VOLUME 4 - SECTOR I

HYDROLOGY

THE STUDY ON INTEGRATED URBAN DRAINAGE IMPROVEMENT FOR MELAKA AND SUNGAI PETANI IN MALAYSIA

FINAL REPORT

VOLUME 4: SUPPORTING REPORT ON FEASIBILITY STUDY

SECTOR I: HYDROLOGY

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SECTOR I

HYDROLOGY

1. RAINFALL ANALYSIS

A large scale of storm flood occurred, on September 03 to 05, 1999, in the objective study area of Sg. Petani as well as other areas in the northeastern part of Peninsular Malaysia. The Study for the Drainage Structure Plan in Vol. 3 had ended in July 1999, and therefore could not incorporate the 1999 flood into its hydrological analysis. The rainfall record at Bayan Lepas Meteorological Station in Penang Island is used as the base of hydrological analysis for Sg. Petani. According to the record, the total rainfall depth of the 1999 flood is 288mm, out of which about 94% concentrated within 12hours on September 04, 1999. The 12-hour rainfall intensity in the1999 flood is the highest among the annual maximums recorded since 1948, and could possibly effect the design hyetograph. From this viewpoint, a supplementary rainfall analysis on the 1999 flood was made to re-evaluate the design hyetograph developed in the hydrological study on the Drainage Structure Plan.

Table I-1 presents a list of annual maximum rainfall intensities for various rainfall duration recorded from 1951 up to 1999. As shown in the Table, the intensities of less than 30-minute duration in the 1999 flood is the lowest in the list. However, the one-hour rainfall intensity of the 1999 flood ranks the middle, and those of 2 to 12 hour duration rises to the top in the list. Thus, the rainfall intensity of the 1999 flood has the lower value for shorter time duration, but the extremely high value for longer duration.

Fig. I-1 show two probable rainfall intensity-duration curves developed by Talbot formula. One is developed excluding the rainfall record of the 1999 floods from a population of the annual maximum rainfall intensities as estimation base. On the other hand, another is subject to inclusion of the 1999 flood into the population. As shown in the comparison of the rainfall intensity-duration listed on Table I-2, the rainfall intensities for shorter time duration tend to reduce but those for longer duration increase if the rainfall data of the 1999 flood is included.

2. BASIN FLOOD RUN-OFF ANALYSIS

2.1 Revision of Estimation Base for Basin Flood Run-off Discharge

The basin flood runoff simulation was made in the Study for Drainage Structure Plan in order to express the flood runoff discharge generated from the design hyetograph in the objective drainage basins. The design hyetograph is developed from the rainfall intensity-duration curve; the detailed procedures for development are as described in the Interim Report. A revision of the rainfall-intensity curve is, however, possibly required due to the aforesaid occurrence of the 1999 flood.

Moreover, the basin flood runoff simulation in the Study for Drainage Structure Plan was based on the topographic map of 1 to 50,000, which has a contour interval of 20m causing difficulties in clarifying detailed sub-basin boundaries particularly in the flat low-lying area. In order to cope with the difficulties, the scope of works for this Feasibility Study included the development of topographic maps covering the four (4) objective study areas with a scale of 1 to 2000 and a counter interval of 0.5m. The supplementary channel survey for four (4) trunk drains of 7,615 m in total and field reconnaissance was also made to scrutinize land use conditions, and flow directions of the existing drainage networks. Based on these supplementary works, the following items were clarified and reflected to the flood runoff analysis for the study areas:

(1) Design Hyetograph

An attempt was made to clarify the difference of flood runoff discharge hydrographs estimated from the above two kinds of rainfall intensity-curves with and without inclusion of the 1999 flood. The flood runoff discharge is assumed to be from a residential area of $1km^2$. As the results, it is clarified that there is no significant difference in the peak flood runoff discharges estimated from the above two kinds of design hyetographs as listed below. However, the flood runoff volume could have a significant increment of about 10% if the design hyetograph is subject to inclusion of the rainfall record of the1999 flood.

Recurrence Period of Flood	With and without the 1999 Flood	Runoff Volume ($10^6m^3/km^2$)	Peak Discharge ($m^3/s/km^2$)
5-year	without 1999 flood	1.16	32.7
	with 1999 flood	1.25	32.3
100-year	without 1999 flood	1.83	47.3
	with 1999 flood	2.04	46.0

In this study, the flood detention facility is highlighted as one of the useful drainage improvement measures, and the flood runoff volume is one of the essential factors to determine the structural size of the facility. From this viewpoint, it is decided that the design hyetograph to be applied for this Feasibility Study is subject to inclusion of the rainfall data of the 1999 flood into the estimation base.

(2) Basin Boundary

Through the aforesaid aerial photographic survey, the channel survey and the field reconnaissance, the entire basin boundary for each of the four (4) objective study areas were delineated in Sector IV of Vol.4. In accordance with the basin boundaries delineated, the total catchment are of each basin estimated in the Drainage Structure Plan is revised as below:

Name of Basin	Catchment Area Estimated (km^2)	
	in Phase 1	in Phase 2
(1) Sg. Air Mendidih in Sg. Petani	3.40	3.62
(2) Line G in Sg. Petani	3.19	3.00*
(3) Pokok Mangga in Melaka	3.71	4.71
(4) Area of Sg. Ayer Salak in Melaka	16.68	20.81

*: Out of $3.00km^2$, $0.27km^2$ is not within the present catchment area, but could be incorporated into the catchment area by slight change of the existing drainage channel alignment.

(3) Land Use

A minor revisions on these land uses in the Study on the Drainage Structure Plan were made through the detailed field reconnaissance, and the extent of each land use classification in each sub-basin was recalculated in accordance with the above revision of the basin boundaries. Detailed approaches and results of these land use study are as described in Sector II of this Vol. 4. The land use could have a significant influence to the basin runoff conditions, and the revised land use is incorporated to this basin runoff simulation.

2.2 Results of Basin Run-off Simulation

The basin runoff simulation was provisionally made for the objective four (4) study areas. The simulation is by the "Quasi-Liner Runoff Model", details of which are as described in the foregoing Sector I of Vol. 3. The simulation model could well express the increment of runoff discharge effected by land development. Moreover, the simulation model has a function to express non-liner characteristics of flood flow over a sloping area. Because of the function, the model could generate not only peak discharge but also the flood hydrograph.

In order to facilitate the simulation, the entire drainage area is divided into several sub-basins as shown in Fig. I-2. The division is based on the topographic conditions, where a particular attention was given to the major roads across the trunk drains as the division boundary. The present and project land use classification is also incorporated to the sub-basin boundary.

As the results of simulation the probable flood runoff discharges of 5-year return period were estimated as shown in Table I-3. The simulation is subject to the present and projected land use conditions in 2020. As described in Sector II of Vol. 4, the objective study areas are projected to have the extensive land development, which has a significant influence on the basin runoff conditions. Due to the land development, the probable discharge in the year of 2020 is simulated to remarkably increase as compared with the present probable discharge.

The above basin-runoff simulation is made on the premises of no proposed flood detention facility in the objective drainage basin, and the further simulation will be made assuming the detention facilities proposed in the alternative drainage improvement plans; the layout of the alternative plans are as described in Sector IV of Vol. 4. Moreover, the objectives of the simulation will be extended to other two study area of Pokok Mangga and Sg. Ayer Salak in Melaka.

3. CHANNEL FLOW AND FLOOD INUNDATION ANALYSIS

3.1 Purpose of Analysis

The present habitual flood inundation area is going to be converted to the highly value-added land, and the drainage channel improvement has to be made to accommodate the design flood discharge and prevent it from overflow. From this point of view, the hydrological study in the Drainage Structure Plan made the channel flow simulation to estimate the probable channel flow discharge on the premises that the channel improvement is made and the river channel will not cause any channel overflow (refer to Sector I of Vol. 3). In another word, the simulated probable channel flow discharge is regarded as the design channel flow capacity and far different from the present actual channel flow discharge.

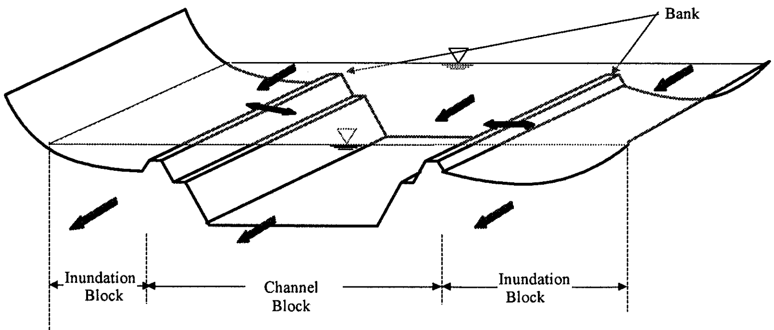
As described above, however, the flood discharge presently often overflows from the trunk drains and stagnates in the low-lying area. As the results, the flood discharge is hardly propagated from upstream to downstream, and the peak flow discharge of the drains is reduced in appearance. Under such conditions, the hydrological study for this Feasibility Study simulates the following flood inundation and channel flow conditions based on the detailed topographic maps newly developed.

(1) Present probable flood inundation extent, inundation depth and inundation duration which could be used as the essential information for evaluation on the effect of the alternative drainage plans, and proposal on the definitive location and structural size of the alternative flood detention facilities and the drainage networks.

(2) Probable channel flow discharge for various alternative drainage improvement plans, which could be used as the design channel flow capacity required to each alternative.

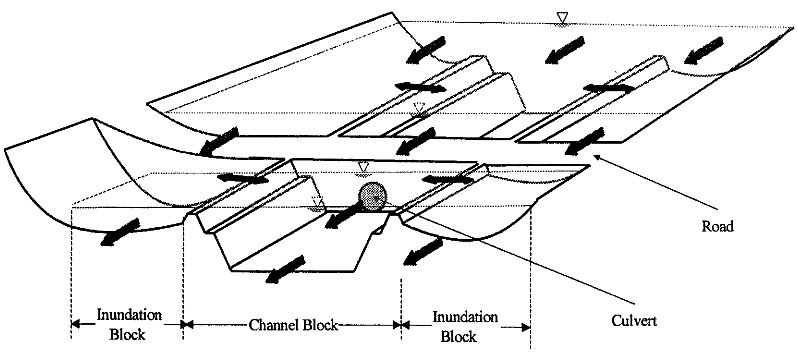
3.2 Simulation Model

Most of the existing trunk drains have the extremely small channel flow capacities which could not cope with even the probable peak flood discharge of 2-year return flood. as described in Sector V of Vol. 4. The bank levels of the trunk drains are almost same or slightly higher that the hinterland ground level, and the flood tends to gradually overflow from the trunk drains and extensively inundates the flat low-lying hinterland. The flood flow will have various flow directions as illustrated in the right conceptual figure.



As illustrated, the flood discharge in the channel will have the flow direction toward downstream, and at the same time toward the inundation area in the hinterland of the channel when the channel water level is higher than the inundation level. The flood discharge in the inundation area also flows toward lower ground level, and a part of the discharge return to channel when the inundation water depth is higher than the channel water level. .

The flood inundation is further accelerated by the roads across the drains. The culvert is often used to pass the road over the drain but its size is far smaller than that of the upstream open channels. As the results, the culvert greatly hinders the channel flow and the flood inundation occurs just upstream of the road. The road is constructed on the embankment that is higher than the ground level along the road. Accordingly, once the flood inundation occurs, the road dams up the flood water increasing the inundation depth. The flood will have various flow directions between the drainage channels and inundation area. At



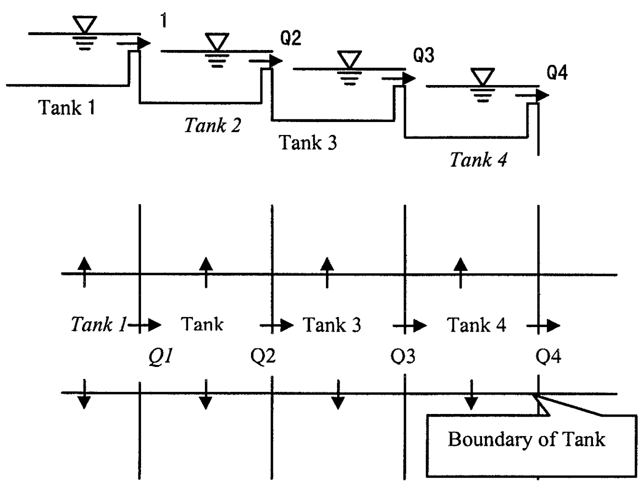
the same time, the floodwater dammed up by the road will have a flow direction toward downstream either over the road or through the culvert as illustrated.

In order to express the above flood channel flow and inundation conditions, the trunk drain and its surrounding possible flood inundation area are divided into the following blocks:

(1) Channel Block : The drainage channel is longitudinally divided into several blocks in accordance with the foregoing sub-basin boundary (refer to Fig. I-3).

(2) Inundation Block : The flat low-lying areas along the trunk drains are assumed as the possible maximum flood inundation area. The determination on boundary of the possible maximum flood inundation is based on the topographic map and the detailed field reconnaissance. The possible maximum flood inundation area is further divided into several blocks in due consideration of the above division of the channel blocks, the confluence of branch drains, and the roads across the trunk drains.

The simulation model assumes both of the channel block and the inundation block as a tank which connects length-wise and breath-wise each other as shown in the right illustration. The model estimates the movement of discharge from a tank to its next tank based on the water level estimated at the preceding time (t – Δ t). Then, the storage volume as well as its corresponding water level is estimated based on the movement of discharge. Thus, the simulation is made, one after another, on the movement of discharge from a tank to the next tank and the water level at the tank.



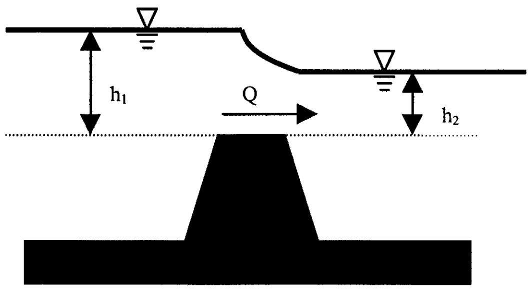
In the simulation, particular attention was given to the calculation time interval. (Δ t). As the calculation time interval is made shorter, the more precise simulation results are given but the calculation volume drastically increases. In due consideration of the computer capacity, this simulation model takes one second as the calculation time interval. The movement volume of discharge from a tank to next tank within the calculation time interval (Δ t) is controlled by the equation of motion, while the storage volume of the tank is by the equation of continuity. In this simulation, the following equations of motion and equations of the continuity are applied:

(1) Equation of Motion

The following equations of motion control the movement depending on the topographic features of the boundaries between the two tanks.

Boundary of Embankment

When the embankment such as road and dike forms the boundary, the discharge flows over boundary, and its movement volume within a unit calculation time (Δ t) is decided by the following formula for overflow discharge.



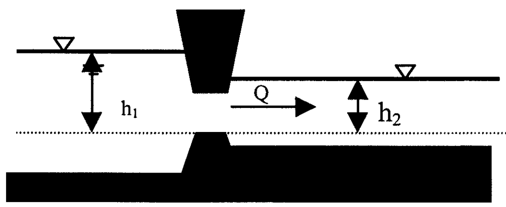
$Q = C{\cdot}B{\cdot}h_1{\cdot}(2gh_1)^{1/2}$ ($h_2/h_1 \leq 2/3$) .... (Eq. 3.1)

$Q = C{\cdot}B{\cdot}h_2{\cdot}\{2g(h_1 - h_2)\}^{1/2}$ ($h_2/h_1 > 2/3$) .... (Eq. 3.2)

Where Q : Overflow discharge
C : Coefficient (assumed as 0.9)
B : Width of the embankment (overflow width)
h1, h2 : The water level of blocks

Boundary of Culvert

When the flow passes the culvert from a tank to next tank, the movement discharge is decided through the following formula as the orifice discharge.



$Q = -C{\cdot}A{\cdot}\{2g(h_1 - h_2)\}^{1/2}$ ($h_1 > h_2$).. (Eq. 3.3)

$Q = +C{\cdot}A{\cdot}\{2g(h_2 - h_1)\}^{1/2}$ ($h_1 \leq h_2$).. (Eq. 3.4)

Where Q : Flow discharge through the culvert
C : Coefficient
A : Cross-sectional area of the culvert
h_1, h_2 : The water levels of the upstream block and downstream block

Other Boundaries

When the boundary does not form embankment nor has culvert, the movement of discharge from tank to the next tank is assumed as the unsteady flow and expressed by the following formula of unsteady flow:

$\delta H/\delta x + n \cdot v \cdot |v| / R^{2/3} + 1/g \cdot \delta v/\delta t + 1/2g \cdot \delta v^2/\delta x = 0$ .... (Eq. 3.5)

$Q = A \cdot v$ .... (Eq. 3.6)

Where H : Water level
n : Coefficient of roughness
v : Flow velocity
R : Hydraulic radius
A : Cross-sectional area of flow

(2) Equation of Continuity

The following equations are given to the inundation block and the channel bloc respectively:

Inundation Block

The storage volume is estimated by the following equation, and its corresponding water level is estimated through the relation curve of the water level - the storage volume. The relation curve is developed from the topographic map newly surveyed in this study period.

$\Sigma Qin - \Sigma Qout = dV/dt$ .... (Eq. 3.7)

Where Qin : Inflow discharge to the block
Qout : Outflow discharge from the block
V : Flood storage volume in the tank

Channel Block

The channel flow area is varied by the channel flow discharge, and therefore, the following equation is applied:

$\delta A/\delta t + \delta Q/\delta x = -q$ .... (Eq. 3.8)

Where A : Cross-sectional area of flow
Q : Channel flow discharge
q : Inflow discharge from the inundation block to the channel block

3.3 Results of Channel Flow and Flood Inundation Simulation

A simulation model for the drainage area of Sg. Air Mendidih was developed as shown in Fig. I-4, where the reference points for the simulated channel peak discharge is arranged as shown in Fig. I-3. The model simulates the movement of flood discharge through the following procedures: (1) basin flood runoff discharge as channel inflow discharge → (2) channel flow/over flow discharge → (3) movement of flood discharge in the inundation area. Among them, the basin flood runoff discharge is estimated through the basin runoff simulation model.

As the results, it is clarified that the probable flood runoff discharge of 5-year return period could cause the following extent of the flood inundation (refer to Table I-4 and Fig. I-5).

Range of Inundation Depth	Extent of Inundation Area for Probable Flood Runoff of 5-year Return Period	
	Under Present Land Use	Under Projected Land Use in 2020
0 – 1m	24.96 ha	33.43 ha
1-2 m	11.97 ha	19.26 ha
Over 2m	1.23 ha	4.97 ha
Total	38.16 ha	57.66 ha

As the land development progresses, the peak flood off discharge tends to increase as shown in Table I-3 and, the extent of flood inundation area will have a drastic increment as listed above. Due to the flood inundation, the channel flow discharge is hardly propagated from the upstream to the downstream, and the peak discharge is reduced in appearance. Should the drainage channel improvement be implemented to eliminate the flood inundation, the flood runoff discharge concentrates into the channels and increases the peak channel flow discharge as shown in Table I-5.

The simulation model could estimate the present actual channel flow and flood inundation. Moreover, the model could also estimate the hypothetical channel flow brought about by any optional flood runoff scale under the different land use and/or the channel condition. The simulation is, however, is still in progress, and will be further extended to the objective study areas other than Sg. Air Mendidih and to various flood scales. The results of simulation will be used as the basic conditions for the following items:

(1) Calibration of Coefficients Applied to the Simulation Model

Hydrological observation on the actual rainfall intensity and its corresponding flood runoff discharge has been made by DID in the catchment area of Sg. Air Mendidih. The observation aims at calibrating the coefficients applied to the foregoing basin runoff simulation model and the channel flow/inundation simulation model. The available observed data for calibration has not been attained yet. The calibration will be made whenever the observed data is made available during a period of the following home office study from January to March 2000.

(2) Cost Comparison on Alternative Drainage Improvement Plans

The Study proposes alternative drainage plans which composed of various combinations of drainage channel improvement and flood detention schemes (refer to Sector VI of Vol. 4). The simulation model could estimate the design flow capacity for the channel improvement as well as the necessary storage capacity of the flood detention facilities proposed in the alternative plans. The results of estimation could be used as the basic information for cost comparison of alternative plans.

(3) Economic Evaluation on the Optimum Drainage Improvement Plan

The optimum drainage plan selected among the alternatives will improve the present drainage conditions in the objective study areas and reduce the extent of flood inundation. The economic evaluation on the optimum plan is based on the comparison of the extent of flood inundation which could occur before and after the optimum drainage plan is implemented. In this connection, the simulation model is useful on the quantitative analysis on the extent of flood inundation. The simulation for probable flood inundation of various recurrence probability will be further made in the following study period.

TABLES

Table I-1 (1/2) Annual Maximum Rainfall Intensities Observed at Bayan Lepas, Penang (for 15 Minutes to 1 Hour-Duration)

Ranking No.	15 min.-Duration		30 min.-Duration		1 hr.-Duration	
	Year	Rainfall Intensity (mm/hour)	Year	Rainfall Intensity (mm/hour)	Year	Rainfall Intensity (mm/hour)
1	1988	214.4	1988	157.0	1969	113.8
2	1982	180.0	1982	136.8	1996	103.2
3	1976	172.0	1978	132.0	1985	102.0
4	1990	170.0	1996	130.0	1958	98.8
5	1963	167.6	1991	128.6	1994	97.7
6	1966	167.6	1966	122.0	1951	97.5
7	1993	157.6	1993	121.2	1978	95.0
8	1978	154.0	1969	119.4	1988	91.3
9	1977	152.8	1985	117.2	1981	84.5
10	1973	147.2	1973	114.2	1973	83.3
11	1996	146.4	1979	112.0	1993	81.6
12	1991	144.8	1986	111.8	1991	81.6
13	1975	144.0	1990	111.6	1983	81.2
14	1953	142.4	1994	110.4	1986	81.1
15	1985	141.2	1977	109.6	1997	80.1
16	1951	137.2	1975	108.4	**1999**	**79.0**
17	1997	136.8	1976	108.0	1972	78.7
18	1959	135.2	1964	106.6	1982	78.1
19	1964	132.0	1971	106.6	1989	77.5
20	1965	132.0	1965	105.6	1977	76.3
21	1969	132.0	1958	104.2	1971	75.2
22	1986	130.0	1963	104.2	1965	74.2
23	1984	128.8	1967	104.2	1992	72.9
24	1981	128.0	1992	103.6	1976	72.5
25	1967	124.8	1997	103.6	1967	72.4
26	1980	124.4	1951	102.2	1964	70.9
27	1979	124.0	1953	102.2	1962	70.6
28	1994	124.0	1989	101.2	1952	69.1
29	1955	122.0	1983	100.6	1970	68.6
30	1995	121.6	1981	100.0	1975	67.7
31	1987	120.0	1980	99.2	1990	67.6
32	1983	118.0	1959	98.6	1966	66.8
33	1968	116.8	1987	97.8	1995	66.5
34	1971	116.8	1972	96.6	1980	66.4
35	1972	116.8	1974	95.6	1955	66.3
36	1952	116.0	1952	95.0	1968	66.0
37	1992	114.4	1970	94.4	1979	63.2
38	1956	113.6	1984	94.4	1956	61.0
39	1962	113.6	1961	94.0	1961	60.5
40	1960	111.6	1968	94.0	1959	59.7
41	1974	111.6	1962	93.4	1963	59.4
42	1970	106.8	1995	92.8	1987	58.9
43	1958	104.8	**1999**	**91.4**	1957	58.9
44	1957	101.6	1956	89.4	1984	58.6
45	1961	101.6	1957	87.8	1953	55.9
46	**1999**	**101.6**	1960	85.4	1974	55.1
47	1954	93.6	1955	81.8	1960	52.6
48			1954	58.0	1954	38.6
Ave.		132.2		105.9		74.9

Table I-1 (2/2) Annual Maximum Rainfall Intensities Observed at Bayan Lepas, Penang
(for 2 to 12 Hour-Duration)

Ranking No.	2 hr.-Duration		3 hr.-Duration		6 hr.-Duration		12 hr.-Duration	
	Year	Rainfall Intensity (mm/hour)	Year	Rainfall Intensity (mm/hour)	Year	Rainfall Intensity (mm/hour)	Year	Rainfall Intensity (mm/hour)
1	1985	76.4	**1999**	**60.1**	**1999**	**43.8**	**1999**	**22.5**
2	1969	70.4	1958	52.7	1958	27.9	1958	19.1
3	**1999**	**68.1**	1985	51.2	1969	26.9	1985	16.1
4	1996	64.8	1969	50.0	1985	25.6	1964	15.2
5	1958	62.6	1996	44.6	1991	24.5	1969	13.7
6	1951	59.8	1951	41.1	1996	22.9	1991	13.5
7	1994	59.5	1994	40.8	1957	22.0	1995	11.8
8	1993	58.6	1983	39.8	1951	21.4	1966	11.8
9	1983	57.4	1993	39.8	1962	21.2	1962	11.7
10	1989	56.2	1955	39.5	1989	21.0	1957	11.5
11	1955	54.6	1989	39.5	1994	20.6	1971	11.5
12	1978	54.0	1962	39.4	1978	20.2	1996	11.4
13	1962	53.9	1978	36.7	1970	20.1	1961	11.2
14	1986	50.2	1957	36.7	1964	20.1	1989	11.1
15	1988	50.0	1988	36.0	1983	20.0	1951	10.7
16	1972	49.7	1972	34.4	1993	19.9	1970	10.3
17	1981	49.5	1952	33.8	1955	19.9	1994	10.3
18	1952	48.0	1986	33.6	1952	19.5	1952	10.3
19	1970	47.0	1981	33.5	1961	19.0	1978	10.1
20	1957	46.3	1973	33.4	1974	18.8	1982	10.0
21	1982	45.8	1970	31.8	1973	18.4	1983	10.0
22	1987	45.3	1982	31.4	1981	18.2	1993	10.0
23	1973	44.7	1966	31.1	1988	18.0	1955	10.0
24	1997	43.5	1995	30.9	1972	17.9	1974	9.5
25	1995	43.1	1971	30.8	1971	17.7	1981	9.4
26	1980	42.0	1987	30.5	1982	17.5	1973	9.3
27	1991	42.0	1975	29.4	1992	17.2	1984	9.3
28	1965	41.9	1991	29.2	1986	16.8	1972	9.0
29	1977	41.8	1997	29.2	1966	16.6	1988	9.0
30	1966	41.6	1977	28.9	1979	16.0	1959	9.0
31	1976	41.5	1964	28.8	1984	15.9	1992	8.7
32	1971	41.0	1992	28.7	1995	15.8	1953	8.4
33	1984	41.0	1965	28.5	1959	15.8	1986	8.4
34	1964	40.5	1967	28.0	1987	15.5	1979	8.1
35	1975	39.7	1984	28.0	1953	15.0	1977	7.9
36	1967	39.0	1980	28.0	1967	15.0	1990	7.8
37	1959	38.3	1976	27.8	1977	14.9	1987	7.7
38	1992	37.7	1959	25.7	1965	14.8	1954	7.6
39	1968	37.0	1974	25.7	1975	14.7	1975	7.6
40	1990	34.8	1968	24.7	1997	14.6	1965	7.5
41	1956	33.8	1953	24.1	1954	14.0	1967	7.5
42	1960	32.8	1990	23.6	1980	14.0	1980	7.4
43	1979	32.7	1956	23.3	1976	13.9	1997	7.3
44	1954	31.9	1960	22.9	1968	13.6	1968	7.1
45	1974	31.8	1979	22.6	1960	13.0	1976	6.9
46	1953	31.0	1954	21.8	1956	12.7	1960	6.9
47	1963	31.0	1963	21.5	1990	12.2	1963	6.9
48	1961	30.6	1961	21.4	1963	11.6	1956	6.8
Ave.		46.5		33.1		18.6		10.2

Table I-2 Comparison of Rainfall Intensities without and with Inclusion of Sep. 1999 Flood into Population of Annual Extreme Values at Bayan Lepas, Penang

Time Duration	5-year Return Period			100-year Return Period		
	Without	With	Increment	Without	With	Increment
(min.)	(mm/hr)	(mm/hr)	(%)	(mm/hr)	(mm/hr)	(%)
15	152	149	-1.9	210	204	-3.8
30	120	120	-0.1	170	171	0.3
45	100	101	0.8	143	147	2.3
60	85	87	1.2	124	129	3.3
120	54	56	1.4	80	86	4.1
180	39	41	1.3	59	65	3.8
240	31	33	1.1	47	52	3.4
300	25	27	1.0	39	43	3.0
360	22	23	0.9	33	37	2.7
720	11	12	0.5	18	20	1.6

Note:

$$\text{Increment} = \frac{\text{(Rainfall Intensity with 1999 Flood) - (Rainfall Intensity without 1999 Flood)}}{\text{Rainfall Intensity without 1999 Flood}} \times 100\%$$

Table I-3 (1/2) Probable Peak Basin Runoff Discharge
(for Sg. Air Mendidih and Line G in Sg. Petani)

(unit : m^3/s)

River Basin	Sub-basin No.	Total Area (ha)	Percentage of Buitup Area		Probable Peak Dischage (5 year return period)	
			Present	Year 2020	Present	Year 2020
Air Mendidieh	AM- 1	6.96	100.0%	100.0%	2.9	2.9
	AM- 2	5.36	100.0%	100.0%	1.8	1.8
	AM- 3	24.53	91.2%	95.1%	8.8	9.1
	AM- 4	8.87	100.0%	100.0%	3.3	3.3
	AM- 5	25.42	67.2%	100.0%	6.6	8.5
	AM- 6	13.69	67.9%	100.0%	3.7	4.8
	AM- 7	54.47	5.6%	100.0%	5.5	20.0
	AM- 8	17.69	31.1%	100.0%	3.3	6.4
	AM- 9	1.59	35.8%	100.0%	0.4	0.7
	AM- 10	9.58	53.8%	100.0%	2.4	3.9
	AM- 11	11.36	96.9%	100.0%	4.0	4.1
	AM- 12	14.35	40.4%	100.0%	2.8	5.2
	AM- 13	15.96	87.5%	100.0%	5.0	5.3
	AM- 14	30.46	93.8%	100.0%	11.0	11.5
	AM- 15	12.43	100.0%	100.0%	4.6	4.6
	AM- 16	19.73	92.6%	100.0%	7.2	7.6
	AM- 17	50.32	69.5%	100.0%	13.7	17.6
	AM- 18	10.93	67.0%	100.0%	3.1	4.3
	AM- 19	25.35	77.4%	100.0%	7.8	10.1
	AM- 20	3.38	73.7%	100.0%	1.1	1.5
	Total	**362.43**	**65.8%**	**99.7%**		
Line G	LG- 1	37.06	0.9%	100.0%	3.4	10.7
	LG- 2	32.65	2.3%	100.0%	3.1	11.8
	LG- 3	6.18	0.0%	98.0%	0.6	2.4
	LG- 4	69.59	83.0%	94.1%	19.5	21.3
	LG- 5	4.86	16.9%	96.3%	0.8	1.9
	LG- 6	17.70	72.0%	100.0%	4.6	5.7
	LG- 7	11.52	37.0%	100.0%	2.3	3.9
	LG- 8	4.61	8.1%	100.0%	0.6	1.8
	LG- 9	28.04	73.1%	98.0%	7.5	9.1
	LG- 10	20.49	12.7%	100.0%	2.7	7.2
	LG- 11	15.72	15.5%	100.0%	2.2	5.7
	LG- 12	7.05	34.2%	100.0%	1.4	2.6
	LG- 13	12.53	27.9%	100.0%	2.3	4.6
	LG- 14	4.79	71.8%	100.0%	1.5	1.9
	Total	**272.78**	**41.0%**	**98.2%**		

Table I-3 (2/2) Probable Peak Basin Runoff Discharge

(for Prt. Pokok Mangga and Sg. Ayer Salak in Melaka)

(unit : m^3/s)

River Basin	Sub-basin No.	Total Area (ha)	Percentage of Buitup Area		Probable Peak Dischage (5 year return period)	
			Present	Year 2020	Present	Year 2020
Pokok Mangga	L- 1	88.17	36.1%	100.0%	7.2	14.6
	L- 2	28.30	16.7%	100.0%	1.7	5.4
	L- 3	50.42	45.5%	100.0%	4.9	8.9
	L- 4	78.43	46.4%	98.8%	7.2	12.7
	L- 5	22.63	66.6%	100.0%	3.3	4.4
	P- 1	28.77	43.4%	98.4%	3.6	6.1
	P- 2	45.14	55.2%	98.0%	5.4	7.7
	P- 3	15.43	87.5%	100.0%	2.8	3.0
	P- 4	34.78	36.1%	100.0%	3.1	6.4
	P- 5	15.47	91.3%	100.0%	3.0	3.1
	P- 6	29.08	62.4%	100.0%	3.9	5.5
	P- 7	34.28	78.0%	100.0%	5.4	6.4
		470.90	**49.6%**	**99.5%**		
Ayer Salak	A1- 1	128.95	0.0%	100.0%	5.9	25.5
	A1- 2	133.04	2.4%	45.6%	6.5	15.9
	A1- 3	91.13	41.8%	93.0%	10.2	18.9
	A1- 4	12.14	57.8%	58.8%	2.0	2.1
	A2- 1	15.27	0.0%	100.0%	0.8	3.5
	A2- 2	82.86	88.6%	100.0%	15.7	17.1
	A2- 3	66.96	1.0%	98.4%	3.4	14.6
	A2- 4	56.02	33.8%	100.0%	6.1	12.9
	AS- 1	179.27	0.0%	93.4%	7.8	34.3
	AS- 2	134.63	1.5%	100.0%	5.7	26.8
	AS- 3	28.38	0.7%	100.0%	1.6	6.2
	AS- 4	46.95	18.2%	100.0%	3.9	10.3
	AS- 5	61.67	50.4%	100.0%	8.2	13.0
	AS- 6	34.93	21.6%	100.0%	3.2	7.6
	AS- 7	137.55	32.4%	81.9%	13.2	24.1
	AS- 8	177.77	44.8%	100.0%	19.4	34.9
	AS- 9	147.51	40.2%	100.0%	15.9	30.5
	AS- 10	108.35	66.2%	100.0%	16.7	22.9
	AS- 11	61.77	17.8%	52.5%	5.0	8.6
	AS- 12	15.83	0.1%	2.7%	1.0	1.0
	Total	**1,720.98**	**26.5%**	**90.3%**		

Table I-4 Probable Extent of Flood Inundation Area

(unit : ha)

Drainage Area	Existing Land Use		Proposed Land Use (Year 2020)	
	5 yr return Period	100 yr return period	5 yr return Period	100 yr return period
Sg.Air Mendidih	39.65	52.64	47.17	61.71
Line G	13.67	24.64	22.69	39.49
Prt.Pokok Mangga	268.40	406.46	319.92	473.81
Sg.Ayer Salak	340.07	395.61	426.50	469.38

Table I-5 Probable Peak Channel Flow Discharge without Overflow

(unit : m^3/s)

Drainage Area	Channel	CH No.	Calculation Point	5 year return period	
				Present	Year 2020
Sg. Air Mendidih	Sg. Air Mendidih	0.0	12	77.0	112.0
		200.0	11	69.0	102.0
		704.5	10	68.0	97.0
		934.4	9	59.0	85.0
		1150.0	8	59.0	85.0
		1400.0	7	31.0	47.0
		1700.0	6	29.0	43.0
		2200.0	5	26.0	28.0
		2500.0	4	20.0	19.0
		2800.0	3	15.0	15.0
		3000.0	2	3.0	3.0
		3200.0	1	0.0	0.0
	Line P	0.0	18	23.0	36.0
		402.3	17	20.0	34.0
		902.5	16	11.0	28.0
		1071.5	15	6.0	25.0
		1291.5	14	4.0	20.0
	Line O	0.0	20	12.0	16.0
		600.0	19	4.0	5.0
Line G	Line G	0.0	10	80.0	99.0
		200.0	9	65.0	91.0
		500.0	8	60.0	92.0
		900.0	7	51.0	74.0
		1480.0	6	34.0	57.0
		1680.0	5	30.0	54.0
		1789.0	4	25.0	45.0
		1889.0	3	12.0	27.0
		2189.0	2	9.0	21.0
		2789.0	1	4.0	11.0
Pokok Mangga	Pokok Mangga	0.0	4	37.0	52.0
		800.0	3	28.0	40.0
		1200.0	2	23.0	33.0
		2200.0	1	9.0	11.0
	M2	0.0	6	5.0	6.0
		200.0	5	3.0	3.0
	M1	0.0	8	8.0	14.0
		200.0	7	7.0	13.0
	Besar Limbongan	0.0	11	15.0	30.0
		1000.0	10	7.0	13.0
		1800.0	9	5.0	9.0
Ayer Salak	Ayer Salak	1400.0	7	82.0	213.0
		2200.0	6	68.0	170.0
		3400.0	5	57.0	170.0
		4800.0	4	51.0	163.0
		5693.0	3	28.0	110.0
		6293.0	2	19.0	88.0
		7293.0	1	8.0	44.0
	AB1	0.0	11	34.0	65.0
		400.0	10	19.0	38.0
		2200.0	9	11.0	22.0
	AB2	0.0	14	17.0	31.0
		1822.0	13	16.0	26.0
		2847.0	12	16.0	18.0