CHAPTER 3. HYDRAULIC AND HYDROLOGICAL ANALYSES

3.1 Rainfall Analysis

3.1.1 Rainfall Gauging Data

The hydrograph of probable flood discharge within a short duration is essential for formulating the drainage structure plan. Hourly discharges are however observed at only one (1) gauging station in Sungai Melaka, so that discharge hydrographs in the study area are scarcely available. Therefore, a flood simulation model was prepared to estimate probable flood discharge from the observed rainfall data. Firstly carried out was rainfall analysis, and then the probable flood hyetographs were developed.

Rainfall depth is observed and recorded by the Department of Irrigation and Drainage (DID) and the Malaysian Meteorological Service (MMS). Among the rainfall gauging stations, the following four (4) stations could provide rainfall intensity on a short term, in and around the study area. Out of the four (4) gauging stations, one is operated by DID in Alor Setar in the State of Kedah, while the other three (3) are by MMS at airports in Alor Setar, Penang and Melaka, respectively. Such rainfall intensities are indispensable to carry out the runoff analysis for the drainage improvement plan.

Name of Gauging Station	Observed by	Location		Observation Period	
		Latitude	Longitude	From	to
Stor JPSAlor Setar	DID	06°07'00"	100°21'25"	Dec. 1964	Present
Kepala Batas (Alor Setar)	MMS	06°12'05"	100°24'45"	Sep. 1936	Present
Bayan Lepas (Penang)	MMS	05°17'50"	100°16'20"	May 1934	Present
Lanpangan Terbang Melaka	MMS	02°16'00"	100°12'50"	Jan. 1946	Present

The hourly rainfall data observed at the above four (4) gauging stations for the recent five (5) years were collected to clarify the temporal variation of rainfall and the dominant rainfall duration. The annual maximum rainfall intensities of various rainfall duration were also collected from the records of the three (3) gauging stations operated by MMS. The collected annual maximum intensities were for the period from 1951 to 1997, and these maximum intensities were used to develop the rainfall intensity-duration curves.

3.1.2 Rainfall Intensity-Duration Curves

Examined was the time length of one sequence of rainfall, through the observed hourly rainfall for the recent 5 years at the foregoing four (4) gauging stations. As the result, it was clarified that the time length of almost all sequential rainfalls are within 12 hours, as shown in Fig. 3-1.

In due consideration of the clarification, rainfall intensity-duration curves were developed on the premise that maximum rainfall continuation time is 12 hours.

The recurrence probability analysis on the point rainfall intensity for various rainfall duration within 12 hours was made by the Gumbel Method using the aforesaid series of annual maximum rainfall intensities at the three (3) gauging stations operated by MMS. As the result, estimated are the probable intensities of point rainfall of various rainfall duration as shown in Table 3-1.

Rainfall intensity-duration curves express the relation between the probable rainfall intensities and their corresponding rainfall duration, and proposed are several equations to express this relation. In this study, selected is the most conformable equation among the following four (4) prominent equations:

(1)	Talbot Type	:	$I = a / (T + b)$
(2)	Sherman Type	:	$I = a / T^{n}$
(3)	Kuno Type	:	$I = a / (T^{0.5} + b)$
(4)	Honer Type	:	$I = a / (T + b)^{n}$

Where; I : Rainfall Intensity
T : Rainfall Duration
a, b, n : Constants

The least-square regression method was used to estimate the constants for the above equations. Compared were the probable rainfall intensities estimated by the equations and the probable rainfall intensities given from the observed data. As the result, confirmed to be most conformable was the Talbot Type of equation.

The rainfall intensity-duration curves at the MMS' three (3) gauging stations developed by Talbot Type are as shown in Fig. 3-2. All curves resemble each other, probably because all gauging stations are located in the same meteorological region of East Coast. Thus, there is no significant difference among the three kinds of rainfall-duration curves; nevertheless, the following curves were applied to each of the study areas, Sungai Petani and Melaka:

(1) Sungai Petani : The study area is located between two gauging stations, Penang (at Bayan Lepas) and Alor Setar, and the rainfall intensity-duration curves could be selected from those developed for the two (2) gauging stations. As shown in Fig. 3-2, the rainfall intensity of Alor Setar is slightly lower than that of Penang. Hence, to avoid

underestimation of rainfall intensity, selected are the rainfall intensity-duration curves of Penang as the principal curves for the study area of Sungai Petani.

(2) Melaka : The rainfall gauging station at Lanpangan Terbang Melaka (Melaka Airport) is located within the study area, and its rainfall intensity-duration curves were applied to the study area.

3.1.3 Model Hyetograph

Developed for flood runoff simulation are model heytographs in various return periods, with the following major factors taken into consideration:

(1) Entire Rainfall Duration

As described in the foregoing subsection, the assumed maximum time length of one sequence of rainfall is 12 hours and therefore, the entire rainfall duration of the model hyetograph is set at 12 hours.

(2) Temporal Variation of Rainfall Depth

The temporal variation of rainfall is one of the important factors governing the storage volume of flood detention ponds. The most critical temporal variation of rainfall for flood detention ponds is such that the rainfall gradually increases reaching its peak at the end of storm rainfall. Such temporal rainfall pattern requires the largest flood detention volume.

Examined through the hourly rainfall observed at the foregoing four (4) gauging stations were the actual temporal variations of peak rainfall depth. The observation period was five (5) years. As shown in Fig. 3-3, more than 5% of rainfall sequences have their peaks at the end of rainfall. Thus, confirmed was the occurrence of the above critical temporal variation of rainfall in the observed rainfall data and therefore, the pattern is applied to the model hyetographs.

(3) Areal Reduction of Rainfall

During a storm, rainfall usually distributes unevenly over the catchment area and tends to decrease with distance from the storm center. That is, as the drainage area increases, the area average rainfall tends to be lower than the value of point rainfall. However, the subject catchment areas of the Study have a small size of 52 km^2 in maximum, as described in the following Subsection 3.2.1. Such a small-sized catchment area would not give any significant difference between the point rainfall and the areal average rainfall. In fact, the "Hydrological Procedures No. 12" published by DID in 1994 recommends a marginal reduction rate of less than 5% for the catchment area of less than 50 km^2. From these points of view, assumed to represent the areal average rainfall is the adopted point rainfall.

Developed were model hyetographs for 5- and 100-year recurrence probabilities, as shown in Figs. 3-4, based on the above three assumptions and the rainfall intensity-duration curves described in the foregoing subsections.

3.2 Runoff Analysis

3.2.1 Basin Runoff Model

The runoff simulation model consists of two component items. The first component is the basin runoff model to express the flood runoff generated by rainfall in the sub-basins. The second component is the channel flow model to express the propagation of channel flow discharge from upstream to downstream. The details of the basin runoff model is described in this subsection, while the channel flow model is described in the following subsection. To facilitate the basin runoff simulation, the study area was partitioned into 12 groups of major river basins (6 for Sungai Petani and 6 for Melaka), as shown in Fig. 3-5. The 12 groups were further divided into 137 sub-basins (69 for Sungai Petani and 68 for Melaka). These basin divisions are as summarized below (refer to Fig. 3-6):

Study Area of Sungai Petani		
Name of Major River Basins	Catchment Area (km^2)	Number of Sub-basins
1. Lalang	25	11
2. Tukang	8	7
3. Layar Besar	4	4
4. Che Bima	3	3
5. Petani	38	34
6. Pasir	23	10
Total	101	69

Study Area of Melaka		
Name of Major River Basins	Catchment Area (km^2)	Number of Sub-basins
1. Lereh	35	8
2. Malim	52	15
3. Melaka	32	17
4. Cheng	37	10
5. Putat	23	9
6. Others*	13	9
Total	191	68

* Group of drainage area directly flowing into the sea

In due consideration of the ongoing drastic change of land use in the study area, the "Quasi-Linear Runoff Simulation Model" was selected as the basin runoff model. The simulation model has a function to express differences in runoff due to variation in land use. Moreover, the model could express non-linear characteristics of flow over a sloping surface. Because of the function, the model could generate not only peak discharge but also the flood hydrograph. The flood hydrograph is indispensable in this study for estimating the capacity of basin flood detention facilities.

The basic equations of the model are composed of continuity equation (Eq. 3-1) and storage equation (Eq. 3-2)

$$dS/dt = re - q \qquad \text{(Eq. 3-1)}$$

$$S = K \times q \qquad \text{(Eq. 3-2)}$$

Where; S : Storage depth of basin (mm)

re : Effective rainfall intensity (mm/hr)

q : Runoff depth (mm/hr)

K : Recession constant

Obtained was the effective rainfall intensity "re", multiplying the model hyetograph with the runoff coefficient for each land use item. The runoff coefficient was determined, referring to the "Urban Drainage Design Standards and Procedures for Peninsular Malaysia", as shown Table 3-2.

In the above equations, the recession constant "K" was experimentally estimated from the observed data in several model basins in Japan and simply expressed as below:

$$K = Tc/2 \qquad \text{(Eq. 3-3)}$$

$$Tc = C \times A^{0.22} \times re^{-0.35} \qquad \text{(Eq. 3-4)}$$

Where; Tc : Concentration time (hr)

C : Coefficient of basin characteristics

A : Catchment Area (km^2)

The above Eq. 3-3 is as proposed by Yoshino based on experiment on the relationship between actual "K" and flood lag time assuming that the concentration time Tc corresponds to half of the flood lag time.

Eq. 3-4 is based on the Kinematic Wave Theory as proposed by Kadaya. The equation could present variation of concentration time according to magnitude of effective rainfall. As to coefficient "C" for basin characteristics in the equation, the "C" for mountainous area was firstly derived as standard value through the experiment with 18 model basins. Then the "C" for other land use was estimated from the standard value through the Kinematic Wave Theory. The "C" value for each land use category is as shown in Table 3-2. Since coefficient "C" changes according to land use, the equation could also present variations of concentration time according to differences of land use.

3.2.2 Channel Flow Model

As described in the foregoing subsection, the model aims at simulating the channel flow discharge propagated from upstream to downstream. In order to facilitate the simulation, the channels in the study area were divided into 72 sections as listed below. Detailed features of the divided channel sections are as shown in Table 3-3.

Study Area of Sungai Petani		
Name of River	Total Channel Length (km)*	Number of Divided Sections
1. Lalang	14.6	7
2. Tukang	3.9	3
3. Layar Besar	3.8	2
4. Che Bima	2.5	2
5. Petani	20.3	18
6. Pasir	10.0	6
Total	55.1	38

Study Area of Melaka		
Name of River	Total Channel Length (km)*	Number of Divided Sections
1. Lereh	11.8	4
2. Malim	12.9	9
3. Melaka	7.9	10
4. Cheng	20.2	6
5. Putat	14.5	5
Total	67.3	34

* Channel Length applied to the simulation model including the length of main stream and major tributaries.

The main parameters considered for the model are the storage function of the river channel and the flood travelling time on the channel. Most of the present river channel flow capacities are extremely low and channel overflow frequently occurs. Under such present conditions, the flood discharge is hardly propagated from upstream to downstream, and the present channel storage function is extremely large in appearance. However, since the present flood inundation land is going to be converted to the highly value-added land, river channel improvement has to be implemented to accommodate the flow discharge and prevent it from overflow. From this point of view, the river channel simulation was made to estimate the probable channel flow discharge that will occur when the river channel will not cause any channel overflow. In other words, the estimated probable channel flow discharge could be regarded as the standard design flood discharge for each river channel.

In order to express the storage function of the river channel, the "Storage Function Model" was applied as the principal channel flow model. The application of this model was, however, subject to the rather large river channel where the channel width is more than 10 times of channel depth. As for the small drainage channel, the storage function was regarded to be nil.

The basic equations of the model are composed of the continuity equation (Eq. 3-5) and the storage equation (Eq. 3-6).

$$dS/dt = Qi(t) - Qo(t) \qquad \text{(Eq. 3-5)}$$

$$S = K \cdot Qo(t)^P \qquad \text{(Eq. 3-6)}$$

Where; S : Storage volume of the basin ($m^3 \cdot hr/s$)

Qi	:	Inflow discharge of the channel (m^3/s)
Qo	:	Outflow discharge of the channel (m3/s)
K, P	:	Parameters

The parameters "K" and "P" are expressed in Manning's Formula for Steady Flow where "P" takes the constant value of 0.6 and "K" takes the value estimated from the following equation:

$$K = L \cdot B^{0.4} \cdot (n/I^{0.5})^{0.6} / 3.6 \qquad \text{(Eq. 3-7)}$$

Where; L : Channel Length (km)

B : Average channel width (assumed from the existing channel width)

n : Manning's roughness coefficient (= 0.035 for non-improved channel, and 0.02 for improved channel)

I : Average channel slope (assumed from the existing channel slope)

In addition to the above channel storage function, assumed was the channel flood travelling time to each of the subject channels using the following "Kraven Formula":

$$T = L / (W \cdot 3.6) \qquad \text{(Eq. 3-8)}$$

Table 3-3 shows the above parameters "K" and "P" for the "Storage Function Model as well as the "T" estimated by "Kraven Formula".

3.2.3 Results of Simulation

Through the aforesaid "Basin Runoff Model" and "Channel Flow Model", simulated are the probable basin-runoff discharge for each sub-basin as well as the probable channel flow discharge for each divided channel (refer to Tables 3-4 to 3-5). The simulation is subject to the recurrence probability of 5-year return period and 100-year return period. Moreover, the simulation is made on the premise of the present land use states and the projected land use states as of 2020. Present land use states are described in the foregoing Section 2.3, while projected land use states are in Section 4.1.

The study area is now being rapidly urbanized, and this has a significant influence on the basin runoff conditions. Due to rapid urbanization, the probable discharge in the year 2020 is simulated to remarkably increase as compared with the present probable discharge (refer to Fig. 3-7). Shown in the table below is the probable discharge of 5-year return period at the down-most point of each river basin to clarify the difference between the present and future probable discharges. As shown below, a remarkable increment of probable discharge is estimated in Lalang and Che Bima in Sugai Petani, and Malim and the upper reaches of the diversion weir of Melaka River (i.e., the Cheng/Melaka 1 in the following table) in Melaka.

Study Area of Sungai Petani		
Name of River	Probable Discharge (m^3/s)	
	Present	In 2020
1. Lalang	193	372 (+ 93%)
2. Tukang	88	132 (+ 50%)
3. Layar Besar	61	69 (+ 13%)
4. Che Bima	33	77 (+130%)
5. Petani	196	239 (+ 22%)
6. Pasir	165	231 (+ 40%)

Study Area of Melaka		
Name of River	Probable Discharge (m^3/s)	
	Present	In 2020
1. Lereh	172	299 (+ 33%)
2. Malim	261	538 (+106%)
3. Melaka 1*	200	538 (+170%)
4. Melaka 2*	221	245 (+ 11%)
5. Cheng	184	333 (+ 81%)
6. Putat	163	193 (+ 18%)

* Melaka 1 is the upstream from the diversion weir of Melaka River, while Melaka 2 is the downstream of the weir.

3.3 Analysis of Hydrological Effect of Basin Flood Detention Facility

3.3.1 Types of Basin Flood Detention Facility

Basin flood detention facilities are of the storage type and the infiltration type. Among these classifications, the infiltration type is not applicable because of the poor infiltration capacity of the surface soil in the study area as described in Subsection 2.7. Accordingly, the analysis was made for the three storage types of flood detention facility; namely, (1) storage in house lot, (2) storage in public open space, and (3) flood detention pond. These three (3) types of storage facilities are potentially capable at contributing to drainage improvement for the following land use categories (refer to Subsection 4.2.1):

Detention facility	Applicable Land Use
Storage in House Lot	Existing residential area
Storage in Public Place	Projected institutional area
Detention Pond	Projected residential area, commercial area, industrial area and road

The storage type is classified into the off-site and on-site types. The off-site storage type could control runoff discharge from a rather extensive catchment area, while the on-site storage type could collect rainfall within its compound. The flood detention pond belongs to the off-site storage type, while the storage in house lot and in public open space are of the on-site storage type.

3.3.2 Estimation of Standard Hydraulic Dimensions of Basin Flood Detention Facility

The standard hydraulic dimensions of the foregoing three (3) basin flood detention facilities were estimated in the study, as shown in Table 3-6. These dimensions were incorporated into the basin runoff simulation model as basic data for the comparative study on alternative drainage improvement plans. The several assumptions for the estimation are as described below.

(1) Storage in House Lot

It is assumed that the average extent of house lot is 200 m^2 with the rooftop of 100 m^2. The size of water tank to collect the rainfall is also assumed to be 2m^3 (2 m^2 in extent by 1 m in height) in due consideration of the average available open space of house lot. The water tank has an outlet hole at side bottom. The size of the outlet hole is estimated on the premise that the water tank will not allow overflow against the probable rainfall of 5-year return. The recurrence probability of 5-year return period is derived from the target design level for drainage improvement, as described in Chapter 4. Based on these assumptions, trial simulation is made through the following equations, and the size of outlet hole is estimated at 4 cm in width and 3 cm in height.

$$dS(t)/dt = Qin(t) - Qout(t) \qquad \text{(Eq. 3-9)}$$

$$Qout\,(t) = 1.7 \cdot B \cdot H(t)^{1.5} \qquad \text{(If } H < 1.2D\text{)} \qquad \text{(Eq. 3-10)}$$

$$Qout(t) = C \cdot B \cdot D \cdot (2g \cdot (H(t) - D/2)^{0.5} \qquad \text{(If } H > 1.8D\text{)} \qquad \text{(Eq. 3-11)}$$

Where;

S(t)	:	Storage volume at time t
Qin(t)	:	Inflow discharge at time t estimated by the foregoing basin runoff simulation (refer to Section 3.2)
Qout(t)	:	Outflow discharge at time t
H(t)	:	Depth of storage at time t
B	:	Width of outlet hole
D	:	Depth of outlet
C	:	Coefficient (= 0.6)

(2) Storage in Public Space

The available storage space is assumed at about 20% of the entire public compound. The maximum storage depth and storage time length are also assumed to be 30 cm and 10 hours, respectively. This assumption is made in due consideration of original purpose of storage space as public utility. The depth of the surrounding drain is assumed to be 50 cm. One outlet is placed at the bottom of the drain. The size of the outlet is estimated on the premise that the storage will not allow overflow against the probable rainfall of 5-year return period. Based on these assumptions, trial simulation was made through the above equations (Eq. 3-9 to 3-11), and the standard size of out-hole was estimated at 20 cm in width and 5 cm in height for the public compound of 20,000 m^2.

(3) Flood Detention Pond

The storage capacity of the flood detention pond could be far larger than the other two types of detention facilities, and proposed is its regulation effect based on the following criteria:

(a) The flood detention pond will regulate the basin runoff from the new land development area and control the increment of peak runoff to be the same as before the land development.

(b) The flood detention pond will maintain the above regulation effect against the probable flood discharges of 5- and 100-year return period. The 5-year return period is proposed as the target design level for urban drainage, while the 100-year return period is the target design level for prevention of river channel overflow, as described in Chapter 4.

(c) In accordance with the Guideline for Detention Pond prepared by the Town and Country Planning Department, it is assumed that 4% of the new land development area could be allocated as the space for the detention pond.

To satisfy the above criteria, the flood detention pond is assumed to have two outlets, as illustrated in Table 3-6. The probable flood of less than 5-year return period is discharged through only the lower outlet, while the probable flood of 5 to 100-year return period is discharged through both of the two outlets.

The trial simulation was made using the above equations (Eq. 3-9 to 3-11), and the standard hydraulic structural features of detention facilities were determined as below (refer to Table 3-6).

Detention facility	Catchment area (m^2)	Storage Volume (m^3)	Width of Outlet (m)	Height of Outlet (m)
Storage in House Lot	200*1	2	0.04	0.03
Storage in Public Place	20,000*2	1200	0.20	0.05
Detention Pond	100,000*3	12,800	0.32 (upper outlet) 0.50 (lower outlet)	0.50 (upper outlet) 0.50 (lower outlet)

*1 : Standard extent of one house lot in existing residential area
*2 : Standard extent of one lot of projected institutional area
*3 : Standard extent of one lot of new land development area including residential area, commercial area, industrial area and road.

3.3.3 Flood Runoff Simulation of Effects of Basin Flood Detention Facility

To incorporate the effect of basin detention facilities into the foregoing basin runoff simulation model and the channel flow simulation model, the sub-basin used in the basin runoff simulation model was further divided into direct and indirect runoff areas, as shown in Fig. 3-8. The runoff discharge from the direct runoff area is not filtered by any detention facility, while the runoff discharge from indirect runoff area is regulated by the basin detention facilities. The storage volume and outlet width of the detention facilities applied to the basin runoff simulation model were determined through the following assumptions:

Storage Volume: $V = Vunit \cdot A / Aunit$ (Eq. 3-12)

Dimension of Outlet : $B = Bunit \cdot A / Aunit$ (Eq. 3-13)

Where;
- V : Storage volume incorporated to the basin runoff simulation
- B : Width of outlet hole
- Vunit : Standard storage volume as estimated in Subsection 3.3.2.
- Bunit : Standard width of outlet as estimated in Subsection 3.3.2.
- Aunit : Standard catchment area as estimated in Subsection 3.3.2.
- A : Catchment area in the basin runoff simulation model

The effect of the detention facilities was firstly confirmed through simulation for a unit catchment area of 1 km^2 that is simply covered with a simple land use. The simulation is based on the foregoing equations for basin runoff model (refer to Eq. 3-1 to 3-8) and the equations for flood storage estimation (refer to Eq. 3-9 to 3-11). As the result, the following effects of the basin detention facilities were confirmed:

Type of Detention Facility	Land Use in the Catchment Area	Flood Return Period	Peak inflow Discharge ($m^3/s/km^2$)	Peak Outflow Discharge ($m^3/s/km^2$)
Storage in House Lot	Existing residential area	1/5 year	44*	29
Storage in Public Place	Projected Institutional area	1/5 year	31	5
Detention Pond	Projected Commercial Area	1/5 year	39	6
		1/100 year	55	12

* Inflow discharge from rooftop where the runoff condition is assumed to be same as the road (paved)

Finally estimated are the probable basin runoff discharge and channel flow discharge effected by the detention facility. This estimation was made with assumptions of various catchment areas of detention facilities extracted from the alternative plan of drainage improvement. Details of the estimation are as described in Chapter 4.

3.4 Hydraulic Channel Flow Analysis

The flow capacities of drainage and river channels are described in Section 2-4 and 2-5, respectively. On the other hand, the estimation bases for the channel flow capacities are as described in this section.

The channel flow capacity of the major existing river and drainage channels were estimated through either the non-uniform calculation method or the uniform calculation method. The estimation was based on the results of channel survey undertaken by DID, and the number of channels surveyed reached 67 items with a total length of about 182 km.

The non-uniform calculation method was applied to the river channels and/or drainage channels that have a significant backwater effect to the channel flow conditions due to the gentle channel bed slope. The total channel length for non-uniform calculation is 111 km. On the other hand, the uniform calculation method was applied to the small drainage channels of about 71 km in total where the backwater effect is judged to be marginal due to the steep channel bed slope.

The Manning's roughness coefficient and the boundary conditions for the channel flow calculation are as described below.

(1) Manning's Roughness Coefficient

In the non-uniform as well as uniform calculation, the following Manning's roughness coefficient were applied referring to the "Urban Drainage Design Standard and Procedures for Peninsular Malaysia" prepared by DID in 1975:

(a)	Natural channel without any channel improvement	:	0.035
(b)	Improved channels with earth bottom	:	0.023
(c)	Concrete Culvert and Concrete lined channel	:	0.015

Most of the river and drainage channels examined in this study are still in their natural conditions without any channel improvement works. These river channels take the Manning's coefficient of 0.035.

However, out of the river channels examined, channel improvement has been completed for 5.7 km in the downstream of Sg. Malim River (i.e., bypass channel) and 7.1 km in the upstream from the diversion point of Sg. Melaka. For these newly constructed bypass and improved river channels, the Manning's coefficient of 0.023 was applied. Channel improvement is also going to be implemented to the downstream of Sg. Ayer Salak, a tributary of Sg. Malim for an improvement length of 5.5km. Since this channel improvement is going to start in the near future, the designed channel cross-sections were regarded as the present channel states, and the Manning's coefficient of 0.023 was applied.

Some of the drainage channels have concrete culvert and/or concrete lined channels. These concrete channels take the Manning's coefficient of 0.015.

(2) Boundary Condition

Applied were the following conditions as the initial river stages for non-uniform calculation:

(a) Channels Directly Flowing to the Sea

Mean high water spring tide level was applied as the initial river stage (refer to Table 3-7). The tidal levels were derived from the tide tables published by the Royal Malaysian Navy and the Department of Survey and Mapping, Malaysia in 1999. The tidal stations for the river channels in Sungai Petani and Melaka are Tanjung Dawai and Kela Melaka, respectively.

(b) Channels Flowing to Mainstream Channel

The following river channels join major river streams having a significant backwater effect from the mainstream; i.e., (1) Sg. Cheng and Sg. Putat joining Sg. Melaka, (2) Sg. Ayer Salak and Sg. Ayer Hitam joining Sg. Malim. As for the non-uniform calculation for these river channels, assumed was the bank level of the mainstream at the confluence point a s the initial river stage.

(c) Upstream of Sg. Melaka from the Diversion Point

The flow discharge from Melaka River is diverted into the bypass channel during flood time. The diversion discharge is dominated by the hydraulic characteristics of diversion weir. Hence, the initial water stage was derived from the rating curve between the river stage and the overflow discharge at the diversion point. The rating curve was presented in the "Melaka Flood Mitigation Project, Designer's Operation Criteria" prepared by DID in 1990.

3.5 Sediment Runoff

The recent intensive land development tends to cause serious soil erosion within the area and bring sediment to the lower reaches. Due to the sediment, the flood mitigation capacity of the drainage facility is remarkably reduced and, at the same time, the environment in and around the drainage facility is seriously deteriorated.

The actual sediment runoff discharge has been gauged in Kuala Lumpur and Penang, although the gauging has not been made in the study area.. The results of gauging on sediment runoff are as given below.

Catchment	Condition of Land Use	Area (km^2)	Rainfall (mm)	Sediment Runoff ($ton/km^2/year$)	Data Source
Sg. Air Hitam, Penang	Tropical rainforest	4.75	2,580	74.49	Wan Ruslan, 1995
Sg. Air Hitam, Penang	Tropical rainforest in upper part, stable urban area in lower	8.87	2,580	376.59	Wan Ruslan, 1995
Sg. Relau, Penang	Disturbed forest and semi-urban	0.553	1,830	911.09	Wan Ruslan, 1995
Sg. Relau, Penang	Rapidly urbanising, quarrying, construction	11.523	1,830	911.09	Wan Ruslan, 1995
Sg. Jinjang (1), Kuala Lumpur	Newly urbanising	10.3	2,400	1,056	Balamurugan, 1991
Sg. Jinjang (2), Kuala Lumpur	Tin mining and urbanising	27.1	2,300	2,283	Balamurugan, 1991
Sg. Kelang (1), Kuala Lumpur	Newly urbanising	14.2	2,400	1,480	Balamurugan, 1991
Sg. Kelang (2), Kuala Lumpur	Newly urbanising and mature urban	29.0	2,300	1,372	Balamurugan, 1991
Sg. Keroh, Kuala Lumpur	Urban and industrial	35.9	2,200	1,759	Balamurugan, 1991
Sg. Batu, Kuala Lumpur	*Forest and urban*	145	2,400	1,265	Balamurugan, 1991
Mengkuang Heights, Kuala Lumpur	Bare, steep construction site	0.21	2,400	330,821	Mykura, 1989
Sg. Sering, Kuala Lumpur	27% bare construction site	6.50	2,400	42,076	Mykura, 1989
Sg. Gombak, Jln Pekeliling, Kuala Lumpur	Forest and urban	140	2,400	1,157	Douglas, 1978

The above table reveals that the natural reserved area yields 75 to 380 ton/km^2 of annual sediment runoff as shown in the case of Sg. Air Hitam in Penang. On the other hand, the annual sediment runoff from the land development area increases to more than 900 $ton/km^2/year$. Mengkuang Height and Sg. Sering in the above table in particular yield a remarkably large sediment runoff of 42,000 to 330,000 $ton/km^2/year$. Deeply weathered rocks, particularly granite, dominate the geological conditions of these two areas where cutting slope causes numerous gully erosions that lead to a large amount of sediment runoff.

As stated before, the sediment runoff volume varies according to the land use in the river basin and the geological conditions (i.e., soil erodibility). Moreover, the sediment runoff volume also could be varied by rainfall intensity, slope length, slope gradient, and erosion control practice. Thus, the sediment runoff volume varies according to various factors, while the gauging record on the actual sediment runoff volume is limited. Under such conditions, the "Urban Drainage

Design Standards and Procedures for Peninsular Malaysia, DID, 1975" specifies 67 cu. yard per acre as the standard sediment runoff volume from land development area for a period of 18 months. This standard volume corresponds to 8,440 ton/km^2/year and is larger than the actual sediment runoff volume of 900 to 2,300 ton/km^2/year gauged in the land development area. In due consideration of variable unknown factors of the study area, however, the standard value could be provisionally applied as the design purpose for sediment control facilities for the following areas:

(a) Areas where intensive land development is made but no erosion control practice is made; and

(b) Areas where geological condition is fare and not dominated by deeply weathered rocks.

If the natural conditions of the area are still preserved without intensive land development, or if erosion control practice is made, the actual sediment runoff volume could be less than 1,000 ton/km^2/year. On the contrary, if land development is induced on an area dominated by deeply weathered rocks, the sediment runoff volume could be far larger than the standard volume of 8,440 ton/km^2/year. This will also vary according to the particularities of the weathered rocks.