CHAPTER 2 QUANTITY DESIGN FUNDAMENTALS

2.1 INTRODUCTION

This chapter provides the fundamentals and appropriate methods required for designing stormwater quantity facilities. They apply to primarily detention pond as well as conveyance. The design ARIs for various types of facilities vary while the critical storm duration might be different due to differing facility operational concepts and mechanisms.

2.2 RAINFALL ESTIMATION

Rainfall data and characteristics are the driving force behind all stormwater studies and designs. Adequacy and significance of the rainfall design is a necessary pre-requisite for preparing satisfactory urban drainage and stormwater management projects. The estimation involves frequency, duration and intensity analyses of rainfall data.

2.2.1 Average Recurrence Interval

Rainfall and subsequent discharge estimate is based on the selected value of frequency or return period, termed as the Average Recurrence Interval (ARI) which is used throughout this Manual. ARI is the average length of time between rain events that exceeds the same magnitude, volume or duration (Chow, 1964), and is expressed as:

$$T_r = \frac{1}{P} \cdot 100 \qquad (2.1)$$

where,

T_r = Average Recurrence Interval, ARI (year); and

P = Annual Exceedance Probability, AEP (%).

As an example, using Equation 2.1, 1% AEP of storm has an ARI of 100 years. According to the definition, a 100 year ARI storm can occur in any year with a probability of 1/100 or 0.01.

The design ARI of a stormwater facility is selected on the basis of economy and level of protection (risk) that the facility offers. ARIs to be used for the design of minor and major stormwater quantity systems are provided in Table 1.1. It is assumed that the design flow of a given ARI is produced by a design storm rainfall of the same ARI. Design rainfall intensity (mm/hr) depends on duration (minute or hour) and ARI (month or year). It is strongly recommended that performance of the designed drainage system must be examined for a range of ARIs and storm durations to ensure that the system(s) will perform satisfactorily.

2.2.2 Time of Concentration

Time of concentration (t_c) is the travel time of runoff flows from the most hydraulically remote point upstream in the contributing catchment area to the point under consideration downstream. The concept of time of concentration is important in all methods of peak flow estimation as it can be assumed that the rainfall occurring during the time of concentration is directly related to peak flow rate. The practice is to select the design storm duration as equal to or greater than the time of concentration (t_c)

In the design of stormwater drainage systems, t_c is the sum of the overland flow time (t_o) and the time of travel in street gutters (t_g), or roadside swales, drains, channels and small streams (t_d). The relevant equations necessary to calculate the t_c is given in Table 2.1 (QUDM, 2007). Calculation of t_c is subject to the catchment properties, particularly length, slope and roughness of the drainage path. The overland flow time t_o can be estimated with proper judgment of the land surface condition due to the fact that the length of sheet flow is short for steep slopes and long for mild slopes. This equation shall be applied only for distances (L) recommended in Table 2.1. Catchment roughness, length and slope affect the flow velocity and subsequently overland flow time t_o. Typical values of Horton's roughness n^* for various land surfaces are given in Table 2.2 (QUDM, 2007). Alternatively, the overland flow time can easily be estimated using the Design Chart 2.A1.

The drain flow time equation should be used to estimate t_d for the remaining length of the flow paths downstream. Care should be given to obtain the values of hydraulic radius and friction slope for use in the drain flow time equation. Note that recommended minimum time of concentration for a catchment is 5 minutes which applies to roof drainage.

Table 2.1: Equations to Estimate Time of Concentration (QUDM, 2007)

Travel Path	Travel Time	Remark
Overland Flow	$t_o = \frac{107.n^*.L^{1/3}}{S^{1/5}}$	t_o = Overland sheet flow travel time (minutes) L = Overland sheet flow path length (m) *for Steep Slope (>10%), L ≤ 50 m* *for Moderate Slope (<5%), L ≤ 100 m* *for Mild Slope (<1%), L ≤ 200 m* n^* = Horton's roughness value for the surface (Table 2.2) S = Slope of overland surface (%)
Curb Gutter Flow	$t_g = \frac{L}{40\sqrt{S}}$	t_g = Curb gutter flow time (minutes) L = Length of curb gutter flow (m) S = Longitudinal slope of the curb gutter (%)
Drain Flow	$t_d = \frac{n.L}{60R^{2/3}S^{1/2}}$	n = Manning's roughness coefficient (Table 2.3) R = Hydraulic radius (m) S = Friction slope (m/m) L = Length of reach (m) t_d = Travel time in the drain (minutes)

Table 2.2: Values of Horton's Roughness n^* (QUDM, 2007)

Land Surface	Horton's Roughness n^*
Paved	0.015
Bare Soil	0.0275
Poorly Grassed	0.035
Average Grassed	0.045
Densely Grassed	0.060

2.2.3 Design Rainfall Estimate

2.2.3.1 Intensity-Duration-Frequency Curves Development

The most common form of design rainfall data required for use in peak discharge estimation is from relationship represented by the intensity-duration-frequency (IDF) curves. The IDF can be developed from the historical rainfall data and they are available for most geographical areas in Malaysia.

Recognising that the rainfall data used to derive IDF are subjected to some interpolation and smoothing, it is desirable to develop IDF curves directly from local raingauge records, if these records are sufficiently long and reliable. The IDF development procedures involve the steps shown in Figure 2.1 while a typical developed curves are shown in Figure 2.2.

Table 2.3: Values of Manning's Roughness Coefficient (n) for Open Drains and Pipes (Chow, 1959; DID, 2000 and French, 1985)

Drain/Pipe	Manning Roughness n
Grassed Drain	
Short Grass Cover (< 150 mm)	0.035
Tall Grass Cover (≥ 150 mm)	0.050
Lined Drain	
Concrete	
Smooth Finish	0.015
Rough Finish	0.018
Stone Pitching	
Dressed Stone in Mortar	0.017
Random Stones in Mortar or Rubble Masonry	0.035
Rock Riprap	0.030
Brickwork	0.020
Pipe Material	
Vitrified Clay	0.012
Spun Precast Concrete	0.013
Fibre Reinforced Cement	0.013
UPVC	0.011

2.2.3.2 Empirical IDF Curves

Empirical equation can be used to minimise error in estimating the rainfall intensity values from the IDF curves. It is expressed as

$$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}} \tag{2.2}$$

where,

i = Average rainfall intensity (mm/hr);

T = Average recurrence interval - ARI ($0.5 \le T \le 12$ month and $2 \le T \le 100$ year);

d = Storm duration (hours), $0.0833 \le d \le 72$; and

λ, κ, θ *and* η = Fitting constants dependent on the raingauge location (Table 2.B1 in Appendix 2.B).

The equation application is simple when analysis is prepared by spreadsheet. Alternatively designers can manually use the IDF curves provided in Annexure 3.

2.2.4 Temporal Patterns

It is important to emphasise that the rainfall temporal patterns are intended for use in hydrograph generation *design* storms. They should not be confused with the real rainfall data in historical storms, which is usually required to calibrate and validate hydrological and hydraulic simulation results.

The standard time intervals recommended for urban stormwater modelling are listed in Table 2.4. The design temporal patterns to be used for a set of durations are given in Appendix 2.C.

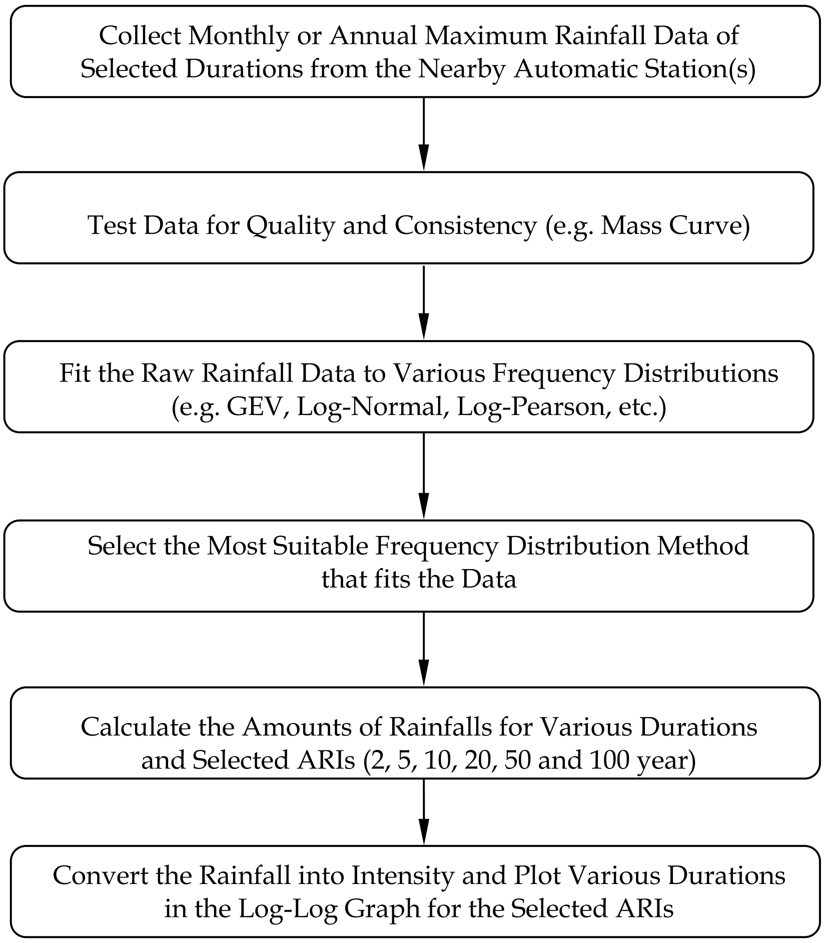


Figure 2.1: Typical Steps to Develop IDF Curves

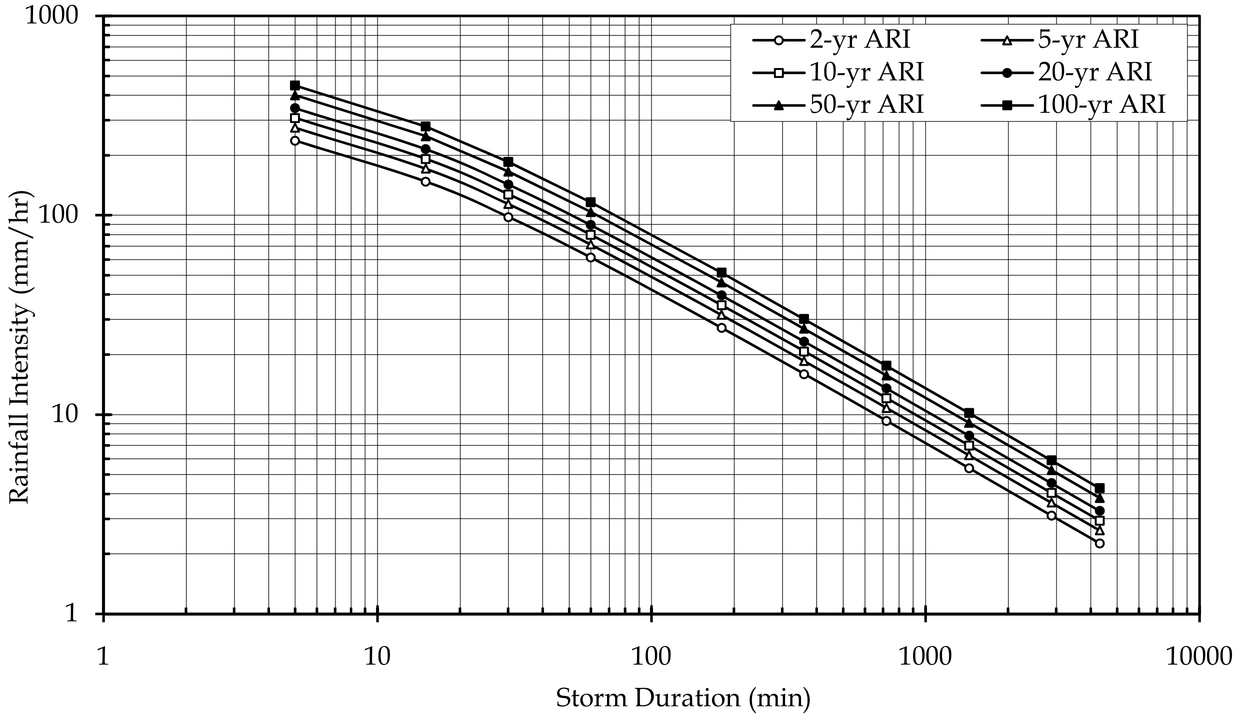


Figure 2.2: Typical IDF Curve

If data available, it is recommended to derive the temporal patterns using the local data following the example given in Appendix 2.D. For other durations, the temporal pattern for the nearest standard duration should be adopted. It is *NOT* correct to average the temporal patterns for different durations.

Table 2.4: Recommended Intervals for Design Rainfall Temporal Pattern

Storm Duration (minutes)	Time Interval (minutes)
Less than 60	5
60 - 120	10
121 - 360	15
Greater than 360	30

Various methods can be used to develop design rainfall temporal pattern. However, it is most important to note that design patterns are not derived from complete storms, but from intense bursts of recorded rainfall data for the selected durations. The method described herein incorporates the average variability of recorded intense rainfalls and also the most likely sequence of intensities. The highest rainfall bursts of selected design storm durations are collected from the rainfall record. It is desirable to have a large number of samples. The duration is then divided into a number of equal time intervals, as given in Table 2.4. The intervals for each rainfall burst are ranked and the average rank is determined for the intervals having same rainfall amount. The percentage of rainfall is determined for each rank for each rainfall burst, and the average percentage per rank is calculated. This procedure is then repeated for other durations. The procedure involves the steps as shown in Figure 2.3.

2.3 PEAK DISCHARGE ESTIMATION

This Section presents the methods and procedures required for runoff estimation. The recommended methods are the Rational Method and Hydrograph Methods. Each method has its own merits. *A simple Rational Hydrograph Method (RHM) is recommended for the design of small storage facilities.*

2.3.1 Rational Method

The Rational Method is the most frequently used technique for runoff peak estimation in Malaysia and many parts of the world. It gives satisfactory results for small drainage catchments and is expressed as:

$$Q = \frac{C.i.A}{360} \tag{2.3}$$

where,

Q = Peak flow (m^3/s);
C = Runoff coefficient (Table 2.5);
i = Average rainfall intensity (mm/hr); and
A = Drainage area (ha).

The primary attraction of the Rational Method has been its simplicity. However, now that computerised procedures for hydrograph generation are readily available, making computation/design by computerised method or software is also simple.

The most critical part of using the Rational Method is to make a good estimate of the runoff coefficient *C*. In general, the values of *C* depend mainly on landuse of the catchment and is very close to its imperviousness (in decimal form). The value of *C* also varies with soil type, soil moisture condition, rainfall intensity, etc. The user should evaluate the actual catchment condition for a logical value of *C* to be used. For larger area with high spatial variabilities in landuse and other parameters, this can easily be done by the use of AutoCAD, GIS or other computer softwares.

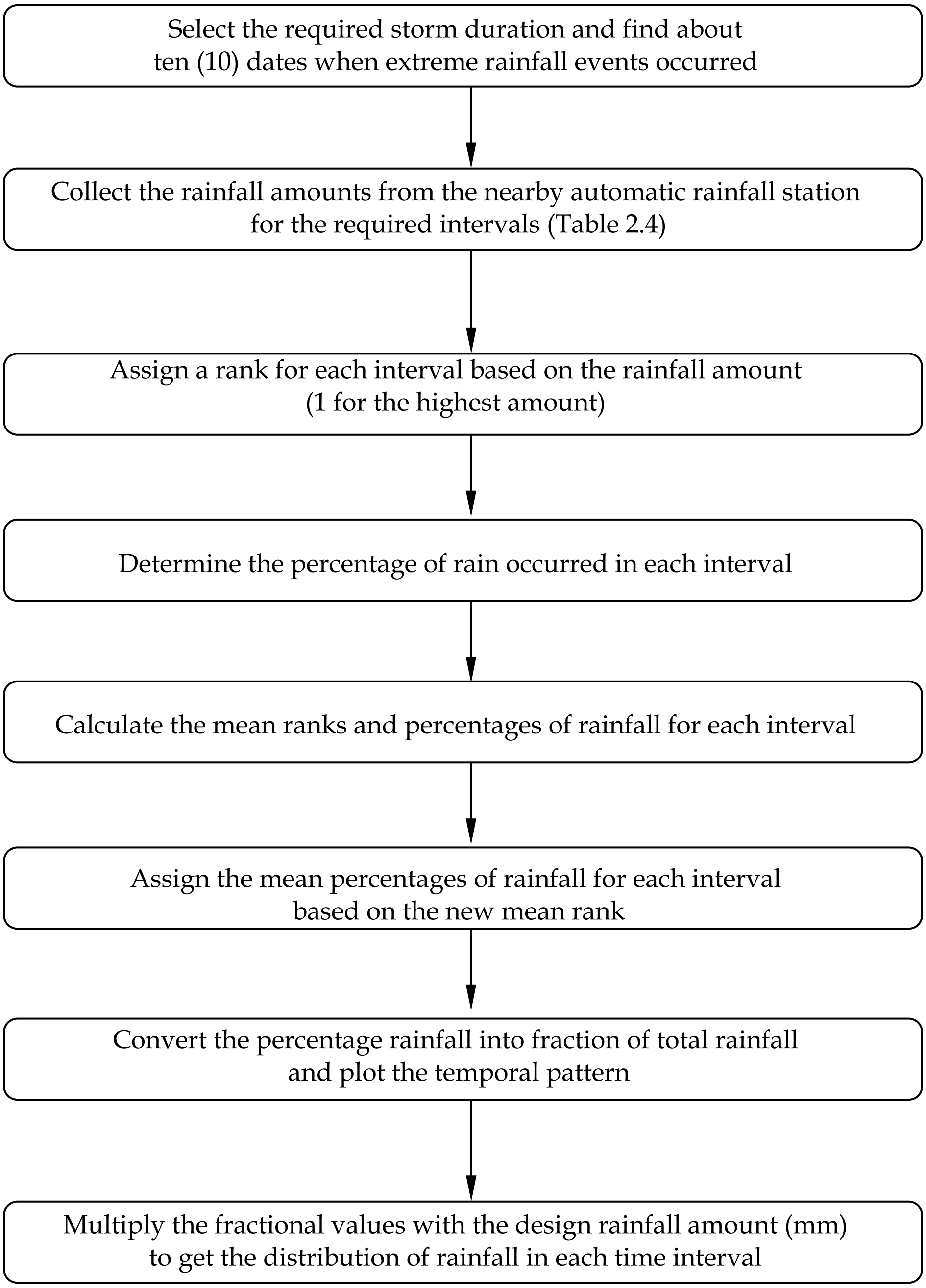


Figure 2.3: Typical Steps for the Development of Design Rainfall Temporal Pattern

2.3.1.1 Runoff Coefficient for Mixed Development

Segments of different landuse within a sub-catchment can be combined to produce an average runoff coefficient (Equation 2.4). For example, if a sub-catchment consists of segments with different landuse denoted by j = 1, 2,……, m; the average runoff coefficient is estimated, C, by:

$$C_{avg} = \frac{\sum_{j=1}^{m} C_j A_j}{\sum_{j=1}^{m} A_j} \tag{2.4}$$

where,

C_{avg} = Average runoff coefficient;
C_j = Runoff coefficient of segment *i;*
A_j = Area of segment *i* (ha); and
m = Total number of segments.

Table 2.5: Recommended Runoff Coefficients for Various Landuses
(DID, 1980; Chow et al., 1988; QUDM, 2007 and Darwin Harbour, 2009)

Landuse	Runoff Coefficient (C)	
	For Minor System (≤10 year ARI)	For Major System (> 10 year ARI)
Residential		
Bungalow	0.65	0.70
Semi-detached Bungalow	0.70	0.75
Link and Terrace House	0.80	0.90
Flat and Apartment	0.80	0.85
Condominium	0.75	0.80
Commercial and Business Centres	0.90	0.95
Industrial	0.90	0.95
Sport Fields, Park and Agriculture	0.30	0.40
Open Spaces		
Bare Soil (No Cover)	0.50	0.60
Grass Cover	0.40	0.50
Bush Cover	0.35	0.45
Forest Cover	0.30	0.40
Roads and Highways	0.95	0.95
Water Body (Pond)		
Detention Pond (with outlet)	0.95	0.95
Retention Pond (no outlet)	0.00	0.00

Note: The runoff coefficients in this table are given as a guide for designers. The near-field runoff coefficient for any single or mixed landuse should be determined based on the imperviousness of the area.

2.3.1.2 Assumptions

Assumptions used in the Rational Method are as follows:

- The peak flow occurs when the entire catchment is contributing to the flow;
- The rainfall intensity is the uniform over the entire catchment area; and
- The rainfall intensity is uniform over a time duration equal to the time of concentration, t_c.

The Rational Method is *not recommended* for use where:

- The catchment area is greater than 80 ha (TxDOT, 2009);
- Ponding of stormwater in the catchment might affect peak discharge; and
- The design and operation of large and more costly drainage facilities are to be undertaken, particularly if they involve storage.

2.3.1.3 Calculation Steps

Steps for estimating a peak flow from a single sub-catchment for a particular ARI using the Rational Method are outlined in Figure 2.4.

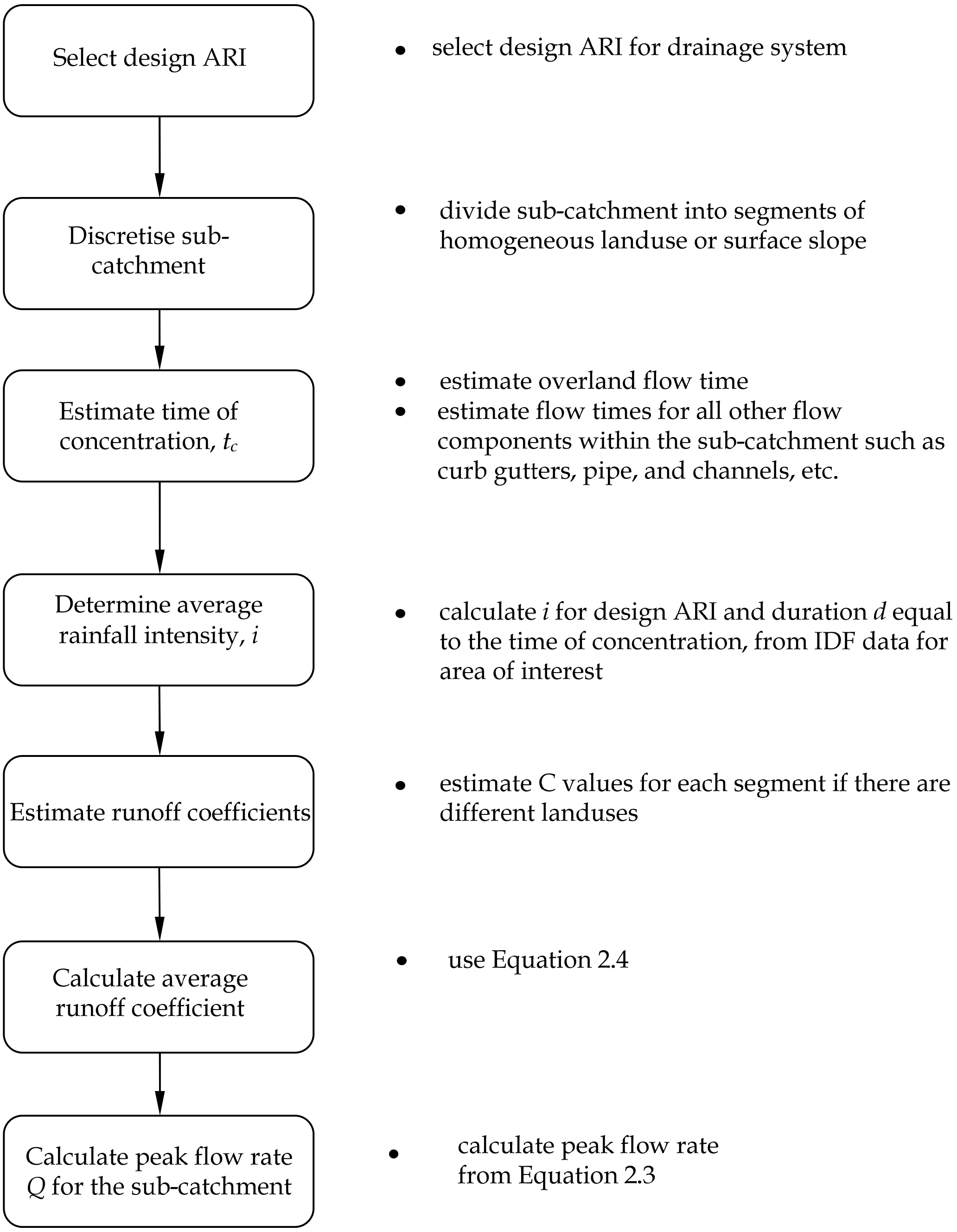


Figure 2.4: General Procedure for Estimating Peak Flow Using the Rational Method (DID, 2000)

2.3.2 Rational Hydrograph Method

This procedure, Rational Hydrograph Method (RHM), extends the Rational Method to the development of runoff hydrographs. For simplicity, this method is recommended for the deriving inflow hydrograph on-site detention (OSD) and small detention pond. However, for complex drainage system and high risk areas, the Time Area Method in Section 2.2.3 or computer models should be used for obtaining the inflow hydrograph.

As illustrated in Figure 2.5, two types of hydrographs are to be used for the sub-catchment using the RHM procedure. Each hydrograph type is a function of the length of the rainfall averaging time, *d*, with respect to the sub-catchment time of concentration, t_c.

Type 1 (d is greater than t_c): The resulting trapezoidal hydrograph has a uniform maximum discharge Q, as determined from the Rational Method. The linear rising and falling limbs each has a duration of t_c.

Type 2 (d is equal to t_c): The resulting triangular hydrograph has a peak discharge Q. The linear rising and falling limbs each have a duration of t_c.

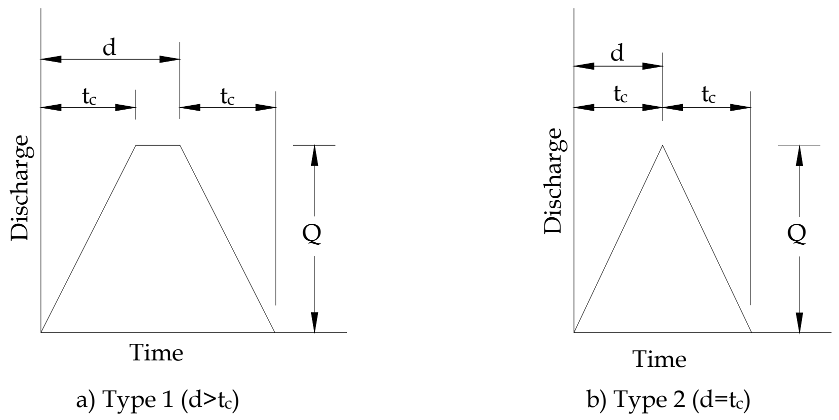


Figure 2.5: Hydrograph Types of the RHM

In summary, hydrograph type in the RHM is determined by the relationship between rainfall duration and the time of concentration of the sub-catchment. Given the hydrograph type, the peak discharge is determined using the Rational Method (Equation 2.3).

2.3.3 Time Area Hydrograph Method

2.3.3.1 Concept

This method assumes that the outflow hydrograph for any storm is characterised by separable subcatchment translation and storage effects. Pure translation of the direct runoff to the outlet via the drainage network is described using the channel travel time, resulting in an outflow hydrograph that ignores storage effects.

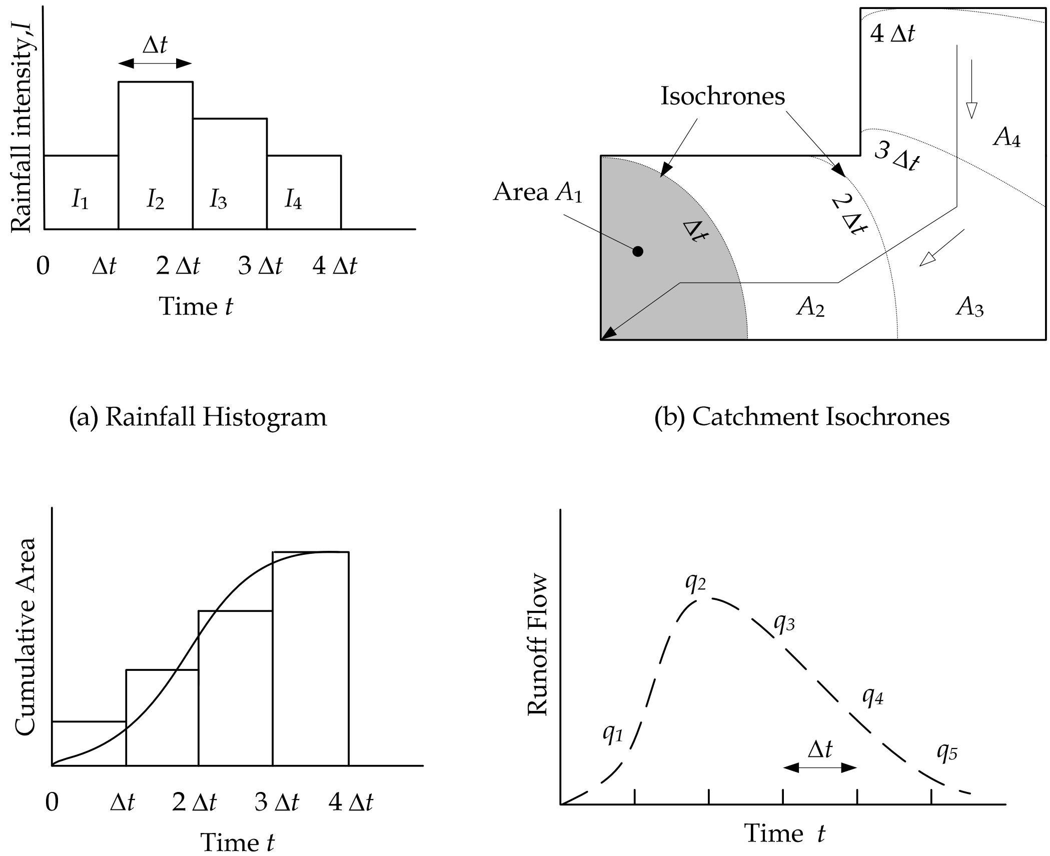
To apply the method, the catchment is first divided into a number of isochrones or lines of equal travel time to the outlet (Figure 2.6b). The areas between isochrones are then determined and plotted against the travel time as shown in Figure 2.6c. Derivation of isochrones is crucial and is illustrated in a worked example in Appendix 2.E2. The translated inflow hydrograph ordinates q_i (Figure 2.6d) for any selected design hyetograph can now be determined. Each block of storm, Figure 2.6a, should be applied (after deducting losses) to the entire catchment; the runoff from each sub-area reaches the outflow at lagged intervals defined by the time-area histogram. The simultaneous arrival of the runoff from areas A_1, A_2,…for storms I_1, I_2,…should be determined by properly lagging and adding contributions, or generally expressed as:

$$q_j = I_j . A_1 + I_{j-1} . A_2 + ... + I_1 . A_j \tag{2.5}$$

where,

q_j = Flow hydrograph ordinates (m³/s);
I_j = Rainfall excess hyetograph ordinates (m/s);
A_j = Time-area histogram ordinates (m²); and
j = Number of isochrone contributing to the outlet.

As an example for j = 3, the runoff from storms I_1 on A_3, I_2 on A_2 and I_3 on A_1 arrive at the outlet simultaneously, and q_3 is the total flow. The inflow hydrograph (Figure 2.6d) at the outlet can be obtained using Equation 2.5.



(a) Rainfall Histogram

(b) Catchment Isochrones

(c) Time-Area Curve

(d) Runoff Hydrograph

Figure 2.6: Time–Area Hydrograph Method

2.3.3.2 Rainfall Excess

Total Rainfall should be deducted by losses, initial or continuous, to calculate the rainfall excess (RE), which will result in the surface runoff hydrograph. The rainfall losses can be assumed constant (for simplicity) or decaying (to be more practical), as shown in Figure 2.7. The parameter values are given in Table 2.6.

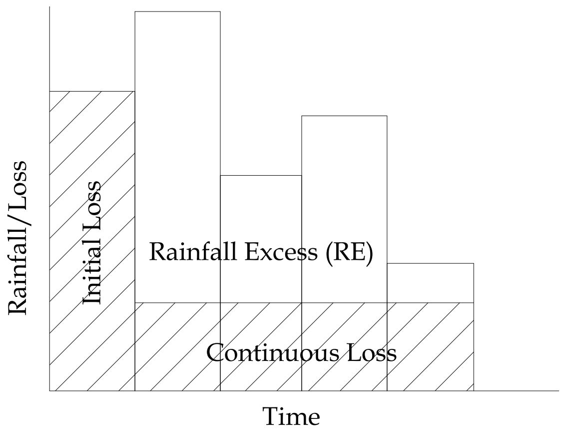


Figure 2.7: Initial and Continuous Loss Concept for Runoff Estimation

Table 2.6: Recommended Loss Values for Rainfall Excess Estimation (Chow et al., 1988)

Catchment Condition	Initial Loss (mm)	Continuous Loss (mm/hr)
Impervious	1.5	0
Pervious	10	(i) Sandy Soil: 10 - 25 mm/hr (ii) Loam Soil: 3 - 10 mm/hr (iii) Clay Soil: 0.5 - 3 mm/hr

2.3.4 Computer Software Application

Various types of simple and complex computer software (models) are available to simulate the runoff peak flow or hydrograph. Prudent use of such softwares can provide more flexibility and opportunity to estimate the runoff hydrograph and volume taking consideration of the variability in rainfall and catchment properties. Wherever and whenever possible, designers should use computer softwares to design and analyse stormwater management component or the whole system train, for more scenarios and reliability at reasonable cost.

Three types of computer methods might be considered, they are:

- Spreadsheets that can be used to implement all of the methods described in this chapter;
- Public domain softwares, such as SWMM-5, RORB and HEC-RAS; and
- Commercial softwares.

All runoff estimation methods will give different peak flow rates. The most practical way to minimise the variations is by calibrating and validating against the recorded rainfall and runoff data.

2.4 OUTFLOW CONTROL

Orifices and weirs outlet are typically used as outlet control structures for ponds and their characteristics must be specified when performing reservoir routing calculations. The relevant equations are given in the following sections.

2.4.1 Orifices

For a single orifice as illustrated in Figure 2.8, orifice flow can be determined using Equation 2.6.

$$Q = C_o A_o (2gH_o)^{0.5} \tag{2.6}$$

where ,

Q = Orifice flow rate (m^3/s);
C_o = Discharge coefficient (0.60);
A_o = X-sectional area of orifice (m^2);
H_o = Effective head of the orifice measured from the centroid of the opening (m); and
g = Gravitational acceleration (9.81m/s^2).

If the orifice discharges as a free outfall, then the effective head is measured from the centerline of the orifice to the upstream water surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the upstream and downstream water surfaces. This latter condition of a submerged discharge is shown in Figure 2.8(b).

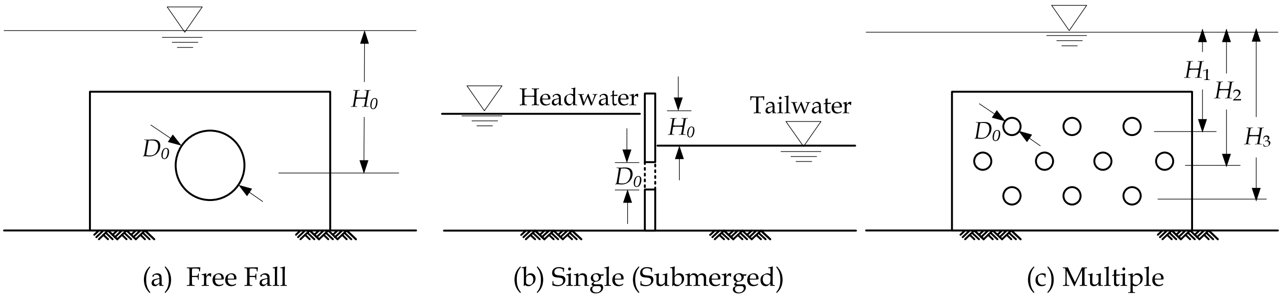


Figure 2.8: Definition Sketch for Orifice Flow (FHWA, 1996)

For square-edged, uniform orifice entrance conditions, a discharge coefficient of 0.6 should be used. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings in corrugated pipe, a value of 0.4 should be used. For circular orifices with C_o set equal to 0.6, the following equation results:

$$Q = K_{or}\, D^2\, H_o^{0.50} \tag{2.7}$$

where,

K_{or} = 2.09 in S.I. units;
D = Orifice diameter (m); and
H_o = Height – $D/2$ for free fall and difference in head and tailwater for submerged orifice.

Pipes smaller than 0.3 m in diameter may be analysed as a submerged orifice as long as H_o/D is greater than 1.5. Pipes greater than 0.3 m in diameter should be analysed as a discharge pipe with headwater and tailwater effects taken into account, not just as an orifice.

2.4.2 Sharp Crested Weirs

Typical sharp crested weirs are illustrated in Figure 2.9. Equation 2.8 provides the discharge relationship for sharp-crested weirs with no end contractions (illustrated in Figure 2.9a).

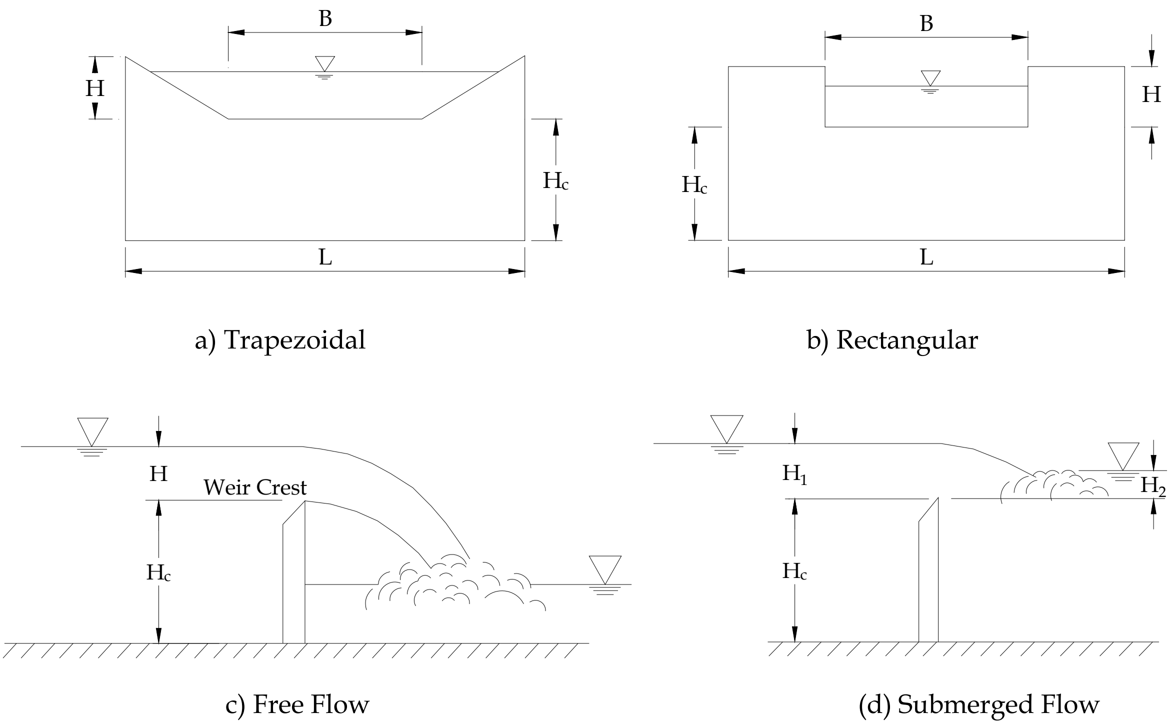


Figure 2.9: Sharp Crested Weirs (FHWA, 1996)

$$Q = C_{SCW} . B . H^{1.5} \tag{2.8}$$

where,

Q = Discharge (m^3/s);
B = Horizontal weir width (m);
H = Head above weir crest excluding velocity head (m); and
C_{SCW} = Weir discharge coefficient = 1.81 + 0.22 (H/H_c).

As indicated above, the value of the coefficient C_{SCW} is known to vary with the ratio H/H_c. Equation 2.9 provides the discharge equation for sharp-crested weirs with end contractions. For values of the ratio H/H_c less than 0.3, a constant C_{SCW} of 1.84 can be used.

$$Q = C_{SCW} (B - 0.2\ H)\ H^{1.5} \tag{2.9}$$

2.4.3 Broad Crested Weirs

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross-section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated in Figure 2.10.

Equation 2.10 presents a relationship for computing the flow through a broad-crested emergency spillway. The dimensional terms used in the equation are illustrated in Figure 2.10.

$$Q = C_{SP}\ B\, H_p^{1.5} \tag{2.10}$$

where,

Q = Emergency spillway discharge (m^3/s);
C_{SP} = Spillway discharge coefficient ($m^{0.5}/s$);
B = Spillway base width (m); and
H_p = Effective head on the spillway weir crest (m).

The discharge coefficient C_{SP} in Equation 2.10 varies as a function of spillway base width and effective head (Table 2.7). Equations 2.11 and 2.12 can be used to compute the critical velocity V_c and critical slope S_c at the control section of an emergency spillway:

$$V_c = 2.14 \left(\frac{Q}{B} \right)^{0.33} \tag{2.11}$$

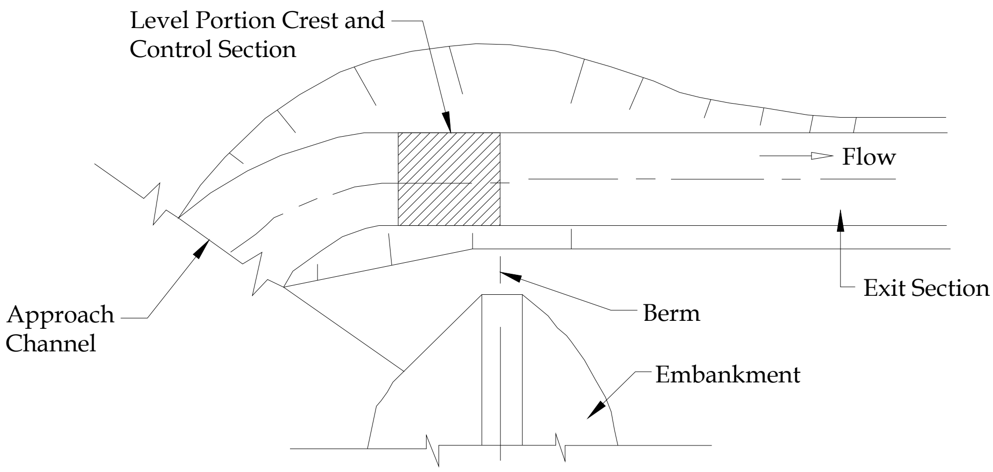
$$S_c = 9.84\ n^2 \left(\frac{V_c\, B}{Q} \right)^{0.33} \tag{2.12}$$

where,

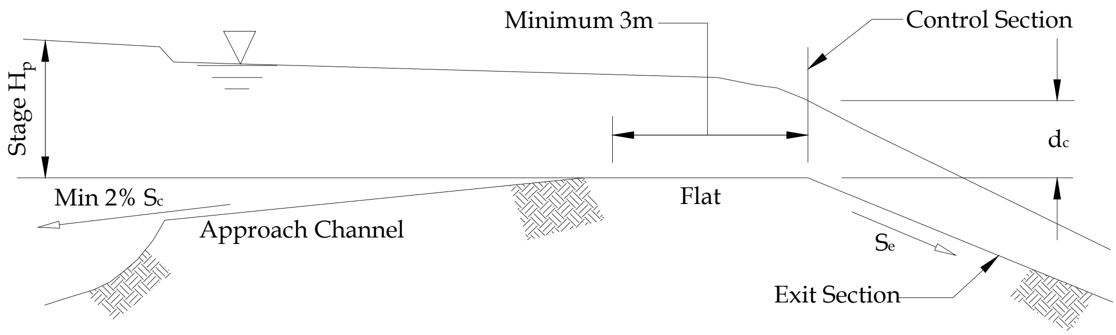
n = Manning's roughness coefficient;
V_c = Critical velocity (m/s); and
S_c = Critical slope (%).

Note that for a given effective head H_p, flattening the exit slope S_e to less than S_c decreases spillway discharge, but steepening S_e greater than S_c does not increase discharge. Also, if a slope S_e steeper than S_c is used, the velocity V_e in the exit channel will increase according to the following relationship:

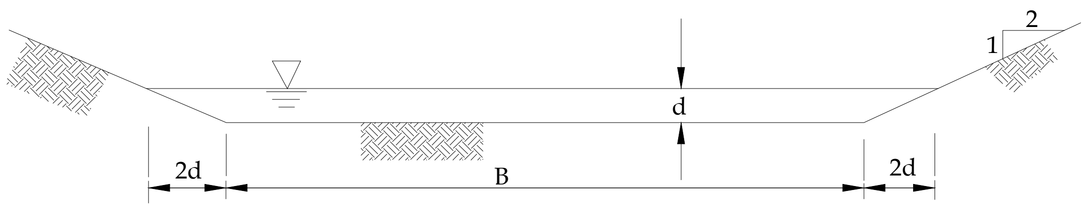
$$V_e = V_c \left(\frac{S_e}{S_c} \right)^{0.3} \tag{2.13}$$



a) Plan View of Excavated Emergency Spillway



b) Profile Along Centerline



c) Cross-Section at Control Section

Figure 2.10: Spillway Design Schematic (FHWA, 1996)

2.4.4 Drawdown Time

It is sometimes necessary to estimate the time it would take to drain a known stored water volume of a pond through an orifice system. The following equation may be used to check that the storage does not take too long time to empty the pond or to return to the normal water (pool) level, after the storm ends:

$$t = -\frac{1}{C_d\, A_o \sqrt{2g}} \int_{H_1}^{H_2} \left(\frac{A_s}{\sqrt{y}} \right) dy \qquad (2.14)$$

where,

t = Time to empty (seconds);

y = Depth of water above the centreline in the storage (m);

A_s = Storage water surface area at depth y (m^2); and

$H_{1,2}$ = Effective heads on the orifice measured from the centroid of the opening (m).

Where the water surface area is constant (i.e. vertical walls in the pond), Equation 2.14 reduces to:

$$t = \frac{2\,A_s}{C_d\,A_o\,\sqrt{2g}}\left(\sqrt{y_1} - \sqrt{y_2}\right) \qquad (2.15)$$

Table 2.7: Broad-Crested Weir Coefficient C_{sp} Values as a Function of Weir Base Width and Head (FHWA, 1996)

Head H_p(m) (1)	Weir Base Width B (m)														
	0.15	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.25	1.50	2.00	3.00	4.00
0.10	1.59	1.56	1.50	1.47	1.45	1.43	1.42	1.41	1.40	1.39	1.37	1.35	1.36	1.40	1.45
0.15	1.65	1.60	1.51	1.48	1.45	1.44	1.44	1.44	1.45	1.45	1.44	1.43	1.44	1.45	1.45
0.20	1.73	1.66	1.54	1.49	1.46	1.44	1.44	1.45	1.46	1.48	1.48	1.49	1.49	1.49	1.45
0.30	1.83	1.77	1.64	1.56	1.50	1.47	1.46	1.46	1.46	1.47	1.47	1.48	1.48	1.48	1.45
0.40	1.83	1.80	1.74	1.65	1.57	1.52	1.49	1.47	1.46	1.46	1.47	1.47	1.47	1.48	1.45
0.50	1.83	1.82	1.81	1.74	1.67	1.60	1.55	1.51	1.48	1.48	1.47	1.46	1.46	1.46	1.45
0.60	1.83	1.83	1.82	1.73	1.65	1.58	1.54	1.46	1.31	1.34	1.48	1.46	1.46	1.46	1.45
0.70	1.83	1.83	1.83	1.78	1.72	1.65	1.60	1.53	1.44	1.45	1.49	1.47	1.47	1.46	1.45
0.80	1.83	1.83	1.83	1.82	1.79	1.72	1.66	1.60	1.57	1.55	1.50	1.47	1.47	1.46	1.45
0.90	1.83	1.83	1.83	1.83	1.81	1.76	1.71	1.66	1.61	1.58	1.50	1.47	1.47	1.46	1.45
1.00	1.83	1.83	1.83	1.83	1.82	1.81	1.76	1.70	1.64	1.60	1.51	1.48	1.47	1.46	1.45
1.10	1.83	1.83	1.83	1.83	1.83	1.83	1.80	1.75	1.66	1.62	1.52	1.49	1.47	1.46	1.45
1.20	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.70	1.65	1.53	1.49	1.48	1.46	1.45
1.30	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.82	1.77	1.71	1.56	1.51	1.49	1.46	1.45
1.40	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.77	1.60	1.52	1.50	1.46	1.45
1.50	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.66	1.55	1.51	1.46	1.45
1.60	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.81	1.74	1.58	1.53	1.46	1.45

(1) Measured at least 2.5H upstream of the weir

2.5 HYDROLOGIC POND ROUTING

The most commonly used method for routing inflow hydrograph through a detention pond is the Storage Indication or modified Puls method. This method begins with the continuity equation which states that the inflow minus the outflow equals the change in storage (I-0=ΔS). By taking the average of two closely spaced inflows and two closely spaced outflows, the method is expressed by Equation 2.16. This relationship is illustrated graphically in Figure 2.11.

$$\frac{\Delta S}{\Delta t} = \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} \qquad (2.16)$$

where:

ΔS = Change in storage (m^3);

Δt = Time interval (min);

I = Inflow (m^3); and

O = Outflow (m^3).

In Equation 2.16, subscript 1 refers to the beginning and subscript 2 refers to the end of the time interval.

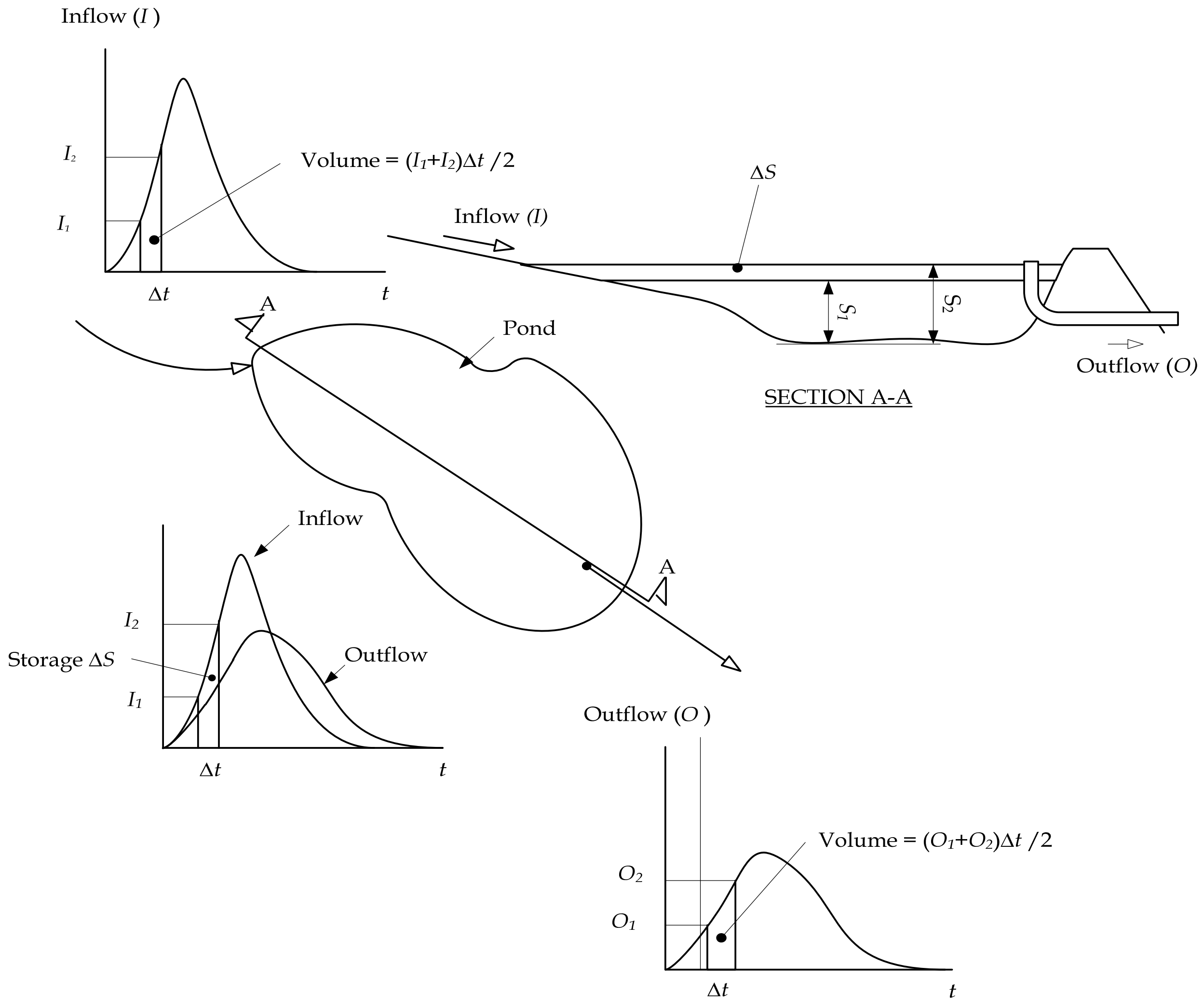


Figure 2.11: Development of the Storage-Discharge Function for Hydrologic Pond Routing

Equation 2.16 can be rearranged so that all the known values are on the left side of the equation and all the unknown values are located on the right hand side of the equation, as shown in Equation 2.17. Now the equation with two unknowns, S_2 and O_2, can be solved with one equation. The following procedure can be used to perform routing through a reservoir or storage facility using Equation 2.17.

$$\frac{I_1 + I_2}{2} + \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 = \left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) \qquad (2.17)$$

Step 1: Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility.

Step 2: Select a routing time period, Δt, to provide a minimum of five points on the rising limb of the inflow hydrograph.

Step 3: Use the stage-storage and stage-discharge data from Step 1 to develop a storage indicator numbers table that provides storage indicator values, S/(Δt) + O/2, versus stage. A typical storage indicator numbers table contains the following column headings:

1 Stage (m)	2 Discharge (O_2) (m^3/s)	3 Storage (S_2) (m^3)	4 $O_2/2$ (m^3/s)	5 $S_2/\Delta t$ (m^3/s)	6 $S_2/\Delta t + O_2/2$ (Storage Indicator Number)

- Discharge (O) and storage (S) are obtained from the stage-discharge and stage-storage curves, respectively.
- Subscript 2 is arbitrarily assigned at this time.
- Time interval (Δt) must be the same as the time interval used in the tabulated inflow hydrograph.

Step 4: Develop a storage indicator numbers curve by plotting the outflow (column 2) vertically against the storage indicator numbers in column 6. An equal value line plotted as $O_2 = S_2/\Delta t + O_2/2$ should also be plotted. If the storage indicator curve crosses the equal value line, a smaller time increment (Δt) is needed (Figure 2.12).

Step 5: A supplementary curve of storage (column 3) vs. $S_2/\Delta t + O_2/2$ (column 6) can also be constructed. This curve does not enter into the mainstream of the routing; however, it is useful for identifying storage for any given value of $S_2/\Delta t + O_2/2$. A plot of storage vs. time can be developed from this curve.

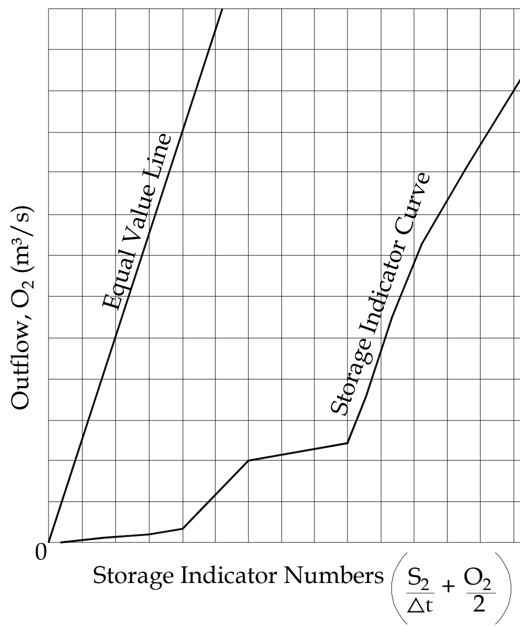


Figure 2.12: Storage Indicator Curve

Step 6: Routing can now be performed by developing a routing table for the solution of Equation 2.17 as follows:

1 Time (hr)	2 Inflow (m^3/s)	3 $(I_1+I_2)/2$ (m^3/s)	4 $(S_1/\Delta t+O_1/2)$ (m^3/s)	5 O_1 (m^3/s)	6 $S_2/\Delta t + O_2/2)$ (m^3/s)	7 O_2 (m^3/s)

- Columns (1) and (2) are obtained from the inflow hydrograph.
- Column (3) is the average inflow over the time interval.
- The initial values for columns (4) and (5) are generally assumed to be zero since there is no storage or discharge at the beginning of the hydrograph when there is no inflow into the basin.

- The left side of Equation 2.17 is determined algebraically as columns (3) + (4) -(5). This value equals the right side of Equation 2.17 or $S_2/\Delta t + O_2/2$ and is placed in column (6).

- Enter the storage indicator curve with $S_2/\Delta t + O_2/2$ (column 6) to obtain O_2 (column 7).

- Column (6) ($S_2/\Delta t + O_2/2$) and column (7) (O_2) are transported to the next line andbecome $S_1/\Delta t + O_1/2$ and O1 in columns (4) and (5), respectively. Because ($S_2/\Delta t + O_2/2$) and O2 are the ending values for the first time step, they can also be said to be the beginning values for the second time step.

- Columns (3), (4), and (5) are again combined and the process is continued until the storm is routeh. Peak storage depth and discharge (O_2 in column (7)) will occur when column (6) reaches a maximum. The storage indicator numbers table developed in Step 3 is entered with the maximum value of $S_2/\Delta t + O_2/2$ to obtain the maximum amount of storage required. This table can also be used to determine the corresponding elevation of the depth of stored water.

- Designer needs to make sure that the peak value in column (7) does not exceed the allowable discharge as prescribed by the stormwater management criteria.

Step 7: Plot O_2 (column 7) versus time (column 1) to obtain the outflow hydrograph.

The above procedure is illustrated in Figure 2.13.

2.6 CRITICAL STORM DURATION

Determination of critical storm duration is important to make the stormwater management facilities safe. Critical storm duration is a function of rainfall intensity, antecedent moisture condition, rainfall temporal pattern, etc. Therefore, it is strongly recommended that the engineer or authority should look into various scenarios that can produce critical storm duration.

Determination of critical storm duration, the one that produces the highest runoff flow rate in the conveyance (pipe or open drain) system, or the highest water level in the storage facility, is required for the design of drainage systems.

2.6.1 Conveyance System

The critical storm duration of a conveyance system is usually close the value of time of concentration (t_c). However, depending on the antecedent moisture condition, variation in the temporal pattern, storm and wind direction, land development distribution of impervious surfaces in the subcatchment, etc. the critical storm duration might be significantly different from that of the t_c. Therefore, rainfall events of various durations and possible runoff contributing areas need to be analysed to determine the critical storm duration for the conveyance system.

Two options can be used to determine the critical storm duration for conveyance. Those are:

- Simple Calculation for catchment < 80 ha: Critical Storm Duration = t_c with possible checks for partial area effects; and

- Computer Model for catchment ≥ 80 ha: Run model for various storm durations and plotting the calculated peak flow rates for various durations to find the critical storm duration, as shown in Figure 2.14.

In order to develop the critical storm duration for a conveyance system, the designer has to select the design ARI and simulate hydrologic and hydraulic calculations for various storm durations together with the rainfall temporal patterns, antecedent moisture condition, etc. to get the peak flow values. The designer must then plot the design peak flow values against the storm durations, as shown in Figure 2.14 to find the critical storm duration for the drain or drainage system

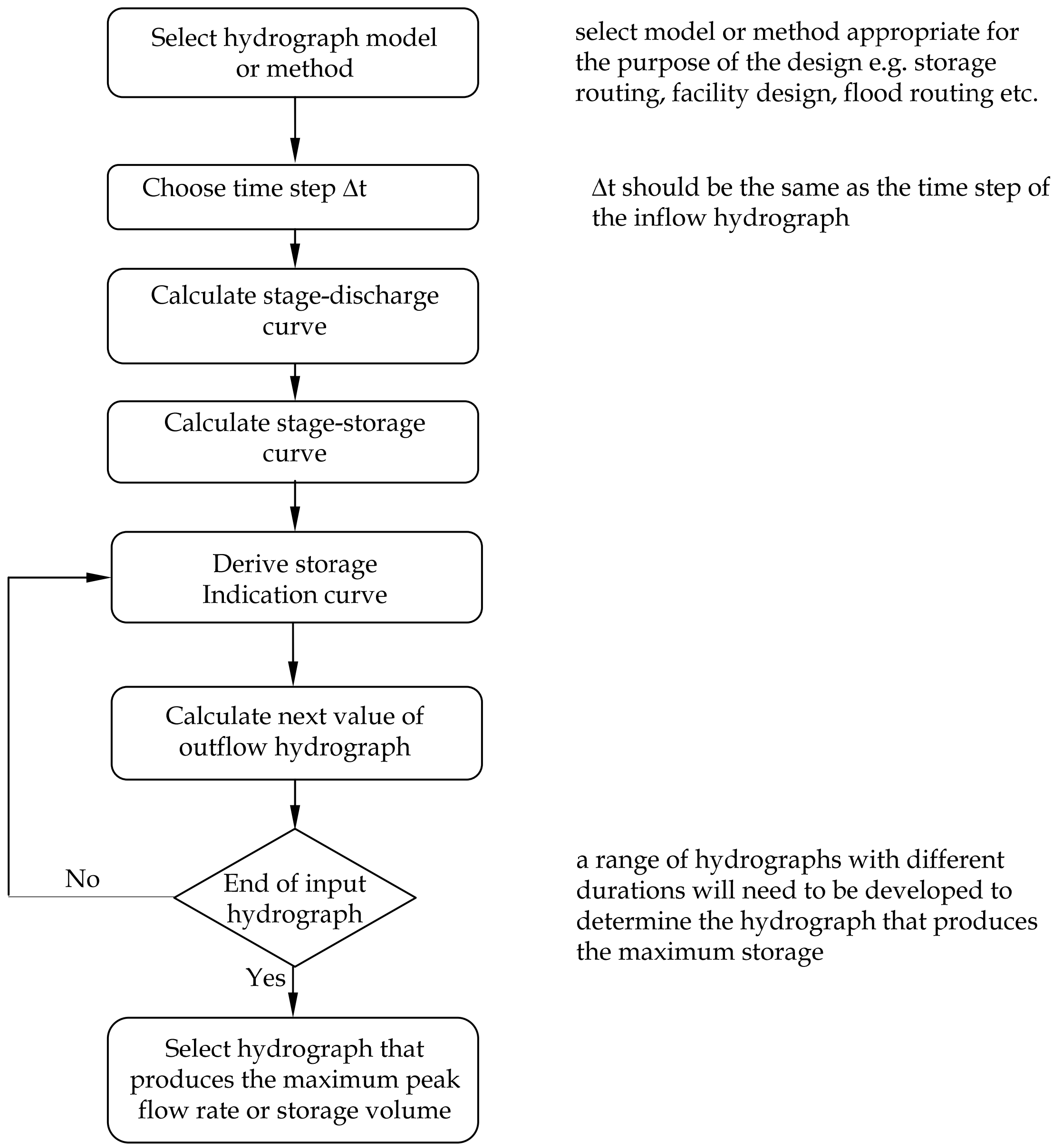


Figure 2.13: General Analysis Procedure for Pond Routing (DID, 2000)

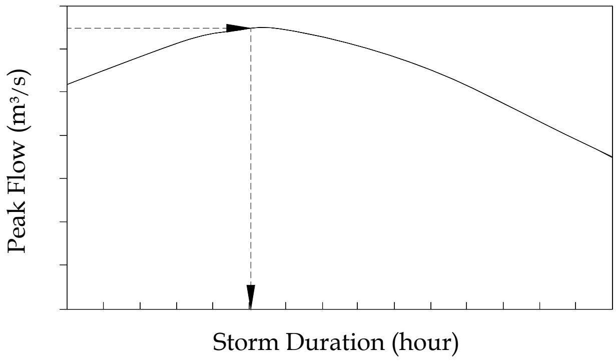


Figure 2.14: Determination of Critical Storm Duration for Conveyance

2.6.2 Storage System

On the other hand, the critical storm duration of any storage facility (OSD, Detention Pond, Wetland, etc.) mainly depends on the event runoff volume, inflow-outflow relationship, initial water level in the system, etc. In short, runoff volume is more critical, instead of just the intensity of the rainfall. Hydrologic and hydraulic routing of various storm durations for various rainfall temporal patterns, antecedent moisture conditions, etc. Is required to define the maximum water level in the storage facility. The designer must then plot the simulated highest water level in the pond, wetland or detention facility against the storm durations, as shown in Figure 2.15, to find the critical storm duration for the storage facilities.

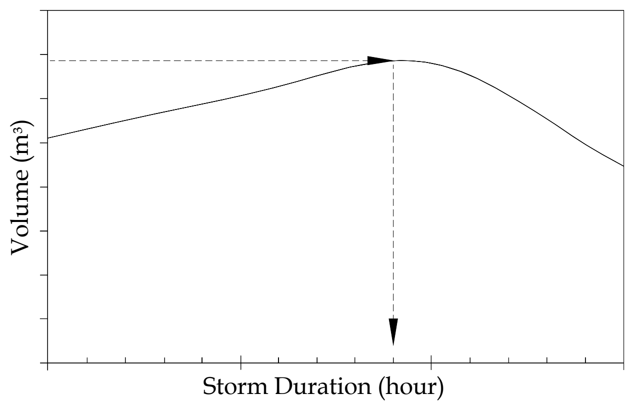


Figure 2.15: Determination of Critical Storm Duration for a Storage Facility

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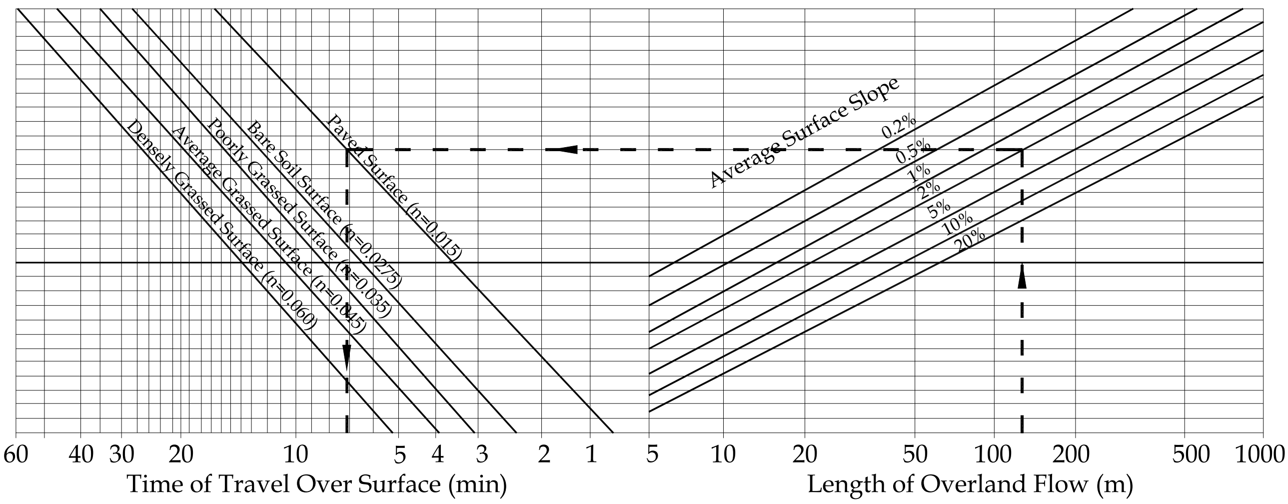
APPENDIX 2.A DESIGN CHART – OVERLAND FLOW TIME



Design Chart 2.A1: Nomograph for the Estimation of Overland Flow Time (t_o) for Sheet Flow (IEA, 1977)

APPENDIX 2.B IDF CONSTANTS

Table 2.B1: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for High ARIs between 2 and 100 Year and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Johor	1	1437116	Stor JPS Johor Bahru	59.972	0.163	0.121	0.793
	2	1534002	Pusat Kem. Pekan Nenas	54.265	0.179	0.100	0.756
	3	1541139	Johor Silica	59.060	0.202	0.128	0.660
	4	1636001	Balai Polis Kg Seelong	50.115	0.191	0.099	0.763
	5	1737001	SM Bukit Besar	50.554	0.193	0.117	0.722
	6	1829002	Setor JPS Batu Pahat	64.099	0.174	0.201	0.826
	7	1834124	Ladang Ulu Remis	55.864	0.166	0.174	0.810
	8	1839196	Simpang Masai K. Sedili	61.562	0.191	0.103	0.701
	9	1931003	Emp. Semberong	60.568	0.163	0.159	0.821
	10	2025001	Pintu Kaw. Tg. Agas	80.936	0.187	0.258	0.890
	11	2033001	JPS Kluang	54.428	0.192	0.108	0.740
	12	2231001	Ladang Chan Wing	57.188	0.186	0.093	0.777
	13	2232001	Ladang Kekayaan	53.457	0.180	0.094	0.735
	14	2235163	Ibu Bekalan Kahang	52.177	0.186	0.055	0.652
	15	2237164	Jalan Kluang-Mersing	56.966	0.190	0.144	0.637
	16	2330009	Ladang Labis	45.808	0.222	0.012	0.713
	17	2528012	Rmh. Tapis Segamat	45.212	0.224	0.039	0.711
	18	2534160	Kg Peta Hulu Sg Endau	59.500	0.185	0.129	0.623
	19	2636170	Setor JPS Endau	62.040	0.215	0.103	0.592
Kedah	1	5507076	Bt. 27, Jalan Baling	52.398	0.172	0.104	0.788
	2	5704055	Kedah Peak	81.579	0.200	0.437	0.719
	3	5806066	Klinik Jeniang	59.786	0.165	0.203	0.791
	4	5808001	Bt. 61, Jalang Baling	47.496	0.183	0.079	0.752
	5	6103047	Setor JPS Alor Setar	64.832	0.168	0.346	0.800
	6	6108001	Kompleks Rumah Muda	52.341	0.173	0.120	0.792
	7	6206035	Kuala Nerang	54.849	0.174	0.250	0.810
	8	6107032	AmpangPadu	66.103	0.177	0.284	0.842
	9	6306031	Padang Senai	60.331	0.193	0.249	0.829

Table 2.B1: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for High ARIs between 2 and 100 Year and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Kelantan	1	4614001	Brook	49.623	0.159	0.242	0.795
	2	4726001	Gunung Gagau	43.024	0.220	0.004	0.527
	3	4819027	Gua Musang	57.132	0.155	0.119	0.795
	4	4915001	Chabai	47.932	0.169	0.108	0.794
	5	4923001	Kg Aring	47.620	0.187	0.020	0.637
	6	5120025	Balai Polis Bertam	61.338	0.168	0.193	0.811
	7	5216001	Gob	41.783	0.175	0.122	0.720
	8	5320038	Dabong	51.442	0.189	0.077	0.710
	9	5322044	Kg Lalok	53.766	0.197	0.121	0.705
	10	5522047	JPS Kuala Krai	39.669	0.231	0.000	0.563
	11	5718033	Kg Jeli, Tanah Merah	72.173	0.196	0.360	0.703
	12	5719001	Kg Durian Daun Lawang	51.161	0.193	0.063	0.745
	13	5722057	JPS Machang	48.433	0.219	0.000	0.601
	14	5824079	Sg Rasau Pasir Putih	51.919	0.216	0.062	0.560
	15	6019004	Rumah Kastam Rantau Pjg	49.315	0.228	0.000	0.609
	16	6122064	Setor JPS Kota Bharu	60.988	0.214	0.148	0.616
Kuala Lumpur	1	3015001	Puchong Drop, K Lumpur	69.650	0.151	0.223	0.880
	2	3116003	Ibu Pejabat JPS	61.976	0.145	0.122	0.818
	3	3116004	Ibu Pejabat JPS1	64.689	0.149	0.174	0.837
	4	3116005	SK Taman Maluri	62.765	0.132	0.147	0.820
	5	3116006	Ladang Edinburgh	63.483	0.146	0.210	0.830
	6	3216001	Kg. Sungai Tua	64.203	0.152	0.250	0.844
	7	3216004	SK Jenis Keb. Kepong	73.602	0.164	0.330	0.874
	8	3217001	Ibu Bek. KM16, Gombak	66.328	0.144	0.230	0.859
	9	3217002	Emp. Genting Kelang	70.200	0.165	0.290	0.854
	10	3217003	Ibu Bek. KM11, Gombak	62.609	0.152	0.221	0.804
	11	3217004	Kg. Kuala Seleh, H. Klg	61.516	0.139	0.183	0.837
	12	3217005	Kg. Kerdas, Gombak	63.241	0.162	0.137	0.856
	13	3317001	Air Terjun Sg. Batu	72.992	0.162	0.171	0.871
	14	3317004	Genting Sempah	61.335	0.157	0.292	0.868

(Continued)

Table 2.B1: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for High ARIs between 2 and 100 Year and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Malacca	1	2222001	Bukit Sebukor	95.823	0.169	0.660	0.947
	2	2224038	Chin Chin Tepi Jalan	54.241	0.161	0.114	0.846
	3	2321006	Ladang Lendu	72.163	0.184	0.376	0.900
Negeri Sembilan	1	2719001	Setor JPS Sikamat	52.823	0.167	0.159	0.811
	2	2722202	Kg Sawah Lebar K Pilah	44.811	0.181	0.137	0.811
	3	2723002	Sungai Kepis	54.400	0.176	0.134	0.842
	4	2725083	Ladang New Rompin	57.616	0.191	0.224	0.817
	5	2920012	Petaling K Kelawang	50.749	0.173	0.235	0.854
Pahang	1	2630001	Sungai Pukim	46.577	0.232	0.169	0.687
	2	2634193	Sungai Anak Endau	66.179	0.182	0.081	0.589
	3	2828173	Kg Gambir	47.701	0.182	0.096	0.715
	4	3026156	Pos Iskandar	47.452	0.184	0.071	0.780
	5	3121143	Simpang Pelangai	57.109	0.165	0.190	0.867
	6	3134165	Dispensari Nenasi	61.697	0.152	0.120	0.593
	7	3231163	Kg Unchang	55.568	0.179	0.096	0.649
	8	3424081	JPS Temerloh	73.141	0.173	0.577	0.896
	9	3533102	Rumah Pam Pahang Tua	58.483	0.212	0.197	0.586
	10	3628001	Pintu Kaw. Pulau Kertam	50.024	0.211	0.089	0.716
	11	3818054	Setor JPS Raub	53.115	0.168	0.191	0.833
	12	3924072	Rmh Pam Paya Kangsar	62.301	0.167	0.363	0.868
	13	3930012	Sungai Lembing PCC Mill	45.999	0.210	0.074	0.590
	14	4023001	Kg Sungai Yap	65.914	0.195	0.252	0.817
	15	4127001	Hulu Tekai Kwsn."B"	59.861	0.226	0.213	0.762
	16	4219001	Bukit Bentong	73.676	0.165	0.384	0.879
	17	4223115	Kg Merting	52.731	0.184	0.096	0.805
	18	4513033	Gunung Brinchang	42.004	0.164	0.046	0.802
Penang	1	5204048	Sg Simpang Ampat	62.089	0.220	0.402	0.785
	2	5302001	Tangki Air Besar Sg Pinang	67.949	0.181	0.299	0.736
	3	5302003	Kolam Tkgn Air Hitam	52.459	0.191	0.106	0.729
	4	5303001	Rmh Kebajikan P Pinang	57.326	0.203	0.325	0.791
	5	5303053	Komplek Prai	52.771	0.203	0.095	0.717
	6	5402001	Klinik Bkt Bendera P Pinang	64.504	0.196	0.149	0.723
	7	5402002	Kolam Bersih P Pinang	53.785	0.181	0.125	0.706
	8	5404043	Ibu Bekalan Sg Kulim	57.832	0.188	0.245	0.751
	9	5504035	Lahar Ikan Mati Kepala Batas	48.415	0.221	0.068	0.692

(Continued)

Table 2.B1: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for High ARIs between 2 and 100 Year and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Perak	1	4010001	JPS Teluk Intan	54.017	0.198	0.084	0.790
	2	4207048	JPS Setiawan	56.121	0.174	0.211	0.854
	3	4311001	Pejabat Daerah Kampar	69.926	0.148	0.149	0.813
	4	4409091	Rumah Pam Kubang Haji	52.343	0.164	0.177	0.840
	5	4511111	Politeknik Ungku Umar	70.238	0.164	0.288	0.872
	6	4807016	Bukit Larut Taiping	87.236	0.165	0.258	0.842
	7	4811075	Rancangan Belia Perlop	58.234	0.198	0.247	0.856
	8	5005003	Jln. Mtg. Buloh Bgn Serai	52.752	0.163	0.179	0.795
	9	5207001	Kolam Air JKR Selama	59.567	0.176	0.062	0.807
	10	5210069	Stesen Pem. Hutan Lawin	52.803	0.169	0.219	0.838
	11	5411066	Kuala Kenderong	85.943	0.223	0.248	0.909
	12	5710061	Dispensari Keroh	53.116	0.168	0.112	0.820
Perlis	1	6401002	Padang Katong, Kangar	57.645	0.179	0.254	0.826
Selangor	1	2815001	JPS Sungai Manggis	56.052	0.152	0.194	0.857
	2	2913001	Pusat Kwln. JPS T Gong	63.493	0.170	0.254	0.872
	3	2917001	Setor JPS Kajang	59.153	0.161	0.118	0.812
	4	3117070	JPS Ampang	65.809	0.148	0.156	0.837
	5	3118102	SK Sungai Lui	63.155	0.177	0.122	0.842
	6	3314001	Rumah Pam JPS P Setia	62.273	0.175	0.205	0.841
	7	3411017	Setor JPS Tj. Karang	68.290	0.175	0.243	0.894
	8	3416002	Kg Kalong Tengah	61.811	0.161	0.188	0.816
	9	3516022	Loji Air Kuala Kubu Baru	67.793	0.176	0.278	0.854
	10	3710006	Rmh Pam Bagan Terap	60.793	0.173	0.185	0.884
Terengganu	1	3933001	Hulu Jabor, Kemaman	103.519	0.228	0.756	0.707
	2	4131001	Kg, Ban Ho, Kemaman	65.158	0.164	0.092	0.660
	3	4234109	JPS Kemaman	55.899	0.201	0.000	0.580
	4	4332001	Jambatan Tebak, Kem.	61.703	0.185	0.088	0.637
	5	4529001	Rmh Pam Paya Kempian	53.693	0.194	0.000	0.607
	6	4529071	SK Pasir Raja	48.467	0.207	0.000	0.600
	7	4631001	Almuktafibillah Shah	66.029	0.199	0.165	0.629
	8	4734079	SM Sultan Omar, Dungun	51.935	0.213	0.020	0.587
	9	4832077	SK Jerangau	54.947	0.212	0.026	0.555
	10	4930038	Kg Menerong, Hulu Trg	60.436	0.204	0.063	0.588
	11	5029034	Kg Dura. Hulu Trg	60.510	0.220	0.087	0.617
	12	5128001	Sungai Gawi, Hulu Trg	48.101	0.215	0.027	0.566
	13	5226001	Sg Petualang, Hulu Trg	48.527	0.228	0.000	0.547
	14	5328044	Sungai Tong, Setiu	52.377	0.188	0.003	0.558
	15	5331048	Setor JPS K Terengganu	58.307	0.210	0.123	0.555
	16	5426001	Kg Seladang, Hulu Setiu	57.695	0.197	0.000	0.544
	17	5428001	Kg Bt. Hampar, Setiu	55.452	0.186	0.000	0.545
	18	5524002	SK Panchor, Setiu Klinik	53.430	0.206	0.000	0.524
	19	5725006	Kg Raja, Besut	52.521	0.225	0.041	0.560

(Continued)

Table 2.B2: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for Low ARIs between 0.5 and 12 Month and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Johor	1	1437116	Stor JPS Johor Bahru	73.6792	0.2770	0.2927	0.8620
	2	1534002	Pusat Kem. Pekan Nenas	62.6514	0.3231	0.1557	0.8212
	3	1541139	Johor Silica	79.5355	0.3363	0.2947	0.8097
	4	1636001	Balai Polis Kg Seelong	61.2124	0.3373	0.2375	0.8427
	5	1737001	SM Bukit Besar	61.3513	0.3027	0.2029	0.8240
	6	1829002	Setor Daerah JPS Batu Pahat	62.1576	0.3055	0.1423	0.8253
	7	1834124	Ladang Ulu Remis	59.1713	0.2935	0.1847	0.8380
	8	1839196	Simpang Masai K. Sedili	71.7947	0.2683	0.1863	0.8071
	9	1931003	Emp. Semberong	66.8854	0.3549	0.2107	0.8384
	10	2025001	Pintu Kaw. Tg. Agas	77.7719	0.3102	0.2806	0.8789
	11	2231001	Ladang Chan Wing	66.1439	0.3236	0.1778	0.8489
	12	2232001	Ladang Kekayaan	66.7541	0.3076	0.2270	0.8381
	13	2235163	Ibu Bekalan Kahang	62.3394	0.2786	0.1626	0.7389
	14	2237164	Jalan Kluang-Mersing	73.2358	0.3431	0.2198	0.7733
	15	2330009	Ladang Labis	65.2220	0.3947	0.2353	0.8455
	16	2528012	Rmh. Tapis Segamat	63.6892	0.3817	0.2586	0.8711
	17	2534160	Kg Peta Hulu Sg Endau	69.9581	0.3499	0.1808	0.7064
	18	2636170	Setor JPS Endau	77.6302	0.3985	0.2497	0.6927
Kedah	1	5507076	Bt. 27, Jalan Baling	62.7610	0.2580	0.3040	0.8350
	2	5704055	Kedah Peak	58.5960	0.3390	0.0640	0.661
	3	5806066	Klinik Jeniang	67.1200	0.3820	0.2380	0.8230
	4	5808001	Bt. 61, Jalan Baling	56.3990	0.3880	0.2520	0.8030
	5	6103047	Setor JPS Alor Setar	67.6410	0.3340	0.2740	0.8280
	6	6108001	Kompleks Rumah Muda	58.4040	0.2780	0.2340	0.8290
	7	6206035	Kuala Nerang	62.9600	0.3080	0.3590	0.8590
	8	6207032	Ampang Padu	70.9970	0.2930	0.3820	0.8630
	9	6306031	Padang Sanai	63.6150	0.3130	0.3090	0.8520

Table 2.B2: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for Low ARIs between 0.5 and 12 Month and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Kelantan	1	4614001	Brook	49.7311	0.3159	0.1978	0.7924
	2	4915001	Chabai	56.2957	0.2986	0.1965	0.8384
	3	4923001	Kg Aring	70.2651	0.3810	0.2416	0.8185
	4	5120025	Balai Polis Bertam	67.7195	0.3271	0.2430	0.8424
	5	5216001	Gob	47.4654	0.2829	0.1531	0.7850
	6	5320038	Dabong	67.7907	0.3777	0.2740	0.8115
	7	5322044	Kg Lalok	67.7660	0.3288	0.2367	0.8188
	8	5522047	JPS Kuala Krai	63.0690	0.4681	0.3096	0.7833
	9	5718033	Kg Jeli, Tanah Merah	73.8139	0.3878	0.1161	0.7600
	10	5719001	Kg Durian Daun Lawang	67.2398	0.3651	0.1822	0.7531
	11	5722057	JPS Machang	57.3756	0.3441	0.1742	0.7085
	12	5824079	Sg Rasau, Pasir Putih	68.5083	0.4079	0.2019	0.7003
	13	6019004	Rumah Kastam Rantau Pjg	65.3650	0.4433	0.1582	0.7527
Kuala Lumpur	1	3015001	Puchong Drop, K Lumpur	68.5873	0.3519	0.1697	0.8494
	2	3116004	Ibu Pejabat JPS	65.9923	0.2857	0.1604	0.8341
	3	3116005	SK Taman Maluri	74.4510	0.2663	0.3120	0.8608
	4	3116006	Ladang Edinburgh	64.5033	0.2751	0.1814	0.8329
	5	3216001	Kg. Sungai Tua	62.9398	0.2579	0.1989	0.8374
	6	3216004	SK Jenis Keb. Kepong	69.7878	0.2955	0.1672	0.8508
	7	3217001	Ibu Bek. KM16, Gombak	66.0685	0.2565	0.2293	0.8401
	8	3217002	Emp. Genting Kelang	66.2582	0.2624	0.2423	0.8446
	9	3217003	Ibu Bek. KM11, Gombak	73.9540	0.2984	0.3241	0.8238
	10	3217004	Kg. Kuala Seleh, H. Klang	64.3175	0.2340	0.1818	0.8645
	11	3217005	Kg. Kerdas, Gombak	68.8526	0.2979	0.2024	0.8820
	12	3317001	Air Terjun Sg. Batu	75.9351	0.2475	0.2664	0.8668
	13	3317004	Genting Sempah	55.3934	0.2822	0.1835	0.8345

(Continued)

Table 2.B2: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for Low ARIs between 0.5 and 12 Month and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Malacca	1	2222001	Bukit Sebukor	78.1482	0.2690	0.3677	0.8968
	2	2224038	Chin Chin Tepi Jalan	66.0589	0.3363	0.3301	0.8905
	3	2321006	Ladang Lendu	64.7588	0.2975	0.2896	0.8787
Negeri Sembilan	1	2719001	Setor JPS Sikamat	60.4227	0.2793	0.2694	0.8540
	2	2722202	Kg Sawah Lebar K Pilah	49.3232	0.2716	0.2164	0.8503
	3	2723002	Sungai Kepis	61.3339	0.2536	0.3291	0.8717
	4	2725083	Ladang New Rompin	65.0249	0.3575	0.3546	0.8750
	5	2920012	Petaling K Kelawang	51.7343	0.2919	0.2643	0.8630
Pahang	1	2630001	Sungai Pukim Sungai	63.9783	0.3906	0.2556	0.8717
	2	2634193	Anak Endau	79.4310	0.3639	0.1431	0.7051
	3	2828173	Kg Gambir	61.1933	0.3857	0.1878	0.8237
	4	3026156	Pos Iskandar	59.9903	0.3488	0.2262	0.8769
	5	3121143	Simpang Pelangai	64.9653	0.3229	0.3003	0.8995
	6	3134165	Dispensari Nenasi	88.6484	0.3830	0.4040	0.7614
	7	3231163	Kg Unchang	71.6472	0.3521	0.1805	0.7886
	8	3424081	JPS Temerloh	62.2075	0.3528	0.3505	0.8368
	9	3533102	Rumah Pam Pahang Tua	80.8887	0.3611	0.4800	0.7578
	10	3628001	Pintu Kaw. Pulau Kertam	63.5073	0.3830	0.2881	0.8202
	11	3818054	Setor JPS Raub	61.3432	0.3692	0.3929	0.8445
	12	3924072	Rmh Pam Paya Kangsar	58.3761	0.3334	0.2421	0.8430
	13	3930012	Sungai Lembing PCC Mill	77.0004	0.4530	0.5701	0.8125
	14	4023001	Kg Sungai Yap	77.1488	0.3725	0.3439	0.8810
	15	4127001	Hulu Tekai Kwsn.”B”	60.2235	0.4650	0.1241	0.8020
	16	4219001	Bukit Bentong	67.6128	0.2706	0.2459	0.8656
	17	4223115	Kg Merting	62.7511	0.2843	0.3630	0.9024
	18	4513033	Gunung Brinchang	42.1757	0.2833	0.1468	0.7850
Penang	1	5204048	Sg Simpang Ampat	59.3122	0.3394	0.3350	0.8090
	2	5302001	Tangki Air Besar Sg Pinang	71.7482	0.2928	0.2934	0.7779
	3	5302003	Kolam Tkgn Air Hitam	56.1145	0.2975	0.1778	0.7626
	4	5303001	Rmh Kebajikan P Pinang	60.1084	0.3575	0.2745	0.8303
	5	5303053	Kompleks Prai P Pinang	49.4860	0.3314	0.0518	0.7116
	6	5402001	Klinik Bkt Bendera P Pinang	68.0999	0.3111	0.1904	0.7662
	7	5402002	Kolam Bersih P Pinang	62.7533	0.2688	0.2488	0.7757
	8	5504035	Lahar Ikan Mati Kepala Batas	60.8596	0.3369	0.2316	0.7981

(Continued)

Table 2.B2: Fitting Constants for the IDF Empirical Equation for the Different Locations in Malaysia for Low ARIs between 0.5 and 12 Month and Storm Durations from 5 Minutes to 72 Hours

State	No.	Station ID	Station Name	Constants			
				λ	κ	θ	η
Perak	1	5005003	JPS Teluk Intan	65.1854	0.3681	0.2552	0.8458
	2	4010001	JPS Setiawan	56.2695	0.3434	0.2058	0.8465
	3	4207048	Pejabat Daerah Kampar	79.2706	0.1829	0.3048	0.8532
	4	4311001	Rumah Pam Kubang Haji	47.8316	0.3527	0.1038	0.8018
	5	4409091	Politeknik Ungku Umar	62.9315	0.3439	0.1703	0.8229
	6	4511111	Bukit Larut Taiping	83.3964	0.3189	0.1767	0.8166
	7	4807016	Rancangan Belia Perlop	57.4914	0.3199	0.2027	0.8696
	8	4811075	Jln. Mtg. Buloh Bgn Serai	63.2357	0.3176	0.3330	0.8462
	9	5207001	Kolam Air JKR Selama	67.0499	0.3164	0.2255	0.8080
	10	5210069	Stesen Pem. Hutan Lawin	53.7310	0.3372	0.2237	0.8347
	11	5411066	Kuala Kenderong	68.5357	0.4196	0.1558	0.8378
	12	5710061	Dispensari Keroh	59.2197	0.3265	0.1621	0.8522
Perlis	1	6401002	Padang Katong, Kangar	52.1510	0.3573	0.1584	0.7858
Selangor	1	2815001	JPS Sungai Manggis	57.3495	0.2758	0.1693	0.8672
	2	2913001	Pusat Kwln. JPS T Gong	65.3556	0.3279	0.3451	0.8634
	3	2917001	Setor JPS Kajang	62.9564	0.3293	0.1298	0.8273
	4	3117070	JPS Ampang	69.1727	0.2488	0.1918	0.8374
	5	3118102	SK Sungai Lui	68.4588	0.3035	0.2036	0.8726
	6	3314001	Rumah Pam JPS P Setia	65.1864	0.2816	0.2176	0.8704
	7	3411017	Setor JPS Tj. Karang	70.9914	0.2999	0.2929	0.9057
	8	3416002	Kg Kalong Tengah	59.9750	0.2444	0.1642	0.8072
	9	3516022	Loji Air Kuala Kubu Baru	66.8884	0.2798	0.3489	0.8334
	10	3710006	Rmh Pam Bagan Terap	62.2644	0.3168	0.2799	0.8665
Terengganu	1	3933001	Hulu Jabor, Kemaman	74.8046	0.2170	0.2527	0.7281
	2	4131001	Kg, Ban Ho, Kemaman	68.6659	0.3164	0.1157	0.6969
	3	4234109	JPS Kemaman Jambatan	75.8258	0.2385	0.3811	0.7303
	4	4332001	Tebak, Kem.	77.2826	0.3460	0.3036	0.7301
	5	4529001	Rmh Pam Paya Kempian	65.2791	0.3642	0.1477	0.6667
	6	4631001	Almuktafibillah Shah	81.8861	0.3400	0.2600	0.7459
	7	4734079	SM Sultan Omar, Dungun	66.4262	0.3288	0.2152	0.7015
	8	4832077	SK Jerangau	81.4981	0.3736	0.4226	0.7586
	9	4930038	Kg Menerong, Hulu Trg	80.9649	0.3782	0.2561	0.7158
	10	5029034	Kg Dura. Hulu Trg	62.7859	0.3495	0.1103	0.6638
	11	5128001	Sungai Gawi, Hulu Trg	59.3063	0.4001	0.1312	0.6796
	12	5226001	Sg Petualang, Hulu Trg	51.7862	0.2968	0.0704	0.6587
	13	5328044	Sungai Tong, Setiu	63.4136	0.3864	0.0995	0.6540
	14	5331048	Setor JPS K Terengganu	67.0267	0.2844	0.2633	0.6690
	15	5426001	Kg Seladang, Hulu Setiu	76.9088	0.4513	0.1636	0.6834
	16	5428001	Kg Bt. Hampar, Setiu	57.9456	0.2490	0.0380	0.6000
	17	5524002	SK Panchor, Setiu	75.1489	0.4147	0.2580	0.6760

APPENDIX 2.C NORMALISED DESIGN RAINFALL TEMPORAL PATTERN

2.C1 Region 1: Terengganu and Kelantan

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.316	0.133	0.060	0.060	0.059	0.070	0.019	0.027	0.021
2	0.368	0.193	0.062	0.061	0.067	0.073	0.022	0.028	0.029
3	0.316	0.211	0.084	0.071	0.071	0.083	0.027	0.029	0.030
4		0.202	0.087	0.080	0.082	0.084	0.036	0.033	0.033
5		0.161	0.097	0.110	0.119	0.097	0.042	0.037	0.037
6		0.100	0.120	0.132	0.130	0.106	0.044	0.040	0.038
7			0.115	0.120	0.123	0.099	0.048	0.046	0.042
8			0.091	0.100	0.086	0.086	0.049	0.048	0.048
9			0.087	0.078	0.073	0.084	0.050	0.049	0.053
10			0.082	0.069	0.069	0.083	0.056	0.054	0.055
11			0.061	0.060	0.063	0.070	0.058	0.058	0.058
12			0.054	0.059	0.057	0.064	0.068	0.065	0.067
13							0.058	0.060	0.059
14							0.057	0.055	0.056
15							0.050	0.053	0.053
16							0.050	0.048	0.052
17							0.048	0.046	0.047
18							0.046	0.044	0.041
19							0.043	0.038	0.038
20							0.039	0.034	0.036
21							0.028	0.030	0.033
22							0.025	0.029	0.030
23							0.022	0.028	0.022
24							0.016	0.019	0.020

2.C2 Region 2: Johor, Negeri Sembilan, Melaka, Selangor and Pahang

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.255	0.124	0.053	0.053	0.044	0.045	0.022	0.027	0.016
2	0.376	0.130	0.059	0.061	0.081	0.048	0.024	0.028	0.023
3	0.370	0.365	0.063	0.063	0.083	0.064	0.029	0.029	0.027
4		0.152	0.087	0.080	0.090	0.106	0.031	0.033	0.033
5		0.126	0.103	0.128	0.106	0.124	0.032	0.037	0.036
6		0.103	0.153	0.151	0.115	0.146	0.035	0.040	0.043
7			0.110	0.129	0.114	0.127	0.039	0.046	0.047
8			0.088	0.097	0.090	0.116	0.042	0.048	0.049
9			0.069	0.079	0.085	0.081	0.050	0.049	0.049
10			0.060	0.062	0.081	0.056	0.054	0.054	0.051
11			0.057	0.054	0.074	0.046	0.065	0.058	0.067
12			0.046	0.042	0.037	0.041	0.093	0.065	0.079
13							0.083	0.060	0.068
14							0.057	0.055	0.057
15							0.052	0.053	0.050
16							0.047	0.048	0.049
17							0.040	0.046	0.048
18							0.039	0.044	0.043
19							0.033	0.038	0.038
20							0.031	0.034	0.035
21							0.029	0.030	0.030
22							0.028	0.029	0.024
23							0.024	0.028	0.022
24							0.020	0.019	0.016

2.C3 Region 3: Perak, Kedah, Pulau Pinang and Perlis

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.215	0.158	0.068	0.060	0.045	0.040	0.027	0.015	0.021
2	0.395	0.161	0.074	0.085	0.070	0.060	0.031	0.020	0.023
3	0.390	0.210	0.077	0.086	0.078	0.066	0.033	0.026	0.024
4		0.173	0.087	0.087	0.099	0.092	0.034	0.028	0.025
5		0.158	0.099	0.100	0.113	0.114	0.035	0.038	0.028
6		0.141	0.106	0.100	0.129	0.166	0.036	0.039	0.031
7			0.104	0.100	0.121	0.119	0.039	0.045	0.044
8			0.098	0.088	0.099	0.113	0.042	0.046	0.049
9			0.078	0.087	0.081	0.081	0.044	0.052	0.058
10			0.075	0.085	0.076	0.066	0.053	0.057	0.063
11			0.072	0.063	0.047	0.046	0.056	0.069	0.074
12			0.064	0.059	0.041	0.036	0.080	0.086	0.081
13							0.076	0.073	0.078
14							0.055	0.060	0.070
15							0.048	0.056	0.058
16							0.044	0.046	0.050
17							0.041	0.045	0.044
18							0.039	0.044	0.044
19							0.036	0.039	0.030
20							0.034	0.035	0.026
21							0.033	0.028	0.025
22							0.032	0.021	0.024
23							0.031	0.017	0.022
24							0.023	0.014	0.008

2.C4 Region 4: Mountainous Area

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.146	0.117	0.028	0.019	0.019	0.041	0.000	0.002	0.005
2	0.677	0.130	0.052	0.019	0.040	0.052	0.002	0.007	0.006
3	0.177	0.374	0.064	0.055	0.045	0.056	0.007	0.018	0.011
4		0.152	0.073	0.098	0.060	0.059	0.009	0.024	0.014
5		0.121	0.106	0.164	0.082	0.120	0.023	0.027	0.018
6		0.107	0.280	0.197	0.390	0.253	0.026	0.033	0.027
7			0.119	0.169	0.171	0.157	0.027	0.037	0.028
8			0.079	0.132	0.062	0.065	0.040	0.043	0.035
9			0.066	0.095	0.054	0.058	0.049	0.053	0.056
10			0.058	0.027	0.041	0.052	0.055	0.062	0.065
11			0.042	0.019	0.020	0.048	0.112	0.080	0.116
12			0.028	0.006	0.016	0.038	0.227	0.204	0.171
13							0.142	0.081	0.127
14							0.060	0.066	0.096
15							0.050	0.057	0.060
16							0.048	0.047	0.039
17							0.034	0.037	0.034
18							0.027	0.036	0.028
19							0.026	0.031	0.023
20							0.023	0.026	0.016
21							0.008	0.018	0.011
22							0.007	0.007	0.009
23							0.001	0.003	0.005
24							0.000	0.000	0.000

2.C5 Region 5: Urban Area (Kuala Lumpur)

No. of Block	Storm Duration								
	15-min	30-min	60-min	180-min	6-hr	12-hr	24-hr	48-hr	72-hr
1	0.184	0.097	0.056	0.048	0.033	0.003	0.003	0.001	0.006
2	0.448	0.161	0.061	0.060	0.045	0.051	0.011	0.011	0.014
3	0.368	0.400	0.065	0.078	0.092	0.074	0.015	0.015	0.019
4		0.164	0.096	0.095	0.096	0.086	0.021	0.018	0.023
5		0.106	0.106	0.097	0.107	0.140	0.025	0.024	0.027
6		0.072	0.164	0.175	0.161	0.206	0.032	0.027	0.040
7			0.108	0.116	0.118	0.180	0.047	0.031	0.049
8			0.103	0.096	0.102	0.107	0.052	0.033	0.050
9			0.068	0.093	0.096	0.081	0.055	0.041	0.054
10			0.065	0.062	0.091	0.064	0.076	0.068	0.067
11			0.058	0.050	0.037	0.007	0.087	0.129	0.072
12			0.050	0.030	0.023	0.003	0.103	0.142	0.110
13							0.091	0.132	0.087
14							0.080	0.096	0.070
15							0.075	0.053	0.060
16							0.054	0.036	0.052
17							0.048	0.033	0.050
18							0.035	0.030	0.047
19							0.027	0.026	0.031
20							0.023	0.020	0.025
21							0.017	0.017	0.022
22							0.012	0.012	0.014
23							0.009	0.004	0.009
24							0.002	0.001	0.003

APPENDIX 2.D EXAMPLE – IDF CURVE DEVELOPMENT

Problem:

Develop IDF curves for 2, 5, 10, 20, 50 and 100 year ARI using the annual maximum rainfall data of 5, 10, 15, 30¸60, 180, 360, 540, 720, 900 and 1440 minutes durations for a raingauge station located at Ampang, Selangor. The required rainfall data is given in Table 2.D1.

Table 2.D1: Annual Maximum Rainfall Data at Ampang Station

Year	Annual Maximum Rainfall (mm) Data for Various Durations (minutes)												
	5	10	15	30	45	60	120	180	360	540	720	900	1440
1980	20.2	35.3	40.8	53.0	59.8	65.4	72.5	72.5	72.5	72.5	72.5	122.4	123.5
1981	34.3	41.0	45.2	49.5	62.6	65.2	76.1	87.2	97.5	113.0	113.0	113.0	114.5
1982	22.3	26.3	35.9	54.9	64.3	69.1	89.0	89.0	89.0	89.0	89.0	89.0	102.5
1983	12.5	15.7	23.5	46.0	65.7	84.7	111.0	111.0	111.0	113.0	113.0	113.0	119.5
1984	38.9	44.5	50.2	67.2	75.5	75.5	75.5	75.5	84.0	90.8	92.5	93.0	93.0
1985	50.1	50.4	50.7	51.6	52.8	58.1	74.8	83.5	84.0	84.0	84.5	89.5	118.5
1986	32.2	36.5	36.5	38.2	54.3	59.0	77.9	89.5	101.7	107.0	108.8	135.2	181.5
1987	8.4	11.0	12.7	25.4	35.0	46.7	64.0	64.0	64.0	64.0	64.0	64.0	74.0
1988	34.5	34.5	34.5	53.3	69.5	85.8	103.0	103.0	103.0	103.0	103.0	103.0	103.0
1989	11.9	23.7	31.5	31.5	38.6	42.0	56.8	64.5	78.5	83.1	91.5	111.0	115.5
1990	18.7	33.0	33.0	33.0	42.4	54.4	59.5	59.5	61.0	66.0	66.0	66.0	88.0
1991	5.2	10.4	15.6	31.2	40.0	47.7	63.2	68.0	86.0	86.0	86.5	86.5	95.2
1992	8.8	17.5	23.7	28.9	42.0	51.7	89.7	107.0	107.0	107.0	107.0	107.0	107.0
1993	10.1	20.3	30.1	54.8	71.4	86.3	105.0	119.0	121.5	121.5	121.5	122.4	124.5
1994	17.0	20.4	24.9	43.0	49.8	54.2	72.0	72.1	77.0	77.0	77.0	77.0	77.0
1995	27.5	30.2	34.0	42.5	50.9	57.4	65.5	66.0	66.5	66.5	67.0	67.0	82.5
1996	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0	87.0
1997	31.3	39.2	47.1	48.5	48.5	50.3	65.4	79.2	109.3	113.5	113.5	113.5	113.5
1998	25.5	29.5	31.8	41.8	51.1	56.0	59.0	59.5	59.7	59.8	60.0	60.1	61.6
1999	26.9	30.2	33.5	44.3	57.6	69.3	84.5	103.5	112.5	112.5	112.5	112.5	119.0
2000	21.1	26.9	35.3	49.8	58.4	65.6	97.3	104.6	111.6	111.7	111.9	112.0	116.1
2001	20.5	29.5	39.0	66.9	87.2	95.6	113.7	114.1	118.1	118.4	119.1	119.3	140.1
2002	19.6	33.4	41.1	61.7	81.3	94.0	115.8	117.5	118.4	118.7	119.1	138.6	139.0
2003	16.5	25.6	35.5	66.0	95.7	102.7	110.0	110.5	110.7	110.9	111.0	111.0	133.4
2004	58.5	58.5	58.5	58.5	67.8	77.9	89.7	91.9	92.3	92.5	92.5	92.5	128.2
2005	14.6	27.0	36.7	62.1	70.5	83.2	90.8	91.0	99.4	103.6	104.3	105.0	110.0
2006	15.1	28.2	39.2	68.7	87.6	111.6	140.7	142.9	144.3	144.6	144.7	144.9	145.0
2007	18.3	29.8	42.8	69.8	90.5	103.2	133.1	137.3	137.7	138.0	138.1	138.3	191.9
2008	18.2	27.6	34.0	61.0	81.0	87.6	90.0	90.1	90.3	90.5	98.1	98.3	98.5
2009	11.8	21.5	27.1	45.5	61.7	72.4	76.0	76.1	76.3	76.4	117.8	139.1	139.4

Solution:

Reference	Calculation	Output
Figure 2.1	Step 1: Collect annual maximum rainfall data of selected durations from the Ampang station, which is given in Table 2.D1.	
Figure 2.1	Step 2: Calculate the cumulative rainfall value for each duration to check the consistencies by mass curve method. A sample mass curve is shown in Figure 2.D1 for the storm duration of 24 hours.	Figure 2.D1
Figure 2.1	Step 3: Fit the Raw Rainfall Data to Various Frequency Distributions. For this purpose, calculate the mean and standard deviation for the annual maximum rainfall values of each duration, as shown in Table 2.D2.	Table 2.D2.
Figure 2.1	Step 4: Select the Most Suitable Frequency Distribution Method that fits the Data. Various statistical distribution should be used to determine the most suitable method that fits the data set best. This step is required to estimate the design rainfall of various ARIs. The Gumble distribution is used in this example, which used the following equation. $RF_T = RF_{mean} + \sigma K$ where, RF_T = The magnitude of the rainfall for a return period of T year; RF_{mean} = The arithmetic mean value of the annual rainfall values of various durations; σ = The standard deviation from the mean; K = The frequency factor for extreme values, which depends on the type of distribution used.	
Figure 2.1	Step 5: Calculate the Amounts of Rainfalls for Various Duration and Selected ARIs (2, 5, 10, 20, 50 and 100 year). Calculate the frequency factors for the required ARIs as given in Table 2.D3. Multiply the standard deviation values with the corresponding frequency factors of various ARIs and add to the mean annual maximum rainfall values to get the design rainfall as given in Table 2.D4.	Table 2.D3 and 2.D4
Figure 2.1	Step 6: Convert the Rainfall into Intensity (Table 2.D5) and Plot Various Durations in the Log-Log Graph for the Selected ARIs. Plot the data of Table 2.D4 to get the IDF curves, as shown in Figure 2.D2. If the graphs are not smooth based on the actual statistical data, adjust the data to produce smooth graphs.	Table 2.D5 and Figure 2.D2

Table 2.D2: Calculation of Mean and Standard Deviation for the Data given in Table 2.D1

Term	Annual Maximum Rainfall (mm) for Various Durations (minutes)												
	5	10	15	30	45	60	120	180	360	540	720	900	1440
Mean	24.6	31.5	37.1	51.2	63.4	72.0	87.0	91.2	95.7	97.4	99.6	104.4	114.8
Std. Dev.	16.9	14.9	13.6	14.2	16.9	18.9	21.9	22.2	21.7	21.8	21.5	23.5	28.7

Table 2.D3: Calculation of Frequency Factors for the Selected ARIs

ARI (Year)	Frequency Factor
2	-0.1681
5	0.7371
10	1.3379
20	1.9134
50	2.6585
100	3.2166

Table 2.D4: Calculation of Design Rainfall Depths for Various ARIs

ARI (Year)	Design Rainfall (mm) Data for Various Storm Durations (minute)										
	5	10	15	30	60	180	540	720	900	1080	1440
2	21.2	29.4	36.7	55.3	67.0	77.0	86.6	87.9	93.9	99.9	110.9
5	23.1	32.0	40.0	60.1	75.3	85.3	96.0	97.2	103.2	109.2	120.2
20	25.5	35.5	44.4	66.4	86.0	96.0	108.2	109.3	115.3	121.3	132.3
50	27.1	37.7	47.1	70.4	92.8	102.8	115.9	117.0	123.0	129.0	140.0
100	28.3	39.3	49.1	73.4	97.9	107.9	121.7	122.7	128.7	134.7	145.7

Table 2.D5: Calculation of Design Rainfall Intensities for Various ARIs

ARI (Year)	Design Rainfall Intensities (mm/hr) for Various Storm Durations (minute)										
	5	10	15	30	60	180	540	720	900	1080	1440
2	216.00	173.93	146.89	99.31	80.68	68.81	41.63	29.16	15.35	10.41	7.99
5	260.62	215.28	181.55	126.47	101.09	85.90	51.55	35.87	18.62	12.60	9.61
20	300.00	247.74	206.54	142.56	114.63	97.25	58.12	40.32	20.80	14.05	10.69
50	348.34	287.74	237.68	159.22	127.61	108.12	64.43	44.59	22.88	15.45	11.72
100	389.05	325.09	274.16	181.55	144.41	122.19	72.58	50.11	25.57	17.25	13.05

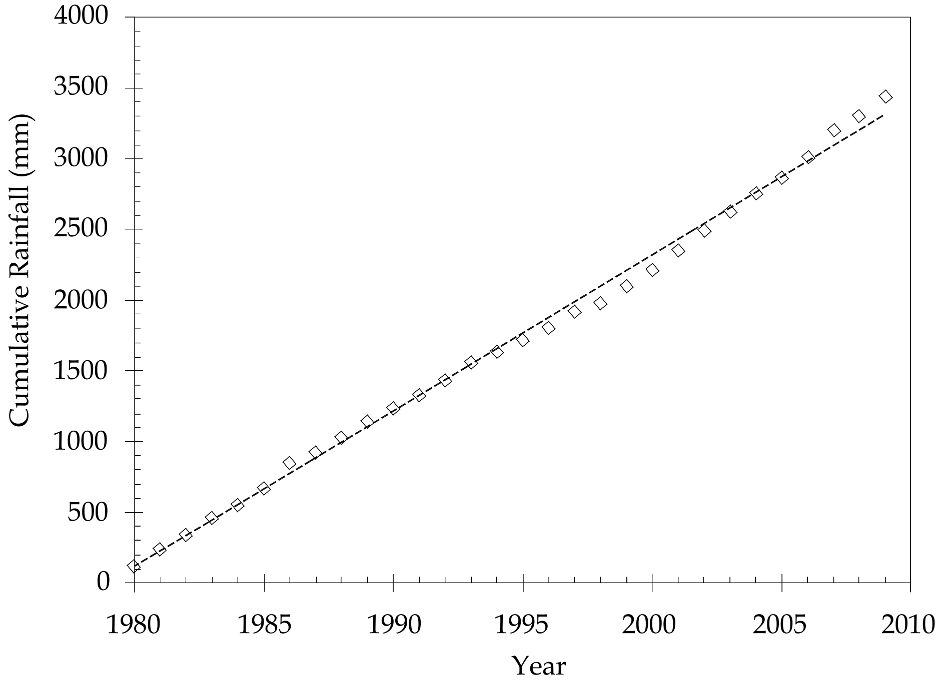


Figure 2.D1: Mass Curve to Check Consistency of the Raw Rainfall Data (24 hours Duration)

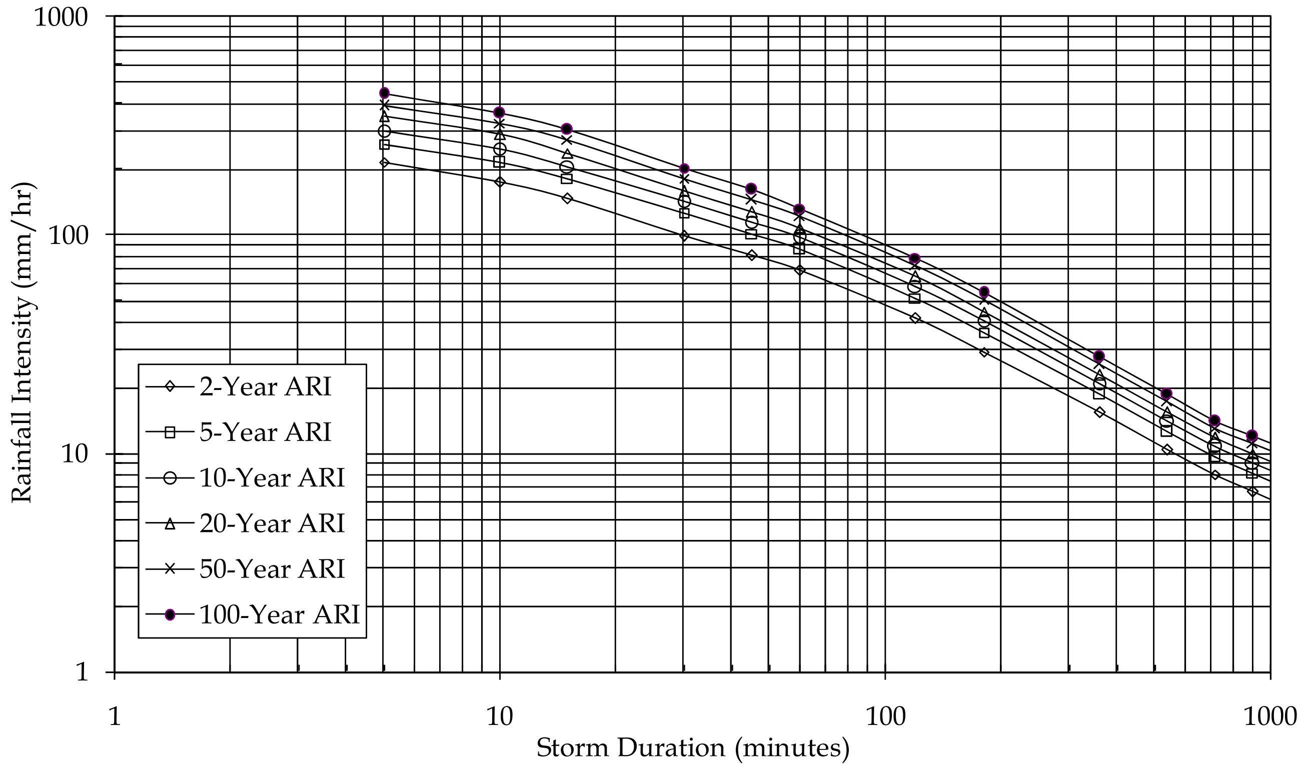


Figure 2.D2: Developed IDF Curves for Ampang Station

APPENDIX 2.E EXAMPLE - DESIGN TEMPORAL PATTERNS

Problem:

Determine the design rainfall temporal pattern for the raw data given in Table 2.E1.

Table 2.E1: Recorded Rainfall Data for Temporal Pattern

A	B	C	D	E	F	G	H
Storm Duration = 25 min Number of Intervals = 5			Rain (mm) at 5-minute Interval				
Date	Total Rain (mm)	Rank	1st	2nd	3rd	4th	5th
12.01.1972	58.7	1	9.2	12.8	12.8	12.8	11.1
07.12.1983	58.3	2	8.0	11.7	14.0	13.3	11.2
21.07.1992	54.0	3	9.5	12.3	11.2	10.9	10.2
03.12.1985	52.3	4	11.2	17.3	9.3	4.0	10.5
19.01.1999	51.1	5	4.8	13.3	12.0	10.7	10.3
27.04.2003	50.0	6	10.7	7.2	10.9	11.2	10.0
14.06.2005	46.7	7	9.3	10.3	9.7	9.0	8.3
30.06.1989	43.7	8	11.2	10.7	9.3	7.0	5.5
04.02.1990	40.9	9	8.1	8.9	9.9	6.9	7.1
17.11.2001	40.2	10	9.6	12.8	10.7	4.0	3.1

Solution:

Reference	Calculation	Output
	Step 1: Select the required storm duration and find about ten (10) dates of extreme most rainfall events, as given in Table 2.E1.	
	Step 2: Collect the rainfall amounts from the nearby automatic rainfall station for the required intervals as given in Table 2.5. The selected highest storm burst with dates and total rainfall amount are collected from the raw rainfall data of 5 minutes interval and listed in Table 2.E1. The Table also shows the distribution of raw rainfall data for the most extreme events in that area. Data of Column D to H are extracted from the five minute rainfall intervals.	
	Step 3: Assign rank for each interval based on the rainfall amount (1 for the highest amount and so on). This arrangement is given in columns I to M of Table 2.E2. For same rainfall amounts in the intervals, the average ranks should be used, as shown in the first event of Table 2.E2.	Table 2.E2
	Step 4: Determine percentage of rain occurred in each interval as given in columns N to P of Table 2.E2	Table 2.E2

Reference	Calculation	Output
	Step 5: Calculate the mean ranks (columns I to M) and percentages of rainfall (columns N to P) for each interval as given in the row (MV).	Table 2.E2
	Step 6: Assign the mean percentages of rainfall for each interval based on the new mean rank as given in the row (NR for columns I to M).	Table 2.E2
	Step 7: Convert the percentage rainfall into fraction of total rainfall and plot the temporal pattern (TPF for column I to M).	Table 2.E2
	Step 8: Multiply the TPF values with the design rainfall amount (mm) to get the distribution of rainfall in each time interval.	

Table 2.E2: Calculation for the Determination of Design Rainfall Temporal Pattern

A	B	C	D	E	F	G	H	I	J	K	L	M	N	M	N	O	P
Storm Duration = 25 min Number of Intervals = 5			Rain (mm) at 5-minute Interval					Rank of Each Rainfall Interval (Mean Rank for the Intervals with Same Rainfall Values)					Percentage of Rain for the Interval				
Date	Total Rain (mm)	Rank	1st	2nd	3rd	4th	5th	1st	2nd	3rd	4th	5th	1st	2nd	3rd	4th	5th
12.01.1972	58.7	1	9.2	12.8	12.8	12.8	11.1	5.0	2.0	2.0	2.0	4.0	16	22	22	22	19
07.12.1983	58.3	2	8.0	11.7	14.0	13.3	11.2	4.0	3.0	1.0	2.0	5.0	14	20	24	23	19
21.07.1992	54.0	3	9.5	12.3	11.2	10.9	10.2	5.0	1.0	2.0	3.0	4.0	18	23	21	20	19
03.12.1985	52.3	4	11.2	17.3	9.3	4.0	10.5	2.0	1.0	4.0	5.0	3.0	21	33	18	8	20
19.01.1999	51.1	5	4.8	13.3	12.0	10.7	10.3	5.0	1.0	2.0	3.0	4.0	9	26	23	21	20
27.04.2003	50.0	6	10.7	7.2	10.9	11.2	10.0	3.0	5.0	2.0	1.0	4.0	21	14	22	22	20
14.06.2005	46.7	7	9.3	10.3	9.7	9.0	8.3	3.0	1.0	2.0	4.0	5.0	20	22	21	19	18
30.06.1989	43.7	8	11.2	10.7	9.3	7.0	5.5	1.0	2.0	3.0	4.0	5.0	26	24	21	16	13
04.02.1990	40.9	9	8.1	8.9	9.9	6.9	7.1	3.0	2.0	1.0	5.0	4.0	20	22	24	17	17
17.11.2001	40.2	10	9.6	12.8	10.7	4.0	3.1	3.0	1.0	2.0	4.0	5.0	24	32	27	10	8
				Mean Value (MV)				3.4	1.9	2.1	3.3	4.3	19	24	22	18	17
				New Rank (NR)				4	1	2	3	5	3	1	2	4	5
				Rainfall Pattern (in % as per the New Rank)				18	24	22	19	17					
				Design Temporal Pattern, in fraction (TPF)				0.18	0.24	0.22	0.19	0.17					

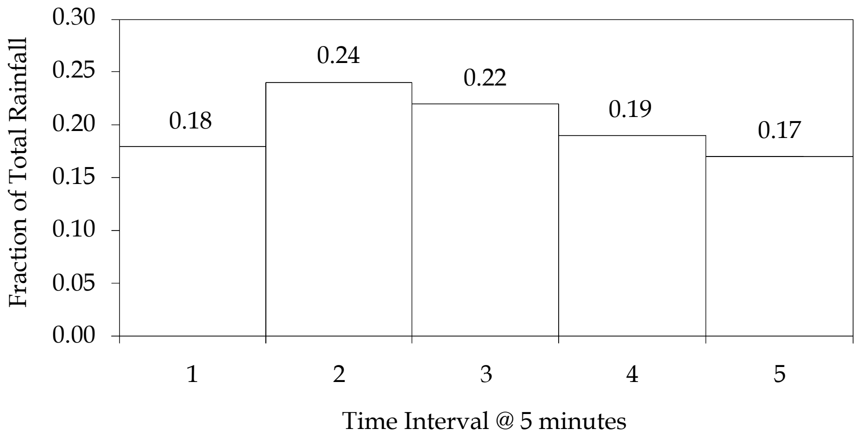


Figure 2.E1: Design Rainfall Temporal Pattern

For a 100 yr ARI rainfall with total 70 mm and 25 min duration, the distribution design rainfall, based on Figure 2.E1, is given in Table 2.E3 below.

Table 2.E3: Distribution of Design Rainfall according to the Temporal Pattern

Storm Duration = 25 min Number of Intervals = 5	Rain (mm) at 5-minute Interval				
Total Design Rainfall (mm) for 100 year ARI	1st	2nd	3rd	4th	5th
70.0	12.6	16.8	15.4	13.3	11.9

APPENDIX 2.F EXAMPLE – RUNOFF QUANTITY ESTIMATION

2.F1 Rational Method and RHM

Problem:

Using Rational Method procedure to calculate a 20 year ARI peak discharge from a subcatchment area of 40.7 ha in Wangsa Maju, Kuala Lumpur (Figure 2.F1). Also develop the runoff hydrograph using the RHM for drain AB based on 5 and 10 minute durations design storms.

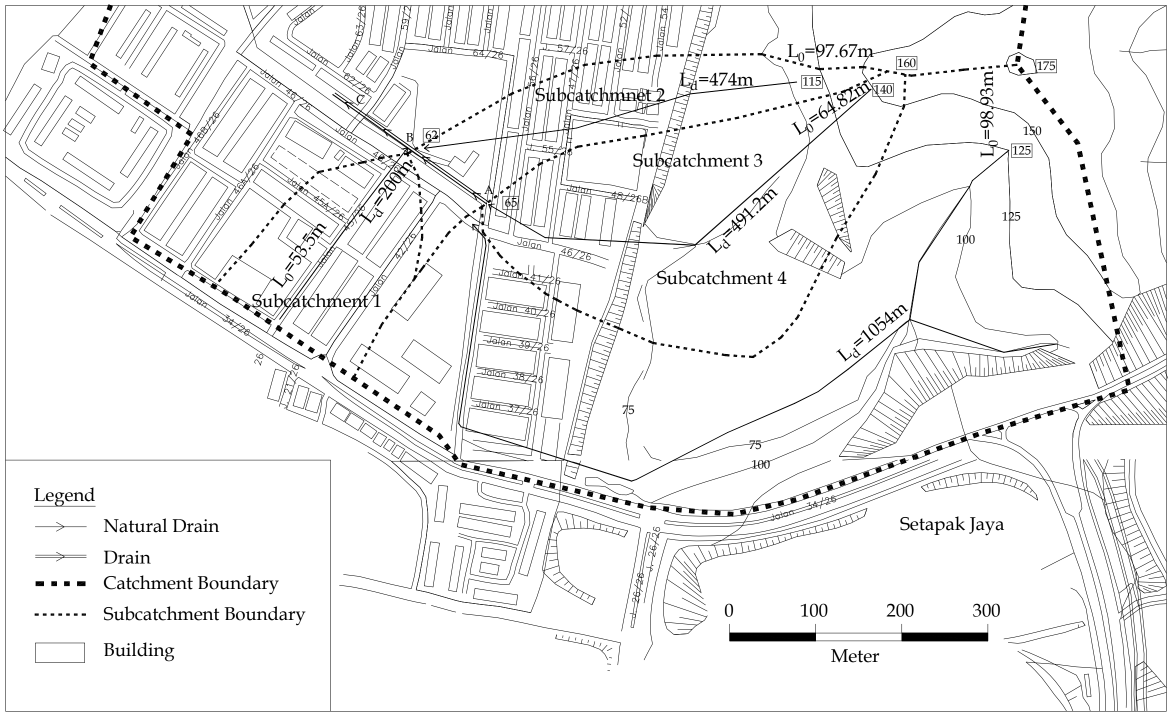


Figure: 2.F1: Drainage Subcatchment Wangsa Maju

Solution:

Reference	Calculation	Output
	Step 1:Delineate the subcatchments, as shown in Figure 2.F1. The subcatchments in this example are identified as 1, 2, 3 and 4 (Table 2.F1).	Table 2.F1
	Step 2: Calculate the subcatchment areas. The area for subcatchment 1 is 3.87 ha, subcatchment 2 is 4.95 ha, subcatchment 3 is 8.61 ha, and subcatchment 4 is 23.22ha.	
Table 2.5	Step 3: Select runoff coefficient (C). The C value for subcatchment 1 is 0.8 for flat and apartment area, 0.4 for open spaces (grass cover), for subcatchment 2 contain two types of landuse, which is 0.8 for flat and apartment area, 0.4 for open spaces (grass cover), for subcatchment 3 contain two type of landuse which is 0.75 for condominium area and 0.5 for open spaces (bare soil), and for subcatchment 4 also contain two types of landuse which is 0.9 for commercial and business centres and 0.4 for open spaces (grass cover).	

Reference	Calculation		Output
Equation 2.4	Step 4: Calculate C_{avg} values. $C_{avg} = [(3.67 \times 0.80) + (0.2 \times 0.4)]/(3.67 + 0.2)$	=	0.78
	Step 5: Determine overland sheet flow path length, L_o for the flow paths in every subcatchment to calculate the time of concentration of each subcatchment. Follow the guideline in Table 2.F2 to estimate L_o.		Table 2.F2
	Step 6: Determine slope of overland surface in percent (%) $S = [(68 - 66)/53.5] \times 100\%$	=	3.74%
Table 2.1 Table 2.2.	Step 7: Calculate t_o. Use the Horton's n^* Value (use n^*=0.015 from Table 2.2). $t_o = \frac{107.n^*.L^{1/3}}{S^{1/5}} = \frac{107 x0.015 x 53.5^{1/3}}{3.74^{1/5}}$	=	4.6 min
Figure 2.F1	Step 8: Determine channel length, L_d for the channels in every subcatchment.		
	Step 9: Calculate area of the channel (triangular shape with slope = 1:2). From the site visit the depth of the channel is assumed at 0.3 m and width = 1.2 m		
	Step 10: Calculate wetted perimeter of the channel (P) $P = 2(0.6^2 + 0.3^2)^{1/2}$	=	1.34 m
	Step 11: Calculate hydraulic radius by, R = A/P $A = 1/2 \times 0.3 \times 1.2$ $= 0.18 \text{ m}^2$ $R = 0.18/1.34$	=	0.134 m
	Step 12: Determine the friction slope of the channel, s (m/m) by dividing the different elevation by the length of channel. $S = (66 - 62)/200$	=	0.02 m/m
Table 2.1 Table 2.3	Step 13: Calculate travel time in channel, t_d (use n=0.015 from Table 2.3). $t_d = \frac{n.L}{60R^{2/3} S^{1/2}} = \frac{0.015 \text{x} 200}{60 \text{x} 0.134^{2/3} \text{x} 0.02^{1/2}}$	=	1.4 min
	Step 14: Calculate time of concentration by using equation below. $t_c = t_o + t_d$ $= 4.6 + 1.4$	=	6.0 min
	Step 15 : Calculate Peak Discharge, Q (Table 2.F3)		Table 2.F3
	Drain AB: This drain discharge water from subcatchments 3 and 4. From Table 2.F1 A_3 = 8.61ha, C_3 = 0.57, and the t_c = 4.4 min, while A_4 = 23.22ha, C_4 = 0.51, and the t_c = 7.5 min. Hence, the total area drained by drain AB is 31.83ha and $\sum CA = C_3A_3 + C_4A_4 = (0.57x8.61) + (0.51x23.22)$	=	16.75ha
Equation 2.3	The time of concentration used is 7.5 min the larger of two drain times. The rainfall intensity, i is taken from Table 2.B1 for 20 year return period storm duration and 7.5min time of concentration at Ibu Pejabat JPS Station, 300.36mm/hr. Calculate peak flow, $Q_{peak.}$		

Reference	Calculation	Output
	Drain BC: This drain conveys flow from all 4 subcatchments. Subcatchment 3 and 4 through drain AB, while subcatchment 2 and 1 directly to point B. There are thus 3 possible paths for water to reach at point B. The time of concentration is the largest of the flow times. The flow time for flowing coming from drain AB is 7.5 min plus 1.18 min travel time = 8.7 min; the flow time from subcatchment 1 and 2 is 6.0 min and 4.8 min, respectively. Thus, the time of concentration for pipe BC is taken as 8.7 min. Then for rainfall intensity and Q_{peak}, use the same method for the previous drain.	

Table 2.F1: Characteristics of the Drainage Catchment

Subcatchment ID	Landuse	Area (ha)		Runoff Coefficient, C (Table 2.5)		Area Weighted C
		Developed Area	Undeveloped Area	Developed Area	Undeveloped Area	
1	Condo	3.67	0.20	0.80	0.40	0.78
2	Apartment	3.22	1.73	0.80	0.40	0.66
3	Terrace	2.37	6.24	0.75	0.50	0.57
4	Industry	5.10	18.12	0.90	0.40	0.51

Table 2.F2: Calculation of Time of Concentration (t_c)

Subcatchment	L_o	S, %	t_o, min	L_d	A	P	R	S, m/m	t_d, min	t_c, min
1	53.50	3.74	4.6	200.00	0.18	1.34	0.134	0.02	1.4	6.0
2	97.67	46.10	3.4	474.00	0.18	1.34	0.134	0.11	1.4	4.8
3	64.82	30.85	3.2	491.20	0.18	1.34	0.134	0.15	1.2	4.4
4	98.93	50.54	3.4	1054.00	0.18	1.34	0.134	0.06	4.1	7.5

Table 2.F3: Calculation for Peak Discharge, Q

Drain	Total Area (ha)	$\sum CA$	t_c (min)	I (mm/hr)	Flow, Q (m^3/s)
AB	31.83	16.750	7.5	300.36	13.98
BC	40.65	23.036	8.7	281.83	18.03

Hydrograph Development using RHM

Reference	Calculation	Output
Table 2.F3	Step 1: Select the drain AB for which the hydrograph need to be generated.	
Figure 2.F1 Figure 2.5	Step 2:Determine whether the storm duration (d) is shorter or longer than the time of concentration (t_c) of the drain. This information is necessary to determine the type of the hydrograph by RHM. In this case, when the d is 5 minutes (d < t_c) it will be type 2 hydrograph and when the d is 10 minutes (d > t_c) it will be type 1 hydrograph.	
Table 2.F3 Figure 2.5(a)	Step 3: Now for the storm duration (d) of 10 minutes which is longer than the t_c, follow similar procedure and construct a trapezoidal hydrograph with height as 13.98 m^3/s and base as d+t_c=10+7.5= 17.5 minutes, as shown in Figure 2.F2(a). The coordinates of the triangular hydrograph are A(0,0), B(7.5,13.98), C(10,13.98) and D(17.5,0).	Figure 2.F2(a)
	Step 4: When d=5 minutes, base of the hydrograph will be 2t_c=2x7.5=15 minutes.	
Figure 2.5(b)	Step 5: Construct a triangular hydrograph with height as 13.98 m^3/s and base as 15 minutes, as shown in Figure 2.F2(b). The coordinates of the triangular hydrograph are A(0,0), B(7.5, 13.98) and C(15,0).	Figure 2.F2(b)

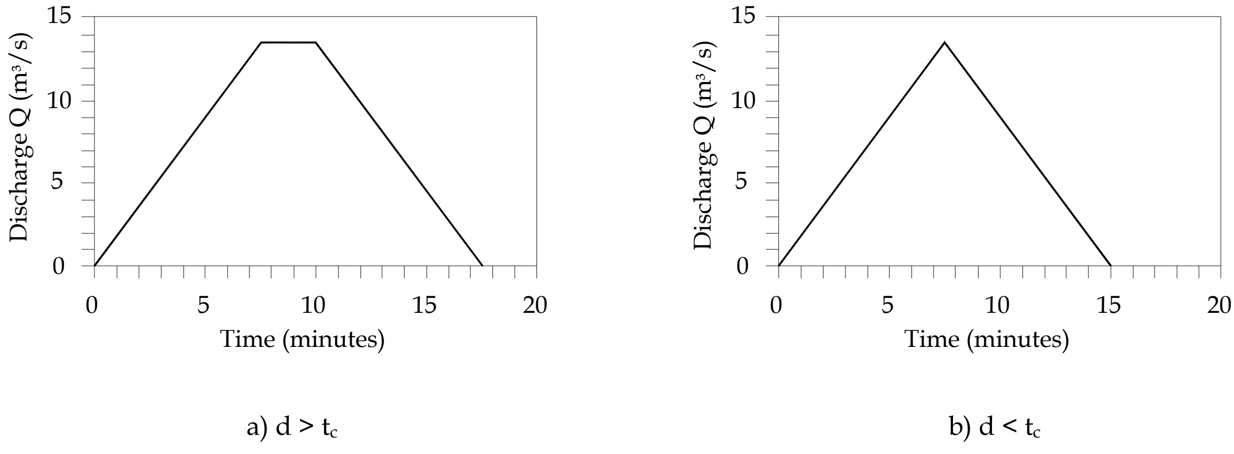


Figure 2.F2 : Hydrographs by RHM

2.F2 Time-Area Hydrograph Method

Problem:

Using the Time-Area Hydrograph Method calculate a 20 year ARI runoff hydrograph from a 97 hectare mixed urban area located in Wangsa Maju, Kuala Lumpur. The study area is shown in Figure 2.F3.

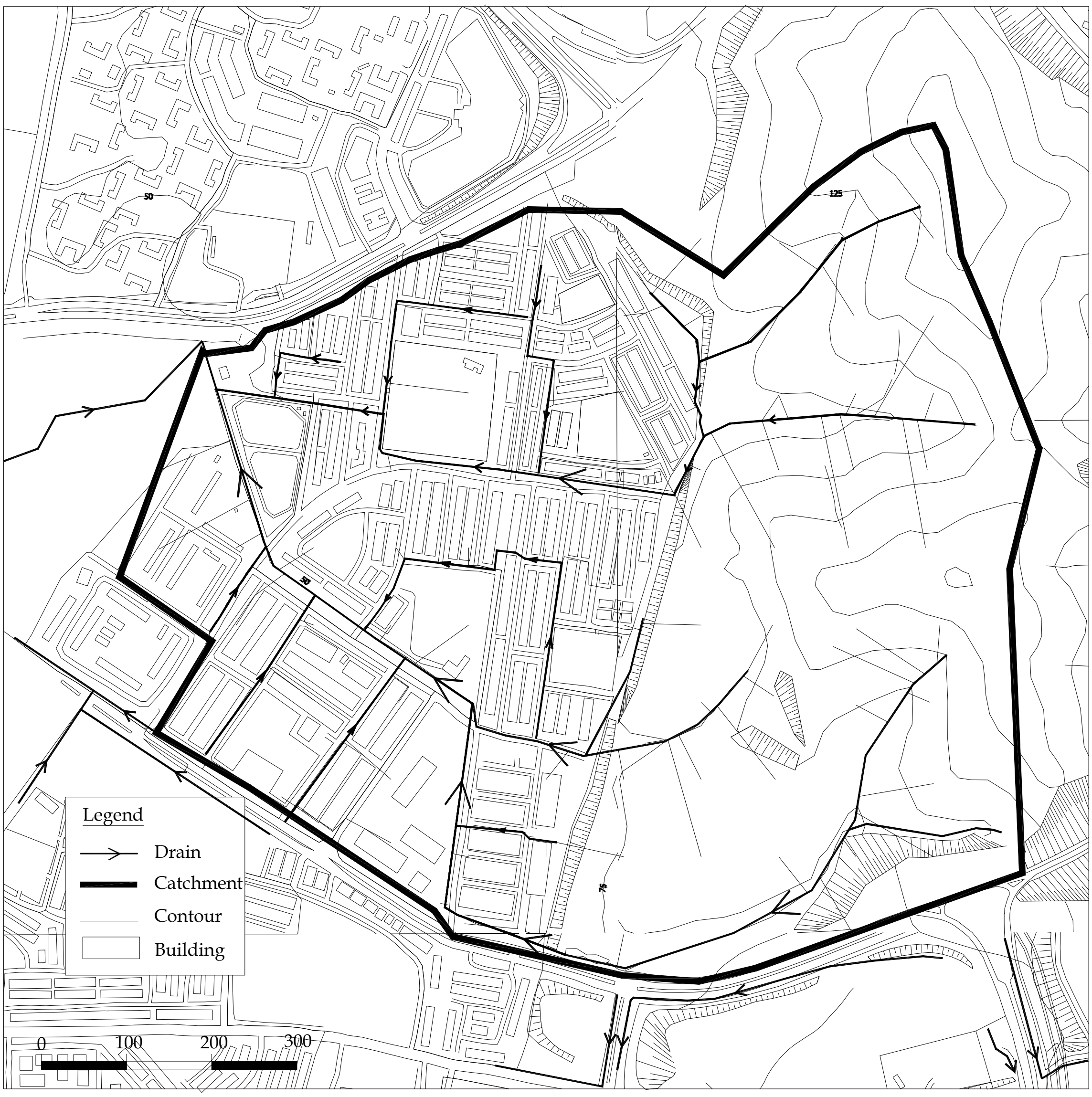


Figure 2.F3: Catchment Area in Wangsa Maju, Kuala Lumpur

Solution:

Reference	Calculation	Output
	A. Isochrone Development Step 1: Setting grid system	Figure 2.F4

Reference	Calculation	Output
Table 2.1	Step 2: Estimate overland flow time, t_o $$t_o = \frac{107.n^* .L^{1/3}}{S^{1/5}}$$	Table 2.F4
Table 2.1	Step 3: Estimate drain time, t_d $$t_d = \frac{n.L}{60R^{2/3} S^{1/2}}$$ However, flow velocity through the drain channel is assumed, in this example, 1m/s for the sake of simplicity.	Table 2.F4
	Step 4: Calculate t_c at each grid point	Table 2.F4
	Step 5: Plot contour of equal t_c for 5 minute interval to produce isochrones; 5, 10, 15, 20, and 25 min	Figure 2.F5
	Step 6: Estimate area between isochrones using AutoCAD	Table 2.F5
	Step 7: Calculate rainfall excess	
Table 2.B1	By using rainfall data of 20 year ARI and 30 minutes duration at station Ibu Pejabat JPS, Kuala Lumpur in Table 2.B1, rainfall intensity is:	
Equation 2.2	$$i = \frac{\lambda T^{\kappa}}{(d+\theta)^{\eta}}$$ $$= \frac{61.976 \times 20^{0.145}}{(0.5 + 0.122)^{0.818}}$$ $= 141.11$mm/hr Total Rainfall = 141.11 x (30/60) = 70.56mm	
Appendix 2.C5	By using Appendix 2.C5, rainfall temporal pattern is obtained from fraction for 30-min storm duration as follows : 0-5 : 0.097 x 70.56 = 6.84mm 5-10 : 0.161 x 70.56 = 11.36mm 10-15 : 0.400 x 70.56 = 28.22mm 15-20 : 0.164 x 70.56 = 11.57mm 20-25 : 0.106 x 70.56 = 7.48mm 25-30 : 0.072 x 70.56 = 5.08mm	
Table 2.6	Using Table 2.6 to assume the losses, in decay form, as follows : 0-5 : 3.5mm 5-10 : 3.0mm 10-15 : 2.5mm	

	15-20 : 2.0mm 20-25 : 1.5mm 25-30 : 1.0mm	
	Rainfall Excess = Rainfall Temporal Pattern – Losses 0-5 : 6.84 – 3.5 = 3.34mm 5-10 : 11.36 – 3.0 = 8.36mm 10-15 : 28.22 – 2.5 = 25.72mm 15-20 : 11.57 – 2.0 = 9.57mm 20-25 : 7.48 – 1.5 = 5.98mm 25-30 : 5.08 – 1.0 = 4.08mm	Table 2.F6
	Step 8: Calculate hydrograph ordinate Step 9: Identify the peak discharge From the Table 2.F7 or Figure 2.F6 the peak discharge is	Table 2.F7 Figure 2.F6 32.81 m^3/s.

Table 2.F4: Calculation Time of Concentration

Grid no.	L_d (m)	L_o (m)	n*	S (%)	t_o (min)	V (m/s)	t_d (min)	t_c (min)
A6	140.44	101.83	0.015	0.5	8.6	1.0	2.3	11.0
A7	202	133.1	0.015	0.5	9.4	1.0	3.4	12.8
B5	78.58	-	-	-	-	1.0	1.3	1.3
B6	167.63	-	-	-	-	1.0	2.8	2.8
B7	258.38	37	0.015	0.5	6.1	1.0	4.3	10.4
B8	375.11	22.25	0.015	0.5	5.2	1.0	6.3	11.4
B9	547.27	8.44	0.015	0.5	3.8	1.0	9.1	12.9
C5	180.81	12.28	0.015	0.5	4.3	1.0	3.0	7.3
C6	191.56	88.22	0.015	0.5	8.2	1.0	3.2	11.4
C7	297.05	71.38	0.015	0.5	7.6	1.0	5.0	12.6
C8	397.1	19.2	0.015	0.5	4.9	1.0	6.6	11.6
C9	470.35	71.03	0.015	0.5	7.6	1.0	7.8	15.5
C10	698.4	-	-	-	-	1.0	11.6	11.6
D5	300.33	-	-	-	-	1.0	5.0	5.0
D6	329.84	20.51	0.018	0.5	6.1	1.0	5.5	11.6
D7	543.55	23.32	0.018	0.5	6.3	1.0	9.1	15.4
D8	457.39	25.15	0.018	0.5	6.5	1.0	7.6	14.1
D9	528.72	49.41	0.018	0.5	8.1	1.0	8.8	16.9
D10	633	78.56	0.018	0.5	9.5	1.0	10.6	20.0
D11	736.21	146.81	0.018	0.5	11.7	1.0	12.3	23.9
E4	514.4	-	-	-	-	1.0	8.6	8.6
E5	312.32	106.01	0.018	0.5	10.5	1.0	5.2	15.7
E6	534.59	-	-	-	-	1.0	8.9	8.9
E7	626.41	24.81	0.018	0.5	6.5	1.0	10.4	16.9
E8	513.82	87.91	0.018	0.5	9.8	1.0	8.6	18.4
E9	598.97	-	-	-	-	1.0	10.0	10.0
E10	700.41	-	-	-	-	1.0	11.7	11.7
E11	795.95	21.73	0.018	0.5	6.2	1.0	13.3	19.4
F3	827.07	35.29	0.018	0.5	7.3	1.0	13.8	21.0
F4	756.59	-	-	-	-	1.0	12.6	12.6
F5	631.7	-	-	-	-	1.0	10.5	10.5
F6	536.7	-	-	-	-	1.0	8.9	8.9
F7	748.69	17.25	0.018	0.5	5.7	1.0	12.5	18.2
F8	869.59	-	-	-	-	1.0	14.5	14.5
F9	954.2	-	-	-	-	1.0	15.9	15.9
F10	745.74	43.69	0.018	0.5	7.8	1.0	12.4	20.2
F11	828.38	19.46	0.018	0.5	6.0	1.0	13.8	19.8
F12	1009.78	15.88	0.018	0.5	5.6	1.0	16.8	22.4
G4	981.95	51.69	0.018	0.5	8.2	1.0	16.4	24.6
G5	624.4	99.51	0.018	0.5	10.3	1.0	10.4	20.7
G6	638.56	42.88	0.018	0.5	7.7	1.0	10.6	18.4
G7	775.02	152.36	0.018	0.5	11.8	1.0	12.9	24.7
G8	966.13	22.37	0.018	0.5	6.2	1.0	16.1	22.3
G9	863.39	-	-	-	-	1.0	14.4	14.4

Table 2.F4: Calculation Time of Concentration (Continued)

Grid no.	L_d (m)	L_o (m)	n	S (%)	t_o (min)	V (m/s)	t_d (min)	t_c (min)
G10	830.13	39.1	0.018	0.5	7.5	1.0	13.8	21.3
G11	899.7	88.56	0.018	0.5	9.9	1.0	15.0	24.9
G12	1108.96	42.6	0.018	0.5	7.7	1.0	18.5	26.2
H3	1000	95.92	0.018	3.43	6.9	1.0	16.7	23.6
H4	944.6	27.04	0.018	2.22	4.9	1.0	15.7	20.7
H5	852.6	-	-	-	-	1.0	14.2	14.2
H6	751.78	16.37	0.018	23	2.6	1.0	12.5	15.1
H7	710.12	76.6	0.018	17	4.6	1.0	11.8	16.5
H8	973.69	76.85	0.018	31.4	4.1	1.0	16.2	20.3
H9	908.11	100.11	0.018	25.4	4.7	1.0	15.1	19.8
H10	908.99	63.86	0.018	11.1	4.8	1.0	15.1	19.9
H11	902.89	157	0.018	3.38	8.1	1.0	15.0	23.2
H12	1225.4	-	-	-	-	1.0	20.4	20.4
I3	1087.2	61.9	0.018	0.9	7.8	1.0	18.1	25.9
I4	1011.6	-	-	-	-	1.0	16.9	16.9
I5	896.86	35.18	0.018	0.4	7.6	1.0	14.9	22.5
I6	880.33	65.93	0.018	34	3.8	1.0	14.7	18.5
I7	796.26	164.71	0.018	37.24	5.1	1.0	13.3	18.4
I8	1030.23	71.77	0.018	22.67	4.3	1.0	17.2	21.5
I9	1024.6	57.79	0.018	12.41	4.5	1.0	17.1	21.6
I10	958.8	123.63	0.018	4.9	7.0	1.0	16.0	23.0
I11	923.51	201.87	0.018	1.1	11.0	1.0	15.4	26.4
I12	1318.7	28.14	0.018	10.7	3.6	1.0	22.0	25.6
J4	1035.5	86.72	0.018	36.0	4.2	1.0	17.3	21.4
J5	995.64	29.53	0.018	37.3	2.9	1.0	16.6	19.5
J6	998.59	70.44	0.018	42.0	3.8	1.0	16.6	20.4
J7	931.68	179.89	0.018	38.1	5.2	1.0	15.5	20.8
J8	1030.23	151.48	0.018	29.0	5.2	1.0	17.2	22.4
J9	1622.78	47.58	0.018	32.6	3.5	1.0	27.0	30.5
J10	1540.87	-	-	-	-	1.0	25.7	25.7
J11	1443.19	26.86	0.018	22	3.1	1.0	24.1	27.2
J12	1385.47	80.71	0.018	7.31	5.6	1.0	23.1	28.7
K3	1169.14	92.02	0.018	60.9	3.8	1.0	19.5	23.3
K4	1056.37	139.61	0.018	64.1	4.3	1.0	17.6	22.0
K5	1083.1	37.39	0.018	54	2.9	1.0	18.1	21.0
K6	1078.29	65.58	0.018	63.5	3.4	1.0	18.0	21.4
K7	1040.3	174.25	0.018	46.8	5.0	1.0	17.3	22.3
K8	1742.31	42.35	0.018	70.8	2.9	1.0	29.0	31.9
K9	1690.2	36.24	0.018	41.4	3.0	1.0	28.2	31.2
K10	1494.02	111.44	0.018	22.43	5.0	1.0	24.9	29.9
K11	1498.53	96.72	0.018	30	4.5	1.0	25.0	29.5

Table 2.F5: Areas between the Isochrones

ID	Isochrones	Area (m^2)
A_1	0 – 5	44449
A_2	5 – 10	79304
A_3	10 – 15	229404
A_4	15 – 20	213852
A_5	20 – 25	160342
A_6	25 >	45306

Table 2.F6: Rainfall Excess for 20 year ARI Design Rainfall

Time (min)	Total Rainfall (mm)	Losses (mm)	Rainfall Excess (mm)
5	6.84	3.5	3.34
10	11.36	3.0	8.36
15	28.22	2.5	25.72
20	11.57	2.0	9.57
25	7.48	1.5	5.98
30	5.08	1.0	4.08

Table 2.F7: Time-Area Hydrograph Method

Time (min)	Area (m^2)	Rainfall Excess (mm)						Hydrograph (m^3/s)
		3.34	8.36	25.72	9.57	5.98	4.08	
0	0	0						0.00
5	44449	0.49	0					0.49
10	79304	0.88	1.24	0				2.12
15	229404	2.55	2.21	3.81	0			8.57
20	213852	2.38	6.39	6.80	1.42	0		16.99
25	160342	1.79	5.96	19.67	2.53	0.89	0	30.83
30	45306	0.50	4.47	18.33	7.32	1.58	0.60	**32.81**
35			1.26	13.75	6.82	4.57	1.08	27.48
40				3.88	5.11	4.26	3.12	16.38
45					1.45	3.20	2.91	6.32
50						0.90	2.18	3.08
55							0.62	0.62

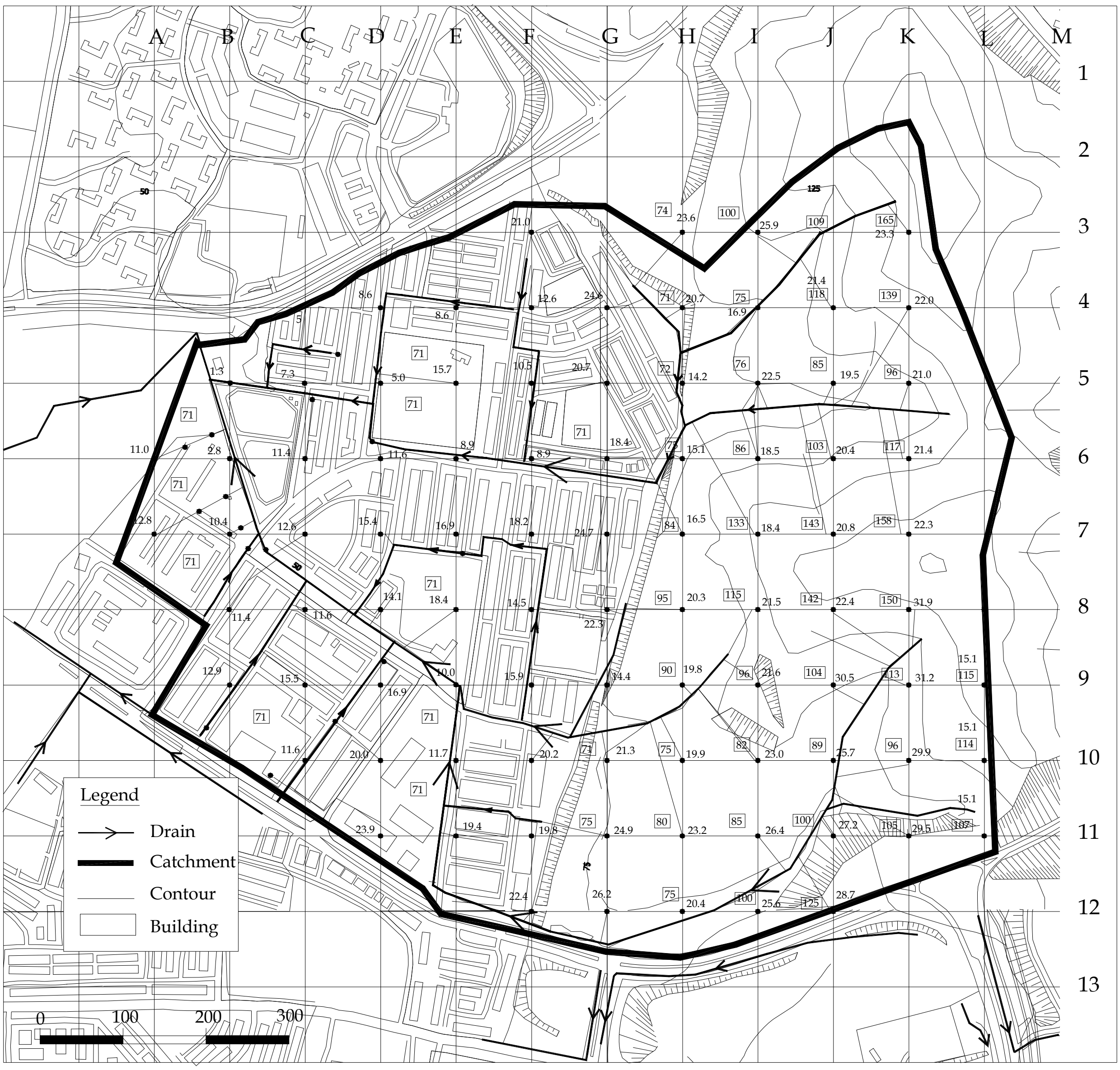


Figure 2.F4: Grid System to Calculate t_c

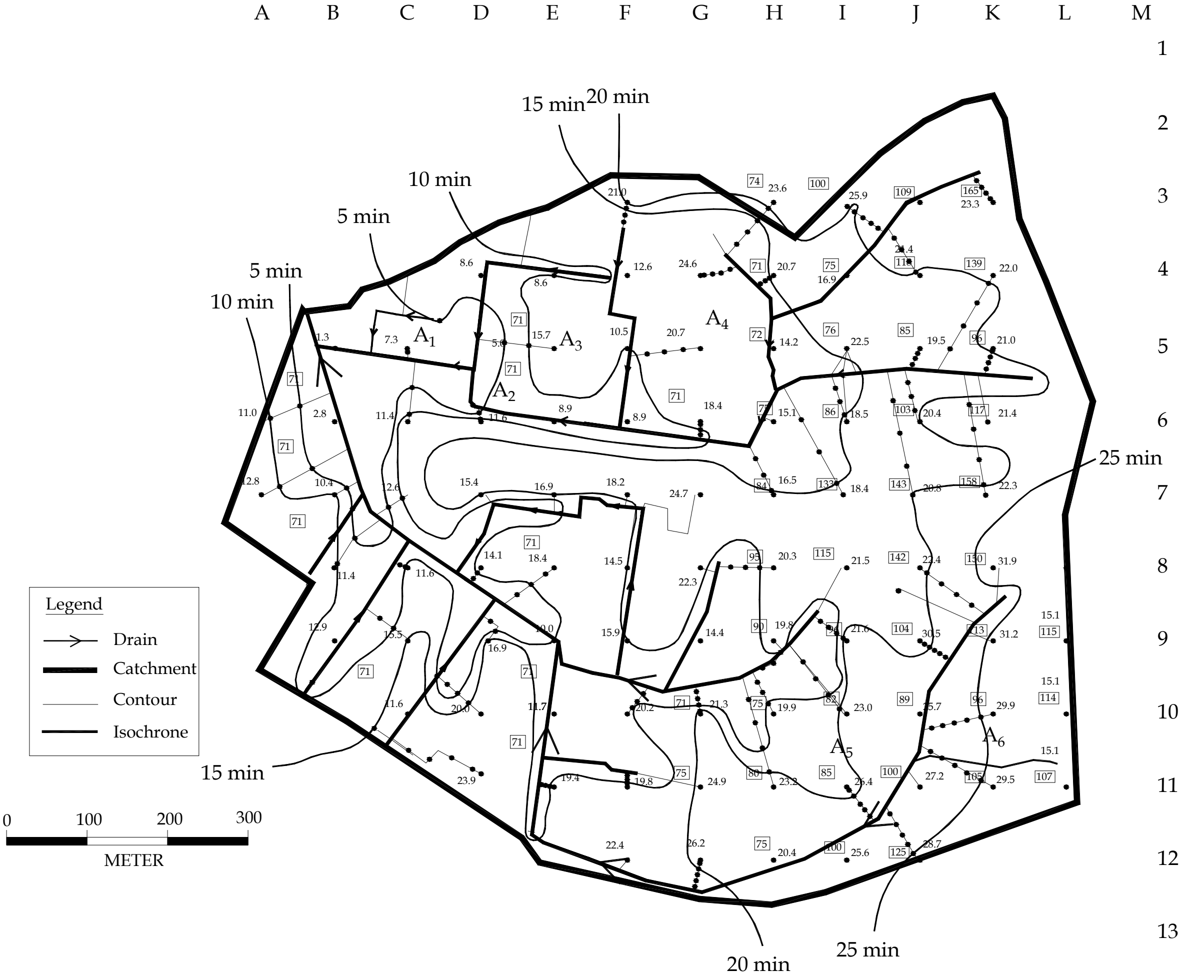


Figure 2.F5: Catchment Area with the Developed Isochrone Lines

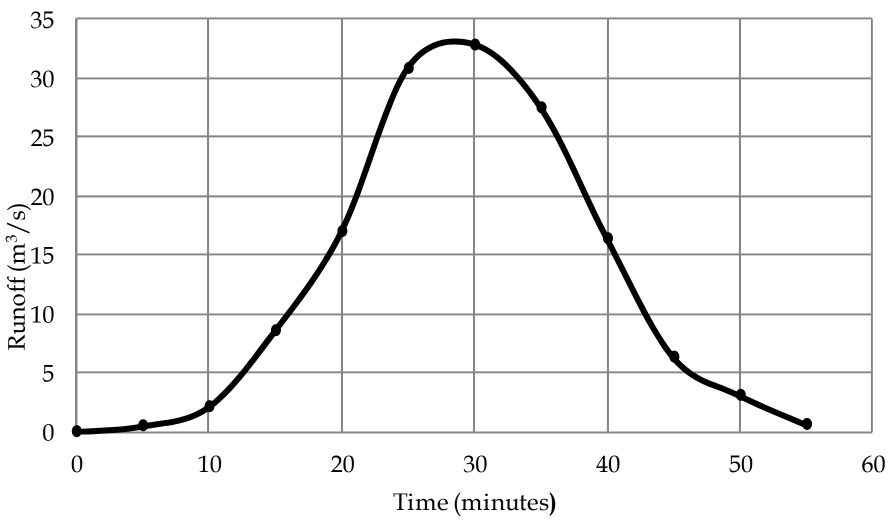


Figure 2.F6: Runoff Hydrograph

APPENDIX 2.G EXAMPLE - POND ROUTING

Problem:

Given is a triangular inflow hydrograph with Q_p = 10.93 m³/s at t_c = 11.65 min (Figure 2.G1). Determine the outflow hydrograph from a storage pond using the routing procedure in Section 2.5. Given are pond stage-storage curve (Figure 2.G2) and stage-discharge curve of the outlet structure, orifice and spillway combined (Figure 2.G3).

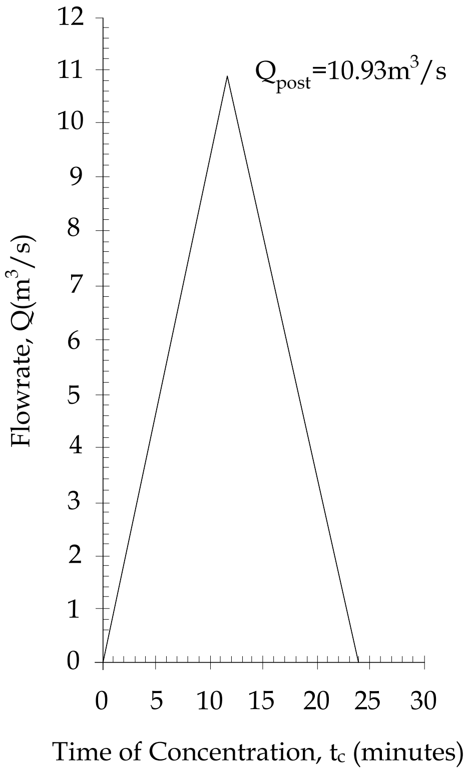


Figure 2.G1: Triangular Hydrograph

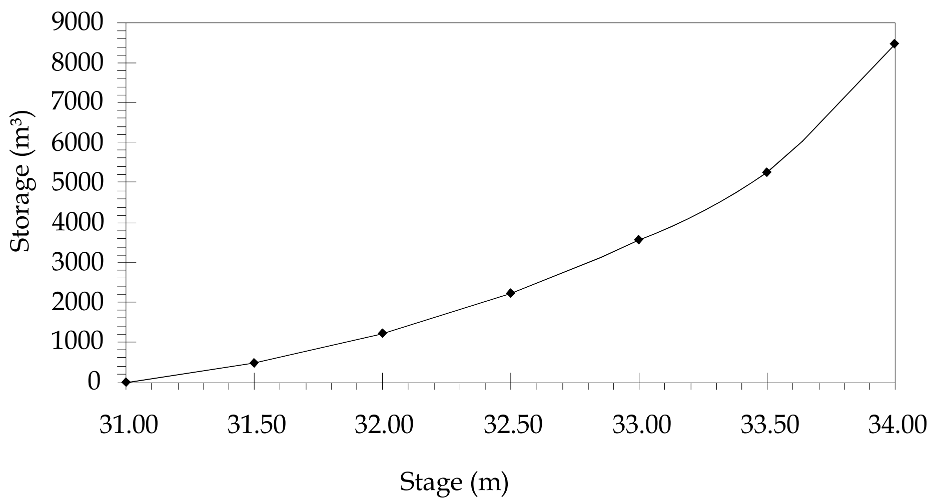


Figure 2.G2: Stage-storage Curve

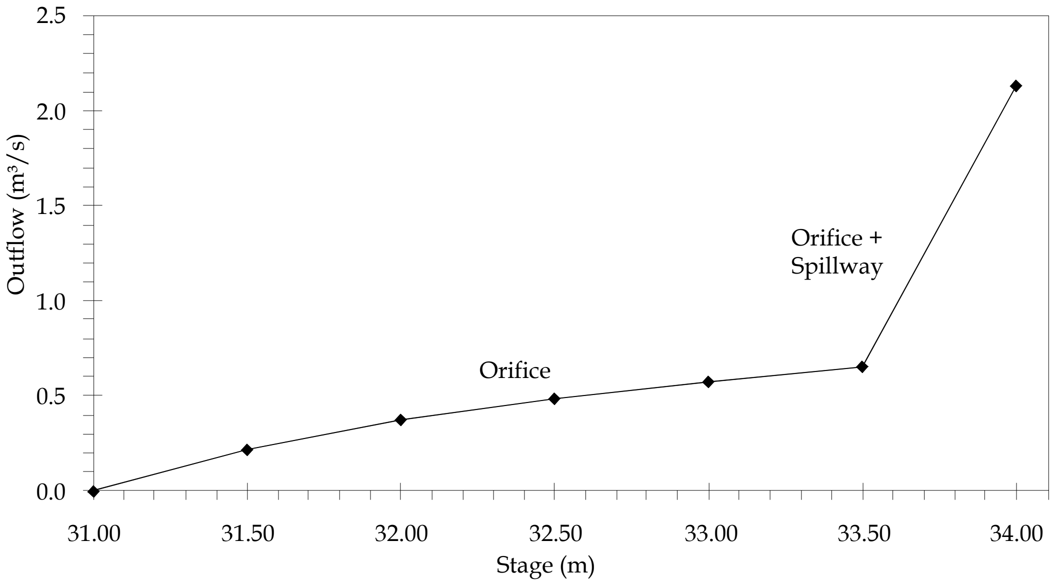


Figure 2.G3: Stage-discharge Curve (Composite)

Solution

Reference	Calculation	Output
	Develop storage indicator curve - For each stage point determine storage(S) and discharge (O) - For each discharge point determine storage indicator ($S_2/\Delta t + O_2/2$)	Table 2.G1 Figure 2.G4
Section 2.5	Develop outflow hydrograph	Table 2.G2 Figure 2.G5

Table 2.G1: Storage Indicator Numbers – Orifice (Δt = 2.5 min. or 150 sec)

Stage (m)	Discharge, O_2 (m^3/s)	Storage Volume, S_2 (m^3)	$O_2/2$ (m^3/s)	$S_2/\Delta t$ (m^3/s)	$S_2/\Delta t + O_2/2$ (m^3/s)
31.00	0.000	0.000	0.000	0.000	0.000
31.50	0.217	486.838	0.109	3.246	3.354
32.00	0.376	1216.161	0.188	8.108	8.296
32.50	0.485	2228.079	0.243	14.854	15.097
33.00	0.574	3562.700	0.287	23.751	24.038
33.50	0.651	5260.137	0.326	35.068	35.393
34.00	2.131	8489.051	1.065	56.594	57.659

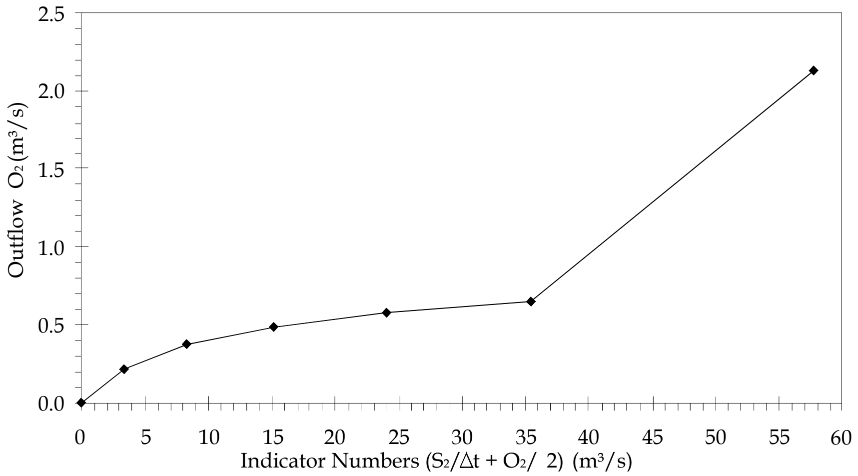


Figure 2.G4: Storage Indicator Curve (Composite)

Table 2.G2: Final Routing Table

Time (hr)	Inflow (I) (m³/s)	$(I_1 + I_2)/2$ (m³/s)	$S_1/\Delta t + O_1/2$ (m³/s)	Outflow (O_1) (m³/s)	$S_2/\Delta t + O_2/2$ (m³/s)	Outflow (O_2) (m³/s)
0.00	0.000	0.000	0.000	0.000	0.000	0.000
0.04	2.350	1.173	0.000	0.000	1.173	0.078
0.08	4.690	3.520	1.173	0.078	4.614	0.463
0.13	7.040	5.866	4.614	0.463	10.017	0.715
0.17	9.390	8.212	10.017	0.715	17.515	0.904
0.21	10.140	9.761	17.515	0.904	26.371	1.040
0.25	7.790	8.963	26.371	1.040	34.295	1.142
0.29	5.440	6.617	34.295	1.142	39.770	1.669
0.33	3.100	4.270	39.770	1.669	42.372	1.971
0.38	0.750	1.924	42.372	1.971	42.324	1.966
0.42	0.000	0.375	42.324	1.966	40.734	1.781
0.46	0.000	0.000	40.734	1.781	38.953	1.574
0.50	0.000	0.000	38.953	1.574	37.379	1.391
0.54	0.000	0.000	37.379	1.391	35.988	1.229
0.58	0.000	0.000	35.988	1.229	34.759	1.148
0.63	0.000	0.000	34.759	1.148	33.611	1.133
0.67	0.000	0.000	33.611	1.133	32.479	1.118
0.71	0.000	0.000	32.479	1.118	31.360	1.104
0.75	0.000	0.000	31.360	1.104	30.257	1.090
0.79	0.000	0.000	30.257	1.090	29.167	1.076
0.83	0.000	0.000	29.167	1.076	28.092	1.062
0.88	0.000	0.000	28.092	1.062	27.030	1.048
0.92	0.000	0.000	27.030	1.048	25.982	1.035
0.96	0.000	0.000	25.982	1.035	24.947	1.022
1.00	0.000	0.000	24.947	1.022	23.926	1.008
1.04	0.000	0.000	23.926	1.008	22.918	0.991
1.08	0.000	0.000	22.918	0.991	21.927	0.975
1.13	0.000	0.000	21.927	0.975	20.951	0.960
1.17	0.000	0.000	20.951	0.960	19.992	0.944

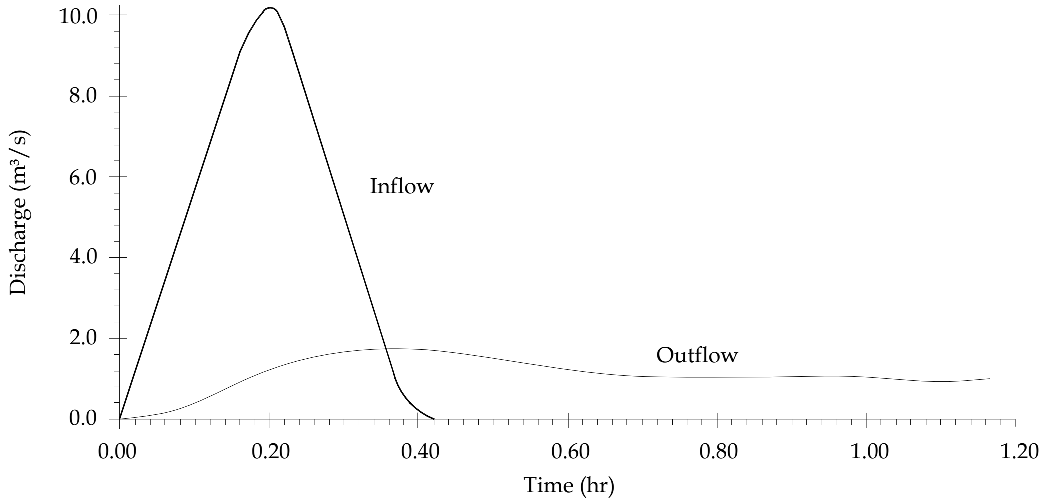


Figure 2.G5: Inflow and Outflow Hydrograph

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