14 FLOW ESTIMATION AND ROUTING

14.1 INTRODUCTION

This chapter presents methods and procedures for the estimation, routing, and attenuation of peak flows and flow volumes for sub-catchments as a prerequisite to the design of stormwater conveyance systems and quantity control facilities. These methods and procedures may be used in conjunction with Chapter 16 to design stormwater system networks.

14.2 DESIGN ARI

Ideally, the design ARI should be selected on the basis of economic efficiency. In practice, however, economic efficiency is typically replaced by the concept of level of protection. The selection of this level of protection (or ARI), that actually refers to the exceedance probability of the design storm rather than the probability of failure of the drainage system, is largely based on local experience. ARIs to be used for the design of minor and major stormwater systems are provided in Chapter 4.

A fundamental assumption, which is made in flow calculations, is that the design flow with a given ARI is produced by a design storm rainfall of the same ARI. Strictly speaking, the ARI of the flow is also influenced by other variable factors such as catchment antecedent wetness. However, the methods presented in this Manual have been designed such that the above assumption is reasonable.

Regardless of the design basis, it is recommended that performance of the drainage system be examined for a range of ARIs to ensure that the system will perform satisfactorily.

14.3 STEPS IN FLOW ESTIMATION

The process of flow estimation generally involves the following main steps. These apply both for estimating design flows, and in estimating flows for historical events.

- determination of time of concentration
- rainfall estimation (see Chapter 13)
- calculation of rainfall excess
- conversion of rainfall excess to runoff
- runoff routing

Modern computer models generally use all of the above steps. Some earlier manual procedures, such as the Rational Method, simplify or combine one or more steps to reduce computations. This simplification is acceptable on small, simple catchments but is a significant source of error in more complex situations.

14.4 TIME OF CONCENTRATION

The *time of concentration* is the flow travel time from the most hydraulically remote point in the contributing catchment area to the point under study. The concept of time of concentration is important in all methods of flow estimation as it can be assumed that the rainfall occurring during the time of concentration is directly related to flow rate.

The time of concentration (t_c) is often considered to be the sum of the time of travel to an inlet plus the time of travel in the stormwater conveyance system. In the design of stormwater drainage systems, this can be the sum of the overland flow time and the times of travel in street gutters, roadside swales, stormwater drains, drainage channels, small streams, and other waterways. A number of methods, mostly using empirical equations, are provided below for estimating the time of concentration for urban catchments.

14.4.1 Components of Flow Time

Depending on the particular location, the calculation of t_c will include one or a number of components as shown in Table 14.1.

Table 14.1 Flow Time Components

Flow Type	Components
Overland or 'sheet' flow	• natural surfaces • landscaped surfaces • impervious surfaces
Roof to drain system	• residential roofs • commercial/industrial roofs
Open channel	• open drains • kerbs and gutters • roadside table drains • monsoon drains • engineered waterways • natural channels
Underground pipe	• downpipe to street gutter • pipe flow within lots including roof drainage, car parks, etc • street drainage pipe flow

14.4.2 Calculation of Flow Time

This section gives procedures for the calculation for each of the flow components in Table 14.1. Some of these procedures are subject to uncertainty; the designer must therefore check that the results are physically meaningful.

For very small, simple catchments, it is acceptable to adopt the simplified assumptions instead of performing detailed calculations.

(a) Overland Flow Time

Overland flow can occur on either grassed or paved surfaces. The major factors affecting time of concentration for overland flow are the maximum flow distance, surface slope, surface roughness, rainfall intensity, and infiltration rate.

Overland flow over unpaved surfaces initially occurs as sheet flow for a short time and distance after which it begins to form a runnel or rill and travels thereafter in a natural channel form.

In urban areas, the length of overland flow will typically be less than 50 metres after which the flow will become concentrated against fence, paths or structures or intercepted by open drains.

The formula shown below, known as Friend's formula, should be used to estimate overland sheet flow times. The formula was derived from previous work (Friend, 1954) in the form of a nomograph (Design Chart 14.1) for shallow sheet flow over a plane surface.

$$t_o = \frac{107.n.L^{1/3}}{S^{1/2}} \qquad (14.1)$$

where,

t_o = overland sheet flow travel time (minutes)
L = overland sheet flow path length (m)
n = Manning's roughness value for the surface
S = slope of overland surface (%)

Note : Values for Manning's '*n*' are given in Table 14.2.

Some texts recommend an alternative equation, the Kinematic Wave Equation. However this theoretical equation is only valid for uniform planar homogeneous flow. It is not recommended for practical application.

(b) Overland Flow Time over Multiple Segments

Where the characteristics of segments of a sub-catchment are different in terms of land cover or surface slope, the sub-catchment should be divided into these segments, and the calculated travel times for each combined.

Table 14.2 Values of Manning's 'n' for Overland Flow

Surface Type	Manning *n*	
	Recommended	Range
Concrete/Asphalt**	0.011	0.01-0.013
Bare Sand**	0.01	0.01-0.06
Bare Clay-Loam** (eroded)	0.02	0.012-0.033
Gravelled Surface**	0.02	0.012-0.03
Packed Clay**	0.03	0.02-0.04
Short Grass**	0.15	0.10-0.20
Light Turf*	0.20	0.15-0.25
Lawns*	0.25	0.20-0.30
Dense Turf*	0.35	0.30-0.40
Pasture*	0.35	0.30-0.40
Dense Shrubbery and Forest Litter*	0.40	0.35-0.50

* From Crawford and Linsley (1966) – obtained by calibration of Stanford Watershed Model.

** From Engman (1986) by Kinematic wave and storage analysis of measured rainfall runoff data.

However, it is incorrect to simply add the values of t_0 for each segment as Equation 14.1 is based on the assumption that segments are independent of each other, i.e. flow does not enter a segment from upstream.

Utilising Equation 14.1, the following method (Australian Rainfall & Runoff, 1998) for estimating the total overland flow travel time for segments in series is recommended. For two segments, termed *A* and *B* (Figure 14.1):

$$t_{Total} = t_{A(L_A)} + t_{B(L_A+L_B)} - t_{B(L_A)} \qquad (14.2a)$$

where,

L_A = length of flow for Segment *A*
L_B = length of flow for Segment *B*
$t_{A(LA)}$ = time of flow calculated for Segment *A* over length L_A
$t_{B(...)}$ = time for Segment *B* over the lengths indicated

For each additional segment, the following time value should be added:

$$t_{add} = t_i(L_{Total}) - t_i(L_{Total} - L_i) \qquad (14.2b)$$

where,

t_{add} = time increment for additional segment
L_{Total} = total length of flow, including the current segment *i*
L_i = length of flow for segment *i*
$t_i(...)$ = time for the segment *i* over the lengths indicated

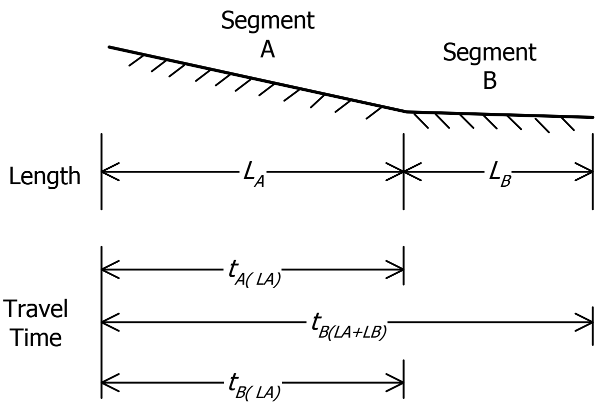


Figure 14.1 Overland Flow over Multiple Segments

This procedure must be applied iteratively because the travel time is itself a function of rainfall intensity.

(c) Roof Drainage Flow Time

While considerable uncertainty exists in relation to flow travel time on roofs, the time of flow in a lot drainage system to the street drain, or rear of lot drainage system is generally very small for residential lots and may be adopted as the minimum time of 5 minutes (Chapter 23). However, for larger residential, commercial, and industrial developments the travel time may be longer than 5 minutes in which case it should be estimated using the procedures for pipe and/or channel flow as appropriate.

(d) Kerbed Gutter Flow Time

The velocity of water flowing in kerbed gutters is affected by:

- the roughness of the kerb, gutter and paved surface
- the cross-fall of the pavement
- the longitudinal grade of the kerbed gutter
- the flow carried in the kerbed gutter

The flow normally varies along the length of a kerbed gutter due to lateral surface inflows. Therefore, the flow velocity will also vary along the length of a gutter. As the amount of gutter flow is not known for the initial analysis of a sub-catchment, the flow velocity and hence the flow time cannot be calculated directly. An initial assessment of the kerbed gutter flow time must be made.

An *approximate* kerbed gutter flow time can be estimated from Design Chart 14.2 or by the following empirical equation:

$$t_g = \frac{L}{40\sqrt{S}} \qquad (14.3)$$

where,

t_g = kerbed gutter flow time (minutes)

L = length of kerbed gutter flow (m)

S = longitudinal grade of the kerbed gutter (%)

Equation 14.3 should only be used for L < 100 metres. Kerbed gutter flow time is generally only a small portion of the time of concentration for a catchment. The errors introduced by these approximate methods of calculation of the flow time result in only small errors in the time of concentration for a catchment, and hence high accuracy is not required.

(e) Channel Flow Time

The time stormwater takes to flow along a open channel may be determined by dividing the length of the channel by the average velocity of the flow. The average velocity of the flow is calculated using the hydraulic characteristics of the open channel.

The Manning's Equation is recommended for this purpose:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad (14.4a)$$

From which,

$$t_{ch} = \frac{n.L}{60} R^{2/3} S^{1/2} \qquad (14.4b)$$

where,

V = average velocity (m/s)

n = Manning's roughness coefficient

R = hydraulic radius (m)

S = friction slope (m/m)

L = length of reach (m)

t_{ch} = travel time in the channel (minutes)

Where an open channel has varying roughness or depth across its width it may be necessary to sectorise the flow and determine the average velocity of the flow, to determine the flow time.

(f) Pipe Flow Time

The velocity V in a pipe running just full can be estimated from pipe flow charts such as those in Chapter 25, Appendix 25.B where the flow, pipe diameter, roughness and pipe slope are known. The time of flow through pipe, t_p, is then given by:

$$t_p = \frac{L}{V} \qquad (14.5)$$

where,

L = pipe length (m)

V = average pipe velocity (m/s)

Where the pipe diameter is not known, the diameter can be first estimated given the flow at the upstream end of the pipe reach and the average grade of the land surface between its ends.

As is the case with kerb gutter flow time, pipe flow time is generally only a small portion of the time of concentration for a sub-catchment. The error in the estimated pipe flow time introduced by the adoption of the wrong diameter or slope, or by the assumption that the pipe is flowing full when in fact it is only flowing part full, will not introduce major errors into the calculated peak flow.

In many situations an experienced user will be able to estimate the velocity of flow in a pipe within a reasonable accuracy. Therefore, the pipe flow time can be estimated directly from Equation 14.5.

14.4.3 Time of Concentration for Natural Catchment

For larger systems times of concentration should preferably be estimated on the basis of locally observed data such as the time of occurrence of flood peaks at or near the catchment outlet compared with the time of commencement of associated storms.

For natural/landscaped catchments and mixed flow paths the time of concentration can be found by use of the Bransby-Williams' Equation 14.6 (AR&R, 1987). In these cases the times for overland flow and channel or stream flow are included in the time calculated.

Here the overland flow time including the travel time in natural channels is expressed as:

$$t_c = \frac{F_c . L}{A^{1/10} S^{1/5}} \qquad (14.6)$$

where,

t_c = the time of concentration (minute)

F_c = a conversion factor, 58.5 when area A is in km^2, or 92.5 when area is in ha

L = length of flow path from catchment divide to outlet (km)

A = catchment area (km^2 or ha)

S = slope of stream flowpath (m/km)

14.4.4 Time of Concentration for Small Catchments

Although travel time from individual elements of a system may be very short, the total nominal flow travel time to be adopted for all individual elements within any catchment to its point of entry into the stormwater drainage network shall not be less than 5 minutes.

For small catchments up to 0.4 hectare in area, it is acceptable to use the minimum times of concentration given in Table 14.3 instead of performing detailed calculation.

Table 14.3 Minimum Times of Concentration

Drainage Element	Minimum t_c (minutes)
Roof and property drainage	5
Road inlet	5
Small areas < 0.4 hectare	10

Note: the recommended minimum times are based on the minimum duration for which meaningful rain intensity data are available.

14.5 RATIONAL METHOD

There are two basic approaches to computing stormwater flows from rainfall. The first approach is the Rational Method, which relates peak runoff to rainfall intensity through a proportionality factor. The second approach starts with a rainfall hyetograph, accounts for rainfall losses and temporary storage effects in transit, and yields a discharge hydrograph. The hydrograph approach is discussed later in this Chapter.

14.5.1 Rational Formula

The Rational Formula is one of the most frequently used urban hydrology methods in Malaysia. It gives satisfactory results for small catchments only.

The formula is:

$$Q_y = \frac{C \, . \, {}^{y}I_t \, . \, A}{360} \qquad (14.7)$$

where,

Q_y = y year ARI peak flow (m^3/s)

C = dimensionless runoff coefficient

${}^{y}I_t$ = y year ARI average rainfall intensity over time of concentration, t_c, (mm/hr)

A = drainage area (ha)

Traditionally, design discharges for street inlets and stormwater drains have been computed using the Rational Method, although hydrograph methods also can be used for these purposes. The primary attraction of the Rational Method has been its simplicity. However, now that computerised procedures for hydrograph generation are readily available, computational simplicity is no longer a primary consideration.

Experience has shown that the Rational Method can provide satisfactory estimates of peak discharge on small catchments of up to 80 hectares. For larger catchments, storage and timing effects become significant, and a hydrograph method is needed. Various methods have been devised to form pseudo-hydrographs based on the Rational Method, but their reliability is uncertain and they should only be used for the design of on-site stormwater detention and retention facilities.

14.5.2 Analysis Procedure

A procedure for estimating a peak flow from a single sub-catchment for a particular ARI using the Rational Method is outlined in Figure 14.2. Peak flow estimates should be obtained for both the minor and major drainage systems. An example of peak flow estimation by the Rational Method for multi-subcatchments is given in Table 16.A9 in Chapter 16.

14.5.3 Assumptions

Assumptions used in the Rational Method are as follows:

1. The peak flow occurs when the entire catchment is contributing to the flow.
2. The rainfall intensity is the same over the entire catchment area.
3. The rainfall intensity is uniform over a time duration equal to the time of concentration, t_c.
4. The ARI of the computed peak flow is the same as that of the rainfall intensity, i.e., a 5 year ARI rainfall intensity will produce a 5 year ARI peak flow.

Experience has shown that when applied properly, the Rational Method can provide satisfactory estimates for peak discharges on small catchments where storage effects are insignificant. The Rational Method is not recommended for any catchment where:

- the catchment area is greater than 80 hectares
- ponding of stormwater in the catchment might affect peak discharge.
- the design and operation of large (and hence more costly) drainage facilities is to be undertaken, particularly if they involve storage.

14.5.4 Rainfall Intensity

The rainfall intensity, I, in the Rational Formula represents the average rainfall intensity over a duration equal to the time of concentration for the catchment. Refer to Chapter 13 for details on IDF relationships for estimating design rainfall intensity.

14.5.5 Runoff Coefficient

The runoff coefficient, C, in Equation 14.7 is a function of the ground cover and a host of other hydrologic abstractions. The runoff coefficient accounts for the integrated effects of rainfall interception, infiltration, depression storage, and temporary storage in transit of the peak rate of runoff. When estimating a value for the runoff coefficient, the roles played by these hydrologic processes should be considered. The runoff coefficient depends on rainfall intensity and duration as well as on the catchment characteristics. During a rainstorm the actual runoff coefficient increases as the soil become saturated. The greater the rainfall intensity, the lesser the relative effect of rainfall losses on the peak discharge, and therefore the greater the runoff coefficient.

Recommended runoff coefficient (C) values for rainfall intensities (I) of up to 200 mm/hr may be obtained from Design Chart 14.3 (urban areas) or 14.4 (rural areas), respectively. These design charts are based on Australian Rainfall & Runoff (1977). For $I \geq 400$ mm/hr, a value of $C = 0.9$ should be used for all types of ground cover. For I values between 200 and 400 mm/hr, use linear interpolation between the applicable C values for I=200 mm/hr and I=400 mm/hr.

Design flow rates for stormwater inlets are calculated for local contributing sub-catchments, while those for pipes and open drains are calculated for the accumulated areas draining through each pipe or open channel section or reach. Except for small lot drainage systems, it is inappropriate to simply add the separate flows from each sub-catchment. This over-estimates flow rates. When times-to-peak differ, the total flow from a number of sub-catchments will be less than the sum of the separate flows from each sub-catchment.

The recommended procedure is to calculate the flow at each point along the drainage line from the Rational Formula, using average rainfall intensity and runoff coefficient values corresponding to the times of concentration at that point.

14.5.6 Variation of Subcatchment Conditions

Segments of different landuse or surface slope within a sub-catchment can be combined to produce an average runoff coefficient. For example, if a sub-catchment consists of segments with different landuse or surface slope denoted by i = 1, 2,……, m; the average runoff coefficient is:

$$C_{avg} = \frac{\sum_{i=1}^{m} C_i A_i}{\sum_{i=1}^{m} A_i} \qquad (14.8)$$

where,

C_{avg} = average runoff coefficient

C_i = runoff coefficient of segment i

A_i = area of segment i (ha)

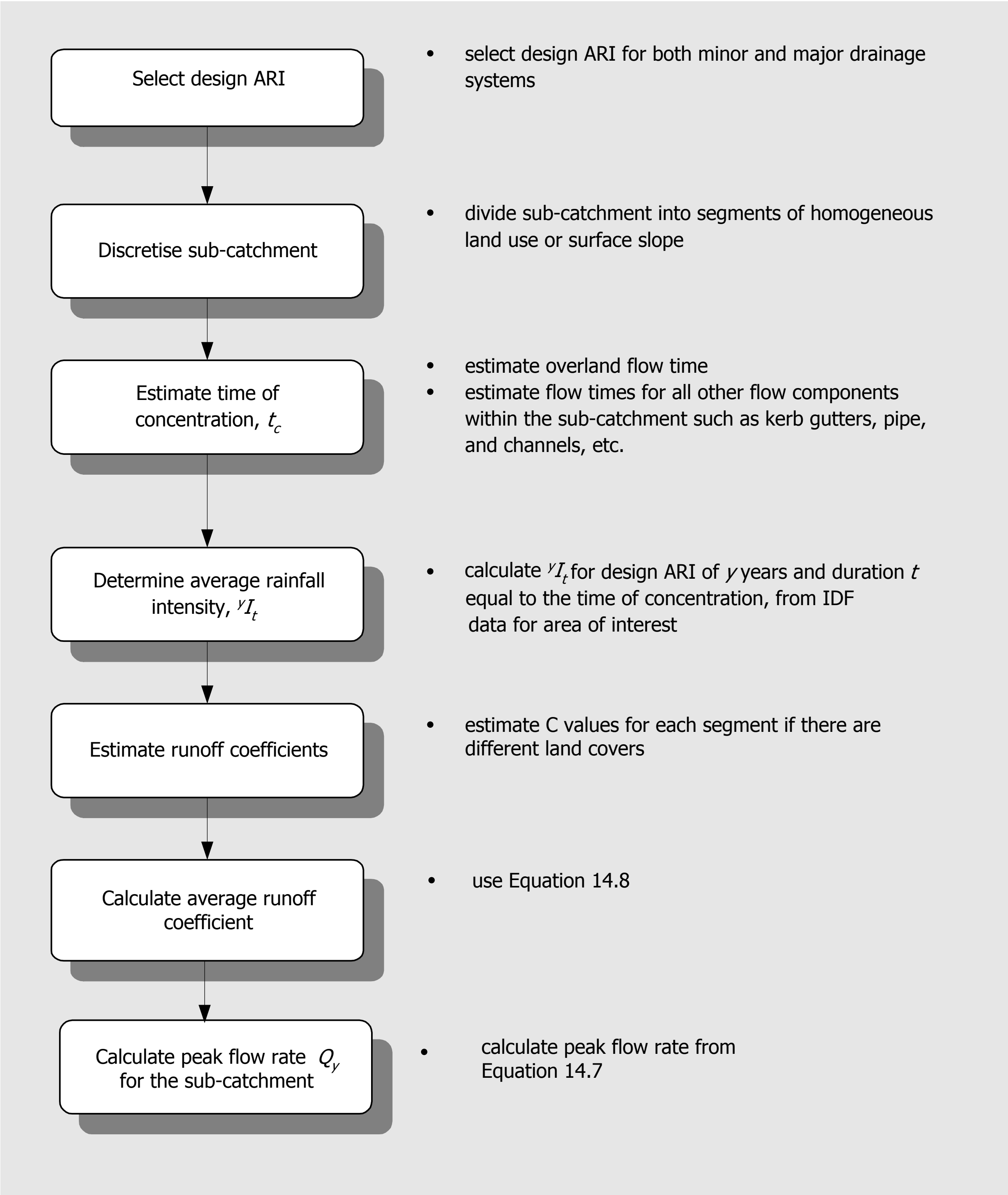


Figure 14.2 General Procedure for Estimating Peak Flow for a Single Sub-catchment Using the Rational Method

14.5.7 The Partial Area Effect

In general, the appropriate t_c for calculation of the peak flow at any point in a catchment is the longest time of flow to that point. However, in some situations, the peak flow may occur when only part of the upstream catchment is contributing, i.e. the product of a lesser *C.A* and a higher intensity ${}^{y}I_t$ (resulting from a lower t_c), produces a greater peak discharge than that if the whole upstream catchment is considered. This is known as the 'partial area' effect.

Usually the above effect results from the existence of a sub-catchment of relatively small *C.A*, but a considerably longer than average t_c. This can result from differences in the shapes and/or surface slopes of sub-catchments within a catchment. Typical catchments that can produce partial area effects are shown in Figure 14.3.

It is important to note that particular sub-catchments may not produce partial area effects when considered individually, but when combined at some downstream point with other sub-catchments, the peak discharge may result when only parts of these sub-catchments are contributing.

The onus is on the designer to be aware of the possibility of the 'partial area effect' and to check as necessary to ensure that the correct peak discharge is obtained.

14.5.8 Limitations

A principal limitation of the Rational Method is that only a peak discharge is produced. Therefore, the Rational Method cannot be used to calculate the volume or shape of the runoff hydrograph, which is required for the design of facilities that use storage such as detention and retention basins.

14.6 HYDROGRAPH METHODS

14.6.1 Basic Concepts

This section discusses methods that should be used to develop a design hydrograph. Hydrograph methods must be used whenever rainfall spatial and temporal variations or flow routing/storage effects need to be considered. Flow routing is important in the design of stormwater detention, water quality facilities, and pump stations, and also in the design of large stormwater drainage systems to more precisely reflect flow peaking conditions in each segment of complex systems.

Hydrograph methods can be computationally involved and computer programs (refer Chapter 17) are usually used to generate runoff hydrographs.

(a) Storm Intensity, Duration and Frequency

Design storm duration is an important parameter that defines the rainfall depth or intensity for a given ARI, and therefore affects the resulting runoff peak. The design storm duration that produces the maximum runoff peak traditionally is defined as the time of concentration. Design storm information is provided in Chapter 13.

The rainfall intensity, ${}^{y}I_t$, used in hydrograph methods represents the *average* rainfall intensity over a particular duration *t* with an average recurrence interval (ARI) of *y*.

Intense rainfalls of short durations usually occur within longer-duration storms rather than as isolated events. These are reflected in the temporal patterns used. It is common practice to compute discharge for several design storms with different durations, and then base the design on the "critical" storm, which produces the maximum discharge. However the "critical" storm duration determined in this way may not be the most critical for storage design.

Recommended practice is to compute the design flood hydrograph for several storms with different durations equal to or longer than the time of concentration for the catchment, and to use the one, which produces the most severe effect on the pond size and discharge for design.

(b) Spatial Distribution

Storm spatial characteristics are important for larger catchments. In general, the larger the catchment and the shorter the rainfall duration, the less uniformly the rainfall is distributed over the catchment. For any specified ARI and duration, the average rainfall depth over an area is less than the point rainfall depth.

The ratio of the areal average rainfall with a specified duration and ARI to the point rainfall with the same duration and ARI is termed the areal reduction factor, and is discussed in Section 13.2.4 of Chapter 13.

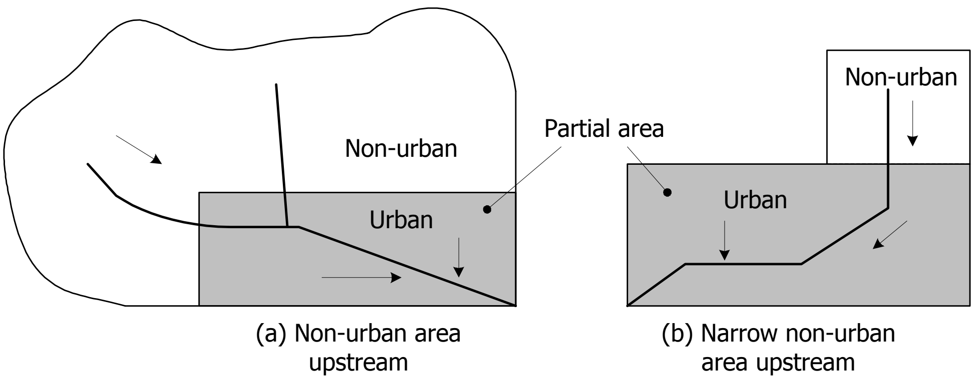


Figure 14.3 Urban Catchments Likely to Exhibit Partial Area Effects

Spatial distributions can be represented in hydrograph methods. However for areas of less than 10 km^2 in urban drainage systems the areal reduction factor can be neglected. It is also common practice to neglect any effects due to storm movement direction.

(c) Temporal Distribution

Commonly used approaches to distribute rainfall within a design storm were discussed in Chapter 13. The temporal distribution adopted can have a significant effect on the shape of the runoff hydrograph, and on the peak discharge. The recommended design temporal patterns are presented in Chapter 13.

(d) Rainfall Excess

Not all of the rainfall that falls on a catchment, produces runoff. Some rainfall losses occur, such as evaporation, infiltration and depression storage. The remaining rainfall after subtracting rainfall losses is called the *rainfall excess*.

In the Rational Method, described in the previous section, the concept of rainfall excess is not used directly. Instead the runoff coefficient allows for rainfall losses. Rainfall excess concepts are used in most hydrograph methods and this discussion focuses mainly on those methods.

The physical processes of interception of rainfall by vegetation, infiltration of water into the soil surface, and storage of water in surface depressions are commonly termed rainfall abstractions. Although these three processes are physically complex, simplified modelling procedures have been found give acceptable results for urban stormwater drainage.

Values can be derived by analysing observed rainfall and runoff data. Since individual values are dependent on the particular rainfall and catchment wetness characteristics of the event, individual values have little meaning except as indicators of those particular events. For design, an average value is usually needed, and since there is no reason for expecting loss rate values for a catchment to conform to any particular distribution, the median of the derived values is probably the most appropriate for design.

In discussing losses it is important to distinguish between *directly connected impervious areas* (DCIA) and pervious areas. The main rainfall losses only apply to pervious areas. Impervious areas that are not directly connected to the pipe system, such as tennis courts and concrete paths that are surrounded by pervious (grassed) surfaces, are also subject to losses because water must pass over these and possibly infiltrate before reaching a point of entry to a pipe or open drain.

(i) Evaporation Losses

Evaporation is generally insignificant and is neglected during the short-duration storms of concern in stormwater drainage design.

(ii) Impervious Area Losses

Impervious surfaces cause small rainfall losses due mainly to depression storage. Recommended values are given in Table 14.4.

(iii) Pervious Area Infiltration Loss Models

The predominant form of loss on pervious surfaces is by infiltration. Some of the most frequently used types of loss models are illustrated in Figure 14.4. These five types of loss models are described as follows:

1. *loss (and hence runoff) is a constant fraction of rainfall in each time period* : this is similar to the Rational Method runoff coefficient concept.
2. *constant loss rate* : where the rainfall excess is the residual left after a selected constant rate of infiltration capacity is satisfied.
3. *initial loss and continuing loss*: which is similar to constant loss rate except that no runoff is assumed to occur until a given initial loss capacity has been satisfied, regardless of the rainfall rate. The continuing loss is at a constant rate. A variation of this model is to have an initial loss followed by a loss consisting of a constant fraction of the rainfall in the remaining time periods (the initial loss-proportional loss model).
4. *infiltration curve or equation* : representing capacity rates of loss decreasing (usually exponentially) with time. The Horton Loss Model is of this type.
5. *standard rainfall-runoff relation* : such as the U.S. Soil Conservation Service relation.

It should be noted that loss values derived according to one of the models are not directly transferable to other models. The choice of loss model therefore depends in part on the choice of flow estimation method. In most urban stormwater drainage applications this is not a serious problem as the losses are generally only small in comparison to rainfall, and therefore a high degree of accuracy in estimation is not necessary.

Loss values are derived by analysing observed rainfall and runoff data. Since individual values are dependent on the particular rainfall and catchment wetness characteristics of the event, individual values have little meaning except as indicators of those particular events. For design, an average value is usually needed, and since there is no reason for expecting loss rate values for a catchment to conform to any particular distribution, the median of the derived values is probably the most appropriate for design.

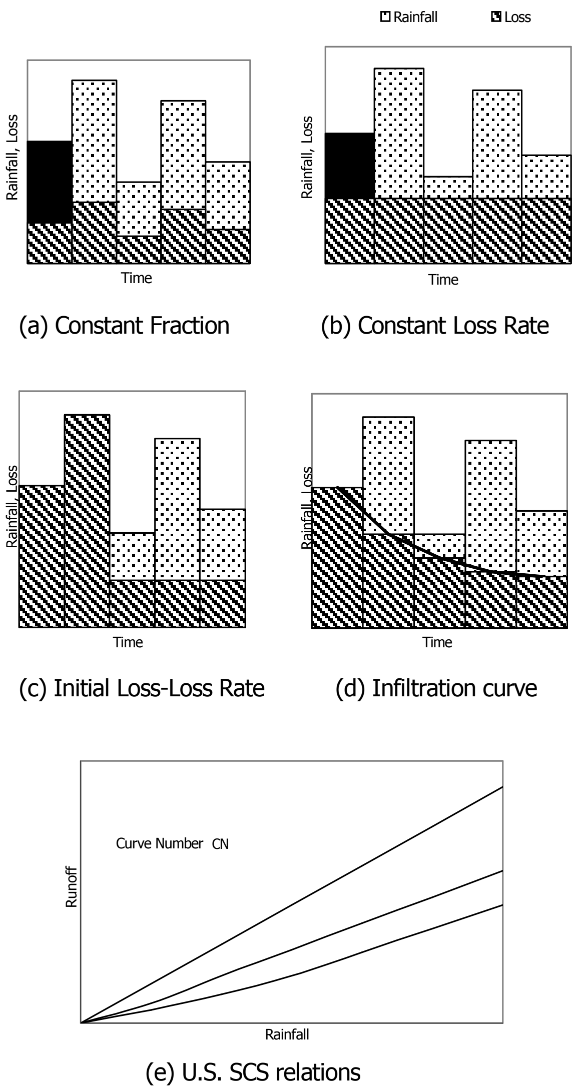


Figure 14.4 Typical Loss Models for Estimating Rainfall Excess

(iv) Choice of Infiltration Loss Model

Choice and validity of the 5 models depend on the data available and the likely runoff processes.

Models (a) and (b) are not often used in current practice. Model (e), the U.S. Soil Conservation Service approach requires further research for Malaysian urban drainage application. Models (c) or (d) are recommended for use in Malaysian urban drainage situations. Table 14.4 contains recommended loss values for use by drainage designers.

If a subcatchment contains areas of different surface condition or landuse, weighted average values of losses for different conditions or landuses, such as proportions of pervious and impervious areas, can be derived using methods similar to those in Section 14.5.6.

Horton's equation is widely used for describing infiltration capacity in a soil. It describes the decrease in capacity as more water is absorbed by the soil, and has the form:

$$f = f_c + (f_0 - f_c) \cdot e^{-kt} \qquad (14.9)$$

where,

f = the infiltration capacity (mm/hr) at time t

f_0 = the initial rate of infiltration (mm/hr)

f_c = the final constant rate of infiltration (mm/hr)

k = a shape factor (per hr)

t = the time from the start of rainfall (hr).

Recommended parameter values for the Horton's equation are given in Table 14.4. The Green-Ampt equation can also be used. (Chow, 1988)

14.6.2 Time-Area Method

Time-area methods utilise a convolution of the rainfall excess hyetograph with a time-area diagram representing the progressive area contributions within a catchment in set time increments. Separate hydrographs are generated for the impervious and pervious surfaces within the catchment. These are combined to estimate the total flow inputs to individual sub-catchment entries to the urban drain network.

The time-area method dates from the research of Ross in 1922. Networked urban drainage adoptions of the procedure however only date back to 1963. This computerised program known as the TRRL Method was developed by the UK Transport and Road Research Laboratory (TRRL), described by Watkins (1963). In the U.S., Terstriep and Stall (1974) further developed the method to include pervious runoff. In South Africa Watson (1981) made a number of additional changes particularly to the way infiltration was estimated. Between 1982 and 1986 Watson's model was used through extensive changes, to formulate a computerised package known as ILSAX (O'Loughlin, 1988). The sub-catchment runoff estimating procedure still utilises the basic time-area method to estimate both pervious and impervious portion of runoff.

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 catchment storage effects.

To apply the method, the catchment is first divided into a number of time zones separated by isochrones or lines of equal travel time to the outlet (Figure 14.5b). The areas between isochrones are then determined and plotted against the travel time as shown in Figure 14.5c.

The translated inflow hydrograph ordinates q_i for any selected design hyetograph (Figure 14.5d) can now be determined. Each block of storm in Figure 14.5a 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:

$$q_i = I_i \cdot A_1 + I_{i-1} \cdot A_2 + ... + I_1 \cdot A_i \qquad (14.10)$$

where,

q_i = the flow hydrograph ordinates (m^3/s)

I_i = excess rainfall hyetograph ordinates (mm/hr)

A_i = time-area histogram ordinates (ha)

i = number of isochrone area contributing to the outlet

For example, 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 total inflow hydrograph (Figure 14.5d) at the outlet can be obtained from Equation 14.10.

Table 14.4 Recommended Loss Models and Values for Hydrograph

Condition	Loss Model	Recommended Values	
Impervious Areas	Initial loss-Loss rate	Initial loss: 1.5 mm	Loss rate: 0 mm/hr
Pervious Areas	Initial loss – proportional loss, or	Initial loss: 10 mm	Proportional Loss: 20% of rainfall
	Initial loss-Loss rate, or	Initial loss: 10 mm for all soils (i) Sandy open structured soil (ii) Loam soil (iii) Clays, dense structured soil (iv) Clays subject to high shrinkage and in a cracked state at start of rain	Loss rate: 10 - 25 mm/hr 3 - 10 mm/hr 0.5 - 3 mm/hr 4 - 6 mm/hr
	Horton model	Initial Infiltration Capacity f_0 A. DRY soils (little or no vegetation) Sandy soils: 125 mm/hr Loam soils: 75 mm/hr Clay soils: 25 mm/hr For dense vegetation, multiply values given in A by 2 B. MOIST soils Soils which have drained but not dried out: divide values from A by 3 Soils close to saturation: value close to saturated hydraulic conductivity Soils partially dried out: divide values from A by 1.5-2.5 Recommended value of k is 4/hr	Ultimate Infiltration Rate f_c (mm/hr), for Hydrologic Soil Group (see Note) A 10 - 7.5 B 7.5 - 3.8 C 3.8 - 1.3 D 1.3 - 0

Note: Hydrological Soil Group corresponds to the classification given by the U.S. Soil Conservation Service. Well drained sandy soils are "A"; poorly drained clayey soils are "D". The texture of the layer of least hydraulic conductivity in the soil profile should be considered. Caution should be used in applying values from the above table to sandy soils (Group A). Source: XP-SWMM Manual (WP-Software, 1995).

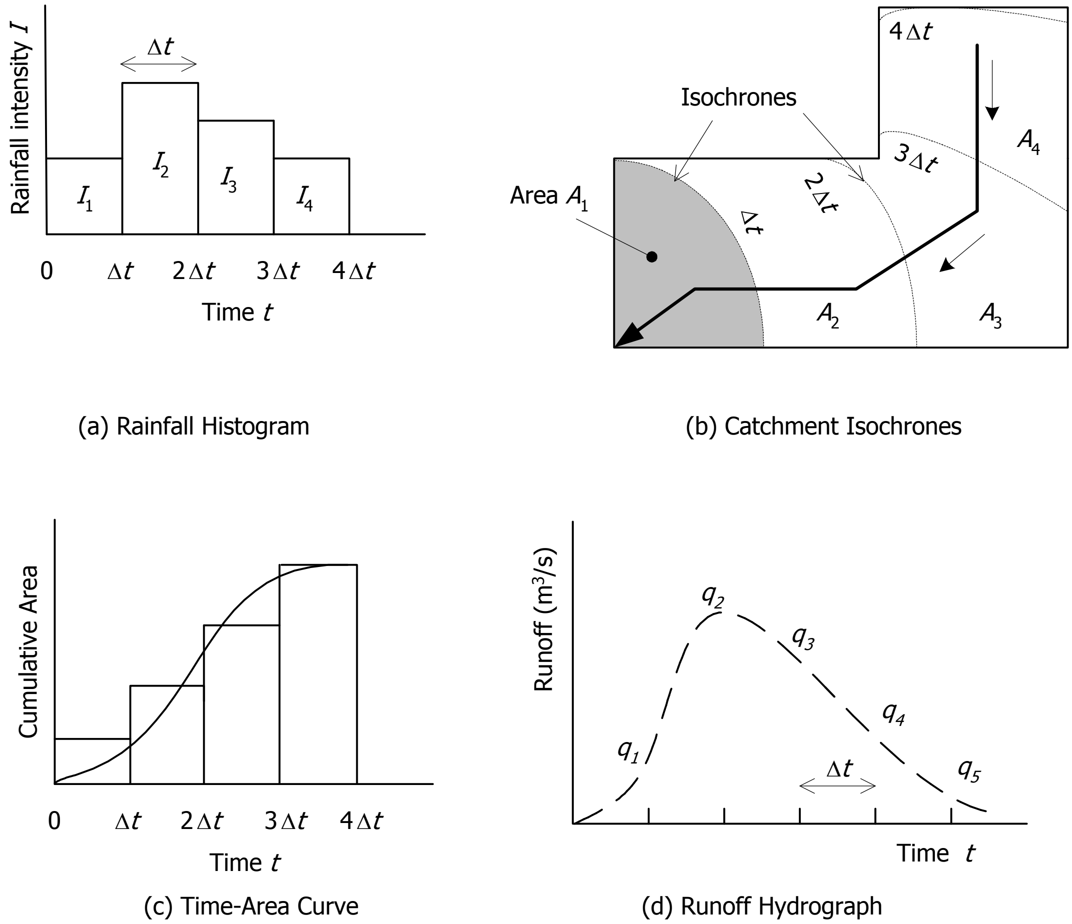


Figure 14.5 Time–Area Method

14.6.3 Kinematic Wave Method

The kinematic-wave method is a hydraulic routing method for routing runoff across planar surfaces and through small channels and pipes. The kinematic-wave formulation couples the continuity equation with a simplified form of the momentum equation that includes only the bottom-slope and friction-slope terms (Chapter 12). Kinematic-wave theory is only valid for one-dimensional overland flow on a planar surface. It has little or no practical use in urban drainage applications.

14.6.4 Non-linear Reservoir Method

In the non-linear reservoir method, the catchment is conceptualised as a very shallow reservoir. The discharge from this hypothetical reservoir is assumed to be a non-linear function of the depth of water in the reservoir. Computer models such as RAFTS and SWMM can optionally use a non-linear reservoir approach to compute surface runoff hydrographs. The SWMM procedure is described here (Huber and Dickinson, 1988).

Figure 14.6 shows the catchment conceptualised as a reservoir with rainfall as inflow, and infiltration and surface discharge as outflows. The depth y represents the average depth of surface runoff, and the depth y_d represents the average depression storage in the catchment. The continuity relationship for this system is:

$$A \frac{dy}{dt} = A \left(I - f \right) - Q \qquad (14.11)$$

where,

A = catchment area

I = rainfall intensity

f = the infiltration rate

Q = the discharge at the catchment outlet

The model assumes uniform overland flow at the catchment outlet at a depth equal to the difference between y and y_d. Based on the Manning friction relationship, the catchment discharge, Q is given by:

$$Q = \frac{W}{n}\left(y - y_d \right)^{5/3} S^{1/2} \quad (14.12)$$

where,

W = a representative width for the catchment

n = Manning roughness coefficient for the catchment

y_d = average depth of depression storage

S = average surface slope

Substituting Equation 14.12 into Equation 14.11 yields a non-linear differential equation for *y*. A simple finite difference form of the equation is used to solve for the depth *y* at the end of each time step. This equation is:

$$\frac{y_2 - y_1}{\Delta t} = \bar{I} - \bar{f} - \frac{C\,W\,S^{1/2}}{An}\left[\frac{y_1 + y_2}{2} - y_d\right]^{5/3} \quad (14.13)$$

where,

Δt = time step increment

y_1 = depth at the beginning of the time step

y_2 = depth at the end of the time step

I = average rainfall rate over the time step

f = average infiltration rate over the time step

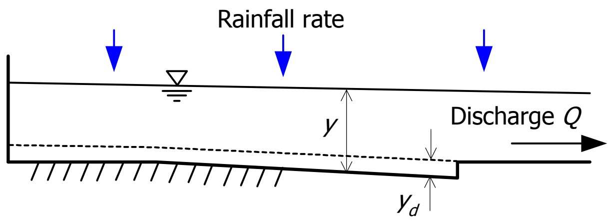


Figure 14.6 Definition Sketch for Non-linear Reservoir Model

For each time step, three separate calculations are performed. First, an infiltration equation is used to compute the average potential infiltration rate over the time step, then Equation 14.13 is solved iteratively for y_2, and, finally, Equation 14.12 yields the corresponding discharge.

Unlike the time-area method, which uses excess rainfall as input, the non-linear-reservoir method couples the processes of infiltration and surface runoff. The non-linear-reservoir model assumes that infiltration occurs at the potential rate over the entire surface area whenever the ponded depth is non-zero. The excess-rainfall models, on the other hand, entirely neglect infiltration of ponded water. This difference becomes important following cessation of rainfall, or whenever the rainfall intensity drops below the potential infiltration rate. In reality, infiltration does continue for some time after rainfall ceases, but the area over which infiltration continues to occurs after rainfall ceases would tend to be underestimated by the non-linear reservoir methods and overestimated by the excess-rainfall methods, though the difference will depend on the degree of discretisation used, since a more detailed schematisation can partially account for the phenomenon of decreasing area of infiltration.

14.6.5 Rational Method Hydrograph Method

This procedure extends the Rational Method to the development of runoff hydrographs. This method is only recommended for the sizing of small on-site detention and retention storages, and must NOT be used under any circumstances for the sizing of community or regional detention and retention facilities.

The RMHM relies on the assumptions of the Rational Method. Therefore, the RMHM is subject to the same misunderstandings and misapplications as the Rational Method. An important additional assumption is that the rainfall intensity averaging time used in the RMHM equals the storm duration. As introduced in the preceding discussion of the Rational Method, this assumption means that the rainfall, and the runoff generated by the rainfall that occurs before or after the rainfall averaging period are not accounted for. Accordingly, the RMHM may underestimate required storage volume. However, for the design of small on-site facilities, this underestimation will not generally be significant enough to warrant using one of the more complex hydrograph methods.

The RMHM also is based on the assumption that the runoff hydrograph can be approximated as being either triangular or trapezoidal in shape. This is equivalent to assuming that the runoff from the on-site catchment area increases linearly with time or that there is a linear area-time relationship for the sub-catchment.

RMHM hydrograph shape assumptions may be explained using the sub-catchment illustrated in Figure 14.7. The sub-catchment has an area of 0.1 hectares, a runoff coefficient C of 0.4 and a time of concentration t_c of 10 minutes. As illustrated in Figure 14.8, three types of hydrographs may be developed for the sub-catchment using the RMHM procedure. Hydrograph type is a function of the length of the rainfall averaging time, *d*, with respect to the sub-catchment time of concentration. The following three types are possible:

Type 1 : d is greater than t_c. The resulting trapezoidal hydrograph has a uniform maximum discharge of $C i_T A$, 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 of $C i_T A$. The linear rising and falling limbs each have a duration of t_c.

Type 3 : d is less than t_c. The resulting trapezoidal hydrograph has a uniform maximum discharge of $C i_T A$ times (d/t_c). The linear rising and falling portions of the hydrograph each have a duration of *d*.

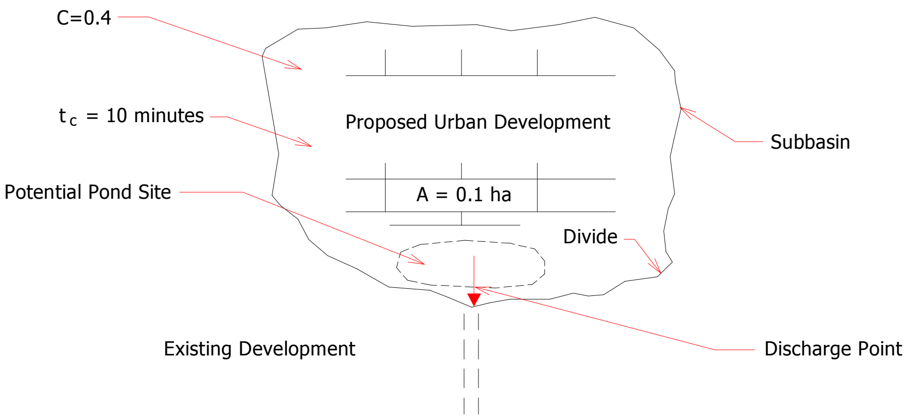


Figure 14.7 Sub catchment and Physical Parameters Needed to Apply the RMHM

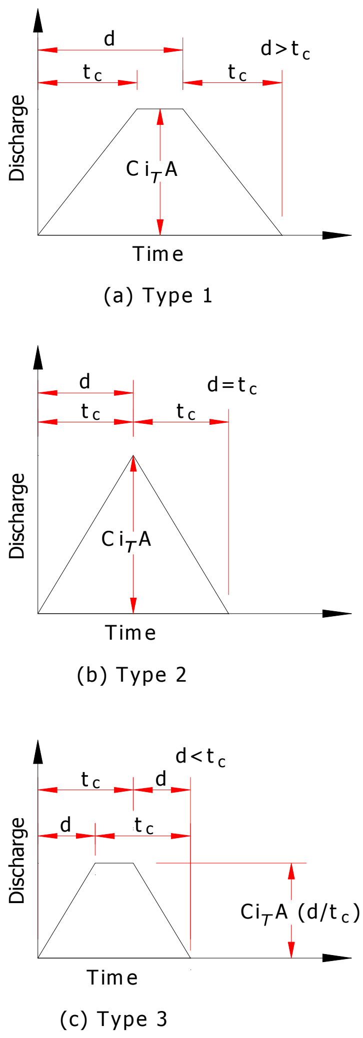


Figure 14.8 Hydrograph Types for the RMHM

In summary, hydrograph type in the RMHM is determined by the relationship between rainfall averaging time and the time of concentration of the sub-catchment. Given the hydrograph type, the peak discharge is determined using the Rational Method formula.

14.6.6 Generalised Analysis Procedure

A procedure for estimating a runoff hydrograph from a single sub-catchment for a particular ARI and duration is outlined in Figure 14.9. Hydrographs should be obtained for both the minor and major drainage systems.

14.7 FLOW ROUTING AND ATTENUATION PRINCIPLES

Flow routing is the process of converting a hydrograph that passes through some part of a flow system to allow for the changes that occur during its passage. There are three main types of flow routing:

- catchment routing, which converts a rainfall excess hyetograph into a hydrograph at the catchment outlet, allowing for the distribution of rainfalls over the catchment surface, and various lags or delays along flow paths;
- channel routing, which allows for the changes in hydrographs as they flow along river or channel reaches, caused by variations in the channel geometry which result in storage effects; and
- reservoir routing, which allows for storage effects in a concentrated, "level pool" reservoir.

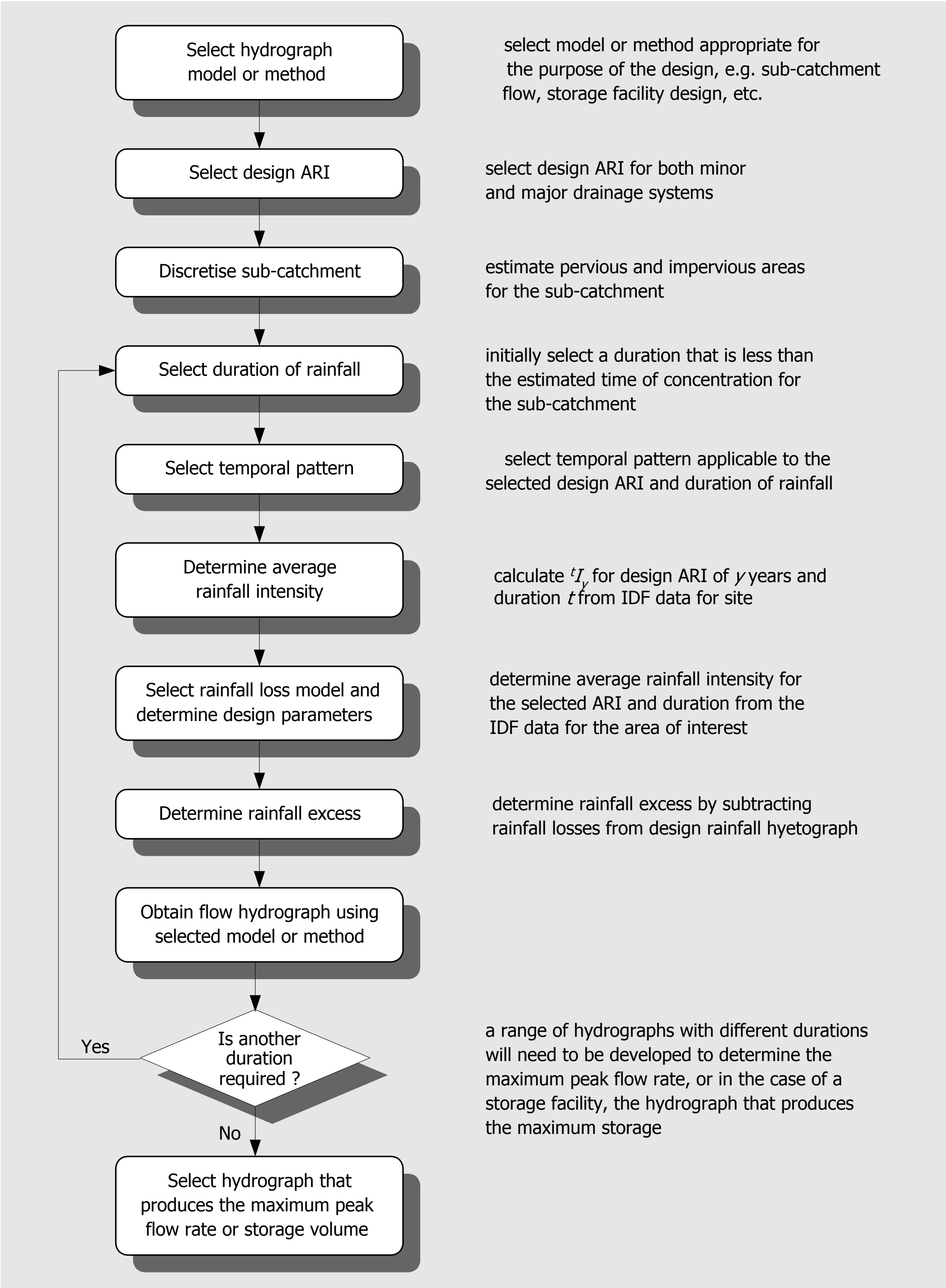


Figure 14.9 General Procedure for Estimating a Hydrograph for a Single Sub-catchment

Broadly defined, flow routing is an analytical procedure intended to trace the flow of water through a hydrological system, pond, conveyance, or porous media, given some runoff event hydrograph as input. The procedure determines the flow hydrograph at a point downstream, from known or assumed flow hydrographs at one or more points upstream. If the flow is a runoff event such as a flood, then the procedure is specifically known as *flood routing*. Routing by lumped system methods is called *hydrological routing*. These methods calculate the flow as a function of time alone. Routing by distributed system methods is called *hydraulic routing*, and the flow is calculated as a function of both space and time throughout the system.

For hydrologic routing the inflow $I(t)$, outflow $O(t)$, and storage $S(t)$ are related by the continuity equation:

$$\frac{dS}{dt} = I(t) - O(t) \tag{14.14}$$

If an inflow $I(t)$ is known Equation 14.14 cannot be solved directly to obtain the outflow $O(t)$, because both O and S are unknown. A second relationship, the storage function, is required to relate I, S, and O. Coupling the continuity equation with the storage function provides a solvable combination of two equations and two unknowns.

The specific form of the storage functions to be employed in hydrologic routing depends on the nature of the system being analysed. In *reservoir routing* by the *level-pool method* (see Section 14.8.1), storage is a non-linear function of O only, i.e. $S = f(O)$, and the function, $f(O)$, is determined by relating reservoir storage and outflow to reservoir water level. In *channel routing* by the *Muskingum method* (see Section 14.10.1), storage is linearly related to I and O. Similarly, in porous media, storage is a function of outflow, which depends on storage in the media and the underlying soils.

14.8 FLOW THROUGH POND AND RESERVOIR

14.8.1 Hydrologic Routing

Level-pool routing is a procedure for calculating the outflow hydrograph from a pond reservoir, assuming a horizontal water surface, given its inflow hydrograph and storage-discharge characteristics. When a reservoir has a horizontal water surface, its storage is a function of its water-surface elevation, or depth in the pool. Likewise, the discharge is a function of the water surface elevation, or head on the outlet works. Combining these two functions yields the invariable single-valued function.

Integration of the continuity equation (Equation 14.14) over the discrete time intervals provides an expression for the change in storage over the j^{th} time interval $j\Delta t$, $S_{j+1} - S_j$, which can be rewritten as:

$$S_{j+1} - S_j = \left(\frac{I_j + I_{j+1}}{2}\right)\Delta t - \left(\frac{O_j + O_{j+1}}{2}\right)\Delta t \tag{14.15}$$

The inflow values at the beginning and end of the j^{th} time interval are I_j and I_{j+1}, respectively, and corresponding outflow values are O_j and O_{j+1}. The values of I_j and I_{j+1}, are known because they are pre-specified (i.e. the inflow hydrograph ordinates). The values O_j and S_j are known at the j^{th} time interval. Hence Equation 14.15 contains two unknowns, O_{j+1} and S_{j+1}, which are isolated by multiplying Equation 14.15 by $2/\Delta t$ and rearranging the result to produce:

$$\left(\frac{2S_{j+1}}{\Delta t} + O_{j+1}\right) = \left(I_j + I_{j+1}\right) + \left(\frac{2S_j}{\Delta t} - O_j\right) \tag{14.16}$$

In order to calculate the outflow O_{j+1} from Equation 14.16, a storage-discharge function relating $2S/\Delta t + O$ and O is needed. The method of developing this function using stage-storage and stage-discharge relationship is shown in Figure 14.10.

The relationship between water-surface elevation and reservoir storage can be obtained using topographic maps or from field surveys. The stage-discharge relationship is derived from hydraulic equations relating head and discharge for various types of spillway and outlet works. The value of Δt is the same as the time interval of the inflow hydrograph.

For a given water-surface elevation, the values of storage S and discharge O are determined. Then, the value of $2S/\Delta t+O$ is calculated and plotted against O. In routing the flow through the j^{th} time interval, all terms in the right-hand-side of Equation 14.16 are known, and so the value of $2S_{j+1}/\Delta t+O$ can be computed. The corresponding value of O_{j+1} can be determined from the storage-discharge function $2S/\Delta t+O$ versus O. To set up the data for the next time interval, the value $2S_{j+1}/\Delta t-O_{j+1}$ is calculated by:

$$\left(\frac{2S_{j+1}}{\Delta t} - O_{j+1}\right) = \left(\frac{2S_{j+1}}{\Delta t} + O_{j+1}\right) - 2O_{j+1} \tag{14.17}$$

The computation is repeated for subsequent routing periods. Input requirements for this routing method are:

- the storage-discharge relationship
- the storage-indication relationship
- the inflow hydrograph
- initial values of the outflow rate (O_1) and storage (S_1)
- the routing interval (Δt)

An analysis procedure for hydrologic routing is shown in Figure 14.11.

14.8.2 Two-Dimensional Hydraulic Routing

Two-dimensional hydraulic routing is especially useful for pond and lake design that involves structures and water quality. The governing equations of a depth-averaged two-dimensional model for a well-mixed shallow pond or lake are derived from the principles of mass and momentum conservation (Chapter 12). In the case of mass conservation, the net mass flux through a control volume is balanced by a change of fluid density. In conservation of momentum, a momentum flux balance is performed on a control volume. Details of the derivation can be found in textbooks (Ippen, 1966).

With the assumption that vertical accelerations and velocities are negligible compared to horizontal ones, and lake water is completely homogeneous, incompressible, and viscous, the continuity and momentum equations for depth-averaged two dimensional flow are given as:

Continuity Equation

$$\frac{\partial \zeta}{\partial t} + \frac{\partial uH}{\partial x} + \frac{\partial vH}{\partial y} = 0 \tag{14.18}$$

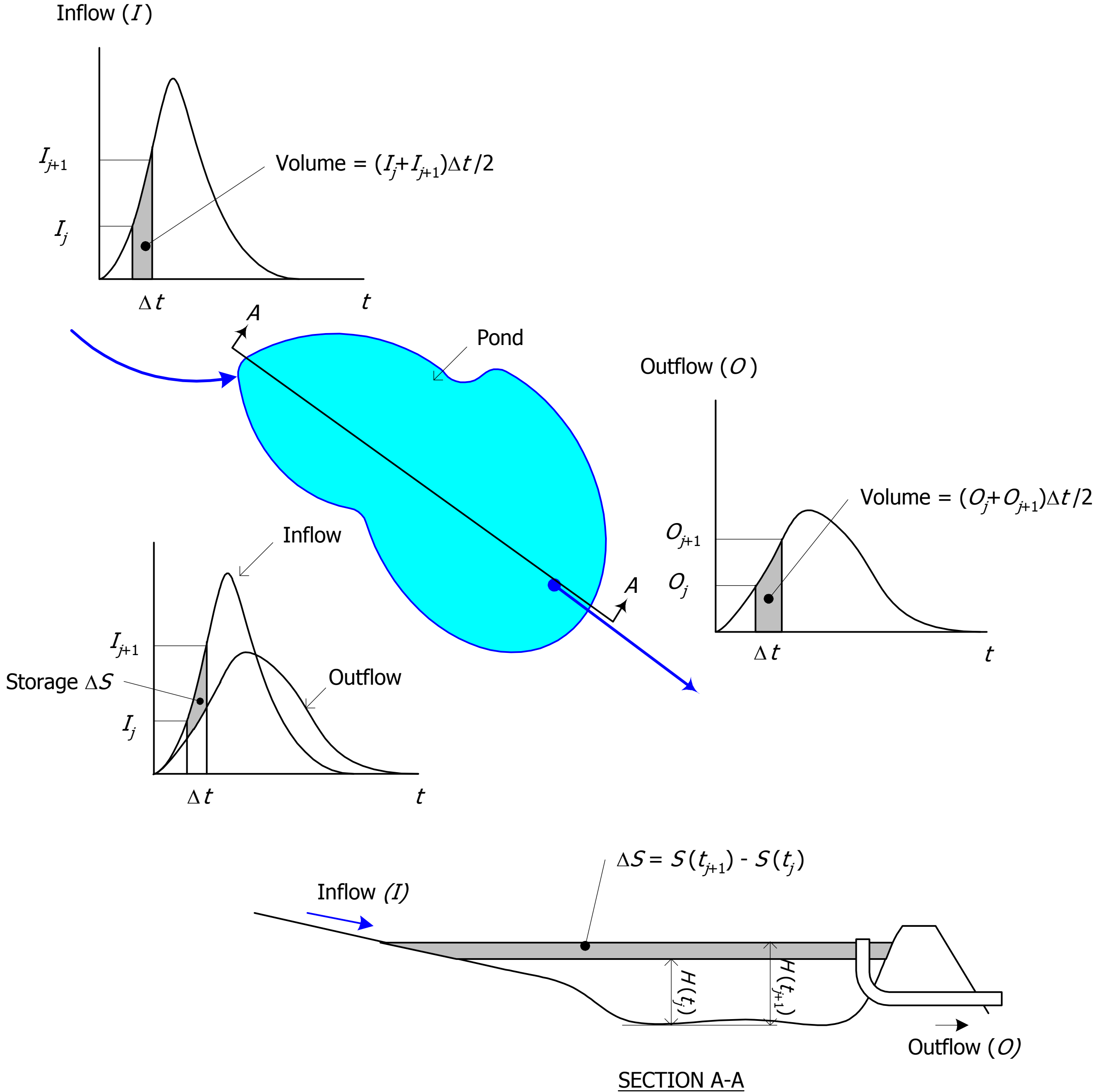


Figure 14.10 Development of the Storage-Discharge Function for Hydrologic Pond Routing

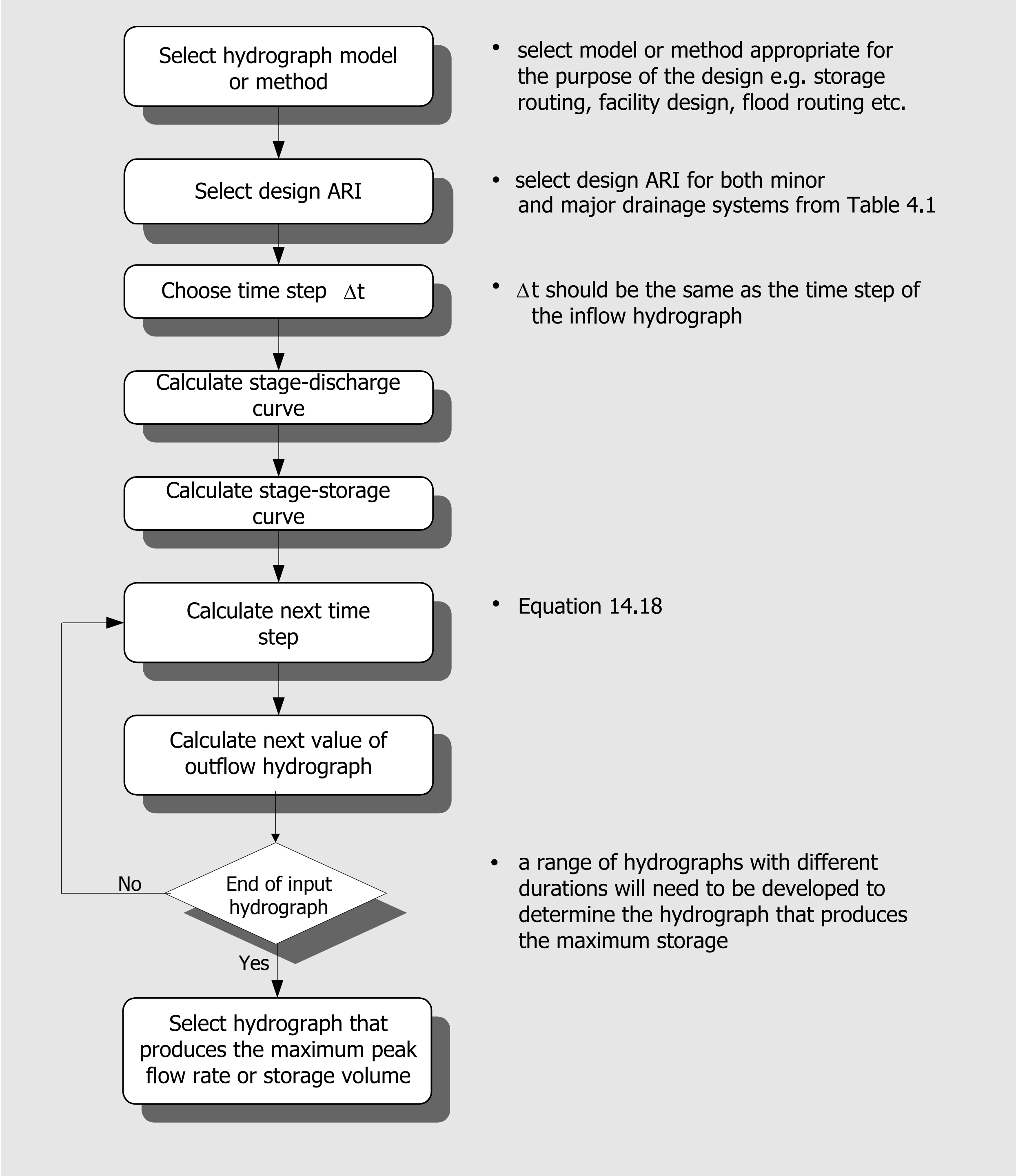


Figure 14.11 General Analysis Procedure for Hydrologic Routing

Momentum Equations

x - direction:

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+g\frac{\partial \zeta}{\partial x}+\frac{1}{\rho H_x}\tau_{bx}-\frac{1}{\rho H_x}\left[\frac{\partial H\tau_{xx}}{\partial x}+\frac{\partial H\tau_{xy}}{\partial y}\right]=0 \qquad (14.19a)$$

y - direction:

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}+g\frac{\partial \zeta}{\partial y}+\frac{1}{\rho H_y}\tau_{by}-\frac{1}{\rho H_y}\left[\frac{\partial H\tau_{xy}}{\partial y}+\frac{\partial H\tau_{yy}}{\partial y}\right]=0 \qquad (14.19b)$$

where,

$H = h + \zeta$

h = still water depth

ζ = free surface displacement

τ_{bx} and τ_{by} are bottom shear stresses in which it is generally assumed that:

$$\tau_{bx} = \frac{\rho g n^2 u \sqrt{u^2 + v^2}}{H_x^{1/3}} \quad (14.19c)$$

$$\tau_{by} = \frac{\rho g n^2 v \sqrt{u^2 + v^2}}{H_y^{1/3}} \quad (14.19d)$$

where,

u and v are depth-averaged velocity components and n is the Manning roughness coefficient

For flows in small and medium size waterbodies such as detention ponds, the effects of wind and earth rotation are insignificant compared to the driving forces. The equations are normally solved by finite differences or finite elements methods. Their non-linearity is handled by the Newton Raphson technique. The flow equations provide convective velocity effects to solving the transport equation for pond water quality analysis and design (see Section 15.6.5).

14.9 FLOW THROUGH POROUS MEDIA

14.9.1 Routing through Infiltration Facility

The constant infiltration model provides a method of analysis and evaluating the hydraulics of soakage pits. This model can be used to calculate the maximum water level occurring in a given infiltration system during a design storm event, thus allowing the required depth of a system to be calculated. This section introduces equations for a maximum water level for both plane and three-dimensional infiltration drainage systems.

Applying the flow balance approach (Equation 14.15) to infiltration drainage facilities where,

S = $S(h)$, the volume of water stored in the infiltration system

Q_{in} = $I(t)$, the inflow of runoff

Q_{out} = $O(t)$, the outflow through infiltration

and,

h = water depth above the bed level of the infiltration system

t = time since the start of the rainfall event

Considering the Storage

The storage available depends on whether or not the facility is rubble filled.

$$S = nV \quad (14.20)$$

where,

V = $V(h)$, the volume of the water filled part of the facility and is a function of the geometry of the system

n = effective porosity of any fill material and is unity if the system is not rubble-filled

The hydrological balance equation may therefore be written as:

$$n \frac{dV}{dh} \frac{dh}{dt} = Q_{in} - Q_{out} \quad (14.21)$$

The water level in the system, $h(t)$, can be found at any time, t, by inserting appropriate functions of $Q_{in}(t)$, $Q_{out}(t)$, and $V(h)$ into Equation 14.21, rearranging for h, and integrating with respect to time.

Considering the Inflow

It is assumed that the facility receives a constant rate of runoff, estimated from the Rational Formula.

$$Q_{in} = C.I.A_D \text{ , for a duration } D \quad (14.22)$$

where,

I = intensity of the rainfall

A_D = impermeable area drained

Considering the Outflow

It is assumed here that the infiltration rate (flow per unit surface area) is constant, thus:

$$Q_{out} = q_f . A_w \quad (14.23)$$

where,

A_W = wetted infiltration surface area of the facility which depends on the geometry of the system and is a function of the water depth, h

q_f = the infiltration coefficient which includes an appropriate factor of safety

With these assumptions, a flow balance equation may be written for h from Equation 14.21:

$$h(t) = \int \frac{I.A_D - q_f.A_w(h)}{n\left(\frac{dV(h)}{dh}\right)} dt \quad (14.24)$$

The design method requires integration of Equation 14.24 in order to derive formulae describing the water level, h, at given times, t. These formulae depend on the geometrical functions $A_W(h)$ and $V(h)$. For simple geometries, Equation 14.24 may be solved analytically. For complex geometries, it can only be solved using an approximate method or numerical integration.

Once a solution to Equation 14.24 is found for a particular type of infiltration system, the highest water level, h_{max}, can be found for a particular storm event. Using the constant inflow function described above, the highest water level will occur at the end of the storm event when t equals the storm duration, D.

14.9.2 Flow through Layered Media

For the two layered materials shown in Figure 14.12, which are typically found in practice, vertical flow (infiltration or filtration) through the layers are:

$$q_{1-2} = AK_1 \frac{\left(\frac{p_1}{\gamma} + z_1\right) - \left(\frac{p_2}{\gamma} + z_2\right)}{z_1 - z_2} = AK_1 \frac{\Delta h_1}{\Delta z_1} \quad (14.25a)$$

$$q_{2-3} = AK_2 \frac{\left(\frac{p_2}{\gamma} + z_2\right) - \left(\frac{p_3}{\gamma} + z_3\right)}{z_2 - z_3} = AK_2 \frac{\Delta h_2}{\Delta z_2} \quad (14.25b)$$

For homogeneous media and continuity of flow:

$$q = A\overline{K}_z \frac{\Delta H}{\Delta Z} \quad (14.26)$$

where,

$$\overline{K}_z = \frac{\Delta Z_1 + \Delta Z_2}{\left(\frac{\Delta Z_1}{K_1} + \frac{\Delta Z_2}{K_2}\right)} \quad , \quad \Delta H = \Delta h_1 + \Delta h_2 \quad ,$$

and $\Delta Z = \Delta Z_1 + \Delta Z_2$

14.9.3 Hydraulic of Aquifer Response to Infiltration

Infiltration drainage systems are found in the unsaturated zone above the water table. A diagrammatic representation of the conceptual model of soakage pit hydraulics in which groundwater flow in the unsaturated zone is taken into account is shown in Figure 14.12. It is envisaged that a 'bulb' of saturation becomes established around the pit. As groundwater flows away from the pit area through which it passes increases due to the three-dimensional nature of the flow and the soil becomes unsaturated. Thus to provide a realistic description of the hydraulic behaviour of infiltration system, both saturated and unsaturated groundwater flows have been considered in this section.

(a) Saturated Groundwater Hydraulics

Allowing for storage to occur in the soil for non-steady flow conditions, the general 3-dimensional equations of motion for homogeneous-isotropic saturated phreatic groundwater flow is:

$$\frac{\partial}{\partial x}\left(h\frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(h\frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(h\frac{\partial h}{\partial z}\right) \pm \frac{N}{k} = \frac{S}{k}\frac{\partial h}{\partial t} \quad (14.27)$$

where,

N = stormwater recharge (+ve) or abstraction (-ve) rate
S = the specific yield/ effective porosity
t = time
h = phreatic water level
k = saturated hydraulic conductivity

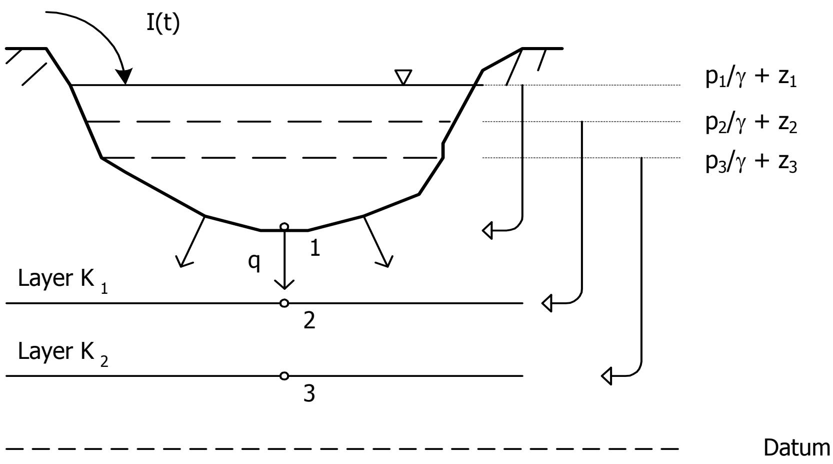


Figure 14.12 Flow through a Saturated Layered System

The equation can be solved numerically for saturated groundwater flow problems if the initial and boundary conditions are defined and the hydraulic conductivity and specific storage are known. Derived velocity values will serve as convective terms in the solution of solute transport in Chapter 15. The equation for saturated confined flow is similar and applicable when stormwater is recharged into a deeper aquifer system.

(b) Unsaturated Groundwater Hydraulics

Unsaturated groundwater flow differs from saturated groundwater flow in that the hydraulic properties of the soil, the hydraulic conductivity and the storage coefficient both depend on the degree of saturation of the soil. As there is both water and air present in the soil, capillary action exerts a force, or soil suction, and so the water in the soil is below atmospheric pressure. The degree of soil suction, or negative pressure, also depends on the degree of saturation and the soil properties.

From Darcy's equation (Chapter 12), the one-dimensional infiltration equation of motion for unsaturated groundwater flow is:

$$\frac{\partial}{\partial Z} k(\psi)\frac{\partial \psi}{\partial Z} + \frac{\partial k(\psi)}{\partial Z} = \frac{\partial \theta}{\partial \psi}\frac{\partial \psi}{\partial t} \quad (14.28)$$

where,

θ = the volumetric moisture content

z = elevation head

k = unsaturated hydraulic conductivity

The equation is often referred to as the modified Richards equations (1931) and can be simply extended to three-dimensions for real world problems. The hydraulic conductivity - pressure head function may be defined as:

$$k(\psi) = k_s\, k_r(\psi) \quad (14.29)$$

where,

k = the hydraulic conductivity

k_s = the saturated hydraulic conductivity

k_r = the relative hydraulic conductivity

The moisture content - pressure head function may be defined as:

$$\theta(\psi) = n\,\theta_r(\psi) \quad (14.30)$$

where,

θ = moisture content

n = porosity

θ_r = relative saturation ($S_r < \theta_r < 1$)

S_r = specific retention, i.e. the moisture content of the soil which will not drain under the influence of gravity

Various analytical models have been proposed to describe the relationship between the relative hydraulic conductivity and relative moisture content with pressure. These functions involve the use of one, two, three or four curve fitting coefficients for a particular soil.

In practice $k(\psi)$ and $\theta(\psi)$ are hysteresis functions depending on whether the soil is wetting or drying at the time. Since the infiltration problem is concerned mainly with wetting then the hysteresis of soil properties is ignored.

(c) Coupled Saturated-Unsaturated Hydraulics

The integrated solution of Equations 14.27 and 14.28 are useful for the planning and design of larger stormwater recharge schemes under more complex environments. They are also important for use in municipal subsurface drainage design especially in vulnerable hillslope vadose areas.

14.10 FLOW THROUGH CONVEYANCE

14.10.1 Hydrologic Routing

A widely used hydrologic method for routing flows in conveyance systems is the *Muskingum method*. It models the storage volume of flow in a channel reach by a combination of *wedge* and *prism storage* (Figure 14.13). When the flood wave is advancing, inflow exceeds outflow and a positive wedge of storage is produced. When the flood is receding, outflow exceeds inflow and a negative wedge results. In addition, a prism of storage is formed by a volume of (approximately) constant cross-section along the length of the channel reach.

Assuming that the cross-sectional area of the flow is directly proportional to the discharge at the section, the volume of prism storage is KO, where K is a coefficient of proportionality. The volume of wedge storage is assumed to be equal to $KX(I-O)$, where X is a weighting factor having the range $0 \leq X \leq 0.5$. The total storage is defined as the sum of the two storage *components* $S=KO+KX(I-O)$ which can be rearranged to give the linear storage function for the Muskingum method:

$$S = K\left[X I + (1 - X)\, O \right] \quad (14.31)$$

The value of X depends on the shape of the modelled wedge storage, X = 0 corresponds to reservoir (level-pool) storage and Equation 14.31 reduces to $S = KOX = 0.5$ for a full wedge. In most natural stream channels, X is between 0 and 0.3, with a mean value near 0.2. Great accuracy in determining X is usually not necessary because the Muskingum method is not sensitive to this parameter.

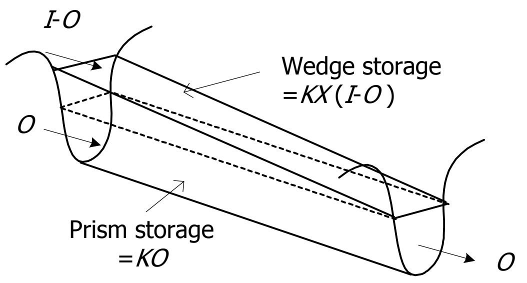


Figure 14.13 Prism and Wedge Storage in a Channel Reach

The parameter K is the time of travel of the flood wave through the reach. Although techniques exist that allow for the values of K and X to vary according to flow rate and channel characteristics (e.g., the *Muskingum-Cunge method*), for hydrologic routing the values of K and X are assumed to be specified and constant throughout the range of flow. The change in storage over the time interval Δt (from j to $j+1$) is:

$$S_{j+1} - S_j = K\left[XI_{j+1} + (1-X)O_{j+1}\right] - \left[XI_j + (1-X)O_j\right] \tag{14.32}$$

The change in storage can also be expressed by Equation 14.16 and combining Equations 14.32 and 14.16 yields the Muskingum routing equation:

$$O_{j+1} = C_1.I_{j+1} + C_2.I_j + C_3.O_j \tag{14.33}$$

where,

$$C_1 = \frac{\Delta t - 2KX}{2K(1-X) + \Delta t} \tag{14.34a}$$

$$C_2 = \frac{\Delta t + 2KX}{2K(1-X) + \Delta t} \tag{14.34b}$$

$$C_3 = \frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t} \tag{14.34c}$$

Note that $C_1 + C_2 + C_3 = 1$.

The values of K and X for a single reach are determined using observed inflow and outflow hydrographs in the channel reach. By assuming various values of X and known values of inflow, successive values of K can be computed using:

$$K = \frac{0.5\Delta t\left[(I_{j+1} + I_j) - (O_{j+1} + O_j)\right]}{X(I_{j+1} - I_j) + (1-X)(O_{j+1} - O_j)} \tag{14.35}$$

Equation 14.35 is derived from Equations 14.16 and 14.32. The computed values of the numerator and denominator of Equation 14.35 are plotted for each time interval Δt, with the numerator on the vertical axis and the denominator on the horizontal axis. This usually produces a graph in the form of a loop (see Figure 14.14). The value of X that produces a loop closest to a single line is taken to be the correct value for the reach. The parameter K, from Equation 14.35 is the slope of the line for the value of X.

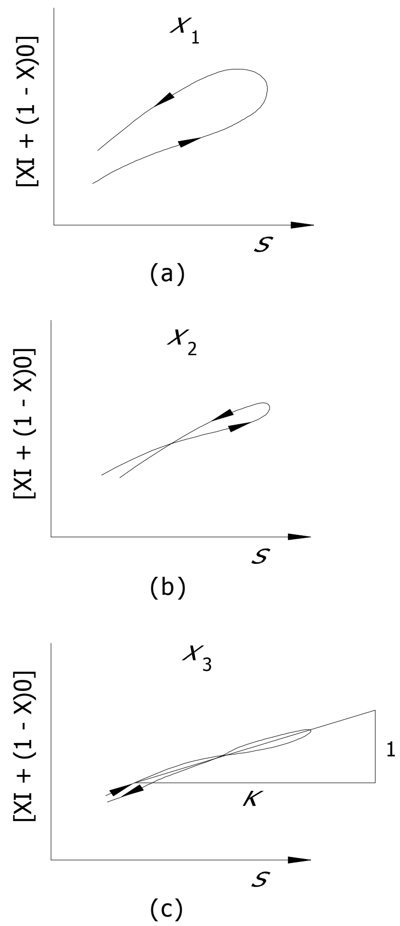


Figure 14.14 Procedure for Determining X and K Values

14.10.2 One-dimensional Hydraulic Routing

Procedures for distributed-flow hydraulic routing are popular because they compute flow rate and water level as functions of both space and time. The methodologies are based upon the Saint-Venant equations of one-dimensional flow. In contrast, the lumped hydrologic-routing procedures discussed in the previous sections compute flow rate as a function of time alone.

Unsteady flow is described by the *conservation form* of the Saint-Venant equations. This form provides the flexibility to simulate a wide range of flows from gradual flood waves in rivers to pipe flows, and can simulate lateral inflows or outflows such as weirs and pumping. The equations are given below (Chow et al, 1988).

Continuity:

$$\frac{\partial Q}{\partial x} + \frac{\partial (A + A_0)}{\partial t} - q = 0 \qquad (14.36a)$$

Momentum:

$$\frac{1}{A}\frac{\partial Q}{\partial t} + \frac{1}{A}\frac{\partial\left(\frac{\beta Q^2}{A}\right)}{\partial x} + g\left(\frac{\partial h}{\partial x} + S_f + S_e\right) - \beta q v_x + W_f B = 0 \qquad (14.36b)$$

where,

- x = longitudinal distance along the conveyance
- t = time
- A = cross-sectional area of flow
- A_0 = cross-sectional area of dead storage (off-channel)
- q = lateral inflow per unit length along the conveyance
- h = water-surface elevation
- v_x = velocity of lateral flow in the direction of flow
- S_f = friction slope
- S_e = eddy loss slope
- B = width of the conveyance at the water surface
- W_f = wind shear force
- β = momentum correction factor
- g = acceleration due to gravity

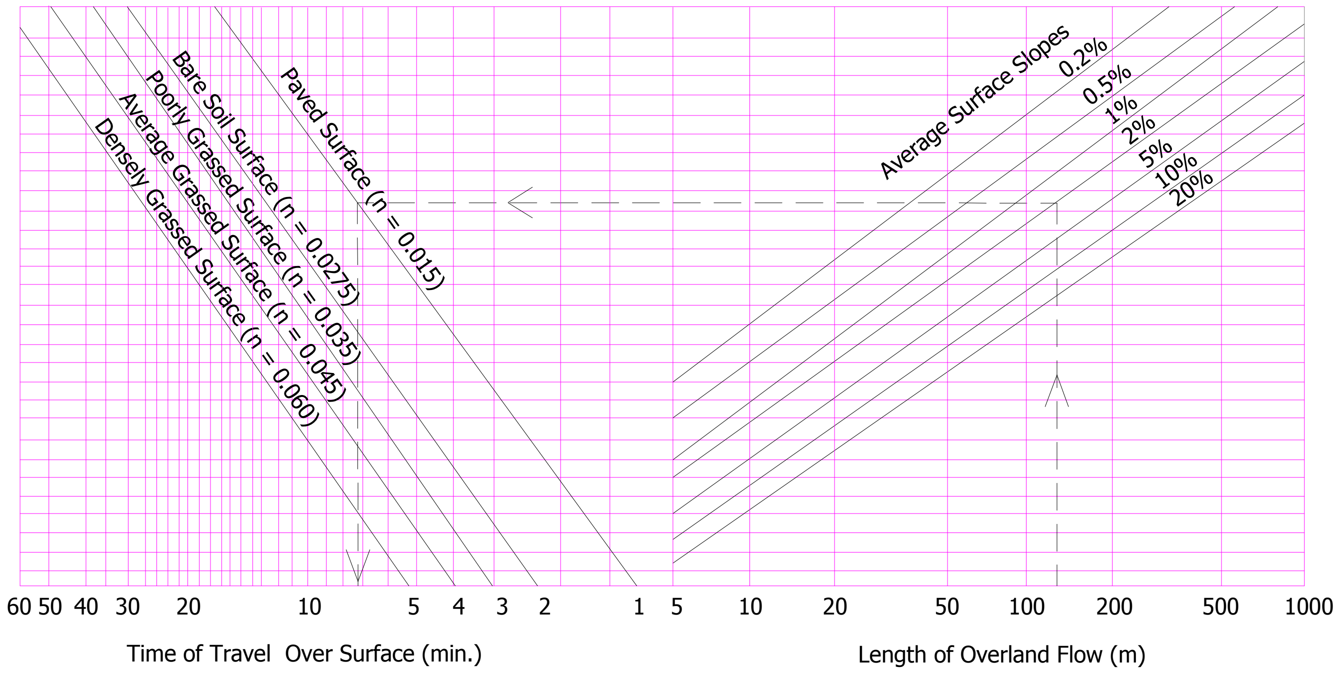
The Saint-Venant equations operate under the following assumptions:

1. The flow is one-dimensional with depth and velocity varying only in the longitudinal direction of the conveyance. This implies that the velocity is constant and the water surface is horizontal across any section perpendicular to the longitudinal axis.
2. There is gradually varied flow along the channel so that hydrostatic pressure prevails and vertical accelerations can be neglected.
3. The longitudinal axis of the channel is approximated as a straight line.
4. The bottom slope of the channel is small and the bed is fixed, resulting in negligible effects of scour and deposition.
5. Resistance coefficients for steady uniform turbulent flow are applicable, allowing for a use of Manning's equation to described resistance effects.
6. The fluid is incompressible and of constant density throughout the flow.

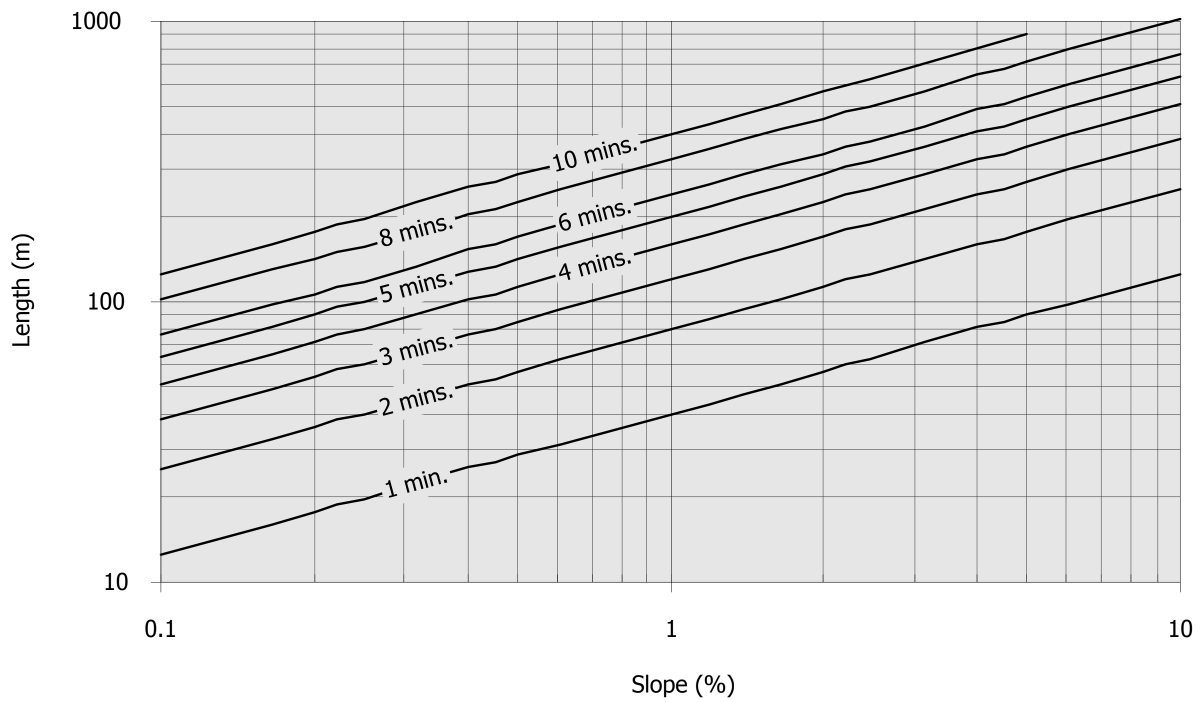
The momentum equation consists of terms for the physical processes that govern flow momentum. When the water level or flow rate is changed at a certain point in a channel with a subcritical flow, the effects of these changes propagate back upstream. These backwater effects can be incorporated into distributed routing methods through the local acceleration, convective acceleration, and pressure terms.

Hydrologic (lumped) routing methods may not perform well in simulating the flow conditions when backwater effects are significant and the drain or channel slope is mild, because these methods have no hydraulic mechanisms to describe upstream propagation of changes in the flow momentum.

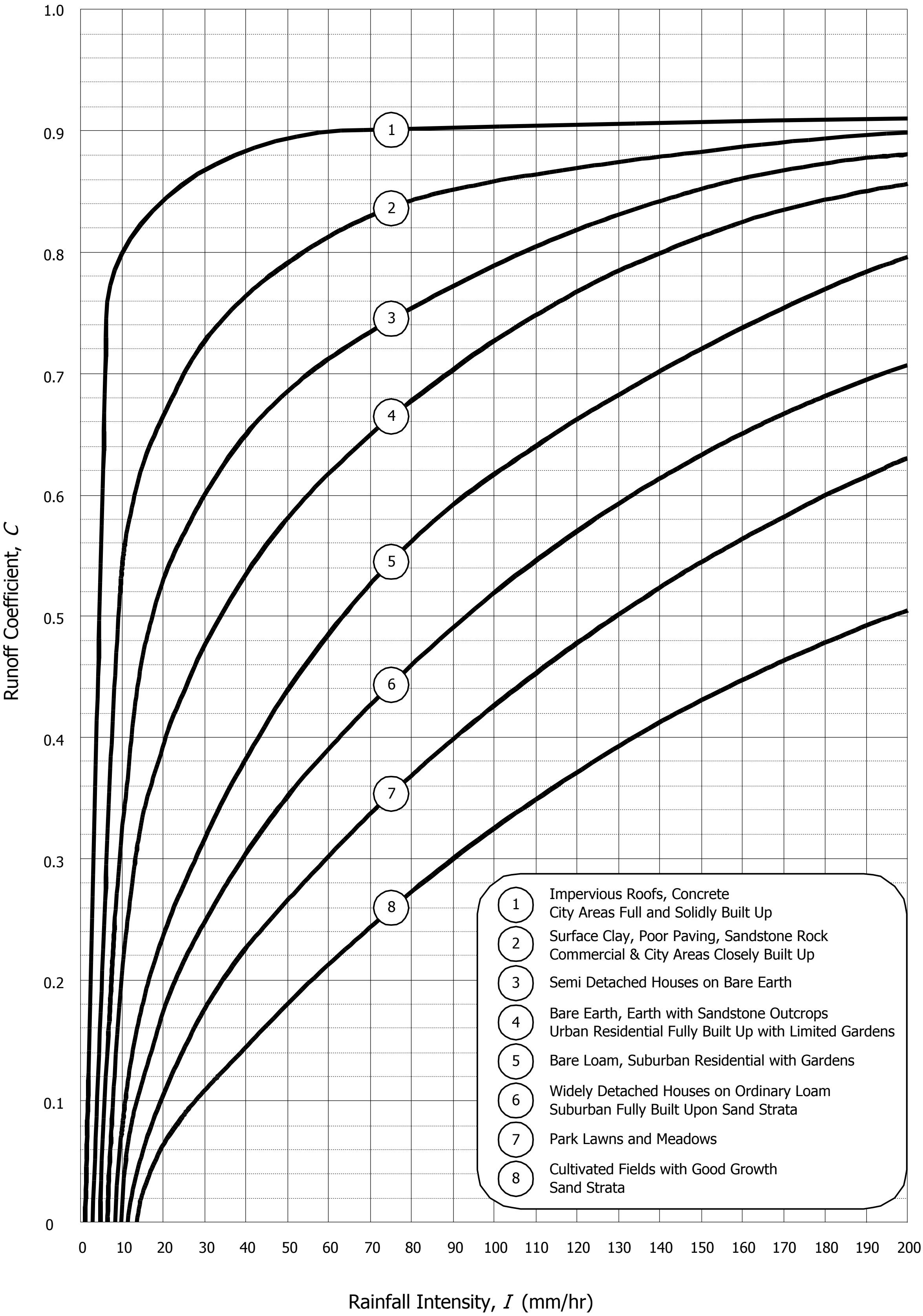
APPENDIX 14.A DESIGN CHARTS



Design Chart 14.1 Nomograph for Estimating Overland Sheet Flow Times (Source: AR&R, 1977)
(Overland Sheet Flow Times - Shallow Sheet Flow Only)

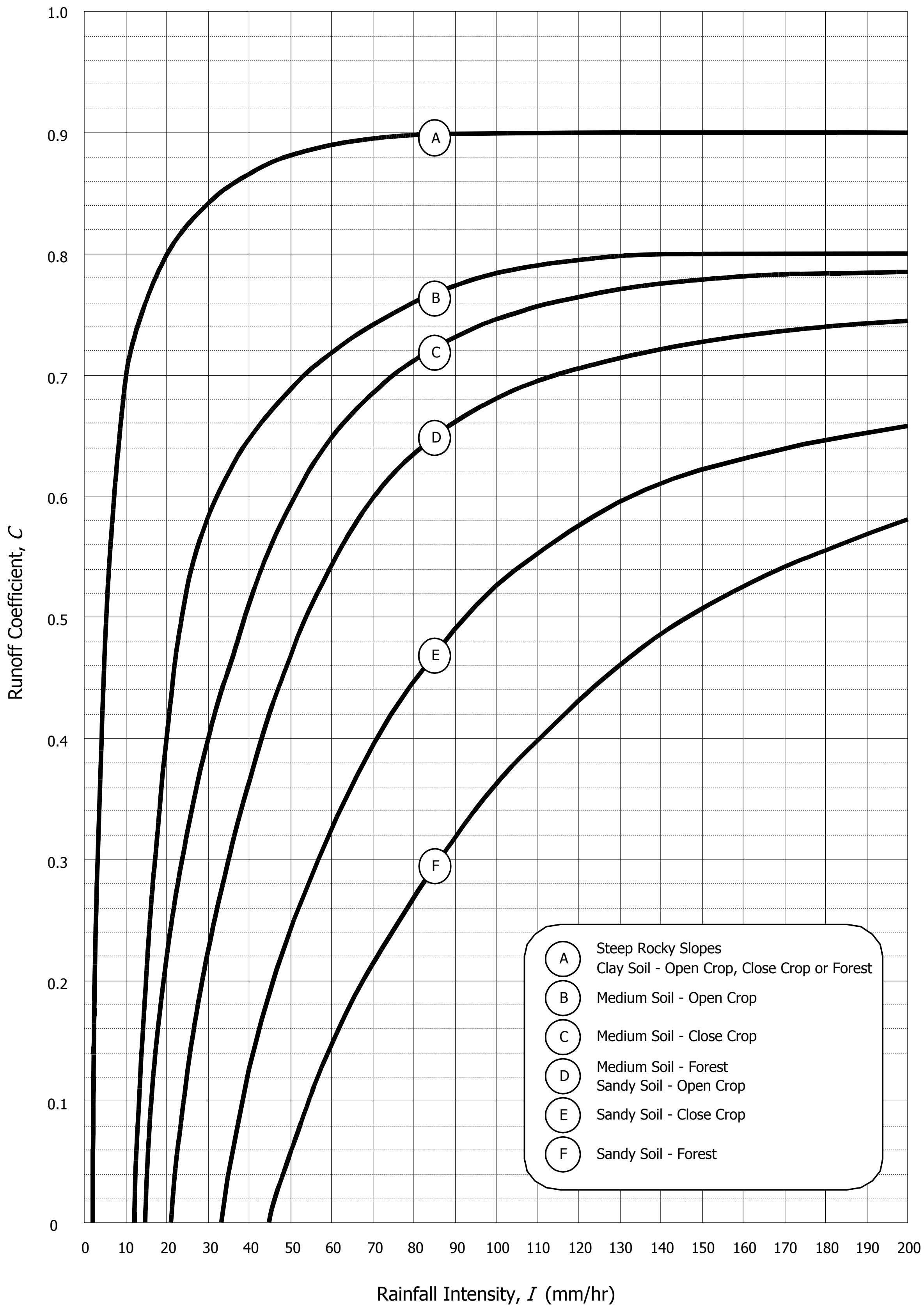


Design Chart 14.2 Kerb Gutter Flow Time



Design Chart 14.3 Runoff Coefficients for Urban Catchments
Source: AR&R, 1977

Note: For I > 200 mm/hr, interpolate linearly to C = 0.9 at I = 400 mm/hr



Design Chart 14.4 Runoff Coefficients for Rural Catchments
Source: AR&R, 1977

Note: For I > 200 mm/hr, interpolate linearly to C = 0.9 at I = 400 mm/hr

APPENDIX 14.B WORKED EXAMPLE

14.B.1 Rational Method Calculation

To determine the design peak for flow generated from a minor drainage of medium density residential area of 10 hectares in Kuala Lumpur. Assume 80 m of overland flow followed by 400 m of flow in an open drain. Catchment area average slope = 0.5%. The catchment is shown in Figure 14.B1.

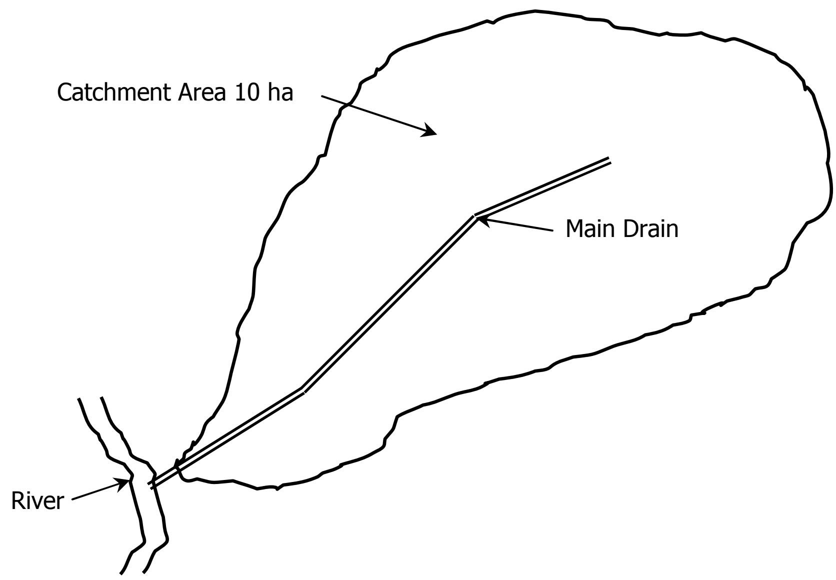


Figure 14.B1 Catchment Area

<u>Solution</u>: From Table 4.1, minor system design ARI = 5 years, major system design ARI = 100 years.

Step (1) : <u>Determine t_c</u>

From Design Chart 14.1 for paved surface,

t_o = 8 minutes.

Average velocity in the open drain should be assessed using Manning's equation. Assume V = 1.0 m/s.

Then $t_d = L/V$ = 400 sec. = 6.7 minutes, say = 7 minutes.

Therefore total t_c = 8 + 7 = 15 minutes.

Step (2): <u>Determine I and C</u>

For ARI of 5 years, from Table 13.A1 using Equation 13.1 for t = 30 minutes;

$$\text{Ln}(I) = 5.1086 + 0.5037\ln(30) - 0.2155[\ln(30)]^2 + 0.0112[\ln(30)]^3 = 4.7698$$

$$^5I_{30} = 117.9 \text{ mm/hr}$$

Similarly $^5I_{60}$ = 75.7 mm/hr

Convert to rainfall depths. $^5P_{30}$ = 115.3/2 = 587.9 mm $^5P_{60}$ = 75.7/1 = 75.7 mm

From Equation 13.3, $^5P_{15}$ = 57.65 – 0.8*(72.9-57.65)

$^5P_{15}$ = 45.5 mm, therefore $^5I_{15}$ = 182 mm/hr

From Design Chart 14.3, C = 0.87 (category (3))

Step (3): Determine Peak Flow (Q)

From Equation 14.7;

$$Q_5 = \frac{C \times {}^5I_{15} \times A}{360} = \frac{0.87 \times 182 \times 10}{360}$$

$$= 4.4 \text{ m}^3\text{/s}$$

14.B.2 Time-Area Calculation

Develop the runoff hydrograph from a 10 hectare medium density residential area located in Kuala Lumpur. The hydrograph is required to design the minor stormwater systems of the area. Given that the time of concentration, t_c at the outlet is 15 minutes. The study area is shown in Figure 14.B2(a).

Solution: From Table 4.1, minor system design ARI = 5 years.

Step (1): Delineate subcatchments based on the isochrones

The isochrones were approximated and drawn to subdivide the whole catchment into subareas, as shown in Figure 14.B2(a). Total area for each subcatchment within the two adjacent isochrones was estimated. The cumulative time-area curve is given in Column E of Table 14.B1.

Step (2): Calculate design rainfall I for the whole catchment

The design rainfall intensity, I was taken as 182 mm/hr. The rainfall distribution (Column B of Table 14.B1) and rainfall losses (Column C, taken from Table 14.4) for the design storm duration (15 minute), were assumed and shown in Figure 14.B2(b). Rainfall losses were deducted from the corresponding rainfall and the resulting effective rainfall is given in Column D of Table 14.B1.

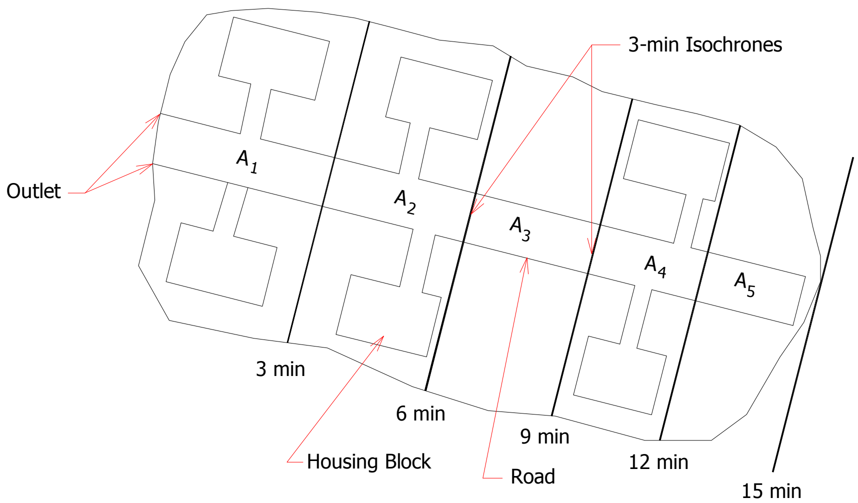
Step (3): Calculate ordinates of the hydrograph

Runoff generated from individual subcatchment (from Column F to J) due to each incremental effective rainfall amount (Column D) is calculated using Equation 14.10. These ordinates are added to get the ordinates of the total hydrograph at the outfall (Column K). The peak runoff discharge is calculated as 4.7 m^3/s. The total hydrograph is shown in Figure 14.B3(c).

Table 14.B1 Calculation of the Time-Area Method

A	B	C	D	E	F	G	H	I	J	K
Time	Rainfall	Losses	ER	Time-Area Curve	Runoff Generated by the Effective Rainfall (in mm)					Hydrograph
(min)	(mm)	(mm)	(mm)	(m^2)	9.9	15.9	9.1	6.8	2.3	(m^3/s)
0	0.0	0.0	0.0	0	0.00					0.00
3	11.4	1.5	9.9	27000	1.48	0.00				1.48
6	15.9	0.0	15.9	50000	1.26	2.39	0.00			3.65
9	9.1	0.0	9.1	69000	1.04	2.03	1.37	0.00		4.44
12	6.8	0.0	6.8	85000	0.88	1.68	1.16	1.02	0.00	4.75
15	2.3	0.0	2.3	100000	0.82	1.42	0.96	0.87	0.34	4.41
18					0.00	1.33	0.81	0.72	0.29	3.15
21						0.00	0.76	0.61	0.24	1.61
24							0.00	0.57	0.20	0.77
27								0.00	0.19	0.19
30									0.00	0.00

Note: ER - Effective Rainfall



(a) Study Area with Subcatchment and Isochrones

Figure 14.B2 Components of Time-Area Method