Table G4.2.2 Monthly Evapotranspiration Type

(mm)

No.	Type 1	Type 2	Type 3
Jan.	118	105	102
Feb.	118	105	102
Mar.	139	124	102
Apr.	124	110	97
May	121	109	97
Jun.	110	98	92
Jul.	118	105	101
Aug.	133	118	111
Sep.	129	115	108
Oct	139	124	108
Nov	131	117	104
Dec	128	114	106
Total	1,508	1,343	1,230

The classifications above and the isohyetal map of annual potential evapotranspiration were applied the typical monthly evapotranspiration in the sub-basins as boundary condition. The applied types are given in **Table G4.2.3**.

Table G4.2.3 Evapotranspiration Type Applied to Sub-Basins

Sub-Basin	Applied Type	Sub-Basin	Applied Type
Komering 1	1	Kelingi	3
Komering 2	2	Harileko	2
Ogan 1	3	Musi 1	1
Ogan 2	3	Musi 2	1
Lematan 1	1	Musi 3	3
Lematan 2	3	Musi 4	3
Semangus	3	Musi 5	3
Lakitan 1	1	Musi 6	3
Lakitan 2	2	Musi 7	2
Rawas 1	2	Musi 8	2
Rawas 2	2	Padang	2

(5) Parameters

(a) Explanation of Parameters

The Runoff Model (NAM Module) is based on physical structures and equations used together with semi-empirical ones. Therefore, some of the parameters can be evaluated from the physical catchment data. A brief description of primary parameters is given below.

(i) Maximum Water Content in Surface Storage (U_{max})

This represents the cumulative total water content of the interception storage (on vegetation), surface depression storage and storage in the uppermost layers (a few centimeters) of the soil. The values at the Musi River Basin are between 10 and 20 mm.

(ii) Maximum Water Content in Root Zone Storage (L_{max})

This represents the maximum soil moisture content in the root zone, which is available for transpiration by vegetation. The values at Musi River Basin are between 80 and 100 mm.

(iii) Overland-Flow Runoff Coefficient (CQOF)

This determines the division of excess rainfall between overland flow and infiltration. The values range between 0.5 and 0.85 at Musi River Basin.

(iv) Time Constant for Interflow (CKIF)

This determines the amount of interflow, which decreases at larger time constants. This parameter is set at 500 at Musi River Basin.

(v) Time Constants for Routing Overland Flow (CK1, CK2)

These determine the shape of hydrograph peaks. The routing takes place through two linear reservoirs (serial connected) with the same time constant (CK1=CK2). High, sharp peaks are simulated with small time constants, whereas low peaks, at a later time, are simulated with large values of these parameters. The values in the range of 5 to 100 hours are at Musi River Basin.

(vi) Root Zone Threshold Value for Overland Flow (TOF)

This determines the relative value of the moisture content in the root zone (L/L_{max}) above which overland flow is generated. The main impact of TOF is seen at the beginning of a wet season, where an increase of the parameter values will delay the start of runoff as overland flow. Threshold value range between 0 (zero) and 20 percent of L_{max}.

(vii) Root Zone Threshold Value for Interflow (TIF)

This determines the relative value of the moisture content in the root zone (L/L_{max}) above which interflow is generated.

(viii) Time Constant for Routing Base Flow (CKBF)

This can be determined from the hydrograph recession in dry periods. In rare cases, the shape of the measured recession changes to a slower recession after some time. To simulate this, a second groundwater reservoir may be included (see the extended components below). This parameter is set at 2,000 at Musi River Basin.

(ix) Root Zone Threshold Value for Ground Water Recharge (TG)

This determines the relative value of the moisture content in the root zone (L/L_{max}) above which groundwater recharge is generated. The main impact of increasing TG is less recharge to ground water storage. Threshold values range between 0 (Zero) and 70 percent of L_{max}. This parameter is set at 0 at Musi River Basin.

(b) Classification for Parameters

The NAM Module function as the rainfall-runoff model for each river basin. For the determination of temporary parameters in the NAM module in the Musi River Basin Study in 1989, runoff at sub-basins was classified into three types: Lowland, Mountain Region and the Area including both Mountain and Lowland, as shown in **Table G4.2.4**.

Table G4.2.4 Temporary Parameters for Hydrological Model

Parameters	Type 1 (Lowland)	Type 2 (Mountain and Lowland)	Type 3 (Mountain Region)
U_{max}	20	10	15
L_{max}	100	80	100
CQOF	0.85	0.7	0.5
CKIF	500	500	500
TOF	10	0	10
TIF	0	0	0
CKBF	2,000	2,000	2,000
Cqlow	0	0	0
Cklow	1	1	1
TG	0	0	0
CK1, CK2	100	18	5
Carea	1	1	1

To establish the simulation model in this study, these characteristics were made as reference to determine the classification of parameters.

Table G4.2.5 Catchment Delineation and Character of Sub-Basins

Sub-Basin	Characteristic of Basin	Selected Parameter Type	Sub-Basin	Characteristic of Basin	Selected Parameter Type
Komering 1	(B)	2	Kelingi	B	2
Komering 2	(L)	1	Harileko	L	1
Ogan 1	B	2	Musi 1	M	3
Ogan 2	L	1	Musi 2	M	3
Lematan 1	B	2	Musi 3	L	1
Lematan 2	L	1	Musi 4	L	1
Semangus	L	1	Musi 5	L	1
Lakitan 1	B	2	Musi 6	L	1
Lakitan 2	(L)	1	Musi 7	(L)	1
Rawas 1	B	2	Musi 8	(L)	1
Rawas 2	L	1	Padang	(L)	1

*1 Source: Musi River Basin Study, 1989
*2 L: Lowland, M: Mountain Region, B: Included both Mountain and Lowland, (L): Decided in this study

4.3 Model Calibration

In the Musi River Basin Study in 1989, the calibration was carried out at four stations: namely, Muara Rupit, Baturaja, Chaya Bumi and Sukaraya, of which locations are shown in **Annex G4.3.1**. The reason why these stations were selected is to be able to construct the natural water flow time series that is not affected by irrigation or other water use at these stations. The calibration period is between 1982 and 1986.

4.4 Simulation Result

The gaps in the time series of natural flow were filled in as a result of the simulation using the established model. The time series will thus be used for the water balance/use analysis in Sector I. Regime of Natural Flow resulting from simulation is summarized in **Table 4.4.1**.

Table G4.4.1 Natural Flow Regime Resulting from Simulation

(m^3/s)

No.	Sub-Basin	C.A. (km^2)	25%	50%	75%	95%	Ave. (Wet)	Ave. (Dry)	Annual Ave.	
									m^3/s	$m^3/s/100km^2$
1	KO1	4,527	72.8	116.5	188.2	305.8	308.3	163.9	235.9	5.2
2	KO1+KO2	9,908	144.4	229.3	390.5	613.4	608.8	283.1	445.6	4.5
3	OG1	3,990	35.3	58.0	101.6	194.9	193.1	71.9	132.4	3.3
4	OG1+OG2	8,222	73.2	116.4	217.9	388.5	389.7	141.0	265.1	3.2
5	LE1	3,930	61.5	87.3	118.1	201.1	223.4	131.2	177.2	4.5
6	LE1+LE2	7,340	103.1	148.0	231.6	376.1	396.5	197.9	297.0	4.0
7	SE	2,146	19.9	32.6	50.9	72.6	71.9	40.1	56.0	2.6
8	LA1	2,290	23.0	37.8	57.1	88.5	91.8	47.8	69.8	3.0
9	LA1+LA2	2,763	28.1	45.9	69.7	106.9	109.8	57.6	83.6	3.0
10	RA1	3,548	40.1	72.6	116.0	189.4	181.5	114.0	147.7	4.2
11	RA1+RA2	6,026	64.4	104.3	164.9	262.4	256.0	151.5	203.6	3.4
12	KE	1,928	20.1	33.3	52.4	79.3	81.2	41.3	61.2	3.2
13	HA	3,765	46.7	83.3	130.4	209.2	195.0	122.7	158.8	4.2
14	Before KE	6,142	124.7	171.2	229.3	358.3	429.5	229.6	329.3	5.4
15	After RA	19,569	329.7	466.6	681.4	1,015.7	1,032.2	562.9	797.0	4.1
16	After LE	34,821	550.0	798.4	1,191.8	1,776.8	1,774.5	944.1	1,358.4	3.9
17	After KO	54,773	868.8	1,271.1	1,911.0	2,976.2	2,920.7	1,440.0	2,178.7	4.0

5. FLOOD ROUTING SIMULATION FOR MUSI RIVER AND TRIBUTARIES

The Kinematic Channel Routing Method is adopted to establish the flood simulation model for solution to define the propagation of hydrograph in the Musi River Basin. Normally, flood simulation of how a flood wave or a hydrograph propagates along a river is based on solving the St. Venant equations (for establishment of hydrodynamic model). This requires cross cross-sectional information of the river, however, the information is inadequate for the establishment of a hydrodynamic model for the Musi River Basin. Thus, the Kinematic Channel Routing Method above said in the first paragraph is applied for the flood simulation because this method does not require cross-sectional information.

5.1 Objective of Flood Routing Simulation and Target Stretch

The Flood Routing Simulation was carried out for the purpose of estimating of the peak discharge of flood and its probability at the downstream edge of tributaries or the representative station in the Musi main stretch.

The flood routing was made along the Musi main stretch (from the confluence with the Komering River to the upstream end) and its tributaries; namely, Komering, Ogan, Lematang, Semangus, Kelingi, Lakitan, Rawas and Harileko River.

5.2 Governing Equation of Kinematic River Routing Method

The governing equations for computation are the following continuity equation (a) and the momentum equation (b):

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \Lambda \; \Lambda \; \Lambda \;\; (a)$$

$$V = \frac{1}{n} R^{2/3} i^{1/2} \; \Lambda \; \Lambda \; \Lambda \;\; (b)$$

where,

Q : Discharge (m^3/s)
A : Flow Area (m^2)
q : Inflow ($m/s^3/m$)
V : Velocity (m/s^3)
n : Coefficient of Roughness
R : Hydraulic Radius (m)
i : Slope of Energy Line

The Kinematic Channel Routing Method is categorized in the hydrologic routing method used to handle variable Discharge-Water Area relationships. Therefore, equation (b) can approximate the exponential function (c):

$$A = K_r Q^{P_r} \Lambda\Lambda\Lambda\Lambda\Lambda\Lambda\Lambda\Lambda\Lambda\Lambda\Lambda\Lambda\Lambda \quad (c)$$

where,

Q : Discharge (m^3/s)

A : Flow Area (m^2)

K_r, P_r : Constant

5.3 Establishment of Flood Routing Model

The Kinematic Channel Routing Module was integrated with the runoff model established in Section 4.2. The schematic diagram of the flood routing model is given in **Figure G5.3.1**.

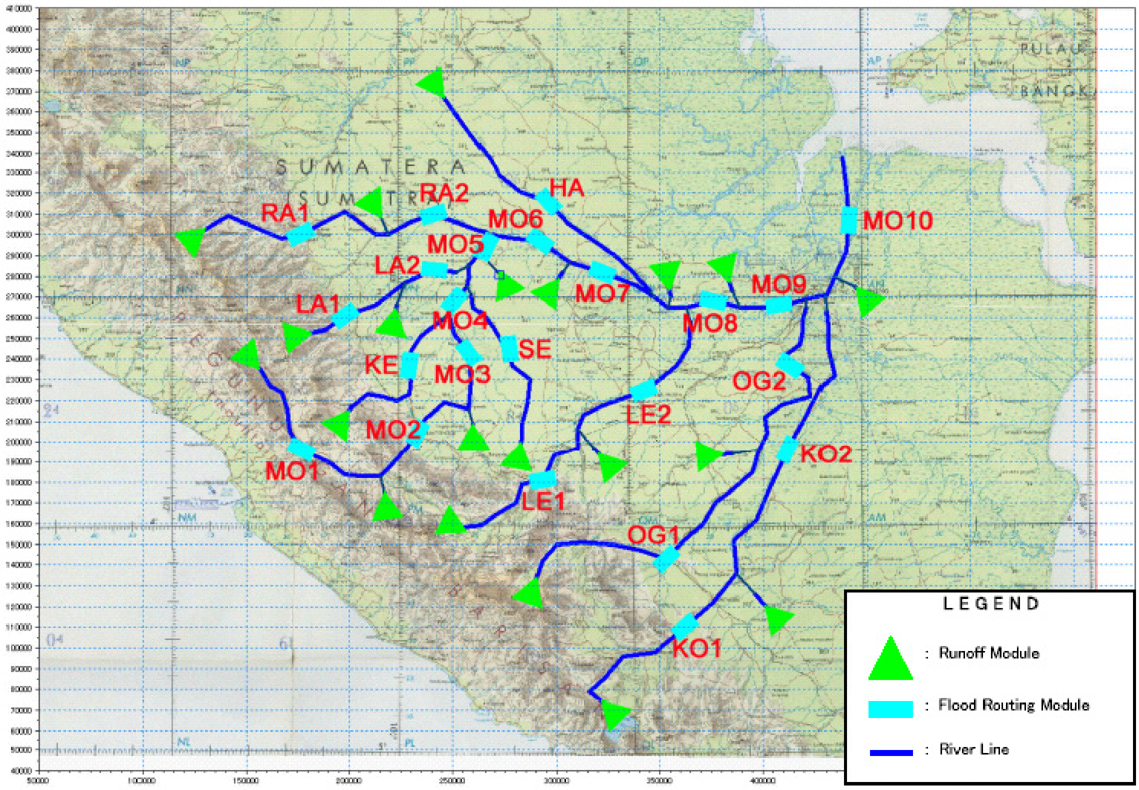

Figure G5.3.1 Schematic Diagram of Flood Routing Simulation

5.3.1 Boundary Condition

(1) Direct Runoff

The time series of discharge generated from the runoff model (NAM model) was used as the direct runoff for the flood routing calculation.

(2) Parameter

As to the parameter Kr, the following standard equation was applied.

$$K_r = B^{0.4}(n/\sqrt{I})^{P_r}$$

where,

B :River Width (m)
n :Roughness Coefficient of Riverbed
I :Slope of Riverbed
P_r :Constant (P_r=0.6)

The setting of N refers to the standard value used in Japan. Values (N, B and I) for the estimation of K and P are summarized in **Table G5.3.1.**

Table G5.3.1 Parameters for Definition of K Value

No.	Sub-Basin	L (m)	N	B (m)	I	No.	Sub-Basin	L (m)	N	B (m)	I
1	M01	125,509	0.040	150	0.01450	13	LA1	97,904	0.045	100	0.01976
2	M02	78,529	0.035	200	0.00059	14	LA2	42,096	0.035	150	0.00099
3	M03	63,802	0.035	200	0.00035	15	RA1	133,817	0.035	200	0.00029
4	M04	20,784	0.030	300	0.00010	16	RA2	74,183	0.040	150	0.00728
5	M05	44,329	0.030	300	0.00009	17	HA	334,000	0.035	150	0.00042
6	M06	46,085	0.030	400	0.00008	18	LE1	162,388	0.040	200	0.00952
7	M07	58,267	0.030	400	0.00007	19	LE2	185,612	0.035	150	0.00029
8	M08	57,769	0.030	500	0.00003	20	OG1	120,357	0.035	200	0.00025
9	M09	42,297	0.030	600	0.00002	21	OG2	192,643	0.030	200	0.00017
10	M10	102,629	0.030	800	0.00002	22	KO1	120,357	0.040	200	0.00516
11	KE	98,000	0.045	150	0.01951	23	KO2	207,643	0.035	200	0.00048
12	SE	183,000	0.035	150	0.00121	-	-	-	-	-	-

5.4 Model Calibration

Examination was made to confirm whether or not the Kinematic Channel Routing Method is applicable as a flood routing method in the Musi River Basin.

5.4.1 Target Flood and Calibration Point

The Model Calibration was carried out in the flood season of 1986 considering the term of availability of data. Water level data at the Tebing Abang station were measured in the flood seasons of 1986, 1994, 1995, 1996 and 2000. Among the flood, the 1986 flood is the most available for calibration because the discharge rating curve was produced in the Musi River Basin Study in 1989 using the past observation data before 1989.

The Tebing Abang station was selected as the representative calibration point, since the station is situated just before the Palembang City which should be protected from inundation by flood. The drainage area in this station is 33,275 km^2.

5.4.2 Calibration Result

Figure G5.4.1 gives a comparison between two discharge hydrographs of the 1986 flood obtained from the hydrodynamic model and the actual discharge that was converted from observed water level and discharge rating curve. Very good agreement was seen between them; hence, it can be said that the Kinematic Channel Routing Method is good enough to be applied for the Musi River and its tributaries.

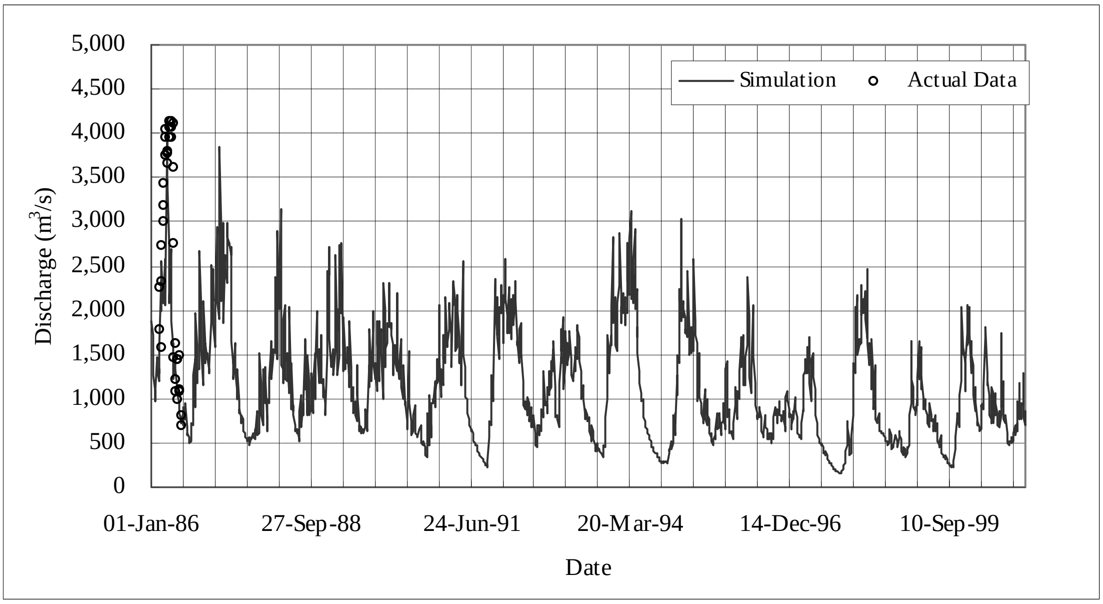

Figure G5.4.1 Calibration Result of Flood Routing Simulation

5.5 Simulation Result

5.5.1 Maximum Discharge

Annual maximum discharges at the Musi River (Tebing Abang and after confluence of Komering River) and the downstream end of main tributaries are shown in **Table G5.5.1**.

Table G5.5.1 Maximum Discharge Resulting from Simulation

(m^3/s)

Year	Musi[1]	Musi[2]	Komering	Ogan	Lematang	Semangus	Kelingi	Lakitan	Rawas	Harileko
1986	3,910	5,082	616	633	727	144	232	159	694	126
1987	3,838	4,626	795	539	892	248	414	277	1534	1297
1988	3,152	4,770	788	665	808	106	203	182	748	523
1989	2,765	4,950	1,209	888	870	116	169	172	1113	804
1990	2,301	3,897	1,016	603	704	199	308	203	882	739
1991	2,557	4,071	994	857	744	219	386	258	1098	849
1992	2,575	4,360	1,158	950	825	129	196	151	479	321
1993	2,824	4,606	1,106	1,079	1,212	136	208	146	701	610
1994	3,123	5,139	1,165	1,119	1,319	254	351	237	482	441
1995	3,031	4,640	1,261	900	1,007	195	273	183	364	336
1996	2,378	3,728	805	607	1,209	58	92	77	280	261
1997	1,692	2,756	693	380	366	140	211	101	637	495
1998	2,474	3,409	652	535	734	194	267	223	449	246
1999	2,026	3,234	733	505	665	72	126	119	342	86
2000	2,053	3,685	1,031	637	858	134	280	174	431	338
Average	2,713	4,197	935	726	863	156	248	178	682	498

Musi[1]:Tebing Abang, Musi[2]: After confluence of the Komering River

5.5.2 Probable Discharge

A statistical analysis was carried out to estimate the probable discharge at the downstream end of tributaries and the representative point of the Musi main stretch based on the maximum discharge summarized in Table G5.5.1. The Gumbel method was applied for this statistical analysis and the results are given in **Table G5.5.2** below.

Table G5.5.2 Probable Discharge Resulting from Simulation

(m^3/s)

Return Period (Year)	Musi[1]	Musi[2]	Komering	Ogan	Lematang	Semangus	Kelingi	Lakitan	Rawas	Harileko
2	2,610	4,078	899	690	823	146	168	233	625	445
3	2,872	4,381	990	783	925	171	192	271	771	580
5	3,165	4,718	1092	886	1,039	199	218	313	934	729
8	3,416	5,008	1,180	976	1,136	223	240	350	1,073	857
10	3,532	5,142	1,221	1,017	1,182	234	251	367	1,138	917
20	3,884	5,549	1,344	1,141	1,319	267	282	418	1,334	1,097
30	4,086	5,783	1,414	1,213	1,398	287	300	447	1,447	1,200
50	4,339	6,076	1,503	1,303	1,496	311	323	484	1,588	1,330
70	4,505	6,267	1,561	1,362	1,561	326	338	508	1,680	1,414
80	4,571	6,343	1,584	1,385	1,587	333	344	518	1,717	1,448
100	4,681	6,470	1,622	1,424	1,629	343	354	534	1,778	1,504

Musi[1]:Tebing Abang, Musi[2]: After confluence of the Komering River

6. INUNDATION ANALYSIS FOR PALEMBANG DRAINAGE PLAN

6.1 Objective Inundation Analysis

To identify the present conditions of the probable flood inundation area, inundation analysis was carried out for the Palembang Drainage Plan. The drainage plan of Palembang in Sector H additionally used the basic hydrological boundary or parameters set up during the establishment of the simulation model for the inundation analysis.

6.2 Target Area

This Chapter 6 describes the runoff simulation done for all the 19 subdivided drainage areas, the results of which were used for the selection of priority improvement area in Sector H. Hence, the flood simulation for the inundation analysis was carried out for the priority area drained by the Buah and Bendung channels.

The target area for runoff simulation covers almost all of Palembang City considering the past inundation-damaged area, the future drainage plan, and the location of infrastructure and houses. The target area of approximately 400 km^2 is shown in **Annex G6.2.1**, and is divided into 19 catchment areas considering the existing drainage network system. The priority area (Buah and Bendung Area) selected in Sector H can be seen also on the same figure.

6.3 Inundation Regime

According to the report entitled “PERENCANNAAN TEKNIS DRAINASE KOTAMADAYA PALEMBANG, FINAL REPORT, 1995/7”, inundation had frequently happened around 10 years ago, spreading from the drainage channel as can be seen in **Annex G6.3.1**. The insufficient flow capacity of the channel, as well as high tide, had been the primary cause of inundation.

The probable inundation area at present is smaller, so that people in Palembang do not recognize the inundation as a serious disaster because of the effect of drainage improvement done so far. However, spot inundations still routinely take place at 59 areas when rainfall intensity becomes higher, as shown in **Annex G6.3.2.** The probable inundation area is 126.9 km^2 and inundation lasts for 4.6 hours at the average inundation depth of 0.3 m. The inundation area is outlined in **Table G6.3.1**.

Table G6.3.1 Outline of Inundation Area

No.	Sub-Basin	CA (km^2)	IA (ha)	Dep. (m)	Duration (hour)	No.	Sub-Basin	CA (km^2)	IA (ha)	Dep. (m)	Duration (hour)
1	Gandus	23.95	-	-	-	11	S. Lincah	4.83	-	-	-
2	Gasing	52.11	1.50	0.25	4.00	12	Borang	71.21	8.26	0.15	5.21
3	Lambidaro	50.52	7.00	0.25	5.00	13	S. Nyiur	22.85	-	-	-
4	Boang	8.67	8.50	0.18	3.00	14	Sriguna	4.91	13.75	0.18	4.13
5	Sekanak	11.40	16.73	0.25	3.29	15	Aur	6.58	9.50	0.18	3.40
6	Bendung	19.19	14.62	0.38	5.43	16	Kedukan	9.32	3.90	0.21	6.25
7	L. Kidul	2.34	-	-	-	17	J. Baring	37.61	3.17	0.15	4.70
8	Buah	10.42	6.30	0.30	2.50	18	Kertapati	25.09	15.00	0.20	6.00
9	Juaro	6.86	13.50	0.40	12.00	19	Keramasan	30.09	-	-	-
10	Batang	5.59	0.80	0.25	4.00						

Source: PROGRAM JANGKA MENENGAH SEKTOR DRAINASE, FINAL REPORT, and 2001/2, PEMERINTAH KOTA PALEMBAN
CA: Catchment Area, IA: Inundation Area, Dep.: Inundation Depth

6.4 Elaboration of Flood Simulation Model

6.4.1 Structure of Flood Simulation Model

The hydrological module (rainfall-runoff module) based on the Storage Function method calculates runoff generated in the 19 drainage areas. The calculated runoff from the drainage area is given to the dynamic flow module as the upstream end model boundary or lateral inflows from branch channels.

For the flood routing in the drainage channel, the hydrodynamic module is used. The hydrodynamic module is a one-dimensional dynamic flow model for which the Saint Venant Equation is applied. By this equation, Hydraulic parameters such as water levels, velocities and discharges can be estimated at any points of the channels.

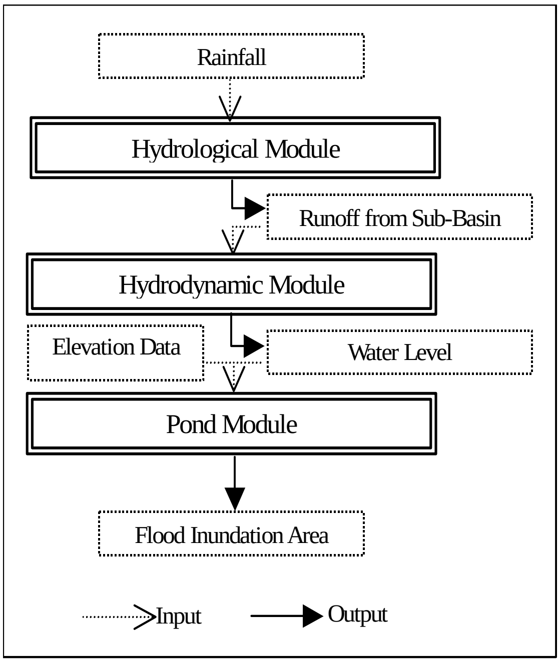

Figure G6.4.1 Flow Chart of Simulation

The calculated water levels are transferred to the Link Channels that compose Pond module for estimation of inundation depth, duration and area. The Pond module expresses the probable inundation area by the relationship between ground elevation and corresponding area. The Link Channels, which transfer the water overflowing from the main channel to the connected pond, form a skeletal structure for the connection among ponds to define flood inundation areas and depths by comparing the river water levels with the ground elevations.

6.4.2 Runoff Model for Estimation of Runoff from Drainage Areas

(1) Concept of Runoff Model

The Storage Function method is employed as the flood run-off model for sub-basins, because this method receives wide recognition as the de-facto standard method for planning flood control in Japan and other Asian countries. The basic image of the Storage Function Model is given in **Figure G6.4.2**.

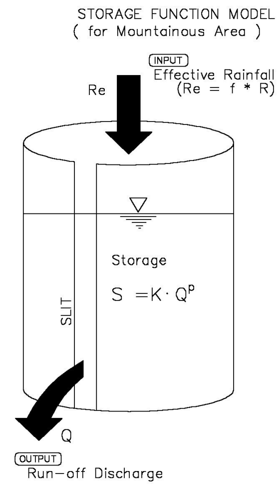

Figure G6.4.2 Image of Storage Function model

Rainfall transforms into discharge, and on the way to transformation there must be some storage. Therefore, to express the non-linear characteristics of the runoff phenomena, the Storage Function Method was developed. This method can simulate the process of transformation from rainfall to runoff on the assumption that there is a one-to-one functional relation between the storage volume in the sub-basin and the runoff discharge.

Calculations of run-off from rainfall are made through the use of the storage volume as the medium function. The relation between the storage volume in the basin and the discharge is expressed as:

$$S = K \times q^P$$

where, S : Depth of storage (mm)
q : Depth of run-off (mm/hr)
K, P : Constants

This formula establishes that run-off is proportional to the exponent of the storage volume. Run-off calculations were performed by the combination of this equation of motion with the following equation of continuity.

$$dS/dt = F \times R(t) - q(t+TL)$$

where, F : Inflow coefficient ($F = F_1$ or $F = 1.0$)
F_1 : primary run-off rate ($F_1 < 1.0$)
$R(t)$: Average rainfall in a watershed (mm/hr)
$q(t+TL)$: Depth of run-off with lag time (mm/hr)
TL : Lag time (hr)
t : Time

The volume of run-off from the basin should be the sum of run-off from both zones plus base flow. Run-off (m^3/s) from the basin is given by the following formula:

$$Q(t) = F_1 \times A \times qt(t)/3.6 + (1-F_1) \times A \times qs(t)/3.6 + Qb$$

where, $Q(t)$: Run-off (m^3/s)
F_1 : Primary run-off rate
A : Catchment area (km^2)
$qt(t)$: Run-off by total rainfall (mm/hr)
$qs(t)$: Run-off by rainfall after saturation (mm/hr)
Qb : Base flow (m^3/s)

(2) Excess Rainfall

To separate excess rainfall from the rainfall abstraction in every sub-basin (including initial loss and infiltration into underground), the F_1-R_{sa} Method is applied. F_1 and R_{sa} are decided from soil, land use, and antecedent rainfall based on various research papers.

Before cumulative rainfall exceeds the saturated rainfall (R_{sa}), only the area of F_1*A causes the run-off. After cumulative rainfall exceeds R_{sa}, run-off may occur even from the remaining part $(1-F_1)$*A due to the rainfall exceeding R_{sa}. Both the run-off zone and the infiltration zone should be calculated separately for the run-off until the end of the flood.

(3) Model Hyetograph

The front-concentrated type of hyetograph was employed as the design storm pattern, considering the rainfall pattern in actual storms described in Subsection 3.1.3 as well as the duration of storm rainfall fixed at 12-hours. On the basis of these results of rainfall analysis, the model hyetograph was established corresponding to each probability of rainfall, as can be seen in **Figure G6.4.3.**

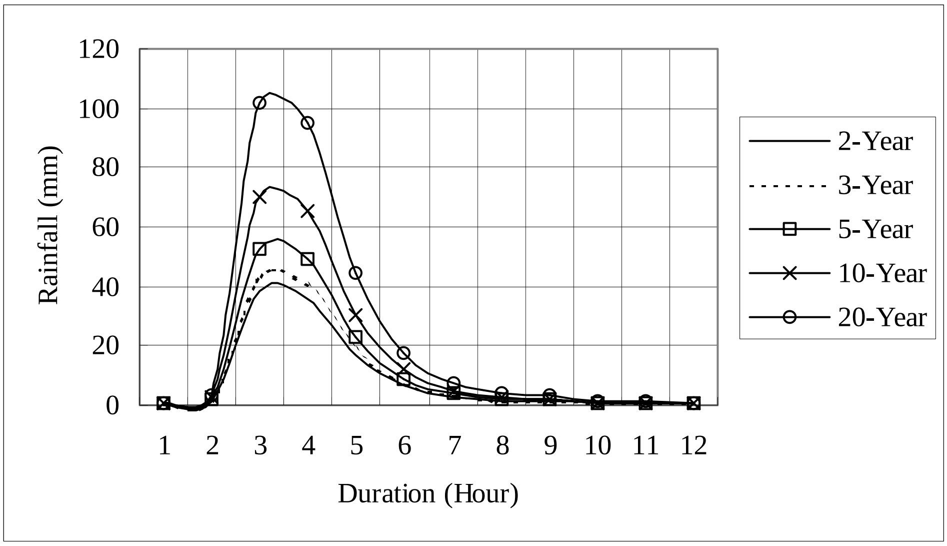

Figure G6.4.3 Model Hyetograph

(4) Parameters for Run-Off Model (Storage Function Model)

As to the parameters K and P in the Storage Function Model for existing land use, the following standard equation was applied.

$$K = 0.1165(N \times L / S^{1/2})^{0.6}$$
$$P = 0.6$$

where, N : Equivalent roughness coefficient (m^{-3}/s)
L : Length of Basin (m)
S : Slope of Basin

S and *L* were obtained from the results of the topographical survey in this study and the maps in the past report (PERENCANNAAN TEKNIS DRAINASE KOTAMADAYA PALEMBANG, FINAL REPORT, 1995/7). The setting of *N* refers to the standard value used in Japan. Values (*N*, *L*, *S*) for the estimation of *K* and *P* are summarized in **Table G6.4.1.**

Table G6.4.1 Parameters for Definition of K Value

No.	Sub-Basin	CA (km^2)	N (m^{-3}/s)	S	L (m)	No.	Sub-Basin	CA (km^2)	N (m^{-3}/s)	S	L (m)
1	Gandus	23.95	2.0	0.00003	7.0	11	S. Lincah	4.83	2.0	0.00003	2.8
2	Gasing	52.11	2.0	0.00003	7.2	12	Borang	71.21	2.0	0.00003	8.1
3	Lambidaro	50.52	2.0	0.00003	13.8	13	SP. Nyiur	22.85	2.0	0.00003	4.7
4	Boang	8.67	2.0	0.00003	4.1	14	Sriguna	4.91	1.0	0.00003	2.4
5	Sekanak	11.40	1.0	0.00003	6.5	15	Aur	6.58	1.0	0.00003	2.0
6	Bendung	19.19	1.0	0.00007	8.6	16	Kedukan	9.32	1.0	0.00003	3.4
7	L. Kidul	2.34	1.0	0.00003	2.7	17	J. Baring	37.61	2.0	0.00003	7.0
8	Buah	10.42	1.0	0.00007	8.1	18	Kertapati	25.09	2.0	0.00003	2.0
9	Juaro	6.86	2.0	0.00003	4.1	19	Keramasan	30.09	2.0	0.00003	5.0
10	Batang	5.59	2.0	0.00003	3.0						

The parameters (K and P) for the run-off module calculated using the equation mentioned above are given in **Table G6.4.2**. F_1 and R_{sa} were also deduced, referring to the standard value corresponding to land use adapted in Japan. Particularly, F_1 was set at a comparatively low value because the swamp area are scattered in many places at the Palembang City.

Table G6.4.2 Parameters for Model (Present Condition)

No.	Sub-Basin	CA (km^2)	P	K	F_1	R_{sa} (mm)
1	Gandus	23.95	0.6	12.51	0.2	250.0
2	Gasing	52.11	0.6	12.72	0.2	250.0
3	Lambidaro	50.52	0.6	18.79	0.2	250.0
4	Boang	8.67	0.6	9.07	0.25	250.0
5	Sekanak	11.40	0.6	7.89	0.3	150.0
6	Bendung	19.19	0.6	7.58	0.3	150.0
7	L. Kidul	2.34	0.6	4.66	0.3	150.0
8	Buah	10.42	0.6	7.32	0.25	150.0
9	Juaro	6.86	0.6	9.07	0.2	250.0
10	Batang	5.59	0.6	7.52	0.2	250.0
11	S. Lincah	4.83	0.6	7.22	0.2	250.0
12	Borang	71.21	0.6	13.65	0.2	250.0
13	SP. Nyiur	22.85	0.6	9.85	0.2	250.0
14	Sriguna	4.91	0.6	4.34	0.25	150.0
15	Aur	6.58	0.6	3.89	0.25	150.0
16	Kedukan	9.32	0.6	5.35	0.25	150.0
17	J. Baring	37.61	0.6	12.51	0.2	250.0
18	Kertapati	25.09	0.6	5.90	0.2	250.0
19	Keramasan	30.09	0.6	10.22	0.2	250.0

CA: Catchment Area

According to the records of Palembang City, the land use of four (4) sub-basins (Lambidaro, Boang, SP. Nyiur and J. Baring) will be developed into one similar to the land use in Sriguna, Aur, Kedukan in 2009. As a future condition, therefore, the parameter F_1 was changed from 0.20 to 0.25 at Lambidaro, Boang, SP. Nyiur and J. Baring.

6.4.3 Elaboration of Hydrodynamic Model for Flood Routing of Channels

The hydrodynamic Simulation model was elaborated for the flood routine of selected Channel in Sector H. Described below is the elaboration of the model for channels.

(1) Concept of Hydrodynamic Simulation

The governing equations of hydraulic simulation (Complete Saint-Venant Equation) are the continuity equation (a) and the momentum equation (b):

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}=q\Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \Lambda\ \cdot (a)$$

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{\beta Q^2}{A}\right)+gA\frac{\partial H}{\partial x}+gAI-\frac{gn^2Q^2}{R^{\frac{3}{4}}A}=0\Lambda\ \Lambda\ \Lambda\ \ (b)$$

where,

- Q : Discharge (m^3/s)
- A : Water Area (m^2)
- q : Lateral Inflow (m^3/s)
- H : Water Depth (m)
- g : Gravity Acceleration (m/s^2)
- I : Bed Slope

The Saint-Venant Equations express conversation of mass and momentum. Conservation of mass leads to the continuity equation that establishes a balance between the rate of rise of water level and the storage in the wedge and prism channel (The wedge means that the cross-sectional shape is wedge-like and the prism means that the cross-sectional shape does not vary along the channel and the bed slope is constant). Conservation of momentum leads to the 'dynamic' equation that establishes a balance between inertia, diffusion, gravity and friction forces. Some other forces, such as the effect of wind or meanders, may also be included but usually these are small.

(2) Prerequisite Condition

(a) Cross-Section of Selected Channels

The cross sections prepared with the interval of about 200 m by the topographic survey in this study were used. The cross sections were measured taking the bottleneck points into consideration.

Table G6.4.3 Dimension of Drainage Channel

Selected Channel	Catchment Area (km^2)	Length (km)	Average Width (m)	Slope	Number of Cross-Section
Bendung Drainage System	19.2	5.40	7	0.0004	30
Buah Drainage	10.4	6.40	5	0.0002	33

(b) Discharge - Time Boundary at Upstream End

The runoff module established in Subsection 6.4.2 calculates the discharge boundary of upstream. The lateral flow, which distribution is explained in Sector H, is considered for flood routing of Buah and Bendung Channel.

(c) Water Level - Time Boundary

The water level - time boundary set at the downstream end was based on the actual water level at the station near Palembang City (see Sector H).

(d) Manning's Roughness Coefficient

In this flood routing simulation, the roughness coefficient for each channel was deduced from the Japanese standard that gives the value of around 0.025 for artificial concrete channel like Bendung and Buah drainage channels.

6.4.4 Pond Model

(1) Concept of Pond Module

The concept of pond model is illustrated in **Figure G6.4.4**, which indicate that the drainage area is divided into the cells. The overflow from a cell to another cell is governed by the continuity equation (a) and the momentum equation (b):

$$F\frac{dH}{dt} = Q_{in} - Q_{out} \Lambda \quad (a)$$

$$Q = cA\sqrt{\Delta H} \Lambda \Lambda \cdot \Lambda \quad (b)$$

where,

Q : Discharge (m^3/s)
A : Water Area (m^2)
H : Water Depth (m)
c : Constant

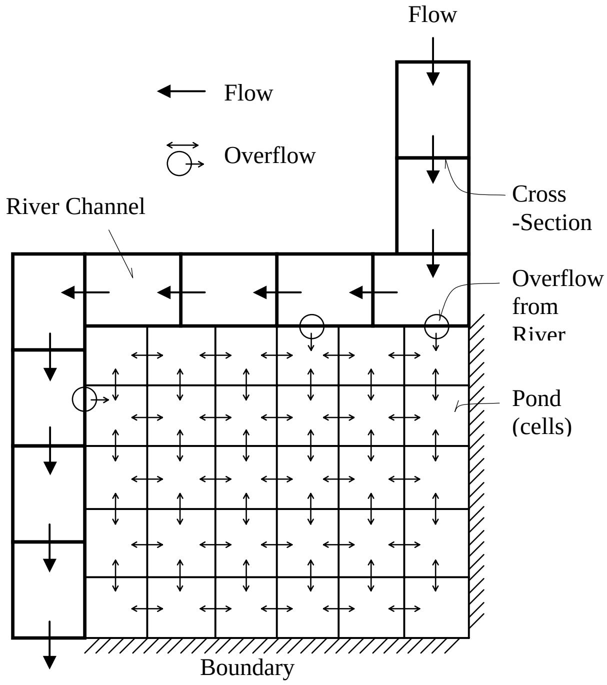

Figure G6.4.4 Concept of Pond Module

(2) Division of Drainage Area to Small Cells

The drainage area of Buah and Bendung channels is divided into 33 and 47 cells respectively (see **Annex G6.4.1**). Cells are separated from each other by walls representing the road, riverbank or other obstructions that prevent overflow from a cell to another cell.

(3) Preparation of H-A Relationship

H-A Relationship (relationship between ground height and area) is given into the cells. The flood inundation area can thus be analysed by comparing the water level with the H-A relationship.

6.5 Simulation Result

6.5.1 Simulation Result of Runoff

The hydrograph corresponding to the return period was calculated by using the establish model, as shown in **Annex G6.5.1.** The drainage planning in this study was carried on the basis of this hydrograph as the boundary condition. The inundation volume also was calculated as summarized for each sub-basin in **Sector H River Conditions, Flooding and Inundation**.

6.5.2 Flood Map

Simulation result (inundation Depth, Volume and Area) corresponding to return period are summarized in **Annex 6.5.2**. Based on the simulation result, the flood map in case of 15-year return period was prepared, as shown in **Annex G6.5.3**.

7. SEDIMENTATION

7.1 Sedimentation in Musi Main Stretch (at Tebing Abang)

The Musi River Basin Study of 1989 had empirically assumed that the average specific annual total sediment load at Tebing Abang (after confluence with the Lematang River) is between 1.0 and 2.0 ton/day/km^2, giving the average total sediment load of between 13 and 25 million ton/year.

7.2 Sedimentation in Upstream of Komering River (at Martapura)

Between 1986 and 1987, the Ministry of Public Works, Institute of Hydraulic Engineering carried out a sediment load survey. The results are given in **Table G7.2.1**.

Table G7.2.1 Annual Sediment Load at Martapura

Item	1986 (mil. ton)	1987 (mil. ton)
Suspended Load	4.64	6.03
Bed Load	0.73	0.86
Total	5.37	6.89

In that survey, the suspended load-rating curve at Martapura was determined at:

$$Q_s = 0.053 \times Q_w^{\ 2.5199}$$

where,

Q_s : Suspended Load (ton/day)
Q_w : Discharge (m^3/s)

Based on the above rating curve and the data collected in the present study, the total amount of sediment between 1988 and 1998 was calculated, as shown in **Table G7.2.2**. The bed load is given as 15% of sediment load based on **Table G7.2.1**.

Table G7.2.2 Estimation Result of Sediment Load at Martapura

Year	Suspended Load (mil. ton)	Bed Load (mil. ton)	Total (mil. ton)	Specific Sediment Load (Ton/day/km^2)
1988	7.78	1.1	8.88	5.60
1991	3.29	0.5	3.79	2.40
1992	2.59	0.39	2.98	1.89
1993	3.63	0.54	4.17	2.64
1998	5.59	0.83	6.42	4.07

The estimation and survey results give 5.02 million tons of annual average total sediment. Hence, the annual average specific sediment load (the catchment area at Martapura is 4,320 km^2) is estimated at 3.18 ton/day/km^2.

7.3 Sedimentation in Lower Musi River

Sediment during low flow is deposited along the Lower Musi River, mainly from Tebing Abang to the sea, because the river flow in this section slows down due to the backwater (tidal) affect. During high flow, however, the riverbed is scoured and most of the sediment is transported into the sea. The remaining sediment deposit after scouring would require dredging, which is presently being carried out annually.

The results of sedimentation studies on the Musi River give the sediment level as follows:

- Frankle USA, 1968: approx. 40 cm/month
- JICA Japan, 1976 : approx. 43 cm/month
- Observation of Pimbagro Faskespel South Sumatra, 1999: approx. 2-4 cm/day
- Observation of Third Pelindo Company, 1999: approx. 2-4 cm/day