2.2 Channel Flow Model

As described in the foregoing sub-section, 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 I-6.

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 to be 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 the 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, as the present flood inundation land is going to be converted to the highly value-added land, the 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 estimated the probable channel flow discharge which will occur when the river channel will not cause any channel over flow. In another word, 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 is, however, subject to the rather large river channel where the channel width is more than 10 time 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. 5) and the storage equation (Eq. 6)

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

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

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

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

The parameters of "K" and "P" are expressed by 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;

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

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, the channel flood travelling time was assumed to each of the subject channel with using the following "Kraven Formula".

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

Table I-6 shows the above parameters of "K" and "P" for the "Storage Function Model as well as "T" estimated from "Kraven Formula".

2.3 Results of Simulation

Fig. I-8 presents the entire simulation diagrams for each river which integrates the foregoing basin run-off simulation and channel flow simulation. Through the diagram, simulated are the probable basin-runoff discharge for each sub-basin as well as the probable the probable channel flow discharge for each divided channel (refer to Tables I-7 to I-8). 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 premises of the present land use sates and the projected land use states as 2020. Present and projected land use states are described in Sector II of Vol. 3.

The study area is now being rapidly urbanized, which has a significant influence on the basin runoff conditions. Due to rapid urbanization, the probable discharge in the year of 2020 is simulated to remarkably increase as compared with the present probable discharge (refer to Fig. I-9). The probable discharge of 5-year return period at the down-most point of each river basin are enumerated as below in order to simply clarify the difference between the present and future probable discharge. As listed below, the remarkable increment of probable discharge is estimated in Lalang and Che Bima in Sugai Petani, and Malim and the upper reaches of diversion weir of Melaka river (i.e., the Cheng/Melaka 1 in the following list) in Melaka.

Study Area of Sungai Petani		
Name of River	Probable Discharge (m3/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. ANALYSIS ON HYDROLOGICAL EFFECT OF FLOOD DETENTION FACILITY

3.1 Types of Flood Detention Facility

The basin flood detention facility is classified into storage type and infiltration type. Among these, the infiltration type is judged to be not applicable due to the poor infiltration capacity of the surface soil in the study area to as described in Sector III of Vol. 3. Accordingly, the analysis was made to the storage types of flood detention facility which cover three kinds of facilities: (1) storage in house lot, (2) storage in public open space and (3) flood detention pond. These three (3) types of facilities contain a potential to contribute to the drainage improvement for the following land use categories (refer to Sector IV of Vol. 3).

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 divided into the off-site and on-site storage types; the off-site storage type could control runoff discharge from a rather extensive catchment area, while the on-site storage type collects the 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 to the on-site storage type.

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 I-9. These dimensions are incorporated into the basin runoff simulation model as the basic data for comparative study on the alternative drainage improvement plans. The several assumptions for the estimation are as described hereinafter:

(1) Storage in House Lot

It is assumed that an 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 $2m^3$ ($2m^2$ in extent by 1m in height) in due consideration of an average available open space of house lot. The water tank has a outlet hole at side bottom. The size of the outlet hole was estimated on the premises 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 Sector IV of Vol.3. Based on these assumptions, trial simulation was made through the following equations, and the size of outlet hole was estimated at 4 cm in width and 3 cm in height.

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

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

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

Where;

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 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 to be about 20% of the entire public compound. The maximum storage depth and storage time length are also assumed to be 30cm and 10hours, respectively. This assumption was made in due consideration of original purpose of storage space as public utility. The depth of the surrounding drain is assumed to be 50cm. One outlet is placed at the bottom of the drain. The size of the outlet was estimate on the premises 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. 9 to 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 flood detention pond could be far larger than other two types of detention facility and, its regulation effect is proposed on the basis of the following criteria:

(a) The flood detention pond will regulate the basin runoff discharge from the new land development area, and control the increment of peak runoff discharge to be 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 periods. 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 Sector IV of Vol. 3.

(c) In accordance with the Guideline for Detention Pond prepared by 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.

In order to perform the above criteria, the flood detention pond is assumed to have the two outlets as illustrated in Table I-9. The probable flood of less than 5-year return period is discharged through only lower outlet, while the probable flood of 5 to 100-year return period is discharged through both of two outlets.

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

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 Flood Runoff Simulation of Effects of Basin Flood Detention Facility

In order to incorporate the effect of basin detention facilities into the foregoing basin runoff simulation model and channel flow simulation model, the sub-basin used in the basin runoff simulation model was further divided into direct and indirect runoff area as shown in Fig. I-10. The runoff discharge from the direct runoff area is not filtered by any detention facilities, 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: $V = Vunit \cdot A / Aunit$

Dimension of Outlet : $B = Bunit \cdot A / Aunit$

Where; V : Storage volume incorporated to the basin runoff simulation

B : Width of outlet hole

Vunit : Standard storage volume as estimated in Subsection 3.2.

Bunit : Standard width of outlet as estimated in Subsection 3.2.

Aunit : Standard catchment area as estimated in Subsection 3.2.

A : Catchment area in the basin runoff simulation model

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

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 detention facility. This estimation was made with assumptions of various catchment area of detention facilities extracted from the alternative plan for drainage improvement. Details of the estimation are as described in Sector IV of Vol. 3.

4. HYDRAULIC CHANNEL FLOW ANALYSIS

The flow capacity of the drainage channels and river channels was described in Sector IV and V of Vol. 3, respectively. In this connection, the estimation bases for the channel flow capacities are described in this Section.

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

The non-uniform calculation method was applied to the river channels and/or drainage channels, which has 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 111km. On the other hand, the uniform calculation method was applied to the small drainage channels of about 71km in total where the backwater effect is judged to be marginal due to 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 with referring to "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

The most river channels and drainage channels examined in this study are still remained as the natural condition without any channel improvement works. These river channels takes the above Manning's coefficient of 0.035.

However, out of the river channels examined, channel improvement has been completed for the downstream of Sg. Malim river with 5.7km in length (i.e., bypass channel) and the upstream of Sg. Melaka from diversion point with 7.1 km in length. As for these newly constructed bypass and improved river channels, the above

Manning's coefficient of 0.023 were applied. The channel improvement is also going to be implemented to the downstream of Sg. Ayer Salak, a tributary of Sg. Malim with a improvement length of 5.5km. Since this channel improvement is going to start in the near future, the designed channel cross-sections are regarded as the present channel states, and the Manning's coefficient of 0.023 was also applied.

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

(2) Boundary Condition

The followings were applied as the initial river stage for the 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 I-10). The tidal levels were derived from the tide tables published by the Royal Malaysian Navy and Department of Survey and Mapping, Malaysia in 1999. The tidal stations for the river channels in Sungai Petani and Melaka is Tanjung Dawai, and Kela Melaka, respectively.

(b) Channels Flowing to Mainstream Channel

The following river channels join to the major river stream having a significant backwater effect from the mainstream; (1) Sg. Cheng and Sg. Putat joining to Sg. Melaka, (2) Sg. Ayer Salak and Sg. Ayer Hitam joining to Sg. Malim. As for the non-uniform calculation for these river channels, the bank level of the mainstream at the confluence point was assumed to be as 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 the flood time. The diversion discharge is dominated by the hydraulic characteristic 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 (refer to Fig. I-12). The rating curve was presented in "Melaka Flood Mitigation Project, Designer's Operation Criteria" prepared by DID in 1990.

5. SEDIMENT RUN-OFF ANALYSIS

The recent intensive land development tends to cause serious soil erosion within the area and bring sedimentation to the lower reaches. Due to the sedimentation, 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 run-off are as listed 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 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 yields a remarkably large sediment run-off of 42,000 to 330,000 $ton/km^2/year$. The geological conditions of these two areas are dominated by deeply weathered rocks, particularly granite, where cutting slope causes numerous gully erosions, leading to a large amount of sediment run-off.

As stated above, 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 could be also 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 $yard^3$ /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 aforesaid actual sediment runoff volume of 900 to 2,300 $ton/km^2/year$ gauged in 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) If the intensive land development is made but any erosion control practice is not made to the area; and

(b) If the geological condition in the area is fare and not dominated by the deeply weathered rocks.

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