAIN SYSTEM

FIGURE 4·6

Construction costs for both Case A and Case B are estimated and summarized in Figure 4.5. The cost for Case A is slightly higher than Case B but not high enough to trade off various advantages expected in Case B. It is also apparent that the expected damage in downstream industrial area due to the Major Storm would be less than that in Case A.

(d) BWD and BWE Drain Systems

The tributary area of the BWD drain extends up to the point near the Ferry Port, with four road corssings in the busiest areas in Butterworth town.

Reconstruction of these existing road crossings would be impractical under the present situation. The possibility of the reduction of the BWD drainage catchment area is studied so that the construction of such crossings can be avoided as much as possible by other suitable alternatives. The study indicates that connections of E-5 and E-6 to E-7 are the best solution to avoid the problem. Also construction of E-7 will be required because of the presently deteriorated drainage situation. For the reasons mentioned above, the connection of these drains should be provided at earlier stage of the drainage programme.

The construction cost by use of the existing the new route are compared as presented in Figure 4.6, but there is no significant difference between the two cases.

In view of the engineering advantages mentioned above, Case A is recommended for the BWD drain system.

(e) PMT, BKC and BKD Drain System (refer to Figure DD-8, Volume V)

Part of the catchment areas of PMT, BKC and BKD is now under development for housing. Runoffs from these three drainage are presently discharged to the Juru River. These are shown in the existing drainage system in Figure DD-2, Volume V.

It is expected that the water level in the upstream of the existing tide gate in the Juru River will be raised as the hinterland is urbanized. The water level is estimated based on following assumptions:

- The existing tide gate will be preserved for the time being.
- The considered region of the Juru River, from Jalan Sungai Rambai to the Juru Tide Gate, is almost flat and has a reserve width more than 100 metres. Therefore, this portion

can be assumed to function as a pond with the gate and the overflow weir at its outlet.

- Accordingly, the water surface throughout this portion is assumed to be level. Nevertheless, there would be head loss between both ends, due to the flow in the river. Because of absence of data as to the grade of water surface in the considered portion of the Juru River, allowances are given for estimating the water level in the upper part.

On the basis of the above assumption, the water level in the Juru River at its portion upstream of the Juru Tide Gate is roughly estimated for the purpose of studying the drainage situation expected at the time of heavy rain under urbanized catchment condition. In Annex 2, detailed estimation of water level is described.

The estimated water levels are:

+ 1.3m (4.3ft) ∿ + 1.6m (5.2ft) .... 5-yr storm under present condition (C=0.15)

+ 1.9m (6.2ft) ∿ + 2.2m (7.2ft) .... 5-yr storm under the year 2000 condition (C=0.4)

Under the present condition, because of low runoff coefficient, the expected water level is relatively low. The ground elevation in the area west of the Juru River is +1.8m (+6.0 ft) at housing scheme portion and about +1.0m (+3.3 ft) in agricultural area including Kampung Bt Tengah. It is understood that the lower agricultural area can not be drained to the upstream of the Juru River when the storm with 5-yr frequency and the tide level of +1.1m (+3.6 ft) occur simultaneously. On the other hand, stormwater from the new housing area can be discharged to the upstream of the tide gate under present condition. However, as the basin is urbanized the runoff will be increased and when the urbanization reaches to the degree envisaged for the year 2000, the housing area can no longer be drained by the existing system to the Juru River.

Taking the above mentioned conditions into consideration, possible alternative countermeasures for the expected situation have been selected and studied as described below:

Alternative I. To lower the water level in the Juru River by increasing the width of the gate:-

The existing tide gate is of the twin slide type. Each gate is 10 ft (3.04 m) wide and 8 ft (2.44 m) high. Beside the tide gate, there is an overflow structure consisting of a series of box culvers of 3.5 ft (1.07 m; width) by 5 ft 1.52 m; height). With the present gate size, the water level will be about +3.0 m (+9.8 ft) when the Major Storm occurs (in this case, a runoff coefficient of 0.65 is used for calculation). If the gate is enlarged by 100 percent, the water level will be lowered to about +2.5 m (+8.2 ft). Thus, it is apparent that a significant lowering in the water level can not be expected by enlargement of the gate.

Alternative II. To drain the right bank area to downstream of the Juru Tide Gate.

An alternative consideration is to separate right bank areas from the upper reach of the Juru River and to connect outlets of the area to downstream of the tide gate. For the separation, embankment along the Juru River is necessary. In this system, the drain PMT is connected to BKD and then to BKC. From the end of BKC to the Juru River, an existing open channel is used as the outlet drain. Because the capacity of the drain is not adequate to carry stormwater from BKC, it is required to store runoffs within upstream drains including BKD and BKC.

It was found that the capacity of the existing drain to which BKC is connected, is about 3 m^3/sec and the required storage volume in BKD and BKC is approximately 150 x 10^3 m^3. This volume is available within the drain reserve of 30 m (100 ft) being set aside by DID.

The elevation of levee crown along the Juru River will be +2.3 m (=7.5 ft), which has been determined on the following basis:

(1) The water level in the river, under a storm of 5-yr frequency and a runoff coefficient of 0.4, is +1.9 m (+6.2 ft) ∿ +2.2 m (+7.2 ft).

(2) The water level at the time of the Major Storm under present land condition (a runoff coefficient of 0.3) is about +2.0 m (+6.5 ft).

(3) The maximum tide level recorded is +1.68 m (+5.5 ft).

The levee crown level should be higher than the highest tide level among those expected and it is determined to be +2.3 m (+7.5 ft) including allowances.

From above two alternatives, it is concluded that the right bank areas of the Juru River should be separated by embankment from the upstream portion of the tide gate and drained to downstream side of the gate. The stormwater generated in the catchment areas has to be stored within the area in order to avoid impact on the undeveloped areas downstream where full scale drainage improvement project is not warranted yet. This storage system can be considered as a tentative measure until the Juru Tide Gate is removed to upper reach or enlarged to the degree enough to lower the water level to drain tributary areas.

4.3 Recommendations and Proposals

On the basis of the results of the previous alternative studies, the best suited drainage system is selected and the recommendations thereof are described in the following paragraphs:

4.3.1 Proposed Drainage System

The proposed drainage facilities comprise open channels, pipe and box culverts, bridges, outfalls to the sea or river, tide gates, and embankment. For some lower portions, land filling has been recommended. Details of recommendations and proposals are described below:

(a) Trunk Drain

Recommended routes and profiles of the trunk drains are shown in Figures DD-5 ∿ 17 of Volume V. These routes are determined based on: (1) data contained in available topographical and other maps, (2) available results of ground surveys, and (3) available data on existing and proposed roads and new housing schemes. Reconnaissance and discussions with the government agencies concerned are carried out for checking and confirming these routes.

The gradients applied for the proposed channels are determined upon existing ground elevation in already developed areas. These gradients are extremely flat in the whole of Butterworth and a part of Bukit Mertajam area. As a result, the design velocity is very low, ranging from 0.8 to 1.0 m/sec in most part of the proposed drain system.

Due to the flat gradient together with the intensive storm, a large cross sectional area is required for proposed channels to accommodate peak discharge (Ref. Figure DD-21 ∿ 27 of Volume V).

The width of reserves for the trunk drains has been decided upon final development of land. The runoff coefficient is considered to be 0.65. Typical cross sections of the recommended reserves are shown in Figure 4.7. The total with of a drain comprises the space for open channel structure and maintenance access.

On either side of the drains, the maintenance access should be preserved in the form of road or green belt. These spaces should be set aside in association with the layout planning of streets in the area now underway by TCP. Land acquisition for maintenance purpose is not included in the land cost estimation because of the variety of the forms with which maintenance space is set aside. For example, existing or planned roads are available for maintenance spaces. In some cases land needed for maintenance will be contributed by private developers when the portion is urbanized, but in other cases the required land will have to be purchased. The required sizes of drain are calculated on the land use pattern for the year 2000 (Ref. Figure SD-9, Volume V).

Availability of right-of-way for drain routes in built-up areas has been examined on the basis of existing physical conditions. No consideration was given to the expected problems relating to land acquisition.

Prior to the final design, any new information on route, availability of right-of-way and land use should be evaluated to reflect the latest condition for the design. The stormwater quantities used for design should be reviewed in the design stage to allow any amendment to the tributary areas.

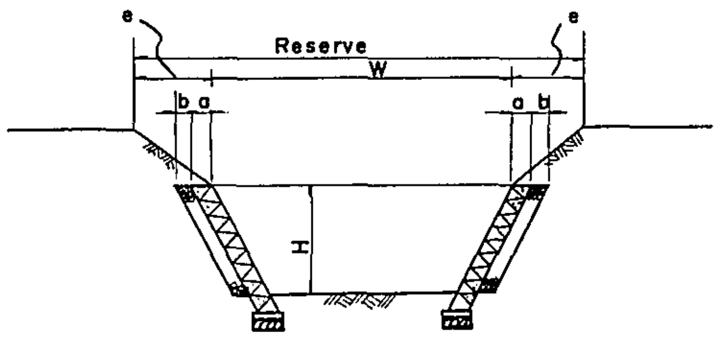
The tailwater of the proposed drains has been estimated for the condition of "Mean High Water Level" (+1.1 m or +3.6 ft). In the higher area, in which water level can be determined without considering the tidal influences, the flood level is set so that the drain can have a freeboard of 10 per cent of the total depth.

Any head losses caused by culverts or bridges in the channels should be minimized. Structures of road crossing should be so designed as not to interfere the flow in the channels. Since the area is flat and low-lying, such losses can not be affordable for the gravitational drainage system proposed herein.

As is described in the alternative studies, the recommended types of channels are of trapezoidal rubble wall, rectangular R.C., and R.C. retaining wall. The routes on which these types are applied, are shown in Figures DD-5 ∿ 8 of Volume V.

As can be seen from Figures DD-5 ∿ 8, the major part of trunk drain system consists of the trapezoidal rubble wall channel. The type has been selected on the basis of analysis on the total cost (construction plus land acquisition costs), availability of

Drain Reserve



Where

- W : width of drain ($C \leq 0.65$) (m)
- a : thickness of rubble wall (m)
- b : thickness of back-filling (m)
- H : depth of drain (m)

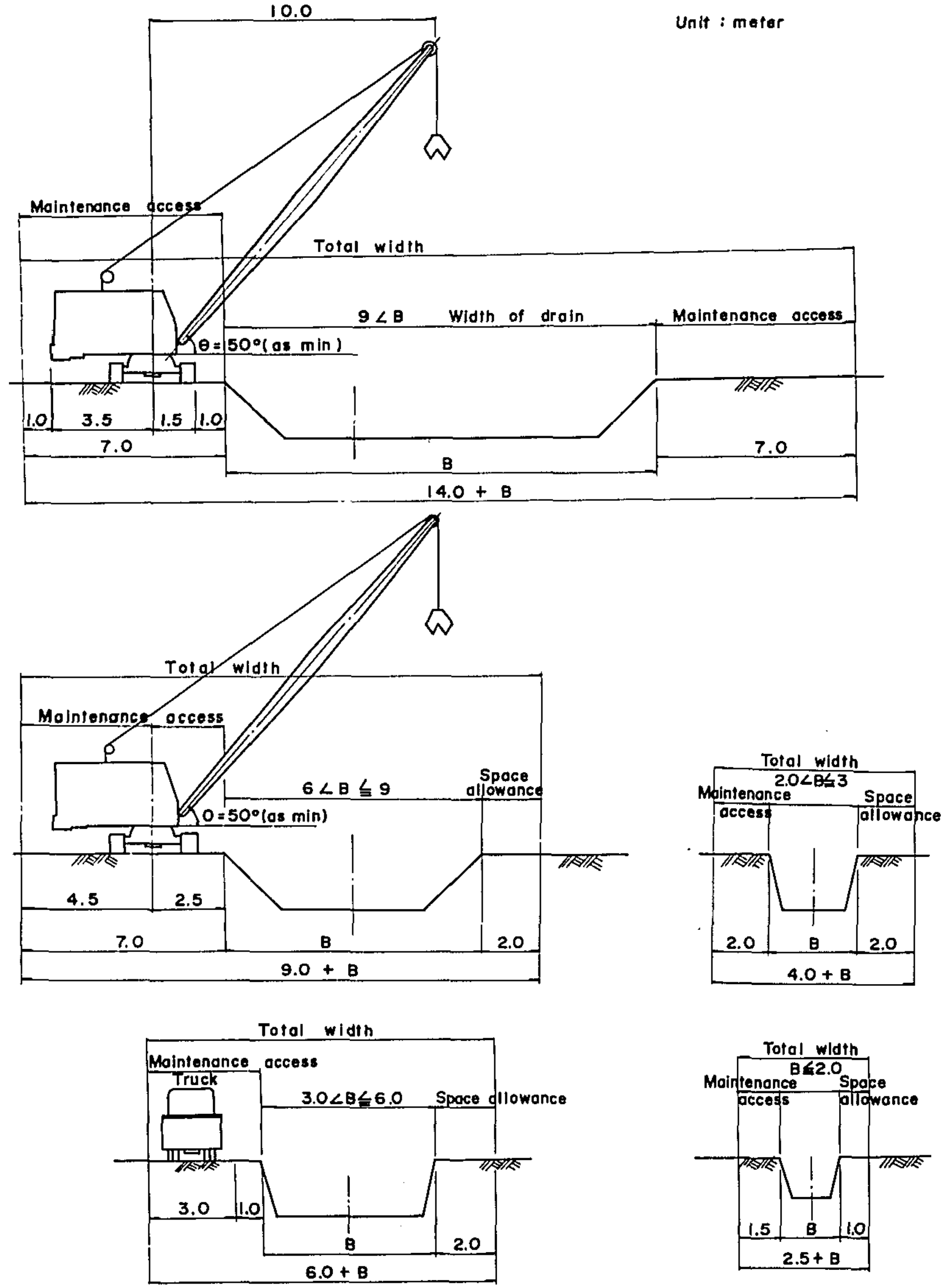
the value of "a" and "b" is :

for $H \geq 2.0^m$ ----- $a = 0.3^m$, $b = 0.25^m$

$H < 2.0^m$ ----- $a = 0.3^m$, $b = 0$

$$\text{Reserve width} = \frac{0.65}{C} \times W + (a + b + e) \times 2$$

Maintenance access could be preserved in the form of road or green belt in either or both sides of drains



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DRAIN RESERVE

FIGURE 4·7

materials and esthetic point of view. The sizes of sections required for individual drains are indicated in Figures DD-21 ∿ 27 of Volume V. Detail of the structures is shown in Figure 3.1. Where the soil is soft, piles should be driven to sustain the weight of the drain structures. To reduce the ground water pressure to rubble walls, drain holes should be provided in every two square metres on the wall.

When part of the channel crosses a roadway, a bridge or box culvert should be used. Many of existing road crossing channels are undersized box culverts and cause flooding under heavy rain. These should be replaced by newly recommended ones with enough capacity.

Box culverts are generally less in construction cost than bridges. In case of larger size channels, however, multicell boxes would be needed, and this type would cause obstruction for channel flows and/or accumulation of floating materials in upstream of culverts. Consequently, the use of bridges is preferable for relatively larger size. It is recommended that bridges be applied to the channel with upper width of 5.0 m (16.4 ft) or more. The locations of bridges are shown in Figures DD-5 ∿ 8 of Volume V. Other road crossings are to be provided with box culverts.

In both cases, a sufficient hydraulic opening area should be provided. Preferably, free board from design tailwater level to the bottom of bridge's beam or culvert slab, should be around 30 cm (1 ft). Under the existing topographic conditions in Butterworth and in the lower portion of Bukit Mertajam, it may be difficult to preserve sufficient opening area. However, bridge beam or culvert slab should at least not be submerged in the water of channels even in the designed tailwater level.

Typical bridge structure and box culvert are indicated in Figures 3.1 and 4.8. These figures are prepared as the base of cost estimation and explanatory purposes. At the time of final design, individual site of the construction has to be investigated further and proper type of structures should be selected respectively.

Outfalls to the sea consist of centrifugally cast reinforced concrete pipes in parallel extending from the seashore. Typical structure of the outfall is shown in Figure 4.8. In order to distribute the load of pipes evenly, concrete base is considered. Pile foundation is applied because the subsoil condition is expected very poor. The diameter and number of pipelines for each outfall is shown below.

Location	Diameter of Pipe (mm)	Number of Pipeline
BWA.A-7	1,800	2
BWA.B-7	1,500	5
SEA.A-3	1,800	1
SEA.B-2	1,800	1
SEA.D-3	1,200	2
SEA.E-3	1,350	2

At BWC.A drain system, overflow weir is applied for the purpose of preventing discharge of dry weather flow into the Prai River upstream of the planned barrage. Details of the weir is shown in Annex 4 and Figure 4.9. An orifice is constructed at the top of BWC.B-1 through which dry weather flow is diverted to BWC.B drain system. At the time of rain, only small quantity of stormwaters can pass through the orifice and the bulk will overflow the weir to the Prai River.

(b) Infrastructural Drain

Layout is determined for the infrastructural drains taking into account the existing conditions of channels and roads. For the areas yet to be developed, the route of the drains is selected from the planned roads. In some parts of the Study Area, street planning has not been developed yet, hence, only construction costs of drain networks have been estimated by applying adjusted unit cost per hectare according to land use of the area.

Figures 4.11 and 4.12 show the typical layout of infrastructural drains proposed in residential and industrial areas.

The recommended infrastructural drains are of "V" shape precast concrete, rectangular reinforced concrete, and trapezoidal rubble wall open channels. Considering the easiness and relative low costs of construction, "V" shape pre-cast concrete channels have been applied to the smaller drains. This type will contribute to preserve the valuable land space. In case of the larger drains, trapezoidal rubble wall or cast-in-place reinforced concrete is adopted.

The recommended sizes of the drains are summarized below:

Name of Channel	Range of Size
"V" shape concrete	0.23 m x 0.23 m - 0.61 m x 0.61 m
Trapezoidal rubber wall	$\frac{1.00\text{ m}}{0.30\text{ m}}$ x 1.00 m - $\frac{6.00\text{ m}}{4.40\text{ m}}$ x 2.40 m
Rectangular R.C.	1.70 m x 1.70 m - 3.70 m x 1.60 m

Trickle channels are provided for the relatively larger drains to convey dry weather flows. The drain with the trickle channel is shown in Figures DD-21 ∿ 27 of Volume V.

At many road intersections, centrifugally cast R.C. pipes or box culverts are applied.

In case of pipe culverts, at least one metre of earth covering is required for avoiding impact of heavy traffic load. For the open channel drainage system, the application of this type of culvert would be restricted for roads crossed by deep channels or utilized by only light traffics and pedestrians. The box culverts of pre-cast type, which is available locally, are applicable to any earth covering, if necessary depth for pavement has to be preserved. This type is flexible for application and has been widely used. However, construction costs of box culverts are higher than pipes. For the purpose of cost estimation box culverts are assumed for application.

In Figure 4.12, typical cross sections of pipe and box culvert are shown.

(c) Land Filling and Tide Gate

Some parts of undeveloped areas lie on an elevation lower than that of the designed tailwater for the proposed drains. The major parts of these low areas are swamps and presently being developed piece by piece basis by private developers. The time schedule of urbanization in the remaining parts is not known. Under this condition, land filling is one of the preferable measures to relieve the areas from flooding. At the time of the implementation of land filling, drainage outlet from upstream zones has to be considered and any adverse effect to the surrounding areas has to be prevented. In some special cases, pumps might be needed temporarily.

The filling elevation should be higher than the designed tailwater level and also the highest recorded tide level of +1.68 m

(+5.5 ft). This level is believed to have been the highest since 1912, when the recording of the sea level initiated. This level is used as the base for drainage planning by DID, Penang State. In the areas of Butterworth and Bukit Mertajam, the filling level is determined by the level of the highest tide recorded which is higher than the designed tailwater level in the areas to be filled up.

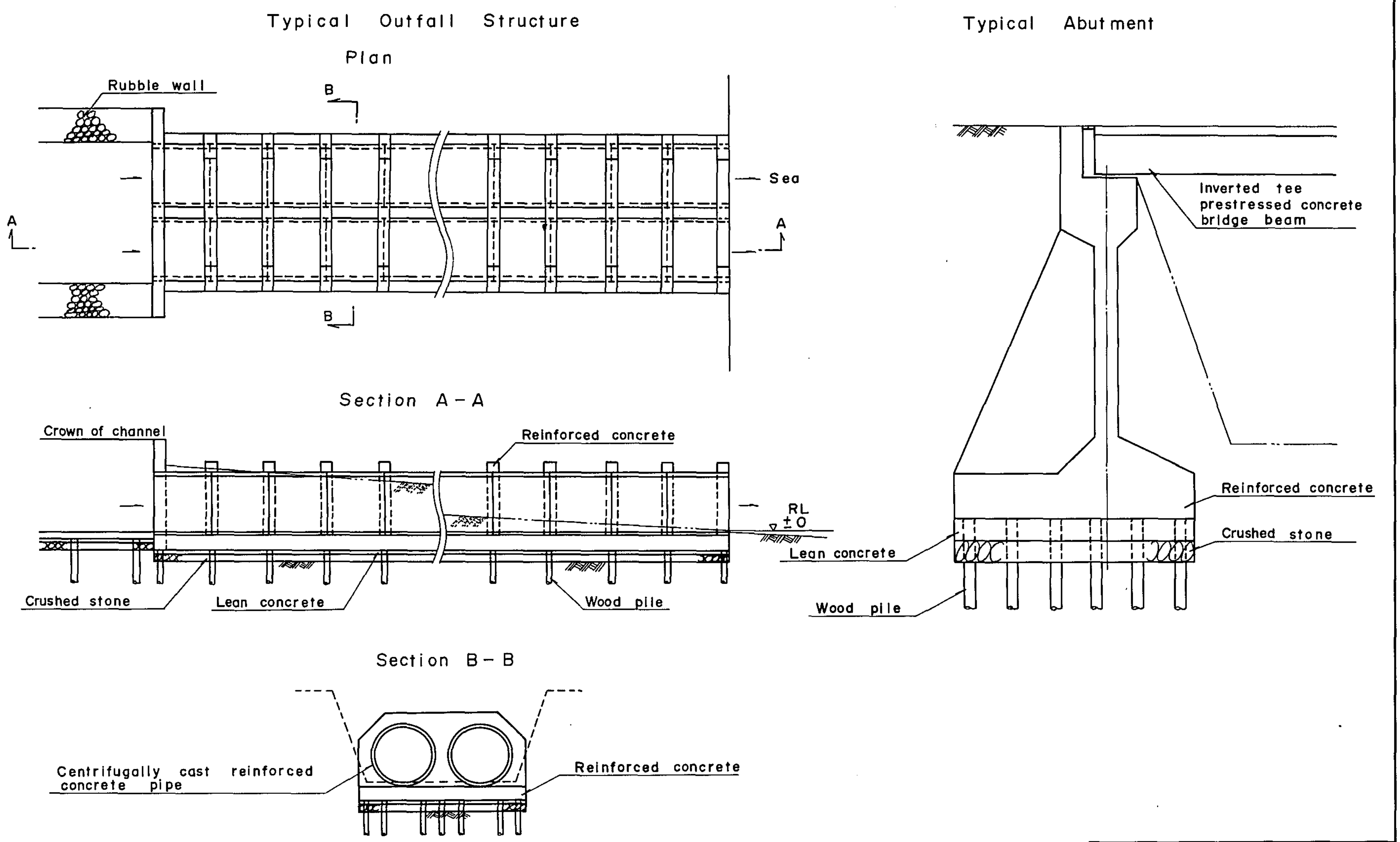
On the assumption that even at the time of the highest tide level there would be some flows from upstream to downstream, the water surface gradient in both of the Prai and Juru rivers was taken into consideration for deciding expected maximum water level. Taking distance from the river mouth to the considered areas into account and put allowances, the ground levels to be filled up are recommended as follows:

For Butterworth Area	+2.0 m (+6.56 ft)
For Bukit Mertajam Area	+2.3 m (+7.5 ft)

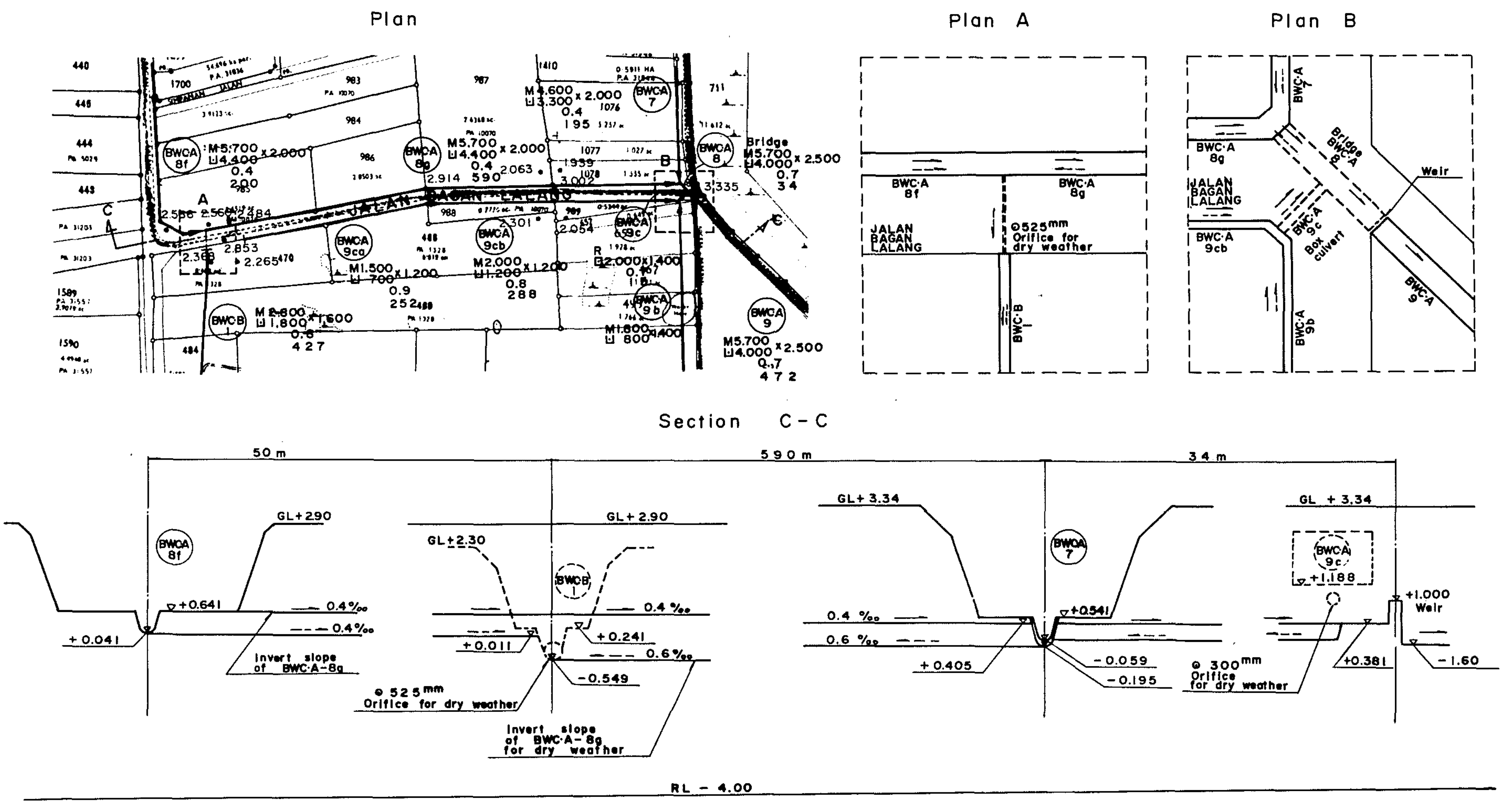
In Butterworth, there is a built-up portion lower than +1.68 m (+5.5 ft). The existing tide gates installed in the major drains receive stormwater not only from lower area but also higher land.

For the proposed trunk drainage system, gates are not recommended, because the sizes of the gates would be limited by the sizes of drains, which are always larger than those actually required. It is recommended that in lower parts, the drain network be separated from surrounding areas by an embankment. In such drainage system, in general, one gate is installed for every catchment area. In Figure DD-18 of Volume V, the area to be filled up and the location of existing and proposed tide gates are shown. However, two gates are provided for Mak Mandin industrial area, one on the right bank and the other on the left. Two main drains, each of which is connected to either of the gate by laying parallel to BWCB-5, are considered to collect all of the stormwater runoff in the area. Because the area will be lower than the surrounding areas, it is necessary to protect the area by banking along the boundary. The elevation of the embankment is recommended to be +2.5 m (+8.2 ft). This value is based on the surrounding ground surface elevation of +2.0 m (+6.5 ft) and allowances as freeboard.

Typical gate structure is shown in Figure 4.13. Flap gates are preferable to tide gates, because the flap gates open automatically to outflow or close against backflow with only a slight difference in head when properly installed. When the flap gates are equipped, continuous maintenance is required for keeping hinges in smooth condition and removing debris, which otherwise obstructs gate's action.



SEWERAGE AND DRAINAGE SYSTEM PROJECT BUTTERWORTH / BUKIT MERTAJAM METROPOLITAN AREA, MALAYSIA	
TYPICAL ABUTMENT AND OUTFALL STRUCTURE	FIGURE 4·8



LEGEND

———	Flow direction for storm water	⊔	Open channel
– – –	Flow direction for dry weather	⊡	Box culvert
BWCA 7	Line No	⊙	Centrifugally cast reinforced concrete pipe
M	Rubble wall drain	4.600 / 3.300 x 2.000	4.600mm : Upper width 3.300mm : Bottom width 2.000mm : Depth
R	Reinforced concrete drain	0.4	0.4‰ : Invert slope 1 / 1.000
		195	195 m : Incremental length

NOTE : Details refer to Section 4.3

SEWERAGE AND DRAINAGE SYSTEM PROJECT
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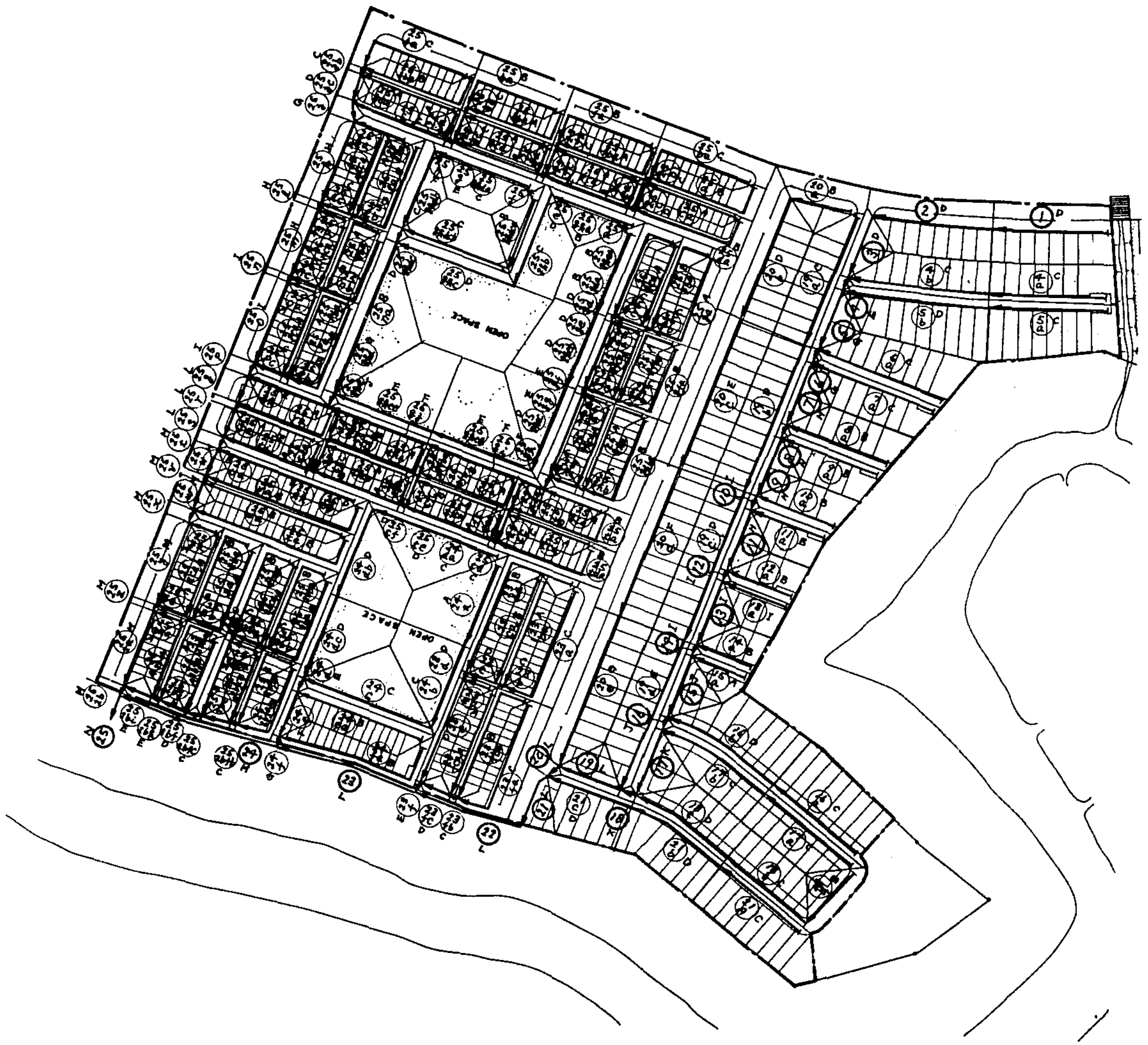
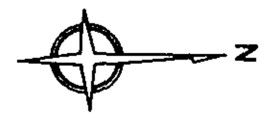
WEIR AND ORIFICE IN BWCA DRAIN SYSTEM

FIGURE 4·9

Drain size
(m)

A	⊔0.300x0.300
B	⊔0.380x0.380
C	⊔0.450x0.450
D	⊔0.600x0.600
E	⊔ $\frac{1.000}{330}$ x1.000
F	⊔ $\frac{1.100}{370}$ x1.100
G	⊔ $\frac{1.200}{400}$ x1.200
H	⊔ $\frac{1.300}{430}$ x1.300
I	⊔ $\frac{1.400}{470}$ x1.400
J	⊔ $\frac{1.500}{500}$ x1.500
K	⊔ $\frac{1.800}{840}$ x1.440
L	⊔ $\frac{2.100}{980}$ x1.680
M	⊔ $\frac{2.400}{1.120}$ x1.920
N	⊔ $\frac{3.200}{1.920}$ x1.920
O	⊔ $\frac{3.600}{2.160}$ x2.160





LEGEND

Limit of area considered

Drainage area boundaries

Proposed drain

Drain number
drain size
(see table "Drain size" on this page)

SEWERAGE AND DRAINAGE SYSTEM PROJECT
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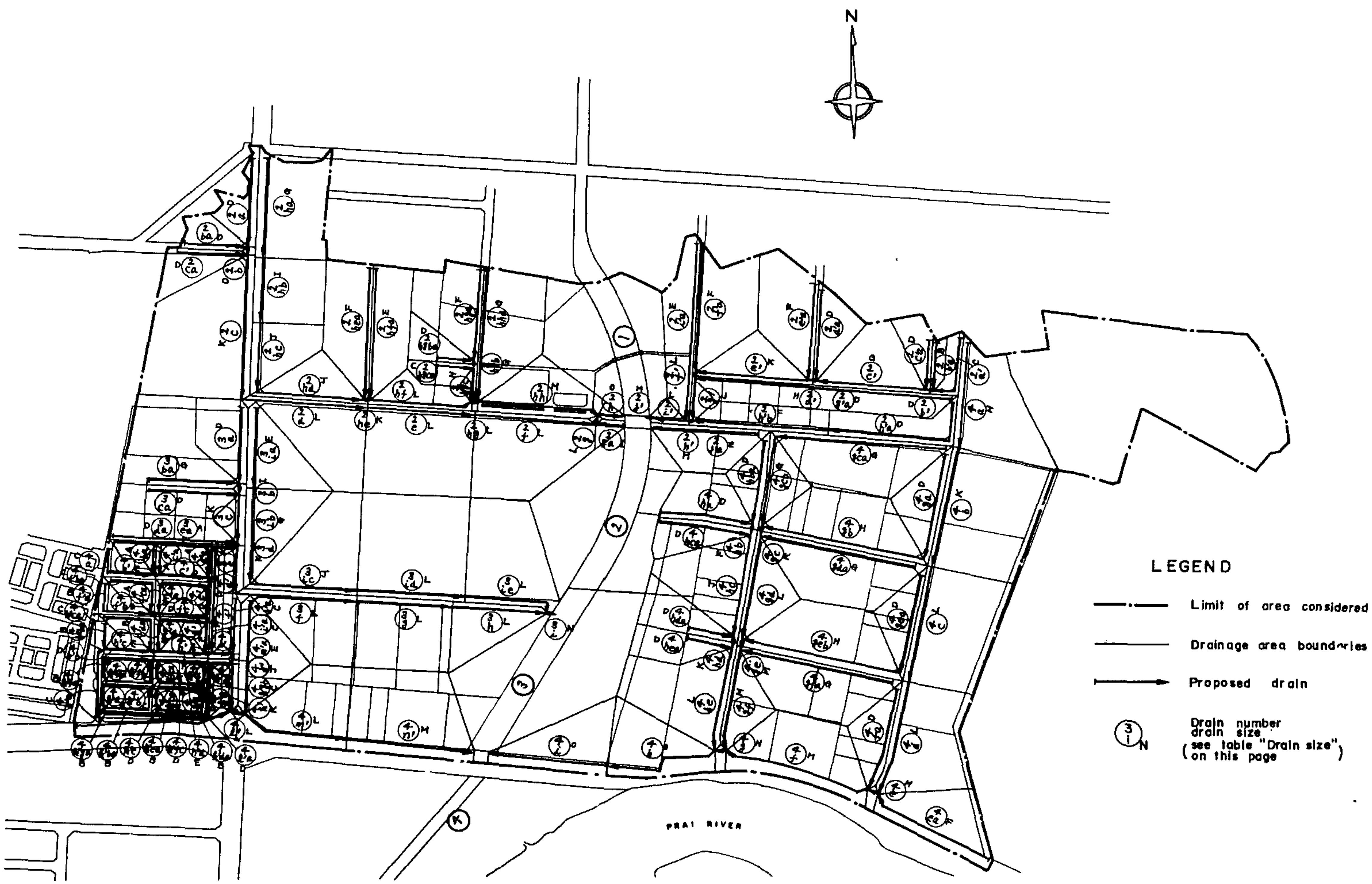
REPRESENTATIVE NETWORK OF SMALLER DRAINS IN RESIDENTIAL AREA

FIGURE 4·10

Drain size (m)

A	⊔0.300x0.300
B	⊔0.380x0.380
C	⊔0.450x0.450
D	⊔0.600x0.600
E	⊔ $\frac{1.000}{330}$ x1.000
F	⊔ $\frac{1.100}{370}$ x1.100
G	⊔ $\frac{1.200}{400}$ x1.200
H	⊔ $\frac{1.300}{430}$ x1.300
I	⊔ $\frac{1.400}{470}$ x1.400
J	⊔ $\frac{1.500}{500}$ x1.500
K	⊔ $\frac{1.800}{840}$ x1.440
L	⊔ $\frac{2.100}{980}$ x1.680
M	⊔ $\frac{2.400}{1.120}$ x1.920
N	⊔ $\frac{3.200}{1.920}$ x1.920
O	⊔ $\frac{3.600}{2.160}$ x2.160



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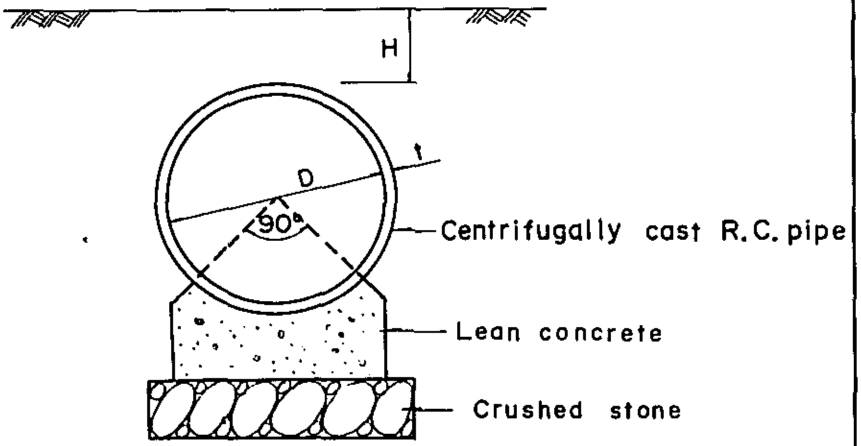
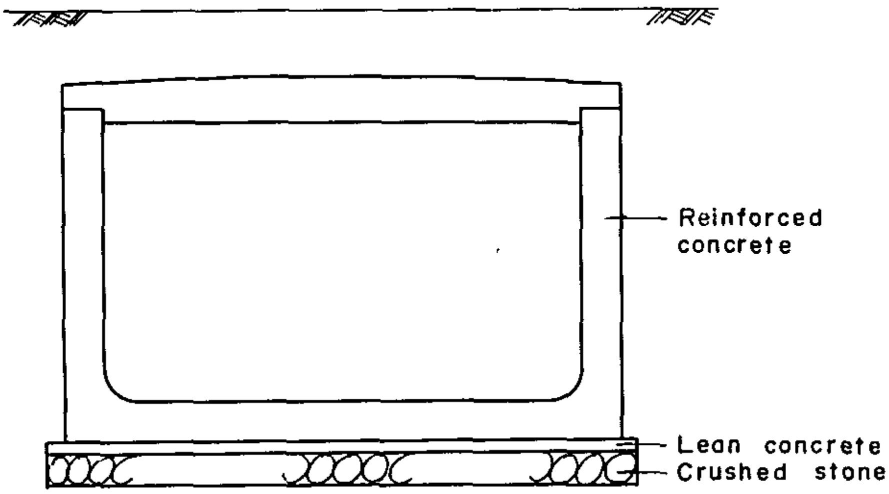
REPRESENTATIVE NETWORK OF SMALLER DRAINS IN INDUSTRIAL AREA

FIGURE 4·11

Pipe Culvert Detail
H > 1.0m



Box Culvert Detail



SEWERAGE AND DRAINAGE SYSTEM PROJECT
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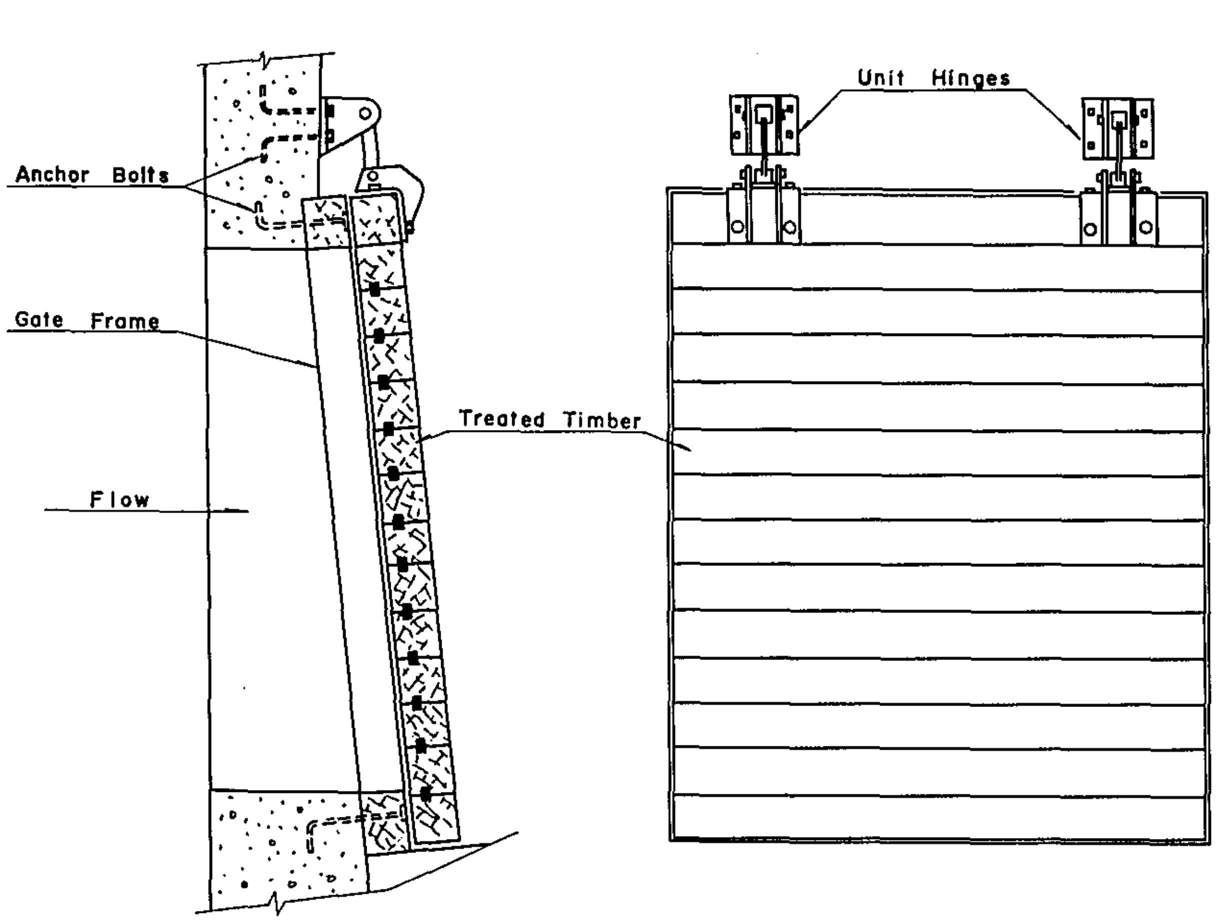
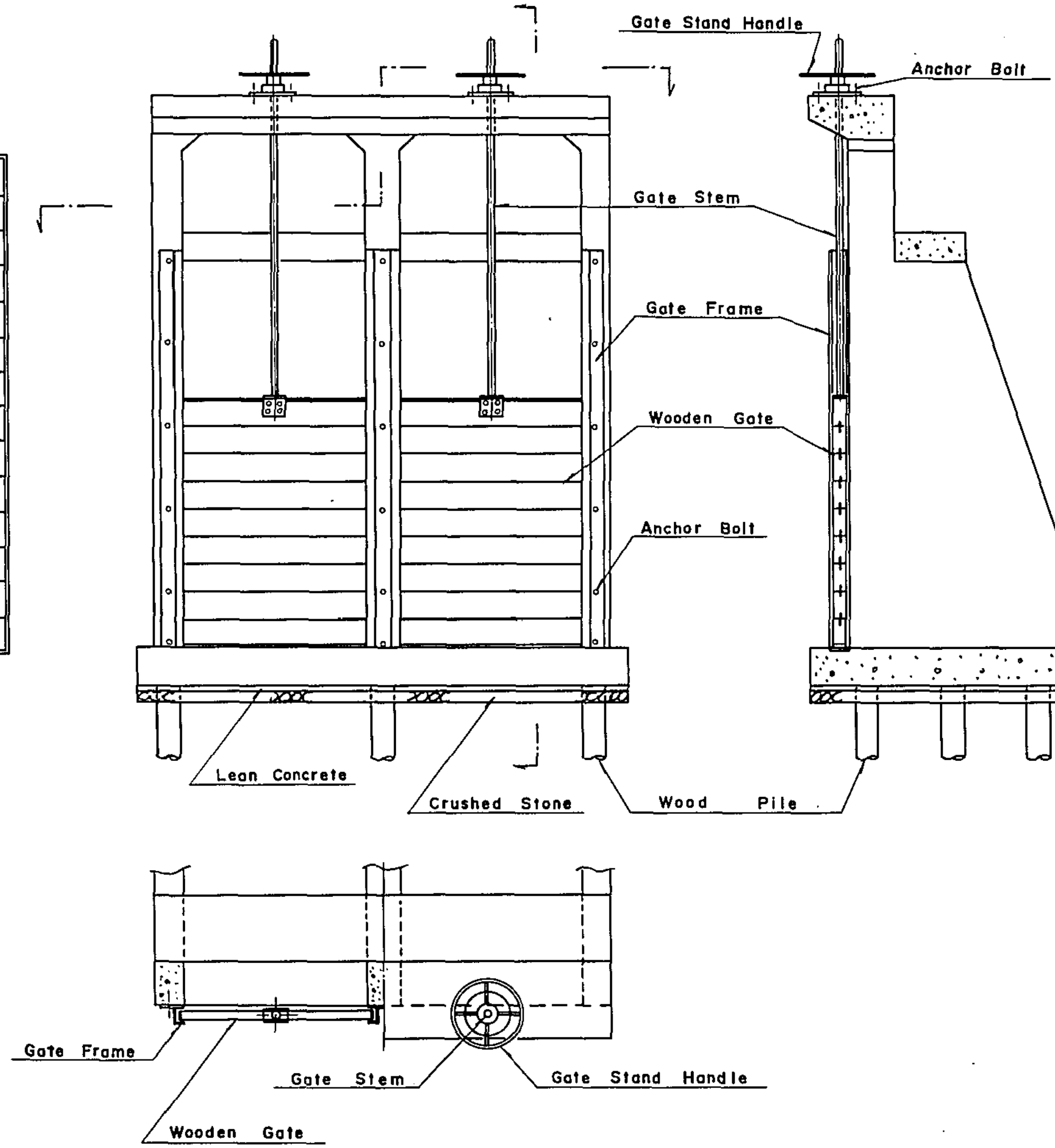
TYPICAL CULVERTS OF INFRASTRUCTURAL DRAINS	FIGURE 4·12

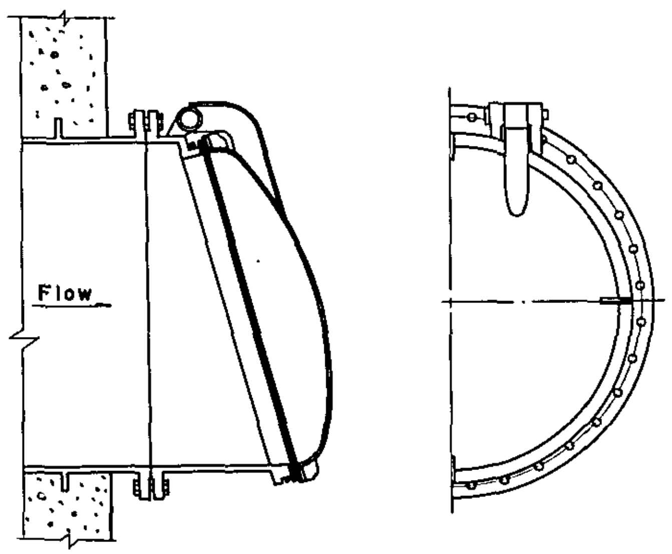
Fabricated Wooden Flap Gate



Sluice Gate



Cast Iron Circular Flap Gate



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TIDE GATES

FIGURE 4·13

4.3.2 The Juru River

Basic hydrological and hydraulic data of the Juru River were not available at the time when the study was conducted.

Urbanization has been in progress in the area upstream of the river, especially in the fringe of Bukit Mertajam where the population density is high. Improvement of the river to discharge the expected increase of the stormwater runoff due to urbanization will not be feasible unless the major part of the tributary area is well developed. As a temporary measure, it will be economical to utilize the available open spaces for stormwater storage and cut the peak flow rate. For this purpose, the existing swamps should be used as stormwater reservoirs.

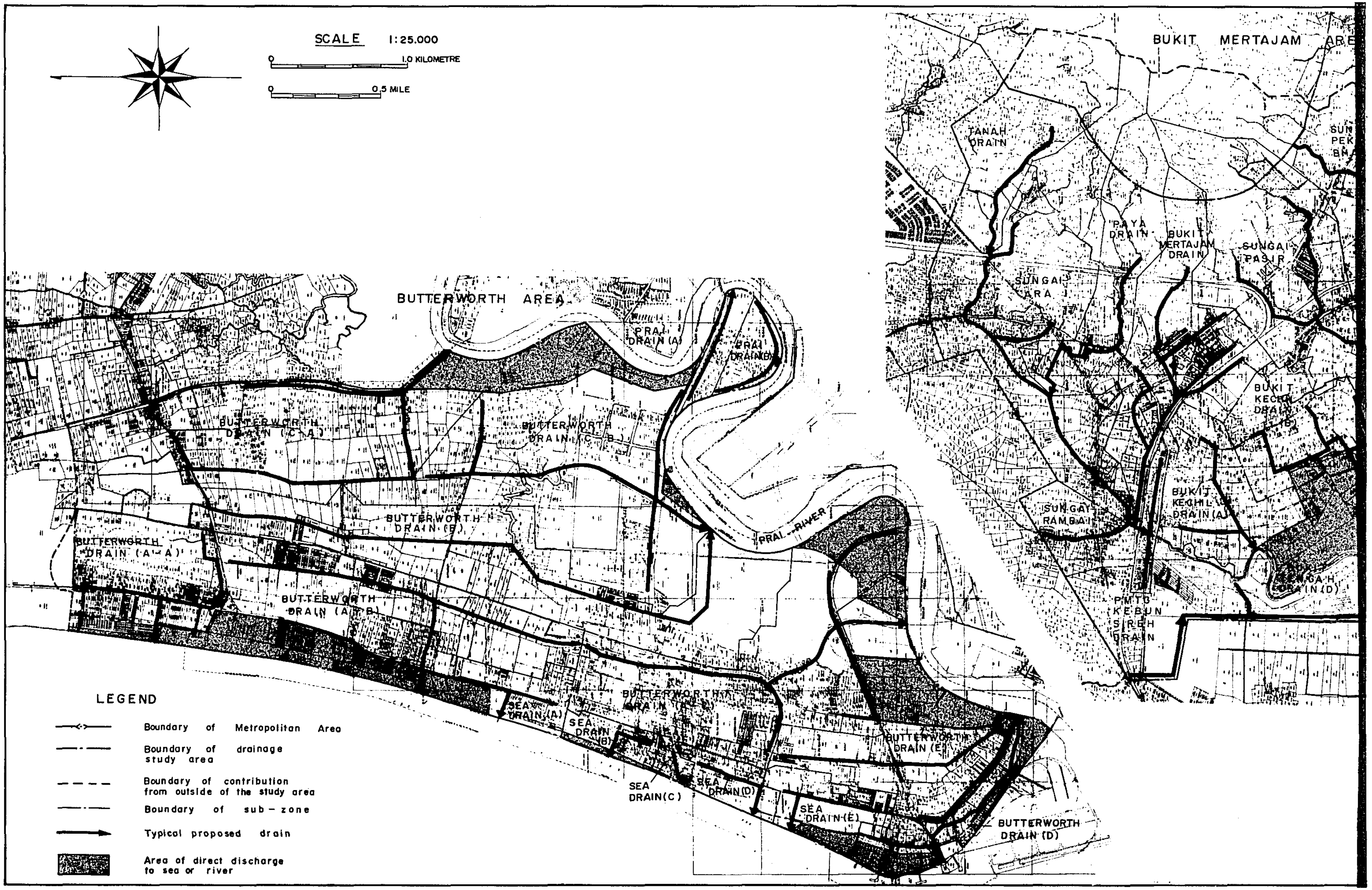
In the areas along the Juru River there is low lying agricultural land protected by the existing tide gates from the saline water back-up. These farm lands are frequently flooded mainly due to the low ground elevations. To solve this problem, embankment will be necessary along the river and pumping facilities to pump up the inner stormwater runoff. The existing drains such as the Sungai Pasir and Sungai Kelang used for irrigation should be improved when the tributary area is urbanized.

It is expected that the water level in these drains will be lowered after the improvement of the river, however, supply of water to the agricultural lands by gravity will become difficult. When the water elevation falls down, it will become necessary to provide some type of weirs at some points to raise the water elevation high enough to supply water to the farm lands by gravity. However, the weirs will interfere the smooth flow and reduce the drain capacity. At the present stage, the time schedule of the urbanization is not known. Hence, prior to the final design, these factors should be carefully checked and a proper measure should be taken.

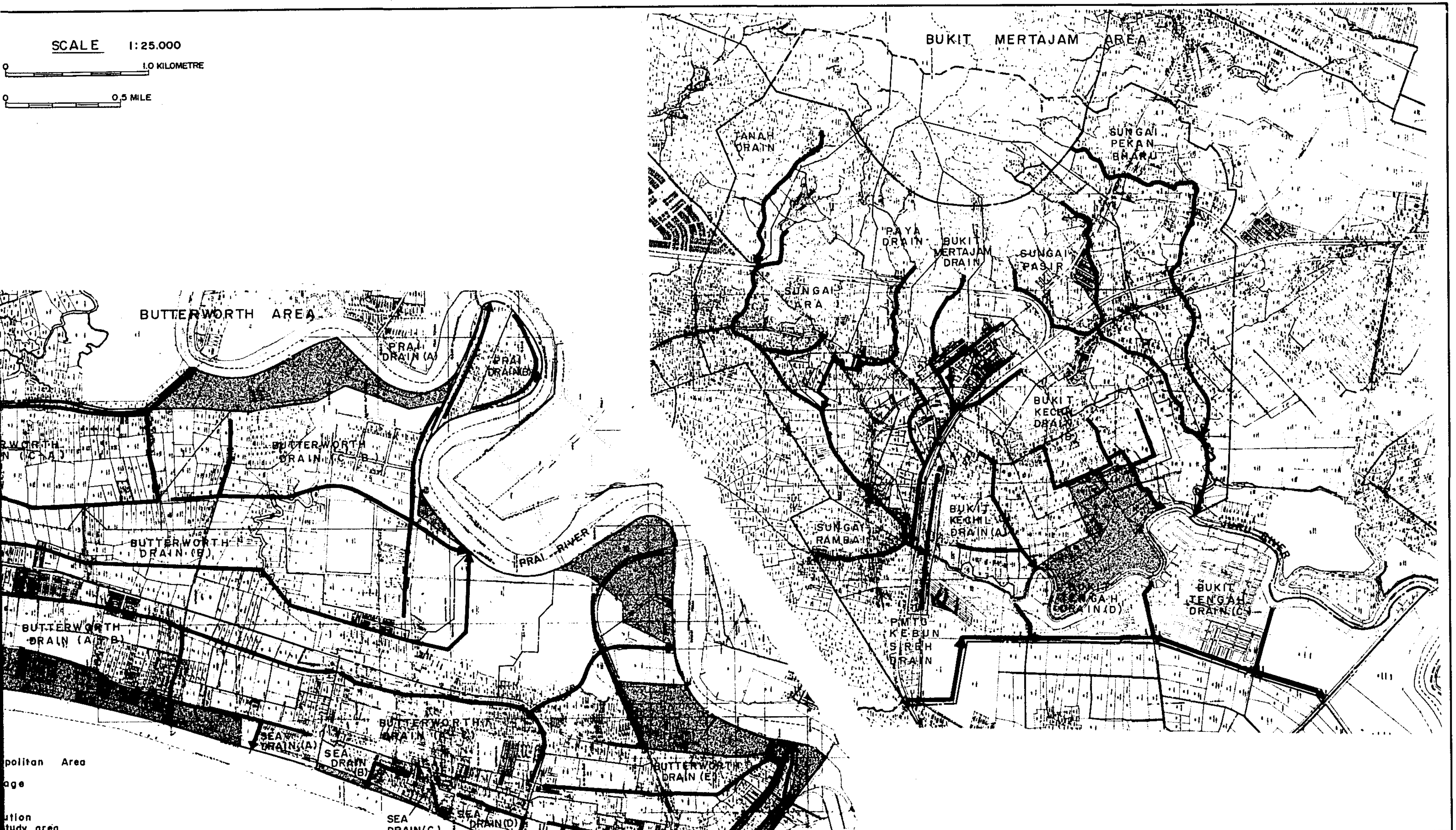
The Juru River water level within the Study Area is controlled by the existing Juru Tide gate. During the dry weather the level is kept at +0.0 m, but during a heavy rain, the stormwater diverted through the gate may exceed the capacity of the gate. A 5-year frequency storm under the present conditions (based on the estimated runoff coefficient of 0.15) will raise the water level up to approximately +1.3 m (4.3 ft) ∿ +1.6 m (5.2 ft). The water level will be about +1.90 m (+6.2 ft) ∿ +2.2 m (7.2 ft) when a 5-year frequency storm occurs, if the area is urbanized to the degree envisaged in the year 2000 and the runoff coefficient increases (with an estimated runoff coefficient of 0.4). The recommended land filling level (+2.3 m or 8.5 ft) is also higher than these water levels caused by the highest tide level ever recorded.

On the basis of the situation mentioned above, the water level in the upstream portion of the Juru Tide gate is set to +2.0 m (6.56 ft) for design purpose. This value is based on the water level of +1.9 m (+6.2 ft) ∿ +2.2 m (7.2 ft) (Re. Section 4.2).

When the Major Storm occurs, the water flow is interrupted by the gate and the water level is raised up to around +3.0 m (+9.9 ft) or higher. Therefore, it may be better enlarge or abolish the gate if the condition allows to do so. The water elevation can be lowered to around +2.5 m (+8.2 ft) even at the time of the Major Storm anticipated in the year 2000 with a runoff coefficient of 0.65 if the gate size is increased by 100 per cent (See Annex 2). The expected water elevation under the present condition (C=0.3) by the Major Storm is about +2.0 m (+6.5 ft). Thus the recommended ground elevation for land filling will be sufficient for the time being to protect the land from the flooding by the Major Storm.



SCALE 1:25.000
0 1.0 KILOMETRE
0 0.5 MILE
BUKIT MERTAJAM AREA
BUTTERWORTH AREA
TANAH DRAIN
PAYA DRAIN
BUKIT MERTAJAM DRAIN
SUNGAI PASIR
SUNGAI ARA
BUKIT KECHIL DRAIN (B)
BUKIT KECHIL DRAIN (A)
SUNGAI RAMBAI
PMTG KEBUN SIREH DRAIN
TENGAH DRAIN (D)
PRAI DRAIN (A)
PRAI DRAIN (B)
PRAI RIVER
BUTTERWORTH DRAIN (C-A)
BUTTERWORTH DRAIN (C-B)
BUTTERWORTH DRAIN (E)
BUTTERWORTH DRAIN (A-A)
BUTTERWORTH DRAIN (A-B)
BUTTERWORTH DRAIN (E)
BUTTERWORTH DRAIN (D)
SEA DRAIN (A)
SEA DRAIN (B)
SEA DRAIN (C)
SEA DRAIN (D)
SEA DRAIN (E)
LEGEND
Boundary of Metropolitan Area
Boundary of drainage study area
Boundary of contribution from outside of the study area
Boundary of sub-zone
Typical proposed drain
Area of direct discharge to sea or river



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

PROPOSED DRAINAGE SYSTEM

FIGURE 4·14

CHAPTER 5

COST ESTIMATES, CONSTRUCTION PROGRAMME AND FINANCING

5.1 Construction Cost

The information and data on materials and labour costs in the Study Area in 1977 have been collected from various sources and are used for reasonable cost estimates of the drainage facilities. The sources of the data and information are:

- "Schedule of Rates," PWD (Central),
- Tender documents for Georgetown sewerage programme,
- Tender documents of DID (Central),
- MPSP,
- PDC,
- Tender documents of Selangor Development Corporation, and
- Information from various local contractors and manufacturers.

Cement, reinforcing steel and aggregate are all available in the area with stable prices. Granite is also produced in Province Wellesley and widely used for rubble local drains. Mechanical equipment are generally imported from various countries though most of the equipment are locally available. Unit costs for the construction works are summarized in Table 5.1.

5.1.1 Unit Cost

(a) Trunk Drain

Using the basic costs estimated previously, cost function curves for various types of drain have been developed. The costs reflected by the curves include excavation, sheeting, dewatering, backfilling, disposal of surplus soil, material and labour for structures, restoration of paving and contractors' profit and overhead. The cost function curves are shown in Figure 5.1

(b) Bridge and Culvert

Unit construction costs of bridge and culvert are estimated on the basis of the proposed structures as shown in Figures 3.1 and 4.8. The unit construction costs for culvert are estimated using the cost function curve in Figure 5.1, while the costs for bridge are estimated on case-by-case basis.

Bridge construction comprises both superstructure and substructure. The superstructure is made of precast reinforced concrete beams and floor slabs with pavement. The substructure consists of retaining abutment with reinforced concrete and piling. The unit construction costs for the elements of the bridge are estimated for the typical bridge structure as shown in Figure 4.8. The unit costs of the element are as follows:

Superstructure:	M$550/$m^2$ of surface area
Substructure:	
abutment	M$3,800/m of width
counterfort (at every 3 m)	M$280/unit
miscellaneous work for piling	MS600 (lump sum)

(c) Infrastructural Drain

Construction costs for reticular drains are estimated on hectare basis. Factors influencing the costs are ground surface conditions and road density of the area concerned. In order to reflect the actual conditions of the Study Area, two typical areas representative for residential and industrial zones are selected and estimated in their construction costs for reticular drains. It is assumed that the character of commercial zone is similar in nature to that of residential zone except for the different return period of rain storm applied for design.

In the sample design, a 5-year return period of the storm is applied for commercial and industrial zones, whereas a 2-year return period is used for residential zone (refer to Chapter 3).

Reticular drains are designed either by V-shaped or rectangular concrete channel, with or without reinforcement. At road intersections, either centrifugally cast reinforced concrete pipe or box culvert is considered.

The unit construction costs for both residential and industrial zones are estimated to be approximately M$13,000/ha. Representative drainage layouts for the selected residential and industrial zones are presented in Figures 4.10 and 4.11 respectively.

5.1.2 Cost Comparison Between the Different Rainfall Frequencies

The proposed drainage system is designed on the basis of the recommended criteria as described in Chapter 3. The trunk and infrastructural drains both in industrial and commercial areas are designed for 5-year return period of storm, whereas in residential areas for 2-year return period is used.

Although these return periods are considered reasonable for drainage design in the Study Area and also supported by the records of other similar cities in Malaysia, further economic justification has been made to confirm that the design is appropriately acceptable.

For cost comparison, construction costs both for the proposed system and the system using 2-year return period of storm are estimated. The relationship between the cross sectional areas of 2-year and 5-year drains is obtained in the form:

$$A_2 = A_5 \times 0.8$$

where

A_2 = required drain cross sectional area designed for 2-year frequency

A_5 = required drain cross sectional area designed for 5-year frequency

From the above relationship, the construction costs for the systems designed for both the proposed and 2-year return periods are estimated as summarized in Table 5.5. Procedures for the estimations are described in Annex 3.

The result of the cost estimation indicates that the cost for the proposed system is about M$91 million while the cost for 2-year return period system is M$87 million. The difference between these two systems is slight and is considered that the adoption of the proposed return period is acceptable for the system since the benefits expected from the system is significant in solving the current flooding problems in the area. The benefits are discussed in Chapter 6.

5.1.3 Construction Cost of Proposed Drainage System

Overall construction cost for the recommended drainage facilities has been estimated on the basis of the procedures described in the previous sections and in Annexes. Construction costs for each of the drainage facilities have been derived from the unit construction costs.

The costs comprise materials and labour, contingency allowances, engineering cost and land acquisition. In the engineering cost, 50 per cent is assumed for engineering design and the remaining for supervision services for construction. As the contingencies, 20 per cent is considered both for materials and labour costs, and 10 per cent of the total cost for engineering. The estimated construction costs are summarized in Table 5.2 and breakdown of the costs by individual drain is given in Tables 5.3 and 5.4.

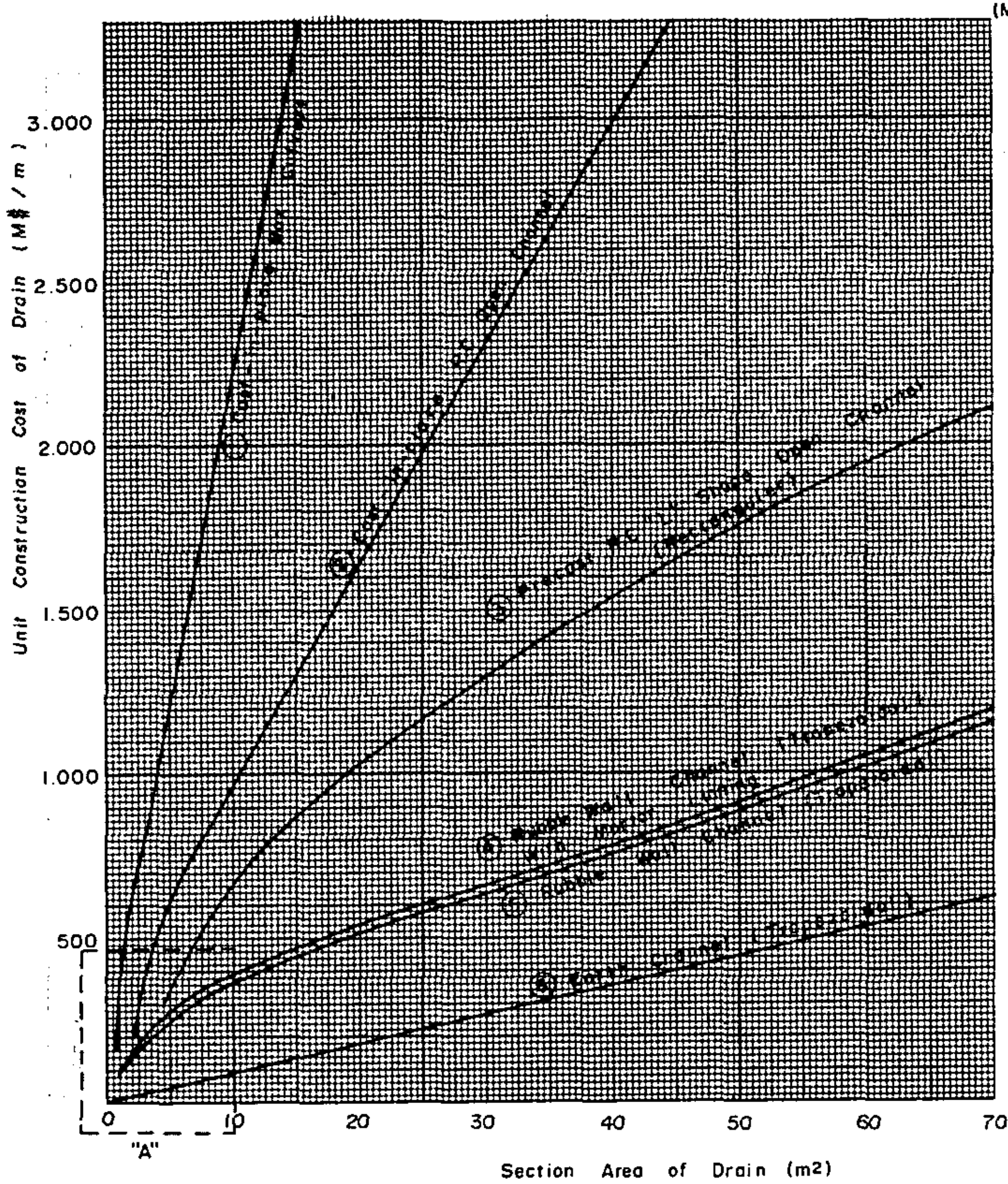
As shown in the tables, the total cost is approximately M$90 million including M$8 million for land acquisition. About 26 per cent of the total cost is for trunk drains with an average per hectare cost of M$26,000.

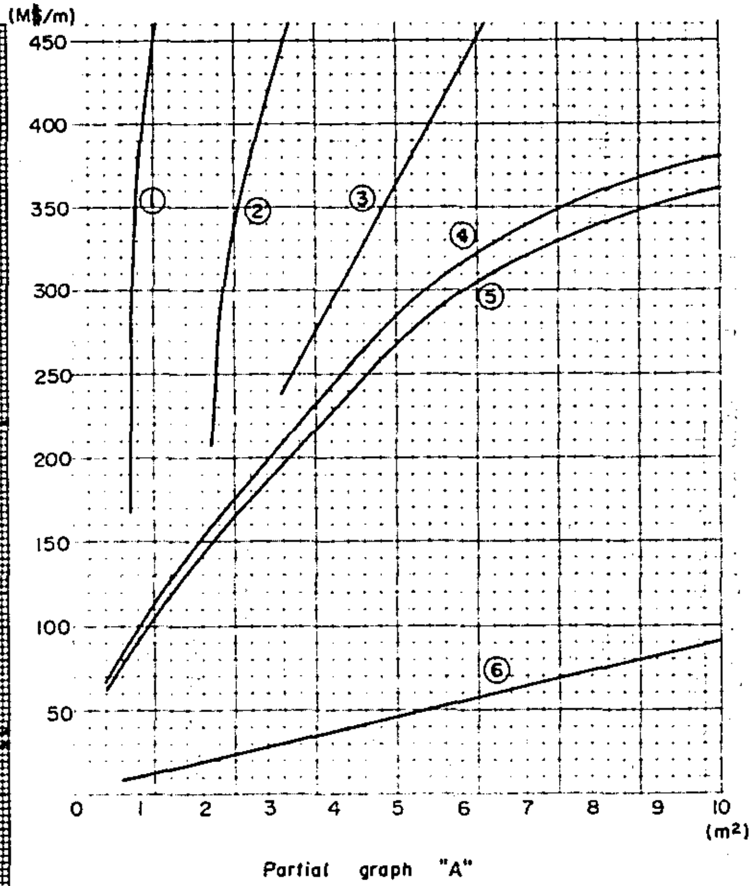
The construction cost by private contribution is estimated on the assumption that the infrastractural drains in undeveloped areas would be provided by private developers, which accounts for about 52 per cent of the total cost. The government will contribute the costs for all trunk drains with catchment area exceeding 40 ha (100 acres) and infrastructural drains in builtup areas.

The drainage area covered by the proposed system is 3,480 ha (8,596 acres), of which about 2,000 ha (4,940 acres) will be covered by the private drainage and the remaining 1,480 ha (3,656 acres) by the government drainage. The area and location of the proposed drainage system are shown in Figures DD-19 and DD-20 of Volume V.

Table 5.1 Schedule of Unit Construction Costs
(including material and labour)

Item	Description	Unit	Cost (M$)
Concrete	1 : 2 : 4 1 : 3 : 6	m^3 "	99.4 81.0
Reinforced concrete	1 : 2 : 4	m^3	250.0
Mortar works	1 : 2 1 : 3	m^2 "	122.5 92.8
Excavation	Bulk excavation (by hand) (by machinery)	m^3 "	3.0 1.8
	Trench excavation		
	0 - 1.5m deep	m^3	3.6
	1.5 - 3.0	"	5.4
	3.0 - 4.5	"	9.2
	4.5 - 6.0	"	13.8
	6.0 - 6.5	"	17.2
	7.5 or more	"	20.5
Sheeting	Excavation depth	m	6.3
	1.0 < H < 2.0m	"	9.0
	2.0 < H < 3.0	"	9.0
	3.0 < H < 4.0	"	18.0
	4.0 < H < 5.0	"	40.5
	6.0	"	112.0
	7.0	"	123.0
	8.0 or more	"	133.0
Surplus soil	Distance L		
	90m < L < 400m	m^3	1.4
	400m < L < 800m	"	2.0
	800 < L <1610m	"	3.2
Back filling and compaction		m^3	2.6
Forming		m^2	7.6
Restoration of paving		m^2	15.6
Dewatering		hr	3.0
Rubble wall		m^3	65





SEWERAGE AND DRAINAGE SYSTEM PROJECT BUTTERWORTH / BUKIT MERTAJAM METROPOLITAN AREA, MALAYSIA	
CONSTRUCTION COST CURVES OF DRAIN	FIGURE 5·1

Table 5·2 Construction Costs of Proposed Drainage System (at 1977 Price Level)

(M$ 1,000)

Items		Government Contribution				Private Contribution			Grand Total
Drain		Trunk Drain	Infrastructural Drain		Total	Infrastructural Drain		Total	
			Principal Drain	Net Work of Smaller Drain		Principal Drain	Net Work of Smaller Drain		
Construction of Structure	Length or Area	30.4 km	37.0 km	104 ha	—	39.9 km	2,017 ha	—	—
	Construction Cost	15,959	11,226	535	27,720	8,094	26,668	34,762	62,483
	Contingency	3,192	2,245	107	5,544	1,619	5,333	6,952	12,496
	Engineering Fee	1,916	1,348	64	3,328	971	3.200	4,171	7,499
	Sub Total	21,067	14,819	706	36,592	10.684	35.201	45,885	82,477
Land Acquisition	Area (ha)	17.2	5.5	—	—	5.2	—	—	—
	Cost	3,053	3,535	—	6,588	1,708	—	1,708	8,296
Total		24,120	18,354	706	43,180	12,392	35,201	47,593	90,773

NOTE : Engineering cost is the sum of engineering design (50%) and engineering supervision (50%)

SEWERAGE AND DRAINAGE SYSTEM PROJECT
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CONSTRUCTION COSTS OF PROPOSED DRAINAGE SYSTEM — TABLE 5·2

Table 5·3 Construction Costs of Proposed Trunk Drain at 1977 Price Level

(M$1,000)

Drainage Basin	Drain Name	Construction of Structure: Length (km)	Construction of Structure: Construction Cost	Construction of Structure: Contingency	Construction of Structure: Engineering Fee	Construction of Structure: Sub Total	Land Acquisition: Area (ha)	Land Acquisition: Cost	Total
Bukit - Mertajam (A part of Basin II)	Tahan Drain	1.2	355	71	43	469	0.5	56	525
	Sungai Ara	3.3	1,753	351	210	2,314	2.5	386	2,700
	Paya Drain	1.7	480	96	58	634	2.6	222	856
	Bukit Mertajam Drain	1.1	222	44	27	293	0.2	29	322
	Sungai Rambai	1.6	700	140	84	924	1.0	198	1,122
	Bukit Kechil Drain (A)	0.2	94	19	11	124	0.1	4	128
	Bukit Kechil Drain (B)	0.7	296	59	36	391	0.3	11	402
	Sungai Pasir	3.2	1,990	398	239	2,627	1.6	223	2,850
	Sungai Pekan Bharu	3.1	1,527	305	183	2,015	1.5	264	2,279
	Total	16.1	7,417	1,483	891	9,791	10.3	1,393	11,184
Butter-Worth (Basin IV)	Butterworth A-A Drain	1.4	1,203	241	144	1,588	0.4	139	1,727
	Butterworth A-B Drain	1.3	888	178	107	1,173	0.5	215	1,388
	Butterworth A-C Drain	1.6	1,091	218	131	1,440	0.1	6	1,446
	Butterworth B Drain	0.7	2,578	516	309	3,403	0.4	682	4,085
	Butterworth C-A Drain	4.4	1,784	357	214	2,355	2.3	262	2,617
	Butterworth C-B Drain	3.1	671	134	81	886	2.4	287	1,173
	Butterworth E Drain	1.8	327	65	39	431	0.8	69	501
	Total	14.3	8,542	1,709	1,025	11,276	6.9	1,660	12,936
Grand Total		30.4	15,959	3,192	1,916	21,067	17.2	3,053	24,120

NOTE Engineering fee is the sum of engineering design (50%) and engineering supervision (50%)

Table 5·4 Construction Costs of Infrastructural Drain at 1977 Price Level

(M$1,000)

Drainage Basin	Items	Drain	Construction of Structure: Length or Area	Construction of Structure: Construction Cost	Construction of Structure: Contingency	Construction of Structure: Engineering Fee	Construction of Structure: Sub Total	Land Acquisition: Area (ha)	Land Acquisition: Cost	Total
Bukit - Mertajam (A part of Basin II)	Government Contribution	Principal Drain	11.2 km	1,722	344	207	2,273	1.2	971	3,244
		Net Work of Smaller Drain	13.8 ha	92	18	11	121	—	—	121
		Sub Total		1,814	362	218	2,394	1.2	971	3,365
	Private Contribution	Principal Drain	21.2 km	4,500	900	540	5,940	2.3	961	6,901
		Net Work of Smaller Drain	930 ha	11,631	2,326	1,396	15,353	—	—	15,353
		Sub Total		16,131	3,226	1,936	21,293	2.3	961	22,254
	Total			17,945	3,588	2,154	23,687	3.5	1,932	25,619
Butterworth (Basin IV)	Government Contribution	Principal Drain	25.8 km	9,504	1,901	1,141	12,546	4.3	2,564	15,110
		Net Work of Smaller Drain	90.6 ha	443	89	53	585	—	—	585
		Sub Total		9,947	1,990	1,194	13,131	4.3	2,564	15,695
	Private Contribution	Principal Drain	18.7 km	3,594	719	431	4,744	2.9	747	5,491
		Net Work of Smaller Drain	1067 ha	15,037	3,007	1,804	19,848	—	—	19,848
		Sub Total		18,631	3,726	2,235	24,592	2.9	747	25,339
	Total			28,578	5,716	3,429	37,723	7.2	3,311	41,034
Grand Total				46,523	9,304	5,583	61,410	10.7	5,243	66,653

NOTE : Engineering fee is the sum of engineering design (50%) and engineering supervision (50%)

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Construction Costs of Proposed Trunk Drain
Construction Costs of Infrastructural Drain

TABLE 5·3 5·4

Table 5·5 Comparison of Construction Costs (2-yr System vs. Proposed System) (at 1977 Price Level) (M$ 1.000)

Items	Drain		Trunk Drain		Infrastructural Drain: Principal Drain		Infrastructural Drain: Net Work of Smaller Drain		Total	
			2-yr System	Proposed System	2-yr System	Proposed System	2-yr System	Proposed System	2-yr System	Proposed System
Government Contribution	Construction of Structure	Construction Cost	14.363	15,959	10.322	11,226	535	535	25.220	27,720
		Contingency	2.873	3,192	2.064	2,245	107	107	5.044	5,544
		Engineering Cost	1.724	1,916	1.239	1,348	64	64	3.027	3,328
		Sub Total	18.960	21,067	13.625	14,819	706	706	33.291	36,592
	Land Acquisition		2.748	3,053	3.244	3,535	—	—	5.992	6,588
	Total		21.708	24,120	16.869	18,354	706	706	39.283	43,180
Private Contribution	Construction of Structure	Construction Cost	—	—	8.024	8,094	26,668	26,668	34.692	34,762
		Contingency	—	—	1.605	1,619	5,333	5,333	6.938	6,952
		Engineering Cost	—	—	963	971	3,200	3,200	4.163	4,171
		Sub Total	—	—	10.592	10,684	35.201	35,201	45.793	45,885
	Land Acquisition		—	—	1.684	1,708	—	—	1.684	1,708
	Total		—	—	12.276	12,392	35,201	35,201	47.477	47,593
Grand Total			21.708	24,120	29.145	30,746	35,907	35,907	86.760	90,773

NOTE : a) Proposed drainage system comprises trunk drain designd 5-yr frequency and Infrastructural drain by 2-yr and others
In case of construction costs for net work of smaller drain, a cost difference between 2-yr system and proposed system (residential: 2-yr, industrial and commercial : 5-yr) is negligible.

b) Engineering cost is the sum of engineering design (50 %) and engineering supervision (50%).

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COMPARISON OF CONSTRUCTION COSTS (2-yr System vs Proposed System)

TABLE 5·5

5.2 Maintenance Cost

The maintenance of the existing drainage system in the Study Area has been performed by DID, PWD and MPSP. MPSP is responsible for the maintenance of the system within the town limit and the remaining system is under DID's responsibility. In some cases, PWD is concerned with roadside ditches. The maintenance costs have been shouldered by the respective agencies. These agencies have been suffering a shortage of engineers but the number of workers is likely to be sufficient to carry out the maintenance of the system.

The operation of tide gates and cleaning of drains have been made daily by MPSP and DID, including repairing of tide gates and desilting of drains. Although the maintenance work is relatively well performed, there seems to be a few points to be improved for better drainage services, i.e., lack of inspection to the system and equipment for desilting and cleaning of drains.

In view of the current situation, it is recommended that a systematic programme for routine inspection of the system and collection of information be set up immediately. The information and data should be recorded, filed and used for establishing the proper maintenance schedule. For these purposes, personnel and equipment are required including; (1) one engineer for inspection and data collection, and preparation of maintenance schedule, (2) one car for site inspection and data collection with a driver and two crew, (3) a fitter for checking and lubrication of tide gates, and (4) a worker for routine jobs.

For dredging or cleaning of the major drainage channels and culverts, a clamshell grabbing crane and a hand rodding machine will be needed. Although most major dredging work for trunk drains will be committed to contractors, a small team should be provided assigning minor or emergency repairing work. The minimum equipment for the work may include hand tools, a concrete mixer with a small capacity and a dump truck. Required work force for the team should consist of a carpenter, a masonry and several labourers.

The maintenance costs to be added to the present recurrent expenditures have been estimated to be M$600 - M$750 per year, approximately 0.4 - 0.5 per cent of capital required for new equipment listed as follows. The above ratio is normally applied to estimate the maintenance cost of sewerage and drainage systems and can be compared with 0.35 per cent applied in estimating sewerage maintenance cost in Kuala Lumpur.

Item		Cost
Clamshell grabbing crane	1 No.	M$100,000
Hand rodding machine	1 No.	M$ 12,000
Concrete mixer	1 No.	M$ 11,000
4-ton dump truck	1 No.	M$ 25,000
Total		M$148,000

As the area is urbanized the requirements for maintenance work will be increased accordingly. The estimated costs are only for the initiation of the systematic maintenance work for the proposed first stage programme. Therefore, it may be necessary to strengthen the manpower and equipment when the system is further extended in the future.

5.3 Construction Programme

The proposed drainage system should be implemented according to the urban development programme and the requirements of drainage. Because the time scheduling for various future urban development programmes are not practicable, it may be unrealistic to plan the construction programme of the drainage system for the entire Study Area at this stage. Consequently, the construction programme is discussed only for the first five years under this section.

A main purpose of the provision of the drainage system is to alleviate the present flooding and to prepare a schedule to meet the requirements for the rapid urbanization in the area. The order of drainage implementation priority is therefore decided taking the present flooding conditions and urbanization of the area into account. The following is the principle applied in planning the system:

- Prepare immediate measures to alleviate the existing flooding conditions, including by rehabilitation or enlargement of the existing drains, and excavation of earthen drains in kampung area.

- Design drains for the areas where the flood is expected to occur due to urbanization.

On the basis of the above principle, a drainage implementation programme for the first stage has been established as described in the following paragraphs:

5.3.1 Elements of Work Included in the First Stage Programme

(a) Alleviation of Existing Flooding

The rehabilitation or improvement of the existing drains is most effective means to alleviate the current flooding. This includes:

- Desilting of BWA C-1 and C-3, reconstruction of box culvert in BWA C-3, conversion of the existing culvert in BWA C-8 to a bridge, widening and deepening from BWD-2 to BWD-4, BWE-7b to BWE-7d and construction of BWE-7, as shown in Table 5.6.

- Construction of RAM-3 and RAM-5, widening and deepening from ARA-9 to ARA-11, and conversion of the existing culvert in ARA-11 to a bridge, as shown in Table 5.6.

(b) Necessary Steps to Cope with Urban Development

As shown in Figure DD-4 of Volume V, a housing development is now underway in the tributaries of BWA.B, BWA.C, SEA.A and BWE drain systems. In these areas, the stormwater runoff is expected to increase as the development proceeds, thus an improvement of the existing drainage system is required.

To meet the requirements, the BWA.B, SCA.A and BWE drain systems need to be improved at an early stage, however, the BWC.A system will be required later. The elements of work thus selected are summarized in Table 5.7.

5.3.2 Implementation Schedule

The elements of work described previously have been arranged in accordance with the order of priority, and an implementation schedule for the first stage programme is then developed. Since the construction is expected to be started in 1981, the detailed engineering design of the facilities has to be prepared by 1980 prior to the commencement of the first stage programme. The implementation programme has been so planned as to spread capital investment evenly in each year throughout the first stage programme period.

In determining the schedule, three sets of alternative construction programme are studied, and advantages and disadvantages are identified. The three alternative cases studied are:

Alternative 1: Minimum improvement requirements, including alleviation of the existing flood problems by the provision of drainage facilities in the urban development scheme areas in BWA.B, at a cost of about M$4 million as shown in Table 5.8, and Figure 5.2.

Alternative 2: Alternative 1 plus upgrading of the BWE and SEA.A drain systems at a total cost of M$6.4 million as shown in detail in Table 5.8, and Figure 5.3.

Alternative 3: Alternative 2 plus upgrading of the BWC drain system at a cost of M$9.3 million as shown in Table 5.8, and Figure 5.4.

Alternatives 2 and 3 stand on an assumption that the provision of drainage system is required only when the housing schemes in the concerned area are implemented. At the present stage, the time schedule of the development for the area has not been fully decided yet; however, if the development is undertaken, the flood damage will be more significant in the tributary of BWE than BWC because of higher population density.

Alternative 1 is considered not sufficient to meet the requirements by the urban development in the foreseeable future. Also, Alternative 3 covers the BWC drain system which is less imminent than other drain systems. In view of the above conditions, Alternative 2 is superior to the others, and the cost is reasonable to implement as the first stage programme. It is recommended therefore that Alternative 2 be implemented as the first stage programme starting from 1981 and ending in 1985. Details of each of the alternatives are presented in Table 5.8.

Table 5.6 Elemental Works for Alleviating Flooding Conditions

Area	Description	Length (m)	Existing size	Improved size	Remarks
Butter-worth	Disilting of		(upper bottom x depth x width)	(upper bottom x depth x width)	
	BWA.C-1	245	4.9m x 3.7m x 0.9m	4.9m x 3.7m x 1.5m	Earth drain
	C-3	270	4.9 x 3.7 x 0.9	4.9 x 3.7 x 1.5	"
	Reconstruction of box culvert in BWA.C-3	-	1.22 x 1.22	3.8 x 1.7	Reinforced concrete box culvert
	Conversion of pipe culvert in BWA.C-8 to bridge	-	ø1.52m x 2 barrel	9.5 x 2.6	Bridge
	Widening and deepening of:				
	BWD-2	303	1.5 x 1.0 x 1.1	3.6 x 0.7 x 1.4	Earth drain
	BWD-3	325	1.5 x 1.0 x 1.1	3.6 x 0.7 x 1.4	"
	BWD-4	420	3.4 x 1.5 x 0.6	3.6 x 0.7 x 1.4	"
	BWE-8b	303	3.0 x 1.2 x 0.6	2.7 x 1.3 x 2.2	"
	BWE-8c	277	3.4 x 0.7 x 1.4	3.0 x 1.4 x 2.4	"
	BWE-8d	405	3.4 x 0.7 x 1.4	3.0 x 1.4 x 2.4	"
	Construction of				
	BWE-7	346	-	3.0 x 0.6 x 1.2	"
Bukit Mertajam	Construction of RAM-3		10.0 x 8.5 x 1.8	15.0 x 13.1 x 2.8	Rubble wall channel

(continued)

Area	Description	Length (m)	Existing size	Improved size	Remarks
Bukit Mertajam	RAM-5		(upper bottom x depth x width) 10.0 x 8.5 x 1.8	(upper bottom x depth x width) 15.0 x 13.1 x 2.8	Rubble wall channel
	Widening and deepening of:				
	ARA-9	570	4.0 x 2.1 x 1.1	9.0 x 5.0 x 2.0	Earth drain
	ARA-10	162	4.0 x 2.1 x 1.1	9.0 x 5.0 x 2.0	"
	ARA-11	553	8.0 x 6.0 x 1.1	11.5 x 7.5 x 2.0	"
	Conversion of pipe culvert in ARA-11 to bridge		ø1.34m x 2 barrel	11.5 x 2.0	Bridge

Table 5.7 Elemental Works to Cope with Urban Development

Area	Description	Length (m)	Existing size	Proposed System	Remarks
Butter-worth	Construction of:		Refer to Figures DD-5 - 8, Volume V.		
	BWA.B system				
	BWE system				
	SEA.A system				
	BWC.A system				

Table 5.8 Alternative Implementation Schedule for First Stage Programme

(at 1977 Price Level)

(case 1)

1980		1981		1982		1983		1984		1985		Total
Description	Cost	Description	Cost	Description	Cost	Description	Cost	Description	Cost	Description	Cost	
Engineering Design	149	Construction of: bridge at, BWA.C-8 ARA-11 box culvert at, BWA.C-3 Rehabilitation of: BWA.C-1,3 BWD-2,3,4 BWE-6 BWE-8b,8c,8d	241 80 37 6 7 17 9	Construction of: drain, RAM-3,5 Rehabilitation of: drain ARA-9,10,11	227 322	Construction of: drain, BWA.B-5,6,7 Construction of bridge at, BWA.B-7	460 235	Construction of: drain, BWA,B-4 BWA.B-3	265 136	Construction of: drain, BWA,B-1,2 BWA,B-5c,5b	206 233	
Construction Cost	-		397		549		695		401		439	2,481
Contingencies	-		79		110		139		80		88	496
Engineering Cost	149		24		33		42		24		26	298
Sub Total	149		500		692		876		505		553	3,275
Land Cost	-		268		107		75		101		124	675
Grand Total	149		768		799		951		606		677	3,950

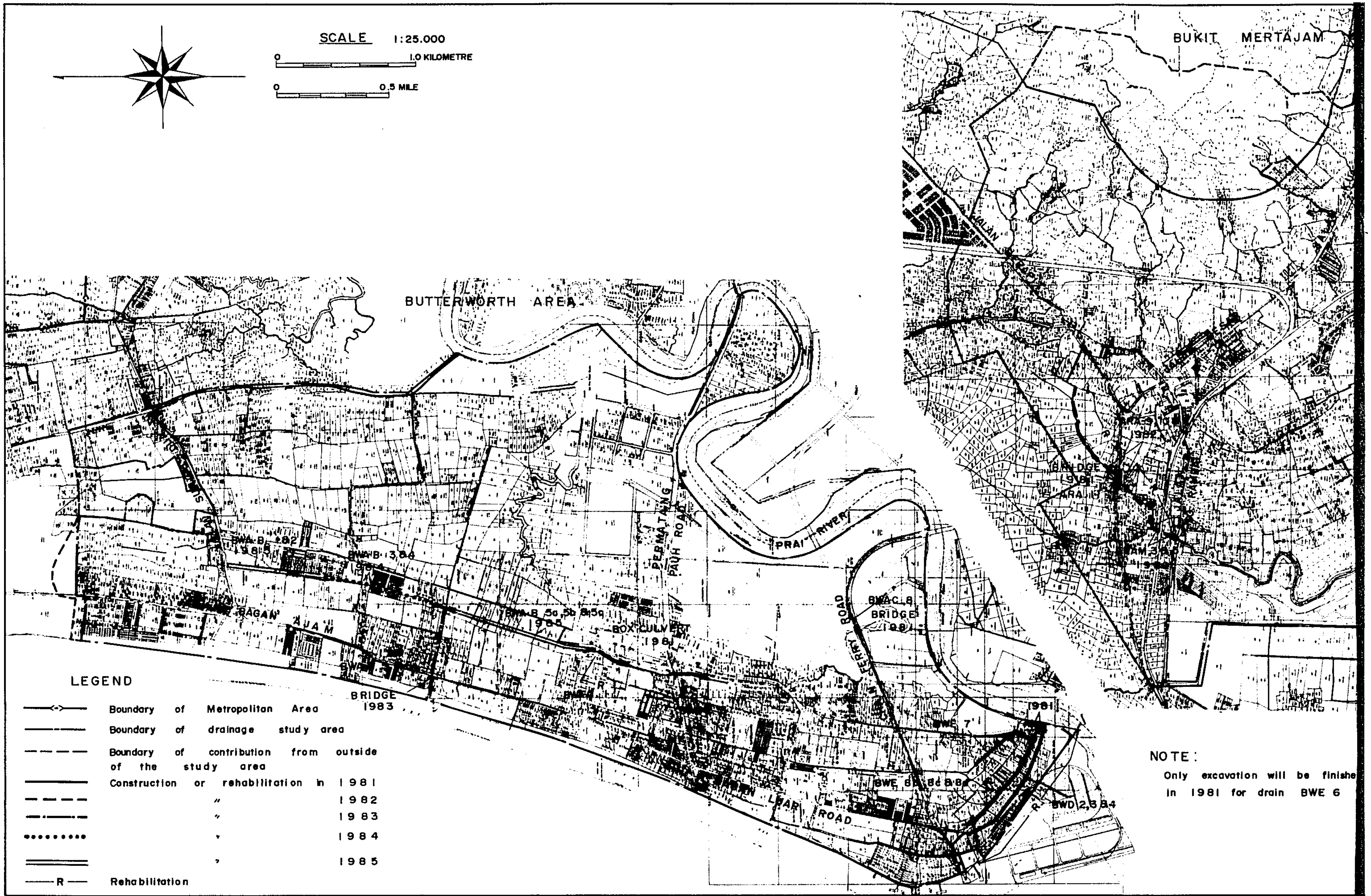
(continued) (case 2)

1980		1981		1982		1983		1984		1985		Total
Description	Cost	Description	Cost	Description	Cost	Description	Cost	Description	Cost	Description	Cost	
Engineering Design	231	Construction of: bridge at, BWA,C-8 ARA-11 RAM-3,5 (drain) box culvert at, BWA.C-3 Rehabilitation of: BWA.C-1,3 BWD-2,3,4 BWE-8b,8c,8d	 241 203 227 37 6 7 9	Construction of: drain, BWA,B-5,6,7 bridge at, BWA.B-7 Rehabilitation of: drain, ARA-9,10,11	 460 235 322	Construction of: drain BWA.B-1,2,3,4 BWA.B-5c,5b	 608 233	Construction of: drain, BWE-5,6,7,8 BWA.A-1,2,3,3b	 532 166	Construction of: drain, BWE-1,2,3,4	570	
Construction Cost	-		730		1,017		841		698		570	3,856
Contingencies	-		146		204		169		140		114	773
Engineering Cost	231		44		61		51		42		34	463
Sub Total	231		920		1,282		1,061		880		718	5,092
Land Cost	-		214		75		224		265		385	1,163
Grand Total	231		1,134		1,357		1,285		1,145		1,103	6,255

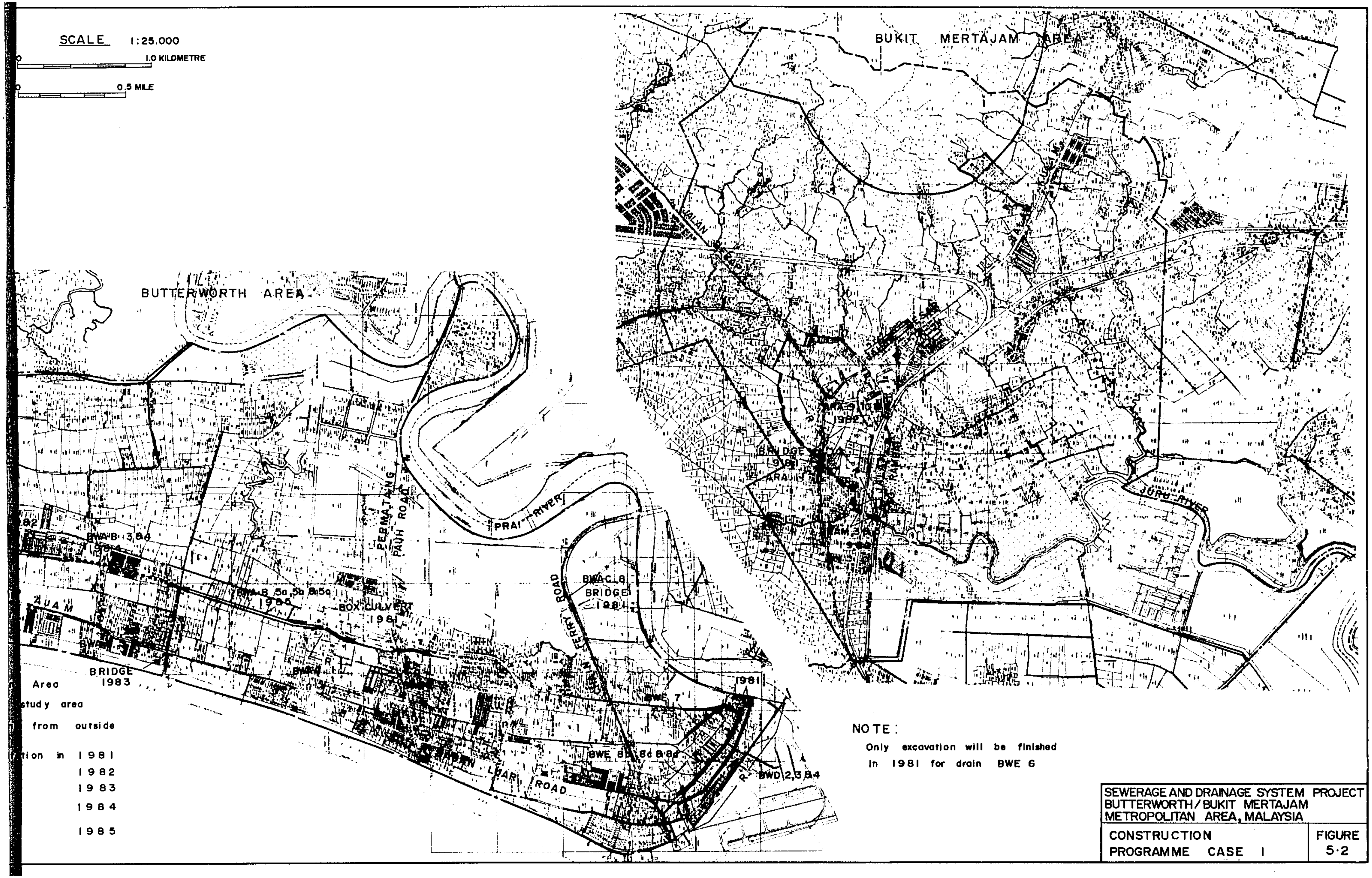
(continued)

(case 3)

1980		1981		1982		1983		1984		1985		Total
Description	Cost	Description	Cost	Description	Cost	Description	Cost	Description	Cost	Description	Cost	
Engineering Design	337	Construction of: bridge at, BWA.C-8 ARA-11 RAM-3,5 (drain) box culvert at, BWA.C-3 Rehabilitation of: BWA,C-1,3 BWD-2,3,4 BWE-8b,8c,8d ARA-9,10,11	 241 203 227 37 6 7 9 322	Construction of: drain, BWA.B-4,5,6,7 BWA.B-5b,5c bridge at, BWA.B-7	 726 233 235	Construction of: drain, BWA.B-1,2,3 BWE-5,6,7,8 SEA.A-1,2,3 3b	 342 531 157	Construction of: drain, BWA.C-6f BWE-1,2,3,4	 455 570	Construction of: drain, BWC.A-8,9 BWC.A-8e,8f,8g bridge at, BWC.8-a'	 291 731 289	
Construction Cost	-		1,052		1,194		1,030		1,025		1,311	5,612
Contingencies	-		210		239		206		205		262	1,122
Engineering Cost	337		63		72		62		62		79	675
Sub Total	337		1,325		1,505		1,298		1,292		1,652	7,409
Land Cost	-		214		219		355		457		172	1,417
Grand Total	337		1,539		1,724		1,653		1,749		1,824	8,826



SCALE 1:25.000
0 1.0 KILOMETRE
0 0.5 MILE
BUKIT MERTAJAM
BUTTERWORTH AREA
PRAI RIVER
PERMATANG PAUH ROAD
FERRY ROAD
BAGAN AJAM
BOX CULVERT 1981
BRIDGE 1983
BRIDGE 1981
BWE 7
BWD 2,3 &4
LEGEND
Boundary of Metropolitan Area
Boundary of drainage study area
Boundary of contribution from outside of the study area
Construction or rehabilitation in 1981
" 1982
" 1983
" 1984
" 1985
R Rehabilitation
NOTE:
Only excavation will be finished in 1981 for drain BWE 6



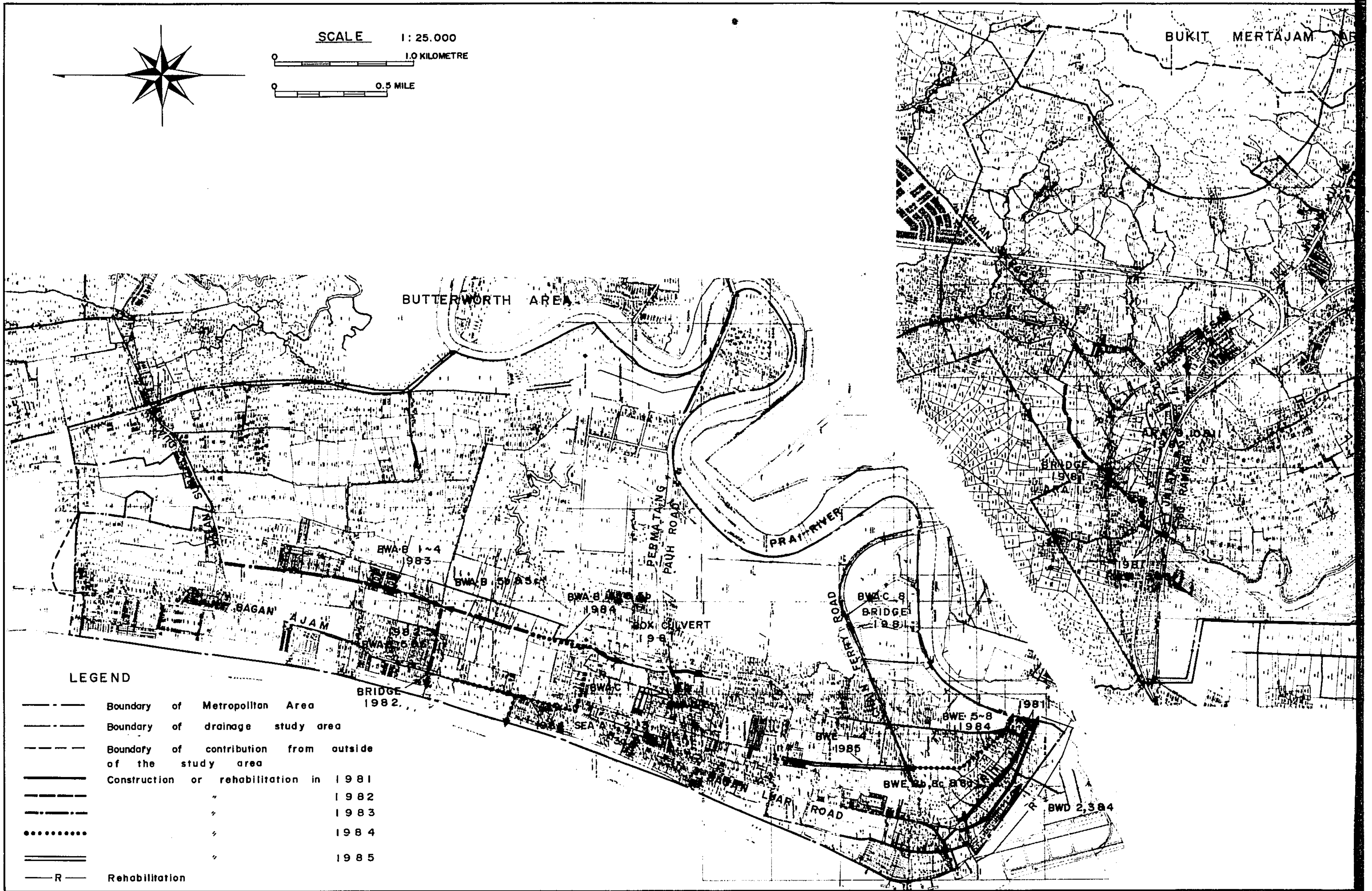
NOTE:

Only excavation will be finished in 1981 for drain BWE 6

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

CONSTRUCTION PROGRAMME CASE I

FIGURE 5·2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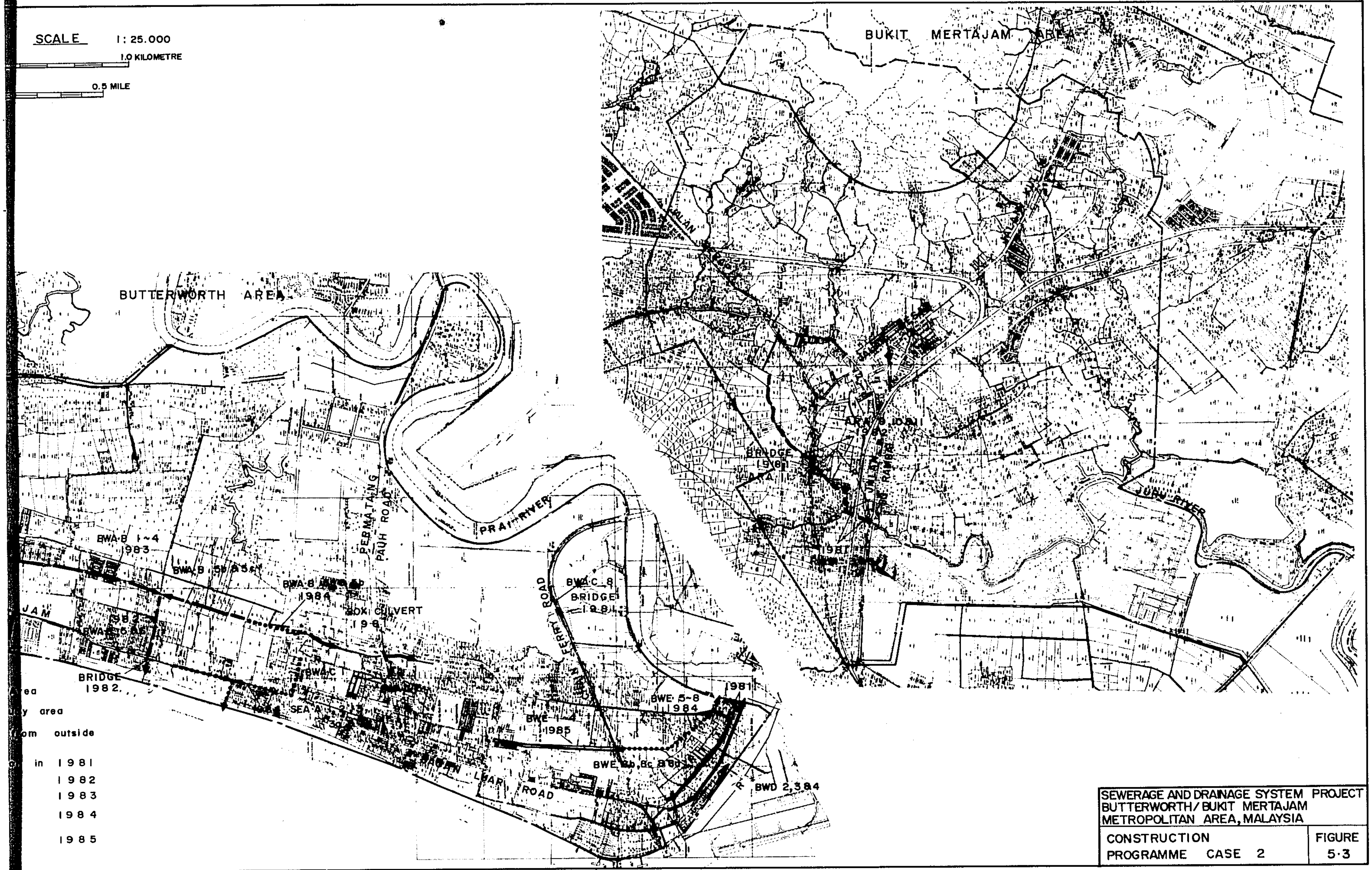
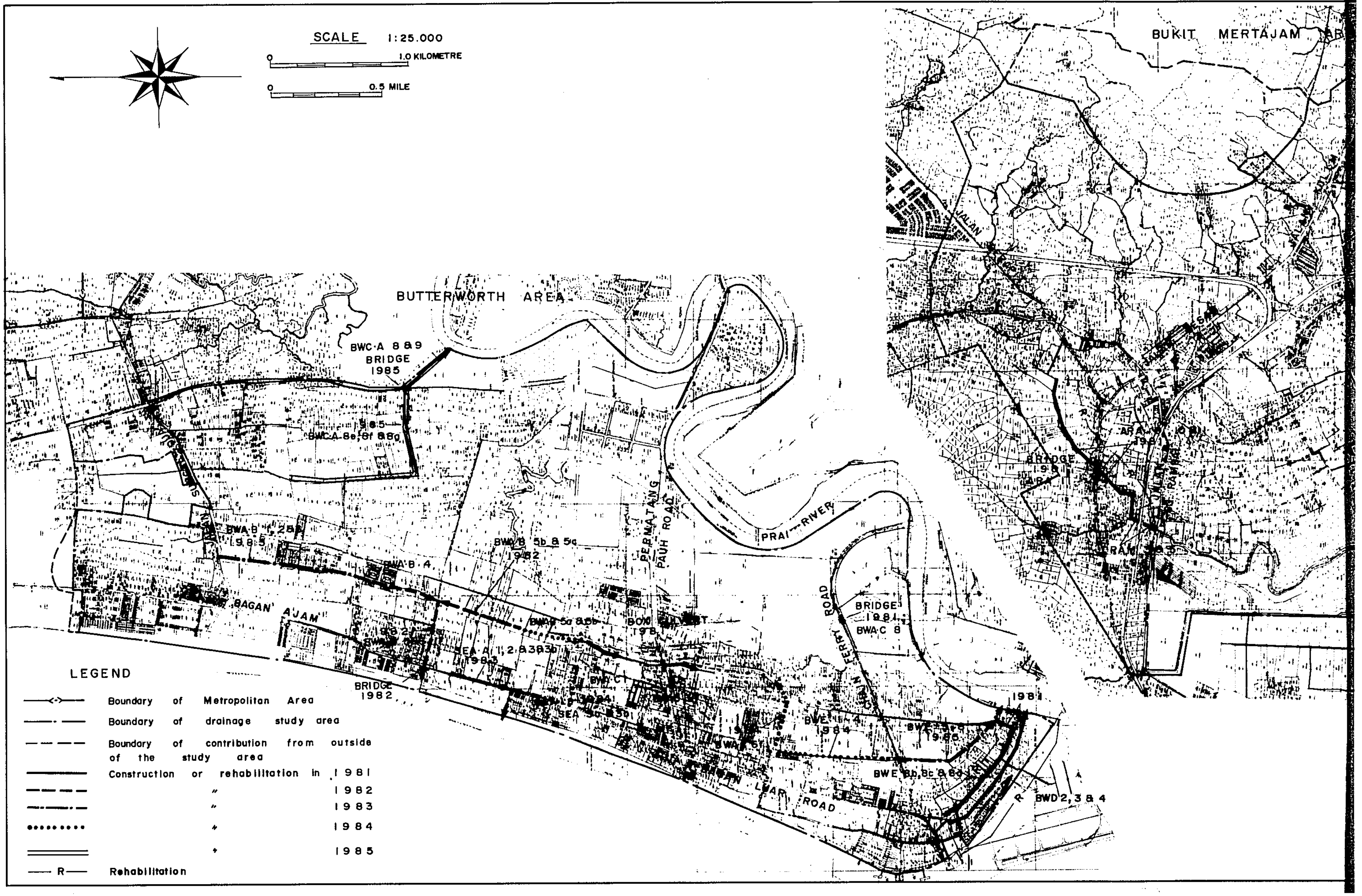
0 0.5 MILE
BUTTERWORTH AREA
BUKIT MERTAJAM
PRAI RIVER
PERMATANG PAUH ROAD
FERRY ROAD
BAGAN AJAM
BWA-B 1~4
1983
BOX CULVERT
BWA-C 8
BRIDGE
1981
BRIDGE
1982
BWE 5~8
1984
BWE 1~4
1985
BWD 2,3,8,4
LEGEND
Boundary of Metropolitan Area
Boundary of drainage study area
Boundary of contribution from outside of the study area
Construction or rehabilitation in 1981
1982
1983
1984
1985
Rehabilitation



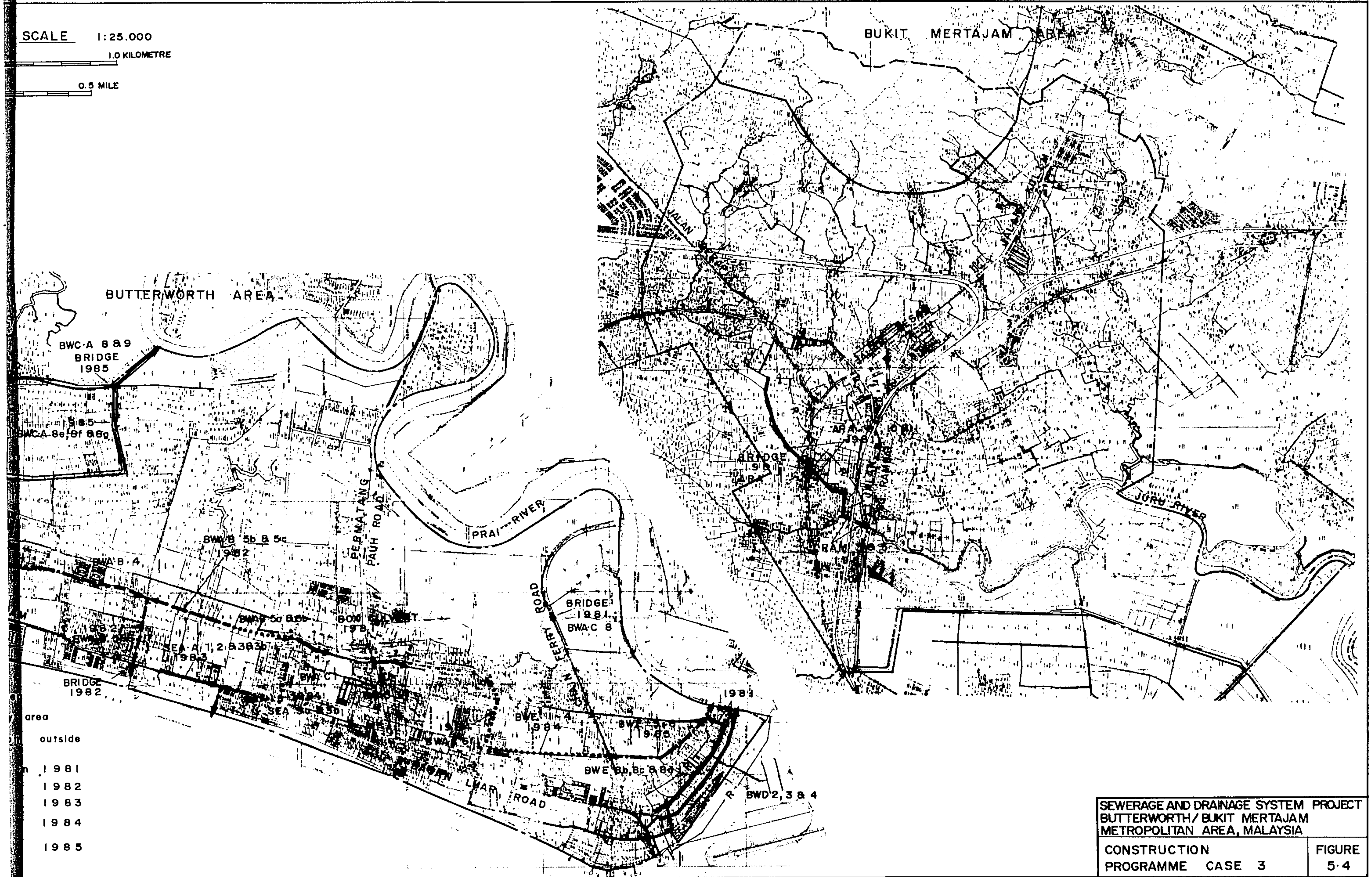
SEWERAGE AND DRAINAGE SYSTEM PROJECT
BUTTERWORTH / BUKIT MERTAJAM
METROPOLITAN AREA, MALAYSIA

CONSTRUCTION PROGRAMME CASE 2

FIGURE 5·3



SCALE 1:25.000
0
1.0 KILOMETRE
0
0.5 MILE
BUKIT MERTAJAM
BUTTERWORTH AREA
BWC-A 8 & 9
BRIDGE
1985
PRAI RIVER
PERMATANG PAUH ROAD
FERRY ROAD
BAGAN AJAM
BRIDGE
1981
BWA-C 8
BRIDGE
1982
BWE 8b, 8c & 8d
BWD 2, 3 & 4
LEGEND
Boundary of Metropolitan Area
Boundary of drainage study area
Boundary of contribution from outside of the study area
Construction or rehabilitation in 1981
" 1982
" 1983
" 1984
" 1985
R Rehabilitation



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

CONSTRUCTION PROGRAMME CASE 3

FIGURE 5·4

5.4 Financing

5.4.1 Investment Requirement

Based on the estimated costs required for the first stage drainage construction, the annually required capital costs, including allowances for price escalation assumed at 5 per cent per annum, have been estimated as shown in Table 5.9.

The total construction cost including engineering cost has been estimated to be M$6,403,000 at 1977 prices and M$8,525,000 at the escalated prices. The operation and maintenance for the proposed drainage system can be undertaken by the presently available personnel and equipment, and will require no additional labour costs except for the maintenance of newly procured equipment. Such expenses as annually required for the maintenance of new equipment is nominal amounting to approximately M$600 - M$750 as detailed in previous Section 5.2, Maintenance Cost.

Although there is a potential requirement for foreign currency to procure some locally available equipment and materials of foreign origin, the amount of such equipment and materials' cost to be represented by foreign currency is negligibly small amounting to only M$1.3 million at 1977 prices, being approximately 20 per cent of the total cost. The construction cost has, therefore, been estimated totally in local currency only.

5.4.2 Financing Sources

The financing for the drainage system construction and maintenance is normally dependent on both the annual budget allocation of the local authority and ad hoc contributions from the national government under national development plan similar to the financing for road provision. The community traditionally regards such an activity of drainage construction and maintenance as a government responsibility since stormwater runoffs are not directly associated with individuals' discharge contributions. It should be emphasized, however, that they are receiving the overall community's benefits from drainage system as mitigation of floods, elimination of general nuisance which will otherwise be arisen. Although the benefits of such nature are intangible and difficult to be quantified, the increase of land price is normally expected in the area where drainage system is provided. The land owners or private land developers, therefore, are required to provide with due financial contributions. The amount of revenue to be derived from such private sector is, however, largely dependent on the awareness of the public of the need for the drainage systems and acceptable means of revenue collection. As for the legal supports necessary

Table 5.9 Construction Cost Estimated by Year

	1980	1981	1982	1983	1984	1985	Total
Construction Cost	-	730	1,017	841	698	570	3,856
Contingencies	-	146	204	169	140	114	773
Engineering Cost	231	44	61	51	42	34	463
Land Cost	-	214	75	224	265	385	1,163
Equipment	-	100	12	36	-	-	148
Total Cost (End, 1977 price)	231	1,234	1,369	1,321	1,145	1,103	6,403
Escalation Factor*	1.158	1.216	1.276	1.340	1.407	1.477	
Total Cost (Escalated Price)	267	1,501	1,747	1,770	1,611	1,629	8,525

* Escalated at 5% per annum on End 1977 price.

to ensure the contribution from such land owners and developers, the local Government Act, 1976, presently in force, empowers MPSP to impose a drainage tax at a maximum rate of 5 per cent of the annual property value to recover the whole or part of the construction cost of the drainage systems. The other regulations such as Street, Drainage and Building Act, 1974 and Town and Country Planning Act, 1976 which will be enforced in due course can also be applied perforce to recover the costs if needs arise since such regulations include some provisions qualifying the local authority to ask due payment for such costs to be paid by the frontagers or developers directly relevant to such drainage systems to be constructed.

The government's endeavour supported by legal enforcement is desirable to enhance the awareness of the public of the benefits from the drainage system and effectuate the revenue collection either by taxation or direct payment which eventually help reduce the financial burden on the Government for the drainage project. The special task division of MPSP is presently undertaking the public relation services including the activities to enhance public concerns to environmental sanitation. This division is desired to inform the public of the advantages and benefits to be derived from the drainage systems by distributing informative documents or by occasional demonstrations in public meeting.

From the information obtained from the State Government officials, the budget allocation amounting to approximately M$3.3 million has been made from the Federal Government towards the drainage construction in urban and industrial areas in the State of Penang during the Second Malaysian Plan (1971 - 1975) and MS$4 million for the same purpose in the Third Malaysian Plan (1976 - 1980). The amount to be allocated for the drainage construction in the Fourth Malaysian Plan (1981 - 1985) will be increased to approximately M$5 million if the constant ratio of increase is assumed.

Annual budget allocation has been made to the respective agency from the general revenue fund of State Government for the operation and maintenance of the existing urban drainage system. The actual amount of such allocation in 1977 is M$2.75 million and M$3.5 million is expected to be allocated in 1978.

5.4.3 Conclusions

It is concluded, from the present practice of the government for the budget allocation for the drainage construction as well as operation and maintenance, that the investment requirement for the proposed drainage construction programme is within the financial capacity of the Government to be physically implemented within the first stage project period. It is, therefore, recommended to the Government to warrant the required funds prior to the initiation of the construction programme.

CHAPTER 6

BENEFITS

6.1 General

Various types of benefits will be derived from the recommended drainage system, including (1) decrease of flood damage, (2) improvement of environment for individual and community life, and (3) decrease of swamps and mosquito breeeding.

Description of these benefits is given in Master Plan Report (refer to Chapter 6, Part IV) covering the entire Project Area. In this chapter, the benefits expected from the first stage programme are discussed.

6.2 Reduction of Flood Damage

The existing flood prone areas, such as Kg. Bagan Dalam, the areas adjacent to the Chartered Bank and watch tower, the down reach of Butterworth drain A, and the Kampung areas along the downstream of the Sungai, will be relieved from flooding if the improvement and rehabilitation for the existing drains and dredging for E-1 drain are completed (see Figure 5.2). In case of flooding in the area adjacent to the Chartered Bank and watch tower, the traffic inconvenience is frequently encountered. If the improvement of the drainage system in Butterworth drain D is carried out, this traffic problem is eliminated and significant benefits, although unquantified, is expected in business activities in the area.

6.3 Improvement of Public Health and Convenience of Community

The alleviation of flooding in the kampung areas will improve the public health conditions and the convenience of the community life. Night soil in these areas is removed by bucket system, but leached into ground in the case of pit privy. When flood occurs, the contents of buckets and privies flow out and spread over the houses. The absence of a drainage system has caused wastewater pondings emanating offensive odour and giving esthetic problem at many places in the kampung areas. When the drainage system is provided in these areas, these insanitary conditions will be improved.

6.4 Decrease of Swamps and Mosquito Breeding

These numerous swamps or pondings in the area contribute to mosquito breeding. MPSP has been controlling the mosquito breeding by chemical spray to swamps and pondings, expending considerable amount of recurrent money. Improvement of the existing drains together with the provision of new drains will decrease the cost which otherwise be required and create more comfortable environment to the residents in these areas.

6.5 Benefits Justification

As has already been described in the previous sections, major portion of the benefits by the drainage improvement is hardly quantifiable in money terms, and therefore no attempt is made to calculate a cost-benefit ratio.

Nevertheless, there will be no doubt high social benefits if the project is completed because the system will make significant flood-free land for further development, upgrade the existing living environment, and also contribute to improving the inconvenience of the community life.

A N N E X E S

ANNEX 1

COST COMPARISON WITH RUBBLE WALL AND REINFORCED CONCRETE

Costs of rubble wall and reinforced concrete channels are estimated and compared each other in their advantages. An advantage of the rubble wall channel is that the construction cost is lower than that of reinforced concrete. On the other hand, the rubble wall channel requires larger cross sectional area and wider right-of-way because of it's higher roughness coefficient of the flow surface and slower side slope of 3 vertical to 1 horizon.

For cost comparison, both the channels are estimated in their construction and land costs in the following procedures:

(a) Basis of Calculation

Item	Rubble Wall	Reinforced Concrete
Shape	Trapezoidal	Rectangular
Invert finish	Paved	Paved
Roughness Coefficient	0.02	0.015

Hydraulic gradient and depth of the channels are assumed to be the same for calculation.

(b) Hydraulic Calculation of Channel

Relation of the two types of channels to the corresponding flow rates is expressed as follows:

$$Q = A_1 \cdot V_1 = A_2 \cdot V_2$$

where

Q = flow rate in channel, m^3/sec

A_1, A_2 = cross sectional areas required to flow 'Q' in rubble and concrete channels respectively, m^2

V_1, V_2 = velocities in rubble and concrete channels respectively, m/sec

Using the above equation, cross sectional areas of channels with the different flow rates are calculated by trial and error, and the required widths of the channels are then estimated as shown below:

A_1 (m^2)	A_2 (m^2)	$W_1^{(*)}$ (m)	$W_2^{(**)}$ (m)
9	8	5.17	4
11.5	10	6.42	5
14.0	12	7.77	6
16.5	14	8.92	7
19.0	16	10.17	8

Note: (*) upper width of rubble channel

(**) upper width of concrete channel

(c) Relationship Between Land Cost and Construction Cost

The economical channel type can be selected by comparing the costs of construction and land between the two types and finding the reflection point of the cost. The point is found when the costs of both types balance as expressed in the following relation:

$$C_{A1} + L\,W_1 = C_{A2} + L\,W_2$$

then

$$L = \frac{C_{A1} - C_{A2}}{W_1 - W_2}$$

where

C_{A1} = construction cost of rubble channel with cross sectional area of A_1, M$/m

C_{A2} = construction cost of concrete channel with cross sectional area of A_2, M$/m

L = unit land cost, M$/m^2

W_1 = upper width of rubble channel with cross sectional area of A_1, m

W_2 = width of rectangular concrete channel with cross sectional area of A_2, m

Using the above equation and also cost function curves as presented in Figure 5.1, cost estimations for five cases have been made and compared each other to find the land cost which balances the costs of both types of channel. The result of the calculation indicates that for the area with land price of M$160/m^2 (M$15/ft^2) or higher, reinforced concrete channel is more economical than rubble channel because of the reduced land cost for concrete channel.

ANNEX 2

ESTIMATION OF WATER LEVEL IN THE JURU RIVER AT THE UPSTREAM OF THE JURU TIDE GATE

The water level in the stream can be calculated by the equation of:

$$V_2 - V_1 = \left(\frac{I_1 + I_2}{2}\right)t - \left(\frac{Q_1 + Q_2}{2}\right)t$$

where

I_1 and I_2 = stormwater inflow at the time of T_1 and T_2 respectively, m^3/sec

V_1 and V_2 = stormwater quantity stored at the time of T_1 and T_2 respectively, m^3

Q_1 and Q_2 = stormwater quantity being discharged at the time of T_1 and T_2 respectively, m^3/sec

(a) Basis of Calculation

- Areas of catch basin : 3,000 ha
- Time of concentration : 150 minutes
- Runoff coefficient
  - 5-yr frequency storm : 0.4
  - 100-yr frequency storm : 0.65
- Time of inflow : 25 minutes
- Rainfall intensity-duration-frequency curve
  - 5-yr frequency : $I_5 = \frac{8,070}{t + 30}$
  - 100-yr frequency : $I_{100} = \frac{13,940}{t + 33}$
- Sea water level applied
  - 5-yr frequency storm : +1.10 m (+3.6 ft)
  - 100-yr frequency storm : +0.15 m (+0.5 ft)

(b) Conditions of Outlet

The Juru Tide Gate

	width		depth		number
the size	3.65 m (10 ft)	x	2.4 m (8 ft)	x	2
invert level	-0.48 m				

The overflow at the side of the gate

This consists of 15 box culverts each with 1.03 m (3.5 ft) width by 1.52 m (5 ft) height.

(c) Results of Calculation

+1.34 m (4.4 ft)	5-yr frequency storm and present land use (C = 0.15)
+1.86 m (6.1 ft)	5-yr frequency storm and the year 2000 land use (C = 0.4)
+2.90 m (9.5 ft)	100-yr frequency storm and ultimate land use (C = 0.65)

If the size of the gate is enlarged by 100 per cent, the water level will be +2.5 m (8.2 ft).

ANNEX 3

COMPARISON OF STORMWATER QUANTITIES BETWEEN 2-YEAR AND 5-YEAR STORMS

Rainfall intensity-frequency-duration curves for 2-year and 5-year storms are as follows:

$$I_2 = \frac{6,270}{T + 32}$$

$$I_5 = \frac{8,070}{T + 30}$$

$$\frac{I_2}{I_5} = \left(\frac{6,270}{8,070} \right) \left(\frac{T + 30}{T + 32} \right)$$

when $T = 10$ minutes,

$$\frac{I_2}{I_5} = 0.78 \times 0.95 = 0.74$$

$T = 60$ minutes,

$$\frac{I_2}{I_5} = 0.76$$

From the above equations, the relationship between I_2 and I_5 is expressed:

$$\frac{I_2}{I_5} = 0.75$$

In the Rational method, factors such as drainage area, runoff coefficient, and storage coefficient are constant, with the same values for both 2-year and 5-year storms. Thus, the relation between the flow rates of 2-year and 5-year frequencies can be expressed:

$$Q_2 = 0.75\ Q_5$$

ANNEX 4

OVERFLOW WEIR AND ORIFICE IN BWC.A DRAIN SYSTEM

(a) Weir

Villemonte's equation is applied for determining the level of the weir used in the outfall of BWC.A drain system, in the form:

$$Q = Q_1 \left[1 - \left(\frac{h2}{h1} \right)^n \right]^{0.385}$$

where

Q = flow rate when the water level upstream of the weir is affected by downstream water level, (m^3/s)

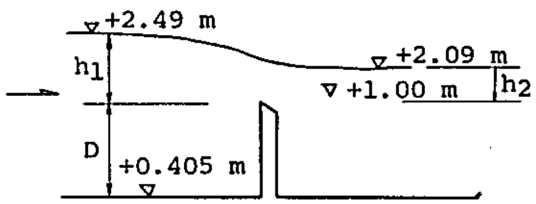
Q_1 = flow rate with free drop, (m^3/s)

h1 = water depth from the weir crest at upstream of the weir, (m)

h2 = water depth from the weir crest at downstream of the weir, (m)

n = constant, 1.5

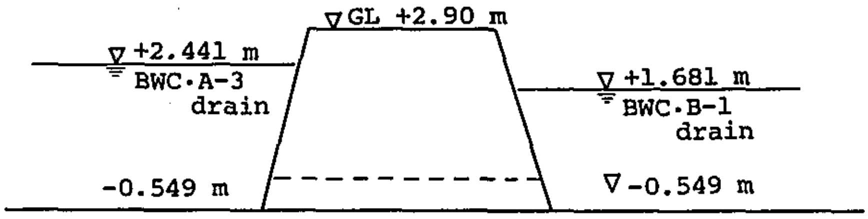
The physical condition near the weir is shown below.



Under the situations, the weir level is calculated to be +1.0 m (3.3 ft).

(b) Orifice

The size of orifice used at BWC.A drain system (Ref. Figure 4.9) is calculated as follows:



The flow rate is calculated by the following equation;

$$Q = AC\sqrt{2gh}$$

where

A = hydraulic area of orifice, m^2

C = constant; 0.603

The hydraulic area "A" is determined on the basis of the expected dry weather flow rate. The results of calculation show that A is 0.02 m^2 and the diameter of the orifice is 525 mm (20 in). At the time of rainfall, the discharge from the orifice is calculated to be about 0.5 m^3/sec.

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