CHAPTER 3 HYDROLOGICAL AND HYDRAULIC ANALYSIS

3.1 General

This Chapter describes the standard methodologies to clarify the hydraulic and hydrological conditions on the urban drainage improvement plan.

Explanation:

The discharge data is indispensable to clarify the present flood runoff conditions in the objective urban area, but its gauging data is hardly obtained due to lack of sufficient gauging network. Moreover, the flood runoff conditions in a drainage area tend to have a dynamic change due to intensive urban development, and one of the important issues on the urban drainage improvement plan is to simulate the flood runoff conditions under the future land use. From these viewpoints, this Chapter prescribes the flood simulation model which is subject to rainfall gauging data as the input data and could estimate the flood runoff discharge both for present and future land use conditions.

In Malaysia, the drainage channel improvement has been conventionally applied as a principal measure for urban drainage. Due to this background, a major hydrological concern has been given to estimation of peak flood discharge which is essential to determine the design scale for drainage channel improvement. However, the present rapid progress of land development causes a drastic increment of the drainage channel flow which is collected into but could be hardly accommodated by the river channel located downstream from the drainage channels. In order to retrieve such adverse effects to the river channel, the basin flood detention and retention (called as "source control") is highlighted as an alternative measure for urban drainage improvement. The plan for basin flood detention and retention requires the hydrological information on not only the peak runoff discharge but also the flood hydrograph in order to clarify the regulation effect by the flood detention and retention facilities. From this viewpoint, this Chapter prescribes the flood simulation model oriented to development of the design flood hydrograph

As for the hydraulic analysis, the contents could be generally classified into "(a) the channel hydraulics" and "(b) the hydraulics of detention and retention facility". The first item (a) has been described in detail in the existing guidelines in Malaysia*[3-1 to 3-2]. In due consideration of the availability of the existing guidelines, this Chapter focuses on the hydraulic design of flood detention and retention facilities. The analysis described in this Chapter includes the following process;.

Fig. 3.1 Process of Hydraulic and Hydrological Analysis

Storm Rainfall Analysis
Development of Design Hyetograph

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Flood Runoff Analysis
Development of Standard Project Flood Hydrograph

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Analysis on Flood Detention and Retention Facilities
Hydraulic Design for the Facilities
Regulation Effect by the Facilities

3.2 Storm Rainfall Analysis

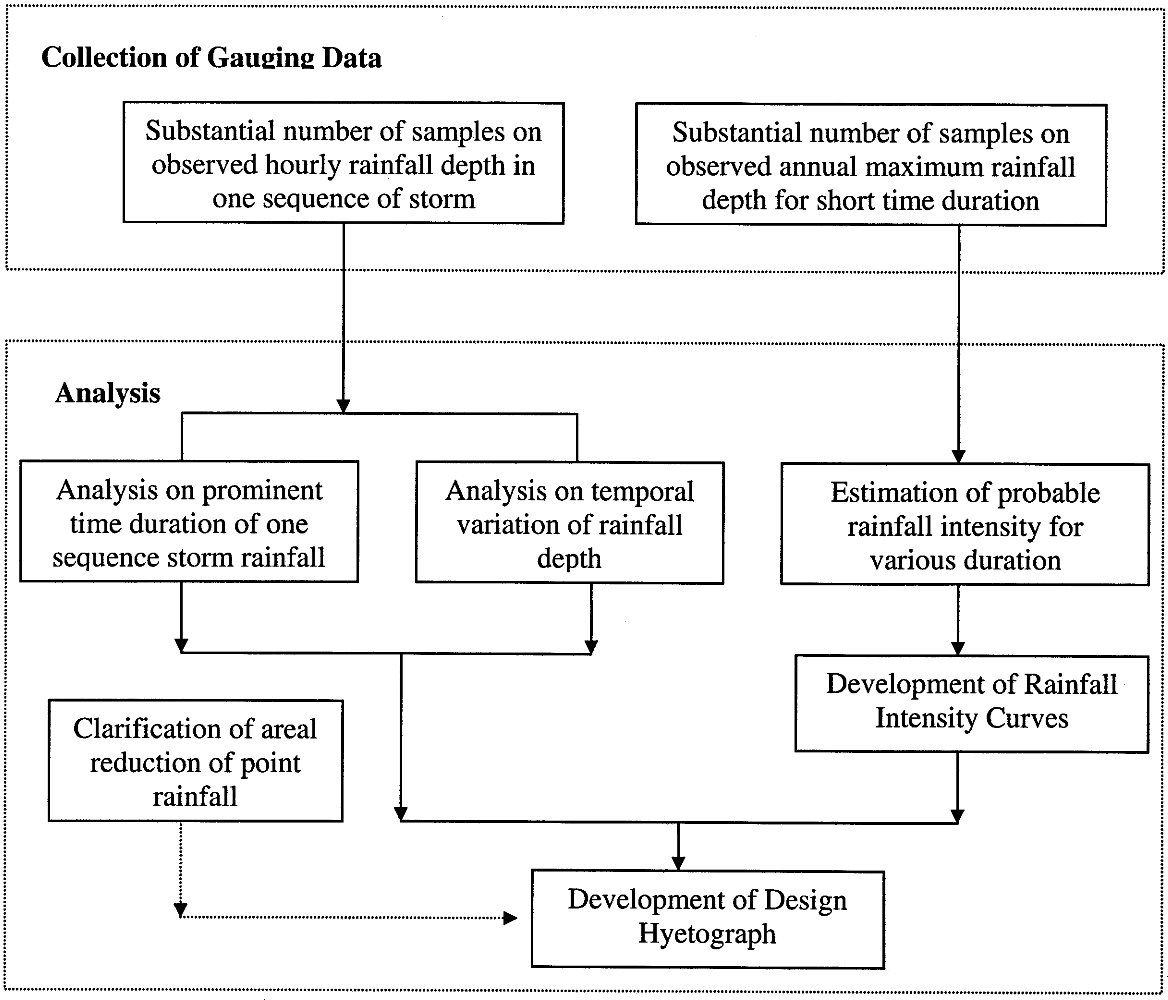
3.2.1 Purpose of Analysis

The rainfall analysis aims at estimating the design hyetograph which is used as the principal input data for simulation of design flood hydrograph.

Explanation:

The design hyetograph is essential for simulation of the standard project flood hydrograph. The design hyetograph has to be expressed as a probable value of a specified return period. In order to estimate such probable value, this sub-section prescribes the methodology to develop the design hyetograph with using the probable rainfall-intensity curves. The standard procedure to develop the design hyetograph is as shown in Fig. 3.2:

Fig. 3.2 Procedure to Develop Design Hyetograph



3.2.2 Collection of Rainfall Gauging Data

The following point rainfall data recorded in and around the objective drainage area should be collected:

(1) Samples of observed hourly rainfall depth in one sequence of storm

(2) Samples on observed annual maximum rainfall intensity for various rainfall duration

Explanation:

The above data in item (a) are required to determine the maximum time length of one sequence of design hyetograph and also to determine the temporal variation of the design hyetograph. The data should contain a substantial number of storm samples so as to represent the features of the storm rainfall in the objective drainage area.

The data in item (b) (i.e., a series of the annual maximum rainfall) are used as the population of extreme values to estimate the probable rainfall intensities for various time duration (15min., 30 min., and 1 hour to several hours). The data of more than 10-year period is required as the available population of extreme values for probability analysis. The data could be given by reading from the automatic rainfall recording sheet.

Reference:

Malaysian Meteorological Service (MMS) has recorded the annual maximum rainfall intensities for rainfall duration of 15 minutes to 48 hours. The data are available at the principle MMS gauging stations in Malaysia, and most of the data has a substantial recording length of decades.

3.2.3 Estimation of Probable Rainfall Intensity

The recurrence probability of point rainfall intensities are estimated from the annual maximum rainfall intensities through the formula of Gumbel distribution.

Explanation:

The Gumbel distribution is derived from the fact such that the series of annual maximum rainfall intensities could suite to extreme exponential value distribution. The basic formula of the Gumbel distribution is expressed as below:

$P(x) = 1 - \exp(-e^{y}) = 1/T$ .... (Eq. 3.1)
$y = a\,(x - x_0)$ or $x = x_0 + y/a$ .... (Eq. 3.2)

Where, $P(x)$: Probability of exceedance
T : Return period
x : Annual maximum value
a, x_0 : Constants (>0)

The constants of "a" and "x_0" are estimated from the following formulas:

$$1/a = Sx/Sy, \quad x_0 = x_{ave} - y_{ave}/a, \quad x_{ave} = 1/N \sum_{i=1}^{i=N} xi, \quad y_{ave} = 1/N \sum_{i=1}^{i=N} yi \quad \text{(Eq. 3.3)}$$

$$Sx = \{1/N \sum_{i=1}^{i=N} (xi - x_{ave})^2\}^{0.5} \quad Sy = \{1/N \sum_{i=1}^{i=N} (yi - y_{ave})^2\}^{0.5} \quad \text{(Eq. 3.4)}$$

3.2.4 Development of Rainfall Intensity-Duration Curves

The rainfall intensity-duration curve is developed through the most conformable formula selected among the following:

(1) Talbot Type	:	$I = a / (T + b)$	(Eq. 3.5)
(2) Sherman Type	:	$I = a / T^{n}$	(Eq. 3.6)
(3) Kuno Type	:	$I = a / (T^{0.5} + b)$	(Eq. 3.7)

Where; I : Probable Rainfall intensity
T : Rainfall time duration
a, b, n : Constants

Explanation:

The rainfall intensity-duration curve expresses the relationship between the point probable rainfall intensities and its corresponding rainfall duration. In order to express the relationship, the above three (3) formulas were developed. The constants in the formulas could be estimated, as below, by a least- square regression method:

(1) Talbot Type:

$$a = \{ [IT] [I^2] - [I^2T] [I] \} / \{N[I^2] - [I] [I]\}$$
$$b = \{ [I] [IT] - N [I^2T] \} / \{N[I^2] - [I] [I]\}$$

(2) Sherman Type:

$$\log a = \{[\log I] [(\log T)^2] - [\log T \cdot \log I] [\log T]\} / \{N [(\log T)^2] - [\log T] [\log T]\}$$
$$n = \{[\log I] [\log T] - N [\log T \cdot \log I]\} / \{N [(\log T)^2] - [\log T] [\log T]\}$$

(3) Kuno Type:

$$a = \{ [IT^{0.5}] [I^2] - [I^2T^{0.5}] - [I^2T^{0.5}] [I] \} / \{N [I^2] - [I] [I]\}$$
$$b = \{ [I] [IT^{0.5}] - N [I^2T^{0.5}] \} / \{N [I^2] - [I] [I]\}$$

Where; [ ] : Total of each indicated index (for example; $[I] = \Sigma I$)
N : Number of samples
I : Probable Rainfall intensity
T : Rainfall duration time

The rainfall intensities estimated through the above three (3) formulas (Eq. (3.5) to (3.7)) are compared with those as estimated in the above sub-section 3.2.3, and the most conformable formula is selected. Based on the selected formula, the rainfall-intensity curve is developed.

Reference:

The probable point rainfall intensities were estimated through the formula of Gumbel distribution (Eq. 3.1 and Eq. 3.2), as an example, on the basis of the annual maximum rainfall intensities of 15 minutes to 12 hours observed at Bayan Lepas, Penang (MMS gauging station) as shown in Table 3.1 and Fig. 3.3. Based on these probable point rainfall intensities, the constants of the three (3) formulas (Eq. 3.5 to 3.7) could be estimated as shown in Table 3.2 and the rainfall intensity-duration curves are estimated as shown in Table 3.3. It is finally concluded that the formula of Talbot type could give the most conformable values as shown in Table 3.3.

Table 3.1 Probable Rainfall Intensities Estimated by Formula of Gumbel Distribution (at Bayan Lepas, Penang)

(Unit : mm/hour)

Return Period	Rainfall Duration 15 min	30 min	45 min	1 hr	2 hr	3 hr	4 hr	5 hr	6 hr	12 hr
2	129.2	102.6	84.5	71.7	44.0	31.0	24.6	20.2	17.3	9.4
3	140.4	110.0	91.5	78.7	49.0	34.6	27.3	22.3	19.1	10.6
5	152.4	118.4	99.2	86.6	54.6	38.6	30.3	24.6	21.1	12.0
8	163.2	125.4	105.9	93.3	59.4	42.1	32.8	26.6	22.8	13.1
10	168.0	128.6	109.1	96.4	61.6	43.7	34.0	27.6	23.6	13.6
20	182.8	138.6	118.4	105.9	68.3	48.5	37.5	30.4	25.9	15.3
25	187.6	141.8	121.3	108.9	70.5	50.1	38.7	31.3	26.7	15.8
30	191.6	144.4	123.7	111.3	72.3	51.3	39.5	32.0	27.3	16.2
50	202.0	151.6	130.5	118.1	77.1	54.8	42.1	34.0	29.0	17.3
100	216.4	161.2	139.6	127.3	83.6	59.5	45.6	36.7	31.3	18.9
200	230.8	170.8	148.7	136.5	90.2	64.2	49.0	39.4	33.6	20.4

Table 3.2 Constants Developed for Rainfall Intensity-duration Curves (at Bayan Lepas, Penang)

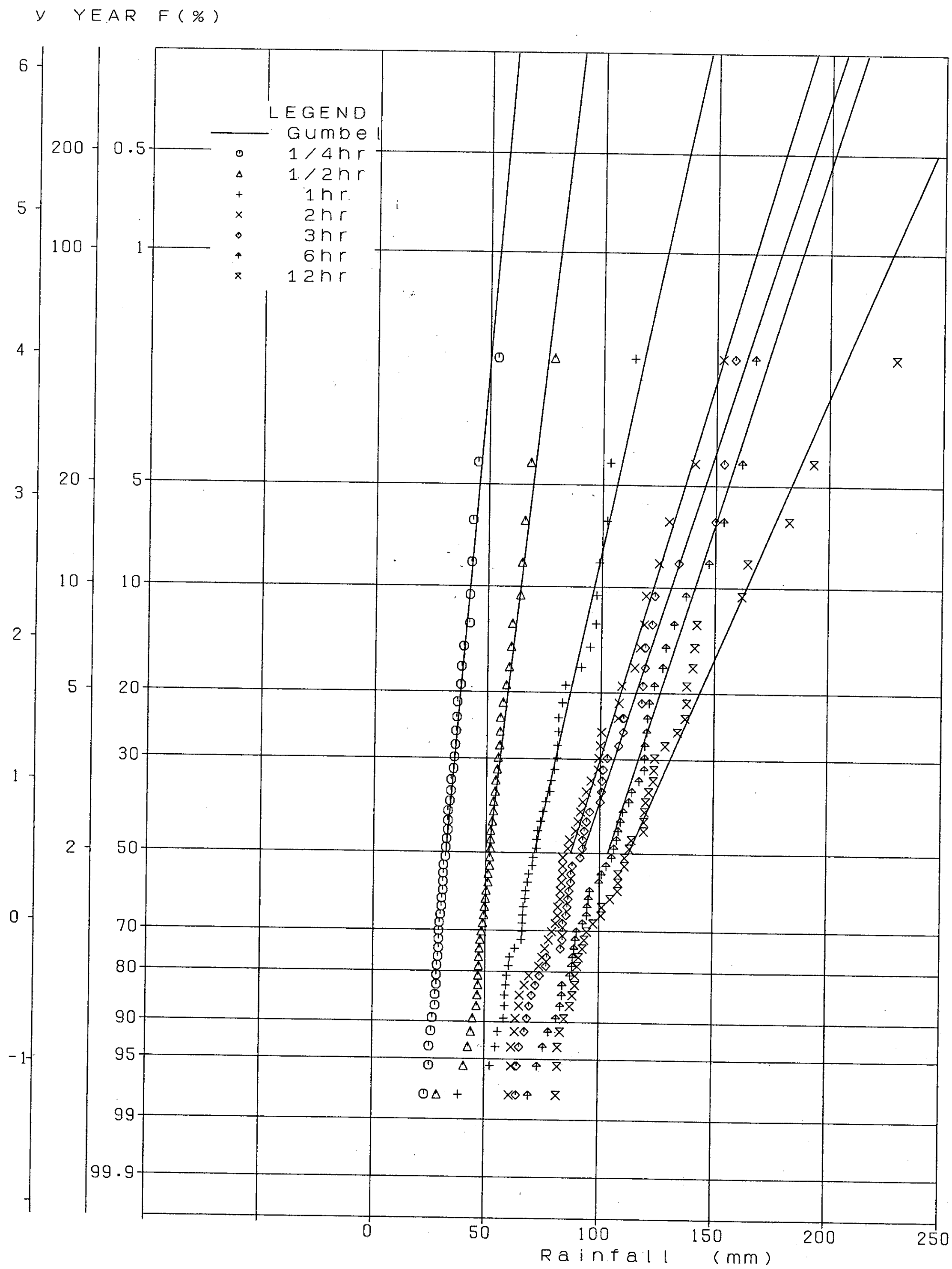
Return Period	(1) Talbot Type	(2) Sherman Type	(3) Kuno Type
2	I= 6914.21/(T+37.80)	$I= 1088.20/T^{0.70}$	$I= 320.89/(T^{0.5}-2.22)$
3	I= 7744.47/(T+40.00)	$I= 1140.26/T^{0.69}$	$I= 358.41/(T^{0.5}-2.13)$
5	I= 8684.35/(T+42.27)	$I= 1195.64/T^{0.68}$	$I= 400.43/(T^{0.5}-2.05)$
8	I= 9466.34/(T+43.66)	$I= 1252.50/T^{0.67}$	$I= 435.83/(T^{0.5}-1.99)$
10	I= 9837.21/(T+44.34)	$I= 1277.30/T^{0.67}$	$I= 452.50/(T^{0.5}-1.96)$
20	I= 10956.11/(T+46.13)	$I= 1351.46/T^{0.66}$	$I= 502.55/(T^{0.5}-1.90)$
25	I= 11319.69/(T+46.67)	$I= 1376.00/T^{0.66}$	$I= 518.95/(T^{0.5}-1.88)$
30	I= 11590.83/(T+46.91)	$I= 1399.13/T^{0.66}$	$I= 531.24/(T^{0.5}-1.87)$
50	I= 12384.09/(T+47.86)	$I= 1457.29/T^{0.65}$	$I= 566.79/(T^{0.5}-1.83)$
100	I= 13474.26/(T+49.09)	$I= 1532.49/T^{0.65}$	$I= 615.58/(T^{0.5}-1.79)$
200	I= 14539.68/(T+50.01)	$I= 1613.28/T^{0.64}$	$I= 663.43/(T^{0.5}-1.75)$

Note : I=Rainfall Intensity (mm/hour)
T=Rainfall Duration (minute)

Table 3.3 Conformity of Rainfall Intensity-duration Curves (at Bayan Lepas, Penang)

Return Period	Rainfall Duration (min)	Rainfall Intensities Estimated From (1) Observed Data (mm/hr)	(2) Eq. of Talbot (mm/hr)	(3) Eq. of Sherman (mm/hr)	(4) Eq. of Kuno (mm/hr)	Difference of Rainfall Intensities (1)-(2) (mm)	(1)-(3) (mm)	(1)-(4) (mm)
2	15	129.2	131.0	164.8	193.8	1.8	35.6	64.6
	30	102.6	102.0	101.7	98.4	0.6	0.9	4.2
	45	84.5	83.5	76.6	71.4	1.0	7.9	13.1
	60	71.7	70.7	62.7	58.0	1.0	9.0	13.7
	120	44.0	43.8	38.7	36.7	0.2	5.3	7.3
	180	31.0	31.7	29.2	28.7	0.7	1.8	2.3
	240	24.6	24.9	23.9	24.2	0.3	0.7	0.4
	300	20.2	20.5	20.4	21.2	0.3	0.2	1.0
	360	17.3	17.4	18.0	19.2	0.1	0.7	1.9
	720	9.4	9.1	11.1	13.0	0.3	1.7	3.6
					Total	6.2	63.9	112.0
					Average	0.6	6.4	11.2

Fig. 3.3 Gumbel Distribution of Annual Maximum Rainfall at Bayan Lepas, Penang



3.2.5 Conversion of Point Rainfall Intensity to Areal Average Rainfall Intensity

The point rainfall intensity should be converted to areal average rainfall with using the areal reduction factors.

Explanation:

The rainfall intensity-duration curves described in the foregoing sub-section 3.2.3 are for the relationship between the point rainfall intensity and its time duration. During a storm, however, rainfall is distributed unevenly over the catchment area, and tends to decrease with the storm center. That is, as the coverage of a drainage area increases, the areal average rainfall tends to reduces from the value of point rainfall. From this view of point, the point rainfall needs to be converted to the areal average rainfall.

Should the short term rainfall gauging data be given from the numerous gauging stations located in the objective drainage area, the areal average rainfall intensities could be estimated through either "Thiessen Polygon Method", "Arithmetic Mean Method" or "Isohyetal Method". It is, however, virtually, difficult to obtain such numerous short term gauging data, and therefore, recommended to adopt the interpolation of the following conversion factors for point rainfall intensity to areal average rainfall intensity as presented in the "Hydrological Procedure No. 1" by DID*[3-3]. The conversion factors are based on the U.S. Weather Bureau recommendations.

Table 3.4 Conversion Factors of Point Rainfall to Areal Average Rainfall

Catchment Area (km2)	Rainfall Duration (hours)				
	1/2	1	3	6	24
0	1.00	1.00	1.00	1.00	1.00
50	0.83	0.88	0.94	0.96	0.97
100	0.73	0.82	0.91	0.94	0.96
150	0.67	0.78	0.89	0.92	0.95
200	0.63	0.75	0.87	0.90	0.93
250	0.61	0.73	0.85	0.89	0.93
300	0.59	0.71	0.84	0.88	0.93
400	0.58	0.68	0.81	0.86	0.92
500		0.67	0.80	0.85	0.92
600		0.66	0.79	0.84	0.91
800		0.65	0.78	0.83	0.91
1,000			0.78	0.83	0.91

Source: "Hydrological Procedure No.1: Estimation of the Design Rainstorm in Peninsular Malaysia" published by DID

3.2.6 Development of Design Hyetograph

The design hyetograph of the design return period for the objective urban drainage improvement plan should be developed in order to simulate the design flood discharge hydrograph.

Explanation:

The design hyetograph should be developed taking the following factors into consideration:

(1) Entire Rainstorm Duration

The entire time length of design hyetograph should be determined on the basis of the observed rainfall depth described in the foregoing sub-section 3.2.1.

(2) Time Interval

The flood concentration time (rounded minutes) should be adopted as the time interval of the design hyetograph. The detailed methodology for estimation of the flood concentration time is as described in sub-section 3.3.4.

(3) Temporal Variation of Rainfall Depth

The most critical temporal variation of rainfall intensity in one sequence of rainfall storm is such that the rainfall intensity gradually increases and reaches to the peak intensity at the end of rainfall. Such temporal rainfall pattern requires the largest flood detention volume. In this connection, the actual temporal variations of the observed rainstorm should be examined. As the results, if the substantial parts of the observed rainstorm are confirmed to have such critical temporal variation, the design hyetograph should apply the variation. However, when non of the observed rainstorm has the critical variation, the dominant variation could be applied to the design hyetograph.

The design hyetograph could be developed with using the rainfall intensity-duration curve and assuming above three (3) factors. The following shows the process of development of design hyetograph which has the peak rainfall intensity at the end of rainstorm.

Rainfall Depth Distribution at Time Interval of "dt"		Time of Occurrence
(a) Highest	$= I(dt) \times dt/60$	Tp (end of rainstorm)
(b) 2nd highest	$= I(dt \times 2) \times dt \times 2/60 - I(dt) \times dt/60$	$Tp - dt$
(c) 3rd highest	$= I(dt \times 3) \times dt \times 3/60 - I(dt \times 2) \times dt \times 2/60$	$Tp - dt \times 2$
(d) 4th highest	$= I(dt \times 4) \times dt \times 4/60 - I(dt \times 3) \times dt \times 3/60$	$Tp - dt \times 3$
	•	•
	•	•
	•	•
	•	•
	•	•

Where; I : Rainfall intensity expressed by the function of rainfall duration ($dt \times 1, 2, 3 \ldots$) (estimated from the rainfall intensity-duration curve)
dt : Time interval of design hyetograph
Tp : Time end of entire rainstorm duration

3.3 Flood Runoff Analysis

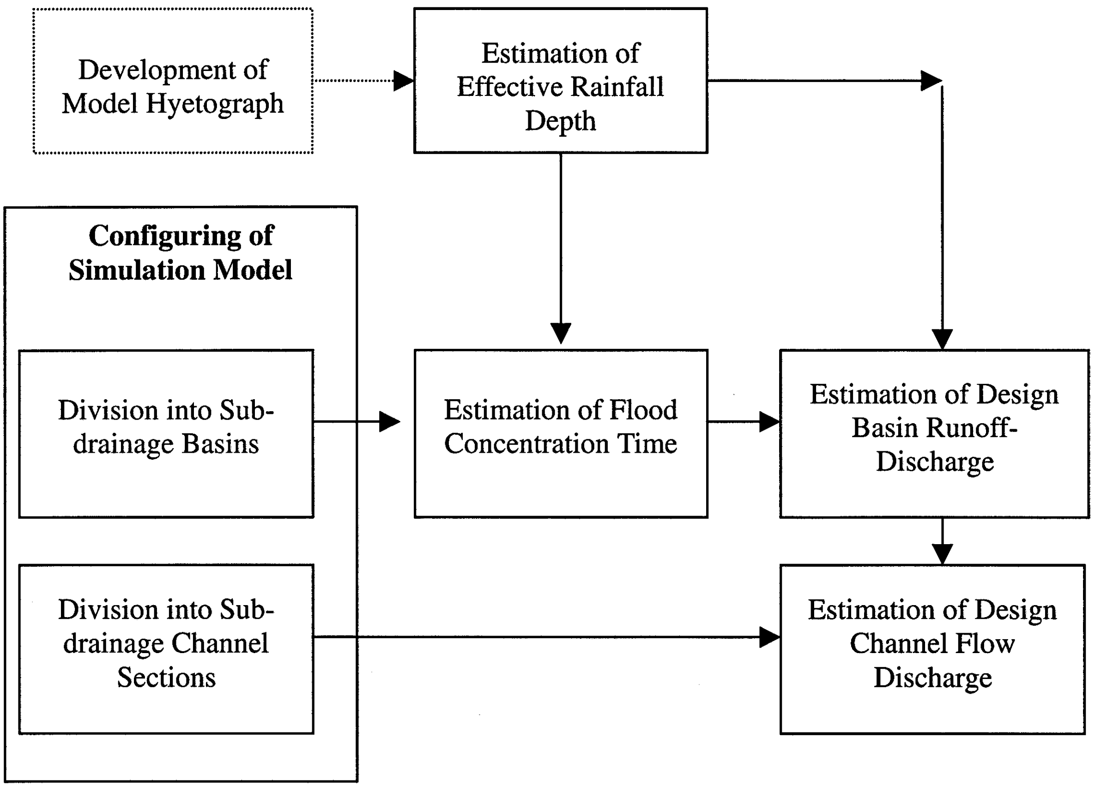
3.3.1 Purpose of Analysis

The analysis aims at estimating the discharge hydrograph of the standard project flood at every design control point and at every design object facilities. The standard project flood is, herein defined as the probable flood discharge which corresponds to the target design level, and subject to no effect by any proposes drainage facilities.

Explanation:

The standard project flood is essential to determine the design flow capacity of drainage channel and the design storage capacity of various flood detention and retention facilities. This sub-section prescribes the methodology for estimation on the discharge hydrograph of the standard project flood. The procedure for the estimation is as shown in Fig. 3.4.

Fig. 3.4 Procedure to Estimate Design Flood Hydrograph



3.3.2 Division to Sub-drainage Basins and Sub-drainage Channel Sections

The objective drainage area should be divided into sub-drainage basins, each of which has a size suitable for the basin runoff simulation model. In accordance with the configuration of the divided sub-basins, the drainage channels should be also divided into sub-drainage channel sections.

Explanation:

In order to estimate the standard project flood at every design control point and at every design object facilities, the objective drainage area is divided into several sub-drainage basins. The boundary of drainage area should be determined in due consideration of the design control point and the design object facilities, and the topographic conditions (such as ground slope and protuberances). In accordance with the division of drainage areas, the drainage channels and/or river channels in the drainage areas should be also divided into sub-channel sections, accordingly.

Reference:

The flood runoff simulation model could be configured by composition of the divided sub-basins and sub-drainage channels. In the "Study on Integrated Drainage Plan for Melaka and Sungai Petani" by JICA, as an example, the simulation model for the drainage area of Sungai Petani located in State of Kedah (catchment area of 38km^2) was configured as shown in Figs. 3.4 and 3.5. In the simulation model, the drainage area was divided into 34 sub-drainage basins, and the drainage channels was also divided into 18 sub-drainage channel sections.

Fig. 3.5 Example of Division of A Drainage Basin to Sub-drainage Basins (Sungai Petani Drainage Area)

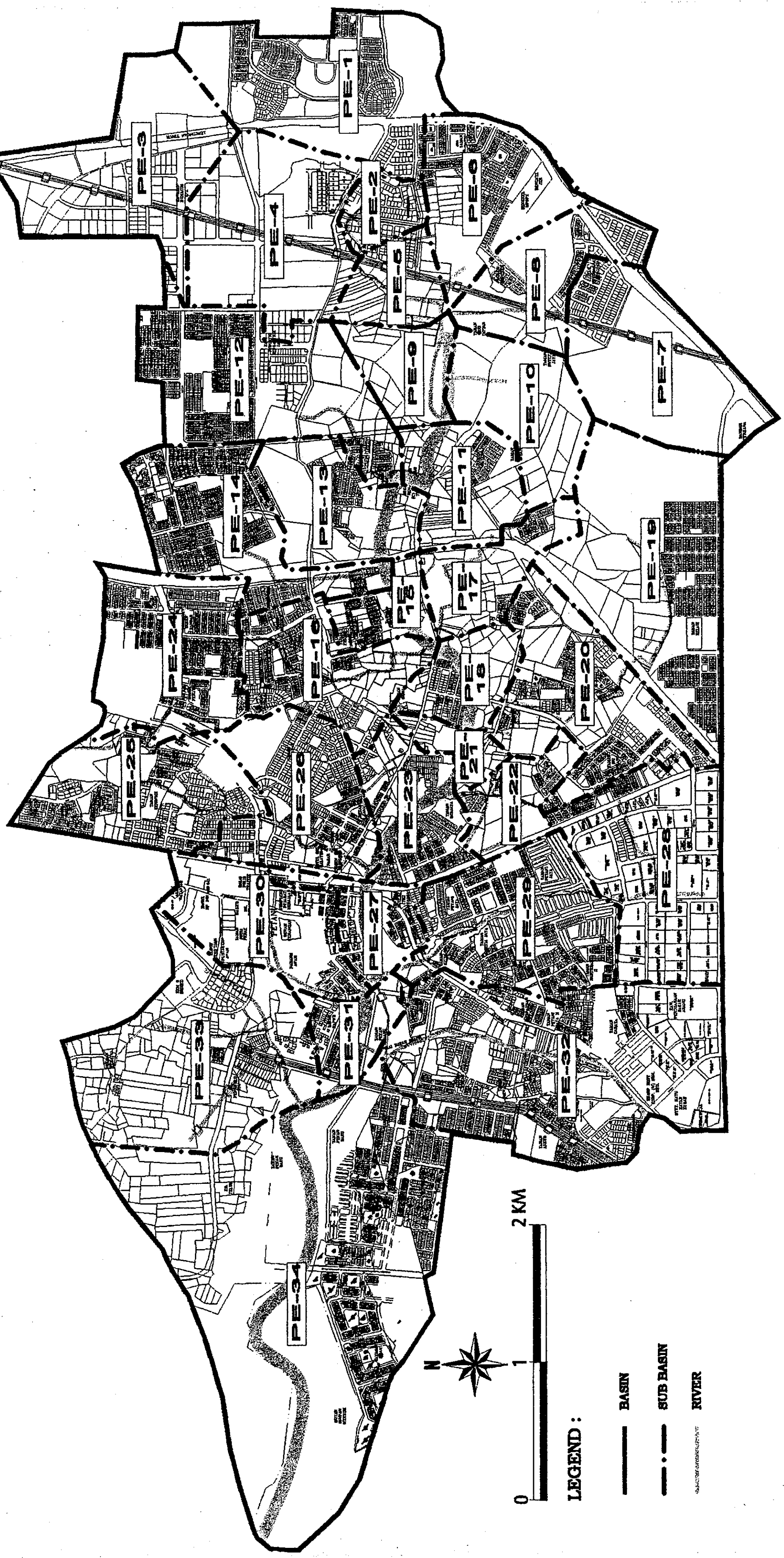


Fig. 3.6 Example of Flood Runoff Simulation Model
(Sungai Petani Drainage Area)