PEAK DISCHARGE ESTIMATION FOR RURAL AREAS USING APSWM AND OTTHYMO MODELS

By

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ABSTRACT

Traditional methods for flood estimation can be categorized as (1) simplified methods, e.g., regression analysis, (2) frequency analysis of streamflow data, (3) design storm-based precipitation-runoff modeling, and (4) continuous precipitation-runoff simulation modeling. The new approach – the Analytical Probabilistic Stormwater Model - was developed as an alternative to provide an efficient way of getting realistic estimation of peak discharges of desired frequencies for use in stormwater management of urban areas. To extend APSWM's application to rural areas, a series of comparisons were made between the calibrated design storm-based OTTHYMO model results, frequency analysis results and APSWM results for the Ganaraska River watershed. Special considerations were given to the transformation of the input parameter values of OTTHYMO model to those of APSWM. Comparable results were obtained for large floods, while APSWM may underestimate peak discharges of low return periods. Upon further testing and development, APSWM may be used for large rural areas.

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A_i - areas of subcatchment i;

a', b', c' - constants in IDF curve's equation;

b, $\bar{b}$ - interevent time and the mean of interevent time of all historical rainfall events(hr);

CN, CN^*- curve number and modified curve number respectively;

f_0 - initial infiltration capacity at the beginning of a storm event (mm/h);

f_c - ultimate infiltration capacity (mm/h);

f_m - maximum infiltration capacity of soil (mm/h);

f_p - infiltration capacity of soil (mm/h);

h - fraction of imperviousness of the subcatchment;

i_{ave} - average intensity (mm/h);

I_a, I_a^* - initial abstraction in CN method and modified CN method respectively ;

i_a - rainfall intensity on the rising limbs (time before the peak) (mm/h);

i_b - rainfall intensity on the falling limbs (time after the peak) (mm/h);

I_{t-1}, I_t - inflow rates at the beginning and the end of a routing interval respectively(cms);

$G_{V_R}(v_r)$ - exceedance probability of the effective rainfall volume;

k_d - decay coefficient for the infiltration capacity recovery curve;

K_n - the storage coefficient of the reservoir in Nash IUH;

K – wave travel time in Muskingum method (hr);

k_h - the infiltration capacity decay coefficient in Horton infiltration equation (h^{-1});

M - catchment characteristics;

n - the number of reservoirs in Nash IUH;

O_{t-1}, O_t - outflow rates at the beginning and the end of a routing interval respectively(cms);

P - rainfall volume (mm);

Q_p - peak discharge rate (cms);

Q - volume of runoff (mm);

r - ratio of the time to peak to the total storm duration in Chicago design storm;

r_k - autocorelation coefficient;

S_{CN}, S_{CN}^* - depression storage in CN method and modified CN method respectively(mm);

S_d - area-weighted depression storage of impervious areas and initial losses of pervious areas (mm);

S_{dd} - difference between initial losses in pervious areas and depression storage in impervious areas (mm);

S_{di} - depression storage of the impervious area (mm);

S_{dp} - depression storage of the pervious area (mm);

S_{il} - initial losses of pervious areas (mm);

S_{iw} - initial soil wetting infiltration depth (mm);

S_n - storage volume of the reservoir in Nash IUH;

$S_{t-1},\ S_t$ - storage volumes at the beginning and the end of a routing interval respectively;

S_{Qp} – maximum storage volume of the detention pond;

t, $\bar{t}$ - rainfall event duration and mean of rainfall event duration of all historical rainfall events (hr);

t_b, t_a - time after the peak and the time before the peak in Chicago design storm (hr);

t_c - time of concentration (hr);

t_d - rainfall duration of design storm (hr);

t_e - time required for the detention pond to empty itself after one runoff event (hr);

t_l - time lag (hr);

t_p - time to peak (hr);

T_{tra} - travel time in variable storage coefficient routing method;

Δt - time interval;

v, $\bar{v}$ - rainfall event volume and average rainfall event volume of all historical rainfall events (mm);

v_r - subcatchment runoff event volume (mm);

X - weighting factor in Muskingum routing;

$\Gamma(n)$ - the gamma function;

θ - average annual number of rainfall events;

ψ - parameter in the probability density function for interevent time b (h^{-1});

ζ - parameter in the probability density function for rainfall event volume v (mm^{-1});

λ - parameter in the probability density function for rainfall event duration t (hr^{-1});

Chapter 1 Introduction

1.1 Traditional Methods for Estimating Peak Flows

Flood discharge rates at specific locations on a stream are important information in structural design, flood and drought mitigation, and water resources planning and management. Traditional methods for flood estimation can be grouped into four categories: (1) use of simplified methods, (2) frequency analysis of streamflow data, (3) design storm-based precipitation-runoff simulation modeling, and (4) continuous precipitation-runoff simulation modeling.

Simplified methods include the use of formulas (e.g., rational method), regression equations, or envelope curves as a basis for making peak discharge estimates. These methods may be especially useful for preliminary estimates or for providing an independent check on estimates developed by other means.

Frequency analysis of actual streamflow records can be performed to develop exceedance frequency relationships for a gauged watershed. Unfortunately, for small watersheds, streamflow data are usually unavailable. Even in a gauged watershed, the records of historical flow or stage are generally too short to clearly define the frequency of large flood flows.

For situations where historical streamflow data are nonexistent or inadequate for frequency analysis, a precipitation-runoff simulation model is commonly used for flood runoff analysis. Compared with frequency analysis of historical streamflow records, the main advantage of the simulation modeling approach is its ability to evaluate flood runoff effects of watershed disturbances such as urban development, channel modification and structural projects.

Simulation models such as SWMM, HEC-HMS, OTTHYMO, MIDUSS are widely used in Canada. In a design storm-based model, precipitation input is a synthetics storm with a given frequency of occurrence. These storms are commonly referred to as design storms. The basic premise behind the use of design storms is that a design storm of a given return frequency will produce a simulated runoff peak and volume having the same return frequency. Although design storm-based models are widely used for water resources planning and design, its validity is questioned by some scholars since the basic premise has not been fully verified. In addition, such models do not perform a continuous water balance calculation, and therefore the initial baseflow and some measure of wetness of the watershed must be provided arbitrarily at the beginning of the simulation.

Continuous precipitation-runoff simulation models use long-term precipitation records as inputs to generate a continuous series of flows. Subsequent frequency analyses on the generated flow series may be conducted to estimate flood peaks of different return periods. Although continuous modeling is believed to provide the most accurate estimates of peak discharge rates of various return periods, the data requirement and onerous calculations limit its applicability in practice.

1.2 The New Analytical Probabilistic Approach for Peak Discharge Estimation

The analytical probabilistic approach for peak flow estimation was developed to overcome the problems of both the design storm-based modeling approach and the continuous simulation approach. The computer software package containing the analytical probabilistic stormwater models (APSWM) was developed and released recently. APSWM is mainly based on the storm event-based meteorological data analysis procedure as described in Adams et al.(1986), derived mathematical expressions for the probability distributions of runoff volume and peak discharge rate from urban catchments as described in Guo and Adams (1998a,b), and derived mathematical expressions for the probability distributions of peak discharge rate downstream of channel reaches and detention ponds as described in Guo (2006).

Rather than employing a design storm, the analytical probabilistic model estimates the frequency distribution of peak flows directly by using derived probability distribution theory. Derived probability distribution theory states that the probability distribution of dependent random variables is fundamentally related to and may be derived from those of the independent random variables using the functional relationship between dependent and independent variables.

To develop the analytical probabilistic model, the probability density functions (PDFs) of the rainfall event characteristics need to be specified first. The functional relationships of rainfall event characteristics (e.g., volume, duration, intensity and interevent time) and runoff characteristics (e.g., peak flow, runoff volume, runoff duration) have to be defined. Once these relationships are established, using the derived probability distribution theory, the PDFs of runoff characteristics (dependent variables) can be derived from those of the rainfall characteristics.

APSWM estimates the frequency distribution of hydrologic variables directly, and eliminates the assumption that the frequency of the design storm and the simulated peakflow are the same in the design storm approach. At the same time, the computation involved is minimal in APSWM, since the estimations are all obtained using a series of closed-form analytical equations. As a result, APSWM has a great potential in providing an easy way of obtaining similar results to those from continuous simulation models.

The validity of this new approach has been tested by performing comparisons with the continuous simulation approach for a number of hypothetical test catchments in Toronto, Canada (Guo and Adams 1998a,b), comparisons between the analytical probabilistic approach, the design storm approach, and the continuous simulation approach for a hypothetical urban test catchment in Chicago, USA (Guo, 2001) and comparison between the analytical probabilistic approach and the design storm approach for a practical design case for a small urban catchment in Kingston, Ontario, Canada (Quader and Guo, 2006).

1.3 Objectives and Methodology

Since APSWM was originally developed for urban stormwater management, it has not been tested for rural area hydrological studies where the waterhshed maybe medium to large in size. To investigate the potential applicability of APSWM for rural areas, the main objective of this study is to evaluate the performance of APSWM for a rural watershed hydrologic study. An actual case, Ganaraska River Hydrology Study (GRHS), is introduced. The study area is located on the north shore of Lake Ontario, 100 kilometers east of Toronto, comprising an area of 277 km^2. A series of comparisons between the design storm-based model Visual OTTHYMO (OTTHYMO) and APSWM are conducted. The peak flows obtained from the two models are further validated with those from frequency analyses of observed streamflow data.

In this study, special considerations are given to the transformation and equivalence of different loss models. A methodology for the transformation from runoff curve-number loss model to APSWM's Horton infiltration loss model is proposed and tested. Addition of runoff curve number loss model may enhance APSWM by providing more options in the precipitation loss estimation.

The description of precipitation input is an important element making APSWM different from other traditional precipitation-runoff simulation models. In using APSWM, the interevent time definition (IETD) and threshold rainfall volume (TRV) are the two parameters which affect input rainfall statistics. Selection of different IETDs and TRVs may lead to different results. In this study, efforts are also made in exploring the impacts of these two parameters.

1.4 Project Report Organization

Chapter 2 presents an overview of the description of precipitation in the design storm-based model and APSWM. Two types of design storms (Chicago design storm and AES design storm) and rainfall statistics with various IETDs and TRVs are generated for use in the case study. Chapter 3 and Chapter 4 describe and compare the methodologies of loss estimation, effective rainfall-runoff transformation and reach routing, which are applied in the design storm-based model and APSWM. The methods of converting hydrologic parameters in OTTHYMO for APSWM use will be developed. Chapter 5 gives a brief review of catchment aggregation strategies in the design storm-based model and APSWM. In Chapter 6, the real case of Ganaraska River Hydrology Study is introduced. APSWM model for selected test catchments and for the entire watershed are setup following the proposed methods described in Chapter 2 through Chapter 4. A series of comparisons are conducted to investigate the performances of APSWM in the rural area, and the results are reported and analyzed. Finally, Chapter 7 summarizes the results obtained in this project and also provides suggestions for future studies.

Chapter 2 Precipitation Description

In a simple conceptualized model illustrated by Fig. 2.1, the runoff process begins with precipitation. After deductions to satisfy evaporation/transpiration, interception, depression and infiltration, the remaining rainfall becomes overland flow which concentrates into stream channels. In stream channels, overland flow combines with interflow and baseflow and is transported further downstream. Surface runoff is attenuated and transformed, and continues to combine with inflows from other drainage areas along the channel. Accordingly, most simulation models conceptualize the rainfall-runoff process with three components: precipitation losses and overland flow generation, reach routing, and runoff combination from different subcatchments (named aggregation). The comparisons between the OTTHYMO model and APSWM with respect to these three components will be conducted subsequently in the following chapters. The precipitation inputs to the two approaches are described in this chapter.

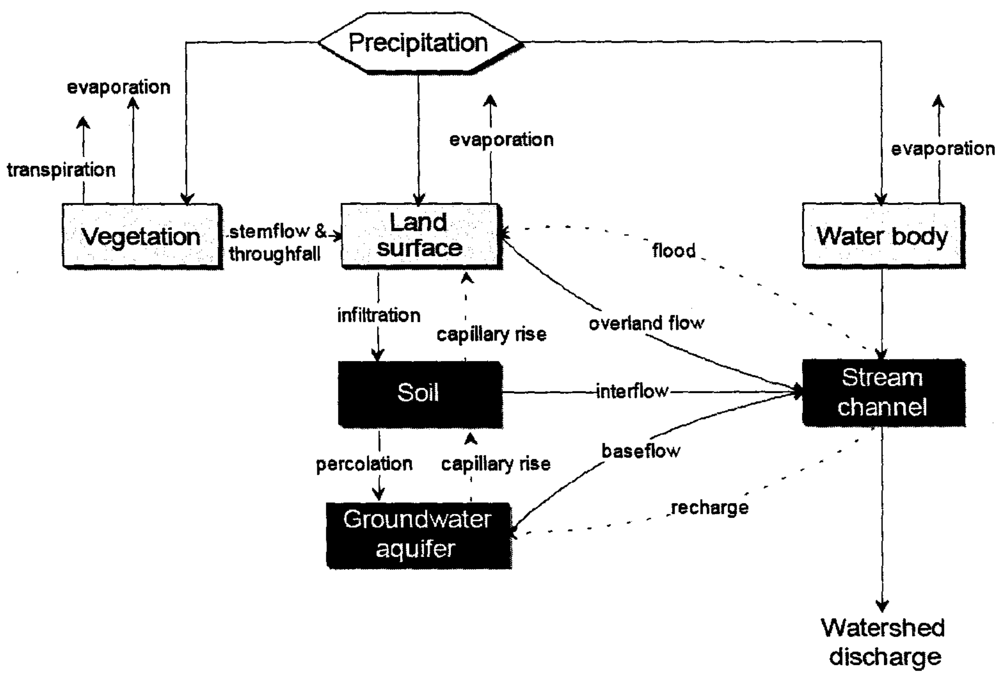


Fig. 2. 1 System diagram of the rainfall-runoff process (Source :HEC-HMS Hydrologic Modeling System User Manual, 2000)

2.1 Precipitation Input Description for OTTHYMO Models

2.1.1 Design Storms

In a design storm-based model, the precipitation input is usually a design storm which is assumed to produce flow of a desired return period. Typically, design storms are developed by statistical analyses of rainfall records. To define a design storm, three components should be considered, i.e., storm duration, total rainfall depth, and temporal distribution of the total rainfall volume within its duration.

(1) Duration of design storm.

Storm duration is an important factor to determine total depth for a given return period. Normally, it is believed that the duration should be long enough to allow runoff from the entire catchment to contribute to the design point. Thus, the typical storm duration is selected to be close to or longer than the time of concentration of the catchment. Generally, large watersheds or small watersheds with storage facilities may require the use of long duration storms such as 12-hour or 24-hour storm. Smaller watershed, such as a typical urban watershed may require a 1 to 3-hour duration storm.

(2) Design storm depth.

The total precipitation depth at a point is a function of storm frequency and storm duration. Rainfall intensity-duration-frequency (IDF) curves provide average rainfall intensity for a given storm duration and selected recurrence interval. The IDF curves published by Atmospheric Environment Services (AES) are available for more than 60 stations in Ontario. IDF information can be expressed in the forms of a table, curves or equations. The design storm depths can be calculated from IDFs by multiplying the average intensity by the selected storm duration. For the area where IDFs are not available, the isohyetal map of 2 year 1 hour rainfall and corresponding rainfall index were developed and published in Ontario Floodplain Management Technical Guidelines (1986).

The point storm depth developed from IDFs or isohyetal maps may be significantly higher than average basin precipitation because of the limited extent of storm cell. The ratios of basin average rainfall to point rainfall values, referred to as areal reduction factors, should be used for basin areas larger than 25 km^2 in Ontario. The adjustment in rainfall depth for a particular point of interest can be calculated by the method developed by the World Meteorological Organization (WMO). The WMO curves of adjustment for various areas are shown in Fig. 2.2

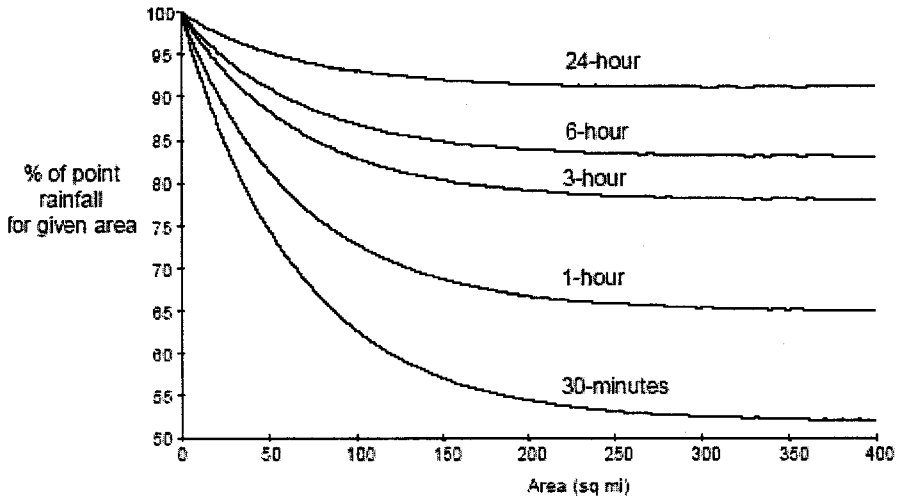


Fig. 2. 2 Point rainfall adjustment factors (Source : Ontario Floodplain Management Technical Guidelines)

(3) Temporal precipitation distribution

The variation of precipitation intensity over the design storm duration is another important factor in determining the timing and the magnitude of the resulting peak flow. The selection of temporal distribution might depend on the local meteorological data. In the past 40 years, several techniques have been developed as summarized in Table 2.1. In some cases, the design storm resulting in the most critical conditions can be selected.

Table 2. 1 Common Design Storm Temporal Distributions*

Design storm	Storm duration	Total rainfall depth	Temporal distribution	Intended application
Uniform	t_c	$i_{ave}*t_c$	uniform	Sewer sizing
Chicago	Usually between 2-6 hr	$i_{ave}*t_c$	Based on IDF curve	Sewer sizing
SCS 24 hr	Long duration usually 12-24 hrs	$i_{ave}*t_c$	Tabulated type 1 & 2 distribution	Rural watershed
SCS 6 hr	6 hr	Given in maps	Tabulated	Design of small dams
ISWS	1 hr	$i_{ave}*t_c$	Huff 1st Quartile 50% distribution	Sewer sizing
AES	1 hr or 12 hr	$i_{ave}*t_c$	Regional charts for 1 and 12 hr durations	Not specified
Urban design storms	1 hr	$i_{ave}*t_c$	Equation/tabulated	Storm water modeling
Yen & Chow	t_c	$i_{ave}*t_c$	Triangular	Drainage facilities in small area

*Note: Adapted and revised from Visual OTTHYMO Technical Manual

The concept of design storm has been widely used in current engineering practices and has achieved almost universal acceptance due to its simplicity and conservatism if properly applied. However the following shortcomings cannot be ignored:

1. The assumption of same exceedance frequency of design storm and simulated peak flow has not been proven to be true.

2. The selected rainfall volume and duration are not the real volume and duration of actual storm events.

3. Since the frequency of rainfall volume and the frequency of intensity are not the same, and the design storm is assumed to have a unique frequency of occurrence, whether it is based on volume or intensity, the frequency cannot represent the frequency of occurrence of all the characteristics of nature rainfall events (Adams & Papa, 2000).

2.1.2 Design Storms Used in the GRHS Case Study

Design storms can be generated from IDF curves (Chicago design storms) or be obtained from analyses of historic storm events (AES design storms). Because both Chicago design storms and AES design storms are generated from local meteorological data, these two types of design storms are suitable for the Ganaraska River Hydrology Study.

(1) Chicago design storm

Chicago temporal distribution of rainfall was proposed by Keifer and Chu in 1957. The duration of Chicago hyetograph is divided into two time periods, the time before the peak (t_a) and the time after the peak (t_b). The ratio of the time to peak to the total storm duration is expressed as r. Values of r are determined individually for a number of historical storms and the mean value is used for the design hyetograph. The values of r for various locations are listed in Table 2.2. In Ontario, r = 0.48 is recommended by Marsalek (1978).

Table 2. 2 Value of *r* for Various Locations

Location	Time to peak/total storm duration (r)	Reference
Baltimore	0.399	McPherson, 1958
Chicago	0.375	Keifer and Chu, 1957
Cincinnati	0.325	Preul et al., 1973
Ontario	0.480	Marsalek, 1978
Philadelphia	0.414	Mcpherson, 1958

The Chicago design storm is developed from IDF relationships. It has a continuous function representation of the instantaneous intensity. In a given duration (t_d), the rainfall volume (P) obtained from IDF by multiplying the average intensity (i_{ave}) by t_d is equal to the rainfall volume derived by integrating the continuous function of instantaneous intensity within that duration.

Therefore, for the IDFs expressed as in Eq. (2.1), the instantaneous intensity of the Chicago design storm can be derived by differentiating P with respect to time.

$$i_{ave} = \frac{a'}{(t_d + b')^{c'}} \tag{2.1}$$

OTTHYMO can generate Chicago design storms automatically when r, t_d and IDFs are defined. The software package accepts the description of IDFs in the forms of three parameter equations such as Eq. (2.1) or a table. The IDFs in Toronto Pearson

International Airport station and in Peterborough STP are available from Atmospheric Environment Services (see Table 2.3 and Table 2.4) and will be used in the case study.

Table 2. 3 IDF Curves of Toronto Pearson International Airport

(Based on 39 year records from 1950 to 1990)

Duration(min)	Rainfall amount for different return period (mm)					
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
5	9	11.8	13.7	16.1	17.8	19.6
10	13.2	17.2	20	23.4	25.9	28.4
15	16.3	21.6	25.2	29.6	32.9	36.2
30	21.5	28.5	33.1	39	43.4	47.7
60	24.3	32.2	37.5	44.1	49	53.9
120	28.4	38.4	45	53.4	59.5	65.7
360	36.9	51.1	60.6	72.4	81.3	90
720	42.3	59.2	70.5	84.7	95.2	105.7
1440	47.4	67.2	80.4	97	109.3	121.5
Fitted equation: $i_{ave} = \frac{a'}{(t_d + b')^{c'}}$						
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Coefficient a'	22.1	29.8	34.9	41.3	46.1	50.8
Coefficient b'	0	0	0	0	0	0
Coefficient c'	0.725	0.711	0.705	0.699	0.696	0.694

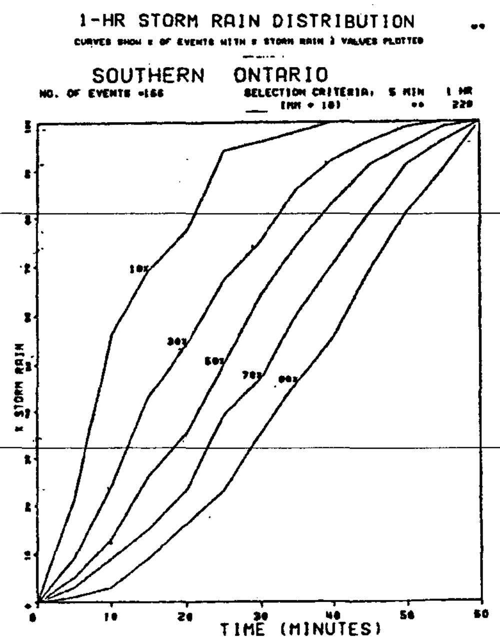
Table 2. 4 IDF Curves of Peterborough STP

(Based on 31year records from 1971 to 2002)

Duration(min)	Rainfall amount for different return period (mm)					
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
5	7.7	10.2	11.8	13.9	15.4	17
10	11.4	14.8	17	19.8	21.9	23.9
15	14	18.4	21.4	25	27.8	30.5
30	18	24.1	28.1	33.1	36.9	40.6
60	22.2	30.4	35.8	42.6	47.6	52.6
120	27.7	39.9	48	58.2	65.7	73.2
360	38.6	51.8	60.6	71.6	79.8	88
720	43.9	57.7	66.8	78.4	86.9	95.4
1440	48.1	62.7	72.4	84.7	93.7	102.8
Fitted equation: $i_{ave} = \frac{a'}{(t_d + b')^{c'}}$						
	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Coefficient a'	20.4	27.3	31.9	37.6	41.8	46.1
Coefficient b'	0	0	0	0	0	0
Coefficient c'	0.684	0.682	0.682	0.681	0.681	0.68

(2)AES Design Storm

Hogg (1980) developed temporal storm distributions for 1-hour and 12-hour durations by examining almost 2000 extreme rain events across Canada. The probability distribution curves were developed for different regions. It was found that differences in the distributions between regions were substantial. The main advantage of the AES design storms is that it is based on actual Canadian data. The disadvantage is that only two durations (1 hour and 12 hour) are provided. For Southern Ontario, the cumulative rainfall distributions for various probability levels were plotted in Fig. 2.3.



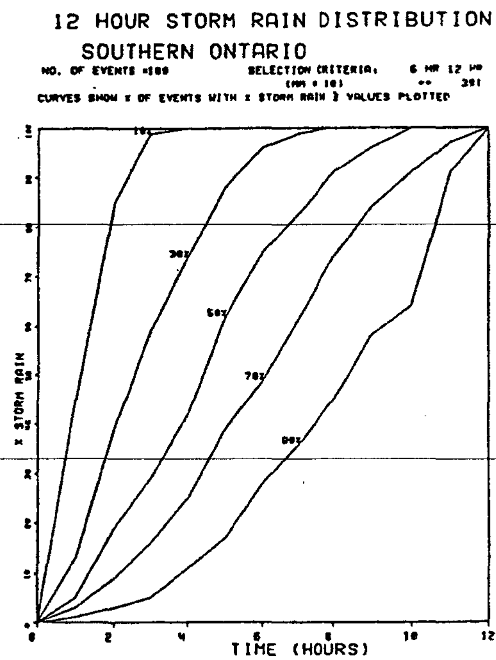


Fig. 2.3 AES 1-hour and 12–hour storm distributions in Southern Ontario (after Hogg, 1980)

Caution should be taken to select the appropriate rainfall distribution graph. It is suggested that the user should analyze local storms to establish appropriate distributions. Where no such data are available, the 30% curve is suggested by the Flood Plain Management Technical Guidelines (1986). Thus, the AES 12 hour 30% distribution will be used for the case study. The temporal distribution and cumulative distribution are tabulated in Table 2.5.

Table 2. 5 AES 12-hour 30% Storm Distribution of Southern ON (DT=60 min)

Time (hours)	Cumulative distribution (%)	Incremental distribution (%)
0	0	0
1	0.18	0.18
2	0.39	0.21
3	0.59	0.2
4	0.74	0.15
5	0.88	0.14
6	0.96	0.08
7	0.99	0.03
8	1	0.01
9	1	0
10	1	0
11	1	0
12	1	0

2.2 Precipitation Input to APSWM

2.2.1 PDFs of Rainfall Event Characteristics

Instead of using an individual event as rainfall input in the design storm-based models, the analytical probabilistic approach uses probability models to represent storm characteristics. To generate probability density functions (PDFs) of the rainfall event characteristics, continuous rainfall series at a specific location must be separated into individual storms. A minimum time period without rainfall or interevent time definition (IETD) is typically used as a criterion. In addition, Guo and Adams (1998a) recommended that the storms with a rainfall volume $\leq$1.0 mm should be removed, since those storms result in a negligible amount of runoff and their removal results in a better fit between analytical PDFs and observed frequencies. This threshold rainfall volume is denoted as TRV. When the separated rainfall event series are obtained, the characteristics such as volume (v), duration (t), and interevent time (b) for individual rainfall events can be determined and statistically analyzed. It is shown that exponential probability density distributions provide a good fit to the histograms obtained from the frequency analyses for each of these event characteristics.

The exponential distributions for rainfall characteristics are as follows:

$$f_V(v) = \zeta e^{-\zeta v}, v \geq 0 \text{ where } \zeta = \frac{1}{\bar{v}} \text{ (mm}^{-1}\text{)}$$

$$f_T(t) = \lambda e^{-\lambda t}, t \geq 0 \text{ where } \lambda = \frac{1}{\bar{t}} \text{ (hr}^{-1}\text{)}$$
$$f_B(b) = \psi e^{-\psi b}, b \geq 0 \text{ where } \psi = \frac{1}{\bar{b}} \text{ (hr}^{-1}\text{)} \tag{2.2}$$

2.2.2 Interevent Time Definition (IETD)

The use of different IETDs will result in different values of the calculated statistics of storm event characteristics. The means of rainfall volume, duration and interevent time increase with increasing IETD.

Adams and Papa (2000) summarized three approaches to determine a suitable IETD:

(1) Autocorrelation analysis. The autocorrelation coefficient of a sample of observation (y) for a lag time of k is given by

$$r_k = \frac{\sum(y_t - \bar{y})(y_{t-k} - \bar{y})}{\sum(y_t - \bar{y})^2} \tag{2.3}$$

The lag time that is sufficiently long to ensure statistical independence between successive rainfall events (i.e., r_k close to 0) may be used as IETD.

(2) Coefficient of variation. Since exponential distributions are used for interevent time b, the IETD can be selected as the one which results in the sample coefficient of variation for interevent times to be equal to or approximately equal to unity.

(3) Examination of the relationship between the IETD and the average annual number of rainfall events. IETD may be selected as that after which increases in the IETD do not result in significant changes in the annual number of events observed.

Adams and Guo (1998a) indicated that the rainfall characteristics become less sensitive for IETD longer than 6 hours. In practice, the interevent time definition should also be governed by the application and the catchment response time. 1 to 6 hours can be used for most urban applications, while 12 to 24 hours is recommended for larger rural areas.

2.2.3 Threshold Rainfall Volume (TRV)

Furthermore, Guo and Adams (1998a) did investigation on Toronto Pearson airport precipitation data and observed that when storms with a rainfall volume ≤ 1.0 mm are removed, the exponential distribution fits observed relative frequency data better for an IETD of 6 hours. Additional comparisons were made in this study using Toronto Pearson International Airport precipitation data with an IETD of 12 hours. Fig. 2.4(a) through Fig. 2.4(f) show the similar results as reported by Guo and Adams (1998a). The rainfall statistics with various IETDs and TRVs are listed in Table 2.6.

Table 2. 6 Rainfall Statistics for the Toronto Pearson Int' Airport Station with Different IETDs and TRVs

	Average volume(mm), ($\bar{v}$)	Average duration(hr) ($\bar{t}$)	Average interevent time(hr) ($\bar{b}$)	Average annual number of rainfall events (θ)
IETD=6 hr, TRV = 0mm	6.906	6.01	73.53	79.8
IETD=6 hr, TRV=1mm	9.425	7.728	103.09	57.2
IETD=12 hr, TRV=0mm	8.43	9.26	93.9	64.7
IETD=12 hr, TRV=1 mm	10.707	11.136	113.636	50.7
IETD=24 hr, TRV=0mm	12.47	18.05	126.58	43.8

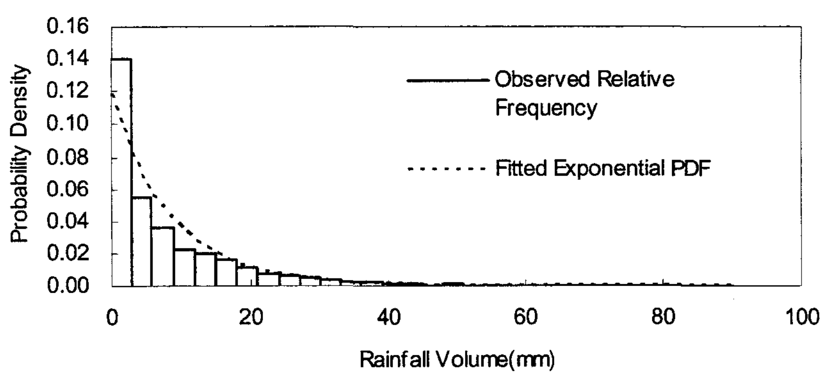


Fig. 2.4 (a) Histogram and pdf of rainfall event volume with all rainfall events included

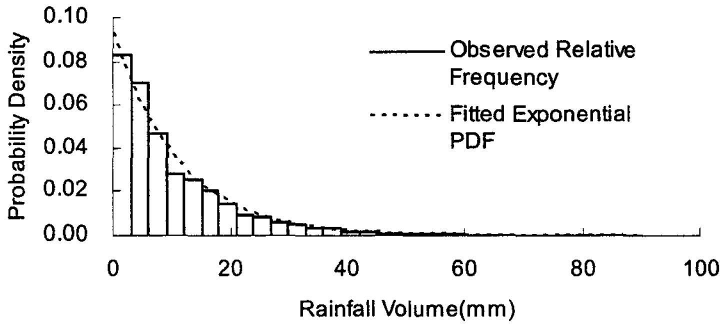


Fig. 2.4 (b) Histogram and pdf of rainfall event volume for rainfall events with v>1.0 mm

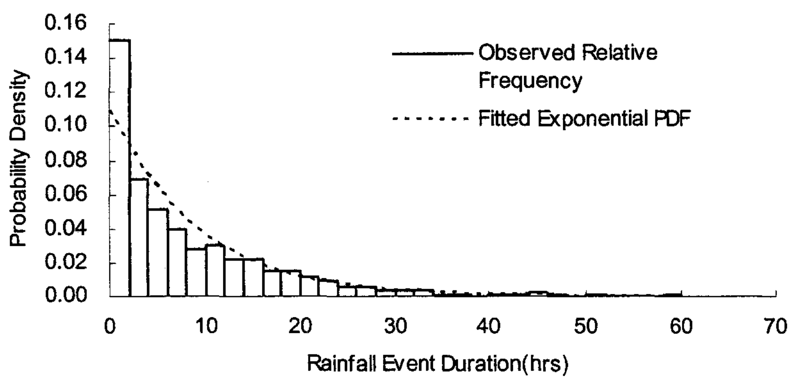


Fig. 2.4 (c) Histogram and pdf of rainfall event duration with all rainfall events included

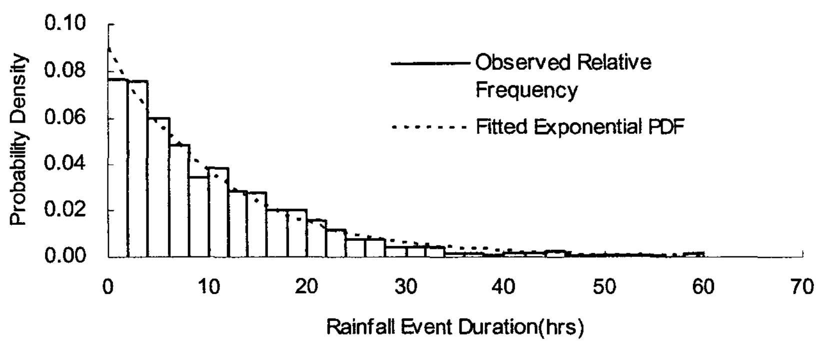


Fig. 2.4 (d) Histogram and pdf of rainfall event duration for rainfall events with v>1.0 mm

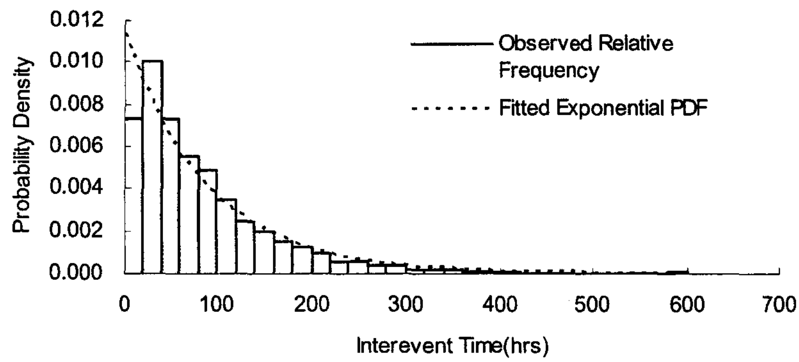


Fig. 2.4 (e) Histogram and pdf of interevent time with all rainfall events included

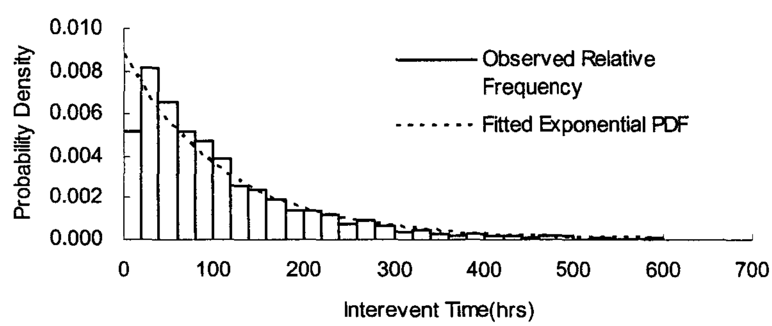


Fig. 2.4 (f) Histogram and pdf of interevent time for rainfall events with v>1.0 mm

2.3 Summary

The fundamental difference between design storm-based models and APSWM is their different descriptions of precipitation input. The design storm-based models describe the precipitation input as a single design storm with a certain exceedance frequency, while APSWM represents precipitation using the probability density functions (PDFs) of rainfall event characteristics. The design storm-based models assume that a design storm of a given return frequency will produce a simulated runoff peak having the same exceedance frequency, while APSWM generates PDFs of runoff peak discharges and volumes directly. Compared with design storms, probabilistic models of storm event characteristics have several advantages as summarized by Guo (2006).

(1) The complete spectrum of the frequency distributions of major storm event characteristics (i.e., v and t) are properly described and represented.

(2) The actual durations and volumes of storm events are studied and characterized.

(3) By describing and inclusion of the frequency distribution of interevent times, the recovery of the soil's infiltration capacity and detention pond's storage capacity between rainfall events can be modeled.

Different temporal distributions of the design storm might result in different results. Two types of design storms developed from local meteorological records, i.e., Chicago and AES -12 hour design storms will be used in the case study. TRV and IETD are the two major parameters affecting the statistics of storm event characteristics. The impacts of these two parameters will be demonstrated in Chapter 6.

Chapter 3 Individual Subcatchment Rainfall-Runoff Transformation

In the case study of GRHS, the modified curve number model is applied for loss estimation and Nash unit hydrograph is employed to transform effective rainfall hyetograph to direct runoff hydrograph for individual subcatchments. Whereas, APSWM loss model is based on Horton infiltration model and subsequently PDFs of runoff volume and peak flow are derived using derived probability distribution theory. The parameters required in OTTHYMO and APSWM for describing subcatchment physical characteristics are different because of the different methodologies. As a result, conversion of parameter values from OTTHYMO model to APSWM is necessary. In this chapter, the procedures of conversion are presented.

3.1 Loss Models

Precipitation losses include evaporation, surface storage and infiltration. Most models omit any detailed accounting of evaporation and transpiration, as these are insignificant during a rainfall event. Surface storage including interception and depression storage is intended to represent the surface storage of water by trees or grasses and local depressions on the ground surface.

The loss/infiltration methods can be categorized as physically based or empirical methods. The physically based models, such as Green and Ampt, Horton's infiltration method, are based on simplified solutions to the Richards equation. Empirical methods, such as the Soil Conservation Service (SCS) curve number (CN) method, are based on correlating parameters estimated from rainfall runoff records to factors affecting loss rates such as soil type and surface cover. The difference between the loss model and the infiltration model is that the loss model estimates total losses and the effective rainfall volume, while infiltration method only considers the soil infiltration portion.

3.1.1 SCS-CN Loss Method and Modified CN Loss Method Used in OTTHYMO

3.1.1.1 CN Loss Method

The Soil Conservation Service Curve Number (SCS-CN) method is an empirical method for estimating direct surface runoff. The basic equation is

$$Q = \frac{(P - I_a)^2}{(P - I_a + S_{CN})} \tag{3.1}$$

Where Q is runoff (mm); P is precipitation (mm); I_a is the initial abstraction that accounts for the short-term losses, such as interception and surface storage, that occur at the beginning of a storm (SCS recommends that $I_a = 0.2S_{CN}$); S_{CN} is the recharge capacity or "potential maximum retention" of the watershed, which is related to CN by

$$S_{CN} = \frac{25400}{CN} - 254 \tag{3.2}$$

The 'curve number' CN is a function of the soil type, the land use and the initial degree of saturation known as the antecedent moisture condition (AMC).

3.1.1.2 Modified CN Loss Method

A lot of engineers believe that, in Ontario, the assumption of $I_a = 0.2\ S_{CN}$ results in over-estimated initial abstraction, especially in areas with low curve numbers. The Modified Curve Number method was first proposed by Paul Wisner & Associates in 1982. Rather than having $I_a = 0.2\ S_{CN}$ in the SCS-CN method, modified CN method fixes I_a at a certain value, and then the CN is modified to CN^* which can generate equal runoff volumes at AMC III conditions.

CN^* can be calculated by the following procedures recommended in Visual OTTHYMO Reference Manual.

(1) Convert the CN (AMC II conditions) to a CN (AMC III conditions);

(2) Determine the largest precipitation volume P that would represent AMC III soil moisture conditions. Normally, 100-year storm is used;

(3) Calculate runoff volume for the 100 year storm event by using Eq. (3.1), where $I_a = 0.2\ S_{CN}$, and CN is the weighted CN at AMC III soil conditions;

(4) Calculate S_{CN}^* using the Eq. (3.1) again to generate the same runoff in step (4), but this time setting I_a^* to 5mm;

(5) Calculate CN^* for AMC III soil conditions from the Eq. (3.2) and convert the CN^* for AMC II soil conditions by using published tables relating CN at AMC II and AMC III conditions.

3.1.1.3 Relationship between *CN* and *CN**

In the Ganaraska River Hydrology study, the rainfall volume of 100-year, 12-hour duration event in the Toronto international airport is 105.4mm, which is used as the largest precipitation. Following the above procedures, S_{CN}* and *CN** are calculated in Table 3.1. At the same time, the related I_a and *CN* are listed.

Table 3. 1 *CN* VS. *CN in Ganaraska River Hydrology Study**

CN	$S_{CN}^{(1)}$	$I_a^{(2)}$	I_a / S_{CN}	CN^*	I_a^*	S_{CN}^*
100	0.00	0.00	-	100.00	5	0
95	13.37	1.04	0.08	98.00	5	0.98
90	28.22	2.12	0.07	93.11	5	7.20
85	44.82	3.24	0.07	87.00	5	13.93
80	63.50	5.02	0.08	79.98	5	25.16
75	84.67	6.93	0.08	73.33	5	37.97
70	108.86	8.96	0.08	66.36	5	52.71
65	136.77	11.15	0.08	60.55	5	69.82
60	169.33	14.33	0.08	52.87	5	97.35
55	207.82	17.85	0.09	45.79	5	132.09
50	254.00	22.82	0.09	37.16	5	190.36
45	310.44	27.35	0.09	29.92	5	255.09
40	381.00	33.87	0.09	22.46	5	374.09
35	471.71	41.56	0.09	15.48	5	571.57

Note: (1) S_{CN} is calculated by Eq. (3.2), where *CN* is for AMC II soil condition;

(2) I_a is calculated from *CN* at AMCIII, so the ratio of I_a to S for AMC II soil condition is not 0.2, but around 0.08.

Examination of the above table, it can be found that there is an approximate linear relationship between *CN* and *CN** (Fig. 3.1) which may be fitted by Eq. (3.3).

$$CN = 1.3612CN^* - 30.239 \tag{3.3}$$

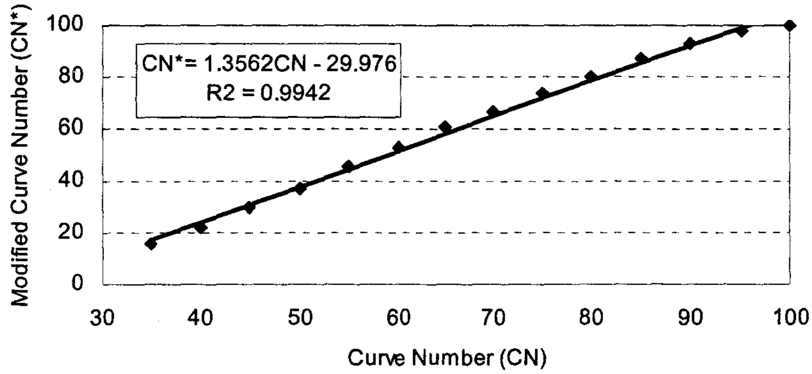


Fig. 3.1 Curve Number VS. Modified Curve Number in OTTHYMO

3.1.2 Horton Infiltration Method and APSWM Loss Model

3.1.2.1 Horton Infiltration Method

Horton infiltration method describes the infiltration process as exponential where the infiltration capacity reduces from an initial maximum rate to an ultimate constant rate as the soil becomes saturated. It is expressed as

$$f_p = f_c + (f_0 - f_c) * e^{-k_h T} \tag{3.4}$$

Where f_p is the infiltration capacity of soil at time T (mm/h); f_c is the ultimate infiltration capacity (mm/h), a function of soil type; f_0 is the initial infiltration capacity at the beginning of a storm event (mm/h); T is the time from the beginning of the storm (hours); k_h is the infiltration capacity decay coefficient (h^{-1}).

The cumulative infiltration can be obtained by integration of Eq. (3.4) with respect to t.

$$F = f_c t + \int_0^t (f_0 - f_c) * e^{-k_h T} dT = f_c t + \frac{(f_0 - f_c)}{k_h}(1 - e^{-k_h t}) \tag{3.5}$$

3.1.2.2 APSWM Loss Model

Similar to the design storm-based models, APSWM considers hydrologic losses as comprised of three parts: interception, surface storage and soil infiltration. Interception and surface storage are lumped together and denoted as depression storage. Depression storage consists of impervious area depression storage (S_{di}) and pervious area depression storage (S_{dp}). Horton infiltration method is used to estimate the soil infiltration loss throughout the duration of the runoff event in pervious areas. Guo and Adams (1998a) revised Eq. (3.5) to Eq. (3.6). Thus, the total infiltration loss in APSWM comprises (1) loss due to constant infiltration rate (f_c) and (2) loss due to initial wetting of soil (S_{iw}) representing the soil infiltration losses when rate of infiltration initially exceeds the constant infiltration rate (f_c).

$$F = f_c t + \frac{(f_0 - f_c)}{k_h}(1 - e^{-k_h t}) = f_c t + S_{iw} \tag{3.6}$$

Because f_0 changes with moisture condition of every storm, APSWM uses equations in the U.S. EPA SWMM model to estimate the recovery of infiltration capacity during dry periods. In APSWM, S_{iw} is approximated as a constant equaling the expected value of S_{iw}.

$$E(S_{iw}) = \frac{(f_m - f_c)k_d}{(k_h + \lambda)(k_d + \psi)} \tag{3.7}$$

Where ψ and λ are parameters of exponential distributions of b and t respectively; k_d is a decay coefficient for the infiltration capacity recovery curve; f_m is the maximum infiltration capacity of soil.

3.1.2.3 The APSWM Loss Model and the Exceedance Probability of Runoff Event Volume

APSWM considers the loss in a very similar way as many continuous or event simulation models. The only difference is that the continuous or event simulation model calculates loss for each time step, while APSWM is rainfall event based. The functional relationship is described in Guo and Adams (1998a) as follows:

$$v_r = \begin{cases} 0 & , \quad v \le S_{di} \\ h(v - S_{di}) & , \quad S_{di} < v \le S_{il} + f_c t \\ v - S_d - f_c(1-h)t & , \quad v > S_{il} + f_c t \end{cases} \tag{3.8}$$

Where v_r is the subcatchment runoff event volume(mm); S_{di} is the depression storage of the impervious area of the subcatchment (mm); S_{il} is initial losses of pervious areas, which is the sum of the depression storage (S_{dp}) and the initial soil wetting infiltration depth (S_{iw}); h is fraction of imperviousness of the subcatchment; f_c is ultimate infiltration capacity of the soil (mm/h); S_d = S_{di} + (1-h)S_{il} is the area-weighted depression storage of impervious areas and initial losses of pervious areas of the subcatchment.

Based on Eq. (2.2) and Eq. (3.8) and the derived probability theory, the exceedance probability of the runoff event volume (denoted as $G_{V_R}(v_r)$) was determined to be:

$$G_{V_R}(v_r) = \begin{cases} \exp(-\zeta S_{di}) & , \quad v_r = 0 \\ \exp(-\zeta S_{di} - \frac{\zeta}{h} v_r) & , \quad 0 < v_r \le hS_{dd} \\ \frac{\lambda}{\lambda + \zeta f_c - \zeta f_c h} \exp(-\zeta S_d - \zeta v_r) + & \\ \frac{\zeta f_c (1-h)}{\lambda + \zeta f_c - \zeta f_c h} \exp\left[-\zeta S_{di} + \frac{\lambda}{f_c} S_{dd} - \frac{1}{h}(\zeta + \frac{\lambda}{f_c}) v_r \right] & , \quad v_r > hS_{dd} \end{cases} \tag{3.9}$$

3.1.3 Conversion of CN Parameters to those for APSWM Use

The SCS-CN method is more popular than the modified CN method in practice. Establishing the relationship between SCS-CN method and APSWM/Horton loss model is more straightforward and will assist in extending APSWM application in practice. Because the linear relationship between CN^* and CN is obvious, the CN^* values can be converted to those of CN and I_a first, and subsequently be converted to the f_c and S_{il} values using the following procedures.

As discussed previously, the cumulative potential loss in APSWM comprises three parts: (1) loss due to interception and depression storage of impervious area, denoted as S_{di}, (2) initial losses of pervious areas (S_{il}) which is the sum of the depression storage (S_{dp}) and the initial soil wetting infiltration depth (S_{iw}), (3) due to infiltration of pervious area at a rate equaling f_c.

The three parts of losses in APSWM model can be obtained from the CN loss method by the following relationships.

(1) Both initial soil wetting of infiltration (S_{iw}) and depression storage (S_{dp}) are treated as constants for all the storm events and subtracted from rainfall volume before runoff occurs, similar to initial abstraction (I_a) in CN method. Thus, S_{iw} and S_{dp} can be lumped to S_{il} which is assumed to be equal to I_a in the SCS-CN method.

(2) Ultimate infiltration capacity f_c is a function of soil type. The relationship between f_c and hydrologic soil group (HSG) was proposed by Musgrave (Maidment, 1993). *CN* is determined by not only soil type, but land use as well. For the purpose of stormwater study, meadow cover is the dominant land use in urban areas. Curve number with meadow land use can be used to establish the relationship between f_c and *CN* associated with HSG. For the Ganaraska River watershed, the main land uses are agricultural and forest. The combined *CN* for 50% wood cover and 50% cultivated land is used to develop the relationship between f_c and *CN* (Table 3.2).

Table 3. 2 Conversion between Curve Number and Ultimate Infiltration Capacity

HSG	Ultimate infiltration capacity f_c (mm/hr)$^{(1)}$	*CN* of meadow cover$^{(2)}$	Combination *CN* for 50% wood cover and 50% cultivated land $^{(2)}$
A (sandy, loamy sand or sandy loam)	76	30	54
B (silt loam or loam)	38-76	58	70.5
C (sandy clay loam)	13-38	71	80.5
D (clay loam, silty clay loam, sandy clay, silt clay or clay)	0-13	78	85

Note: (1) Source: *Handbook of Hydrology* by Maidment (1993)

(2) Source: U.S. Soil Conservation Service National Engineering Handbook (1972)

3.2 Subcatchment Rainfall – Runoff Routing

3.2.1 Unit Hydrograph and Nash IUH in OTTHYMO

In a design storm-based model, runoff hydrograph is usually obtained from excess rainfall hyetograph through convolution calculations using the unit hydrograph (UH) technique. A unit hydrograph represents direct runoff at the outlet of a basin resulting from one unit of excess rainfall fallen uniformly over the basin at a constant rate. The underlying assumption of the UH procedure is that the runoff process is linear. When the duration of the input rainfall event approaches zero, an instantaneous unit hydrograph (IUH) is obtained. Unit hydrographs can be developed and verified with recorded precipitation and corresponding streamflows in gauged watersheds. For ungauged basins, synthetic unit hydrographs can be generated from measurable basin characteristics. Three synthetic IUH options are provided in OTTHYMO. They are Nash IUH, William IUH and Urban IUH. In the GRHS case study, Nash IUH is employed.

Nash instantaneous unit hydrograph method conceptualizes a catchment as a series of *n* identical linear reservoirs such that the outflow from one reservoir is the inflow to the other. This is illustrated in Fig. 3.2.

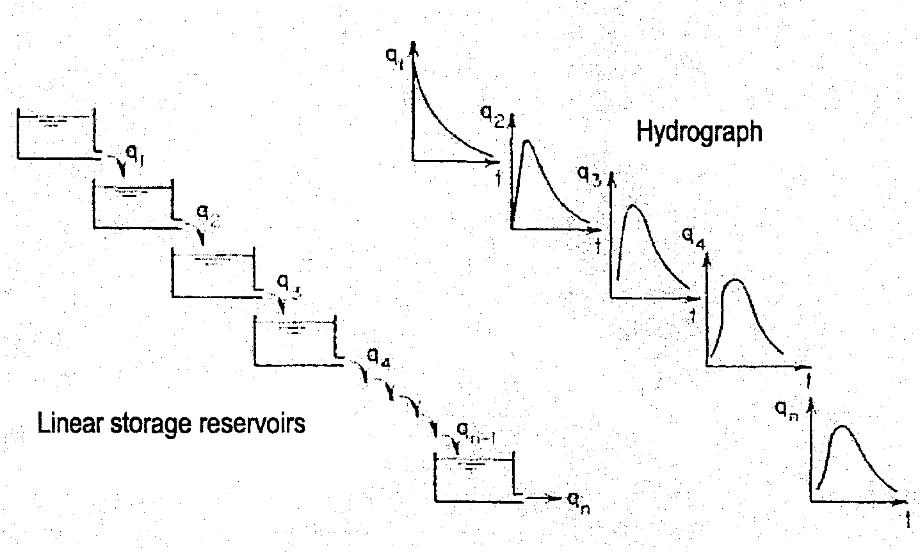


Fig. 3.2 Nash concept for deriving the instantaneous unit hydrograph (after Chow, 1964)

The behavior of each reservoir is governed by the following laws:

Continuity law:
$$\frac{dS_n}{dt} = I(t) - Q(t) \tag{3.10}$$

Where S_n is the storage volume of the reservoir; *I(t)* is the rate of inflow to the reservoir, and *Q(t)* is the rate of depletion (drainage).

Storage law:
$$S_n(t) = K_n Q(t) \tag{3.11}$$

Where K_n is the storage coefficient of the reservoir and is a constant. Substitute Eq. (3.11) into Eq. (3.10),

$$K_n \frac{dQ}{dt} + Q = I \tag{3.12}$$

The general expression of the ordinates of the Nash instantaneous unit hydrograph is obtained as:

$$Q(t) = \frac{1}{K_n \Gamma(n)} \left(\frac{t}{K_n} \right)^{(n-1)} e^{\left(\frac{-t}{K_n} \right)} \tag{3.13}$$

Where Γ(n) is the gamma function; n is the number of reservoirs.

By differentiating Eq. (3.13) with respect to t/K_n and equating it to zero, the time to peak t_p in terms of n and K_n is obtained:

$$t_p = (n-1)K_n \tag{3.14}$$

The time lag (t_l) is defined as the difference between the centroids of excess rainfall hyetograph and the resulting surface runoff hydrograph. The relationship between t_l and K_n was found as (Chow,1964):

$$t_l = nK_n \tag{3.15}$$

Then, combining Eq (3.14) and Eq (3.15),

$$t_p = \frac{n-1}{n} t_l \tag{3.16}$$

3.2.2 Subcatchment Rainfall-Runoff Routing in APSWM

Instead of transforming hyetograph with a certain return period to its corresponding hydrograph in the design storm-based model, APSWM derives the probability distribution of peak flows directly. In APSWM, the generation of runoff over a catchment is based on two assumptions: (1) individual runoff hydrographs are approximately triangular in shape; and (2) the time base of individual runoff hydrograph is the sum of the rainfall duration and the time of concentration of the subcatchment. Thus, peak discharge rate Q_p of a runoff event can be estimated as:

$$Q_p = \frac{v_r}{t + t_c} \tag{3.17}$$

Based on the above relationship and derived probability distribution theory, the exceedance probability of peak flow for type I subcatchments (i.e., subcatchment where $f_c < S_{dd} / t_c$) and type II subcatchments (i.e., subcatchment where $f_c \geq S_{dd} / t_c$) are presented in Eq. (3.18) and Eq. (3.19) respectively. S_{dd} is the difference between initial losses in pervious area and depression storage in impervious area.

$$P[Q_p > q_p] = \begin{cases} \dfrac{2h\lambda}{2h\lambda + \zeta q_p} \exp(-\zeta S_{di} - \dfrac{\zeta t_c}{2h} q_p) & , \quad q < 2f_c h \\ \dfrac{2\lambda\zeta(1-h)(q_p - 2f_c h)}{(2h\lambda + \zeta q_p)(2\lambda + \zeta q_p + 2f_c\zeta - 2hf_c\zeta)} \exp\left[-\dfrac{(\zeta S_{il} - \lambda t_c - f_c\zeta t_c)q_p - 2f_c\zeta h S_{di} + 2\lambda h S_{dd}}{q_p - 2f_c h} \right] & \\ + \dfrac{2\lambda}{2h\lambda + \zeta q_p} \exp(-\zeta S_{di} - \dfrac{\zeta t_c}{2h} q_p) & , \quad 2f_c h < q_p < \dfrac{2hS_{dd}}{t_c} \\ \dfrac{\lambda}{\lambda + \zeta(\dfrac{q_p}{2} + f_c - f_c h)} \exp(-\zeta S_d - \dfrac{\zeta t_c}{2} q_p) & , \quad q_p \geq \dfrac{2hS_{dd}}{t_c} \end{cases} \tag{3.18}$$

$$P[Q_p > q_p] = \begin{cases} \dfrac{2h\lambda}{2h\lambda + \zeta q_p} \exp(-\zeta S_{di} - \dfrac{\zeta t_c}{2h} q_p) & , \quad q < \dfrac{2hS_{dd}}{t_c} \\ \dfrac{2\lambda\zeta(1-h)(2f_c h - q_p)}{(2h\lambda + \zeta q_p)(2\lambda + \zeta q_p + 2f_c\zeta - 2hf_c\zeta)} \exp\left[-\dfrac{(\zeta S_{il} - \lambda t_c - f_c\zeta t_c)q_p - 2f_c\zeta h S_{di} + 2\lambda h S_{dd}}{q_p - 2f_c h} \right] & \\ + \dfrac{2\lambda}{2\lambda + \zeta(q_p + 2f_c - 2f_c h)} \exp(-\zeta S_d - \dfrac{\zeta t_c}{2} q_p) & , \quad \dfrac{2hS_{dd}}{t_c} < q_p < 2f_c h \\ \dfrac{\lambda}{\lambda + \zeta(\dfrac{q_p}{2} + f_c - f_c h)} \exp(-\zeta S_d - \dfrac{\zeta t_c}{2} q_p) & , \quad q_p \geq 2f_c h \end{cases} \tag{3.19}$$

3.2.3 Conversion of Time to Peak in Nash IUH to Time of Concentration for APSWM Use

Two input parameters of Nash IUH are required in OTTHYMO, i.e., the number of reservoirs (n) and time to peak (t_p). In the study case of GRHS, these two parameters have been calibrated. APSWM requires the input of time of concentration. Thus, conversion of t_p and n in Nash IUH to time of concentration for APSWM is needed.

Design storm-based models always divide a watershed into a large number of subcatchments and the hydrograph from one subcatchment is routed downstream or superimposed with others joining at the same point. On the contrary, APSWM treats all the area upstream of a location of interest as one lumped catchment. Triangular hydrograph assumption used for individual subcatchment rainfall-runoff routing is extended to the lumped catchment. Hydrologically equivalent t_c of the lumped catchment is computed by APSWM internally. Recalling that in the Nash model, a watershed is conceptualized as a series of n identical linear reservoirs. Thus, Nash conceptual model can be considered as a lumped catchment with a series of n identical detention ponds in APSWM, as illustrated in Fig. 3.3. Guo (2006) described the derivation about pond flood routing in APSWM. The flood peak reduction effect of a detention pond may be accounted for by increasing the time of concentration of the upstream subcatchment. The routing of hydrographs through an upstream detention pond for each runoff event is depicted in Fig. 3.4. The relationship of maximum storage volume utilized S_{Q_q} and maximum outflow Q_q is as follows:

$$S_{Q_q} = \frac{1}{2} Q_q t_e \tag{3.20}$$

Where t_e is the time required for the pond to empty itself after one runoff event.

It is recognized that the behavior of the pond is similar to the reservoir in the Nash conceptual model, where the storage volume is defined as in Eq. (3.11):

$$S_n(t) = K_n Q(t)$$

Combining the above two equations, we have

$$K_n = \frac{1}{2} t_e \tag{3.21}$$

If n reservoirs are used in a Nash model, correspondingly n ponds should be lumped in APSWM. Then, the above equation may be revised to

$$nK_n = \frac{n}{2} t_e \tag{3.22}$$

Recalling that the relationship between time lag (t_l) and storage coefficient of the reservoir (K_n) in the Nash model is described as $t_l = nK_n$. Combining this with Eq. (3.22), we have

$$2t_l = nt_e \tag{3.23}$$

Time of concentration of a lumped catchment will be the sum of t_c of all the individual components (such as ponds or subcatchments) along the longest pathway. For the extreme scenario in which t_c of inflow hydrograph in Fig. 3.4 is 0, t_c of a catchment comprised of n ponds is equal to $n \times t_e$. The relationship of the time lag in Nash IUH and the time of concentration in APSWM can therefore be obtained as

$$t_c = nt_e = 2t_l = 2\frac{n}{n-1} t_p \tag{3.24}$$

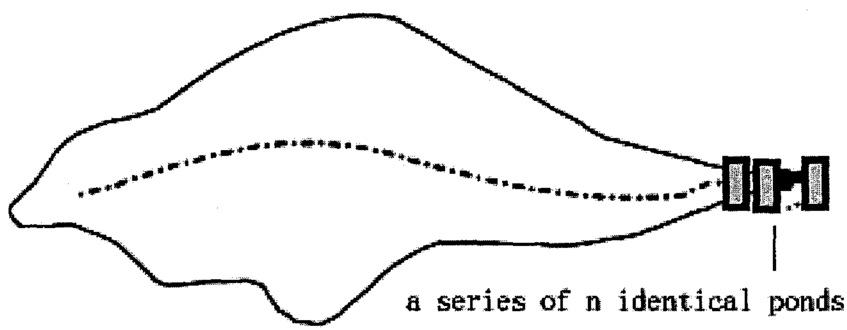


Fig. 3.3 A catchment followed by a series of *n* identical ponds

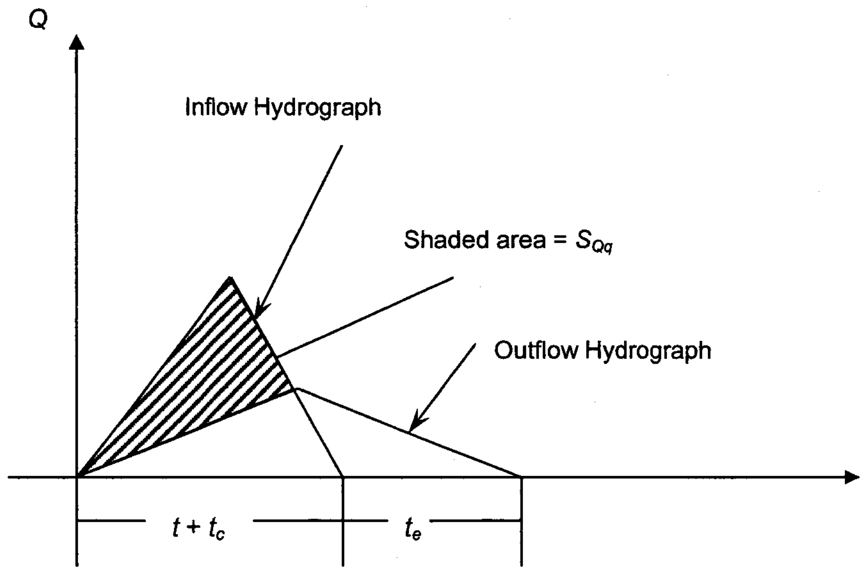


Fig. 3.4 Routing of runoff hydrograph through detention ponds (after Guo, 2006)

3.3 Summary

In this chapter, the methodologies for the estimation of effective rainfall and peak flow rate used in OTTHYMO and APSWM are briefly reviewed. This review facilitates the conversion of parameter values from OTTHYMO to ASPWM.

In the OTTHYMO model, the modified curve number loss model is used in the study case, while APSWM loss model is derived from Horton's infiltration equation. To determine the parameter values for APSWM use, *CN** is converted to *CN* based on the linear relationship as expressed in Eq. (3.3); the ultimate infiltration capacity (f_c) is then estimated from *CN* values according to Table 3.2, and at the same time the S_{il} value is assumed to be equal to the initial abstraction I_a in the CN loss model.

Nash IUH is used to transform effective hyetograph to runoff hydrograph in the OTTHYMO model, in which the time to peak and the number of reservoirs are two input parameters. The relationship between time lag and these two parameters is $t_p = \frac{n-1}{n} t_l$. In APSWM, generation of the frequency distribution of peak flow is based on the estimation of time of concentration and the assumption of triangular hydrograph. It was found that the t_c required in APSWM is two times of t_l in Nash IUH.

Chapter 4 Reach Routing

Two hydrologic routing methods are reviewed in this Chapter. One is the Muskingum routing method, since APSWM makes use of the parameters defined in Muskingum routing method to quantify the peak attenuation effect of a channel reach. The other is the variable storage coefficient routing method which was applied in the OTTHYMO model for Ganaraska River. The determination of equivalent parameter values for APSWM use is described in this Chapter as well.

4.1 Reach Routing Methods Used in Design Storm-Based Models

Channel reach routing in a design storm-based model determines the entire outflow hydrograph from the known inflow hydrograph and channel reach characteristics. The calculation processes are time step-by-time step. Routing methods can be classified as hydrologic routing and hydraulic routing methods. Hydrologic routing employs the continuity equation and storage-discharge relationship of the channel reach, while hydraulic routing uses the continuity equation and the momentum equation. Two hydrologic routing methods are reviewed as follows.

4.1.1 Muskingum Routing Method

The Muskingum routing model uses the continuity equation in the following finite difference form,

$$\frac{I_{t-1}+I_t}{2}-\frac{O_{t-1}+O_t}{2}=\frac{S_{t-1}-S_t}{\Delta t} \tag{4.1}$$

In which, Δt is the calculation time interval; I_{t-1} and I_t are the inflow rates at the beginning and the end of a routing interval respectively; O_{t-1} and O_t are the outflow rates at the beginning and the end of a routing interval respectively; S_{t-1} and S_t are the storage volumes at the beginning and the end of a routing interval respectively.

The storage in a reach is modeled as the sum of prism storage and wedge storage.

As shown in Fig. 4.1, prism storage is the volume defined by a steady-flow water surface profile, while wedge storage is the additional volume under the profile of the flood wave. When a flood wave advances into a reach segment, inflow exceeds outflow and a wedge of storage is produced. As the flood wave recedes, outflow exceeds inflow in the reach segment and a negative wedge is produced. Thus, the Muskingum model defines the storage-discharge relationship as:

$$S_t = K\ \left[XI + (1-X)O\right] \tag{4.2}$$

Where K is the storage time constant for the reach; X is a weighting factor that varies from 0 to 1.

If the storage in the channel is controlled by downstream conditions, then X=0.0, that means outflow are highly correlated with storage. If X=0.5, equal weight is given to inflow and outflow, it results in pure translation without attenuation.

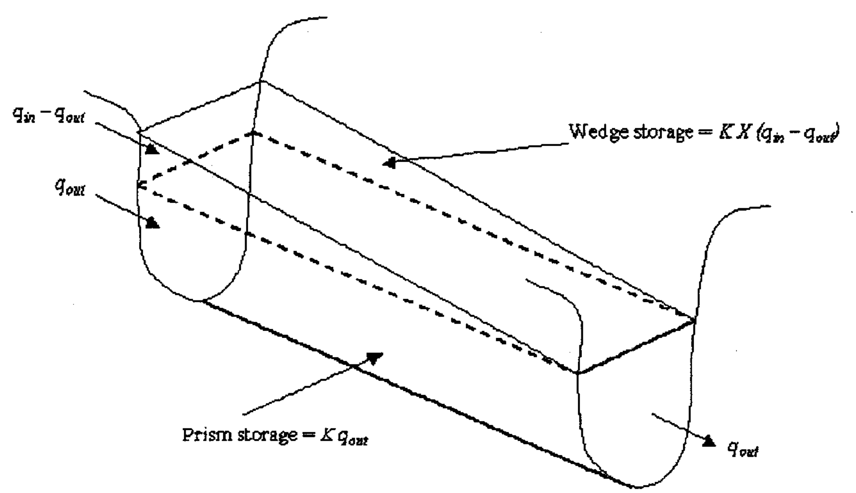


Fig. 4.1 Wedge storage and prism storage of Muskingum routing (after Chow, 1988)

Muskingum routing equation is derived by combining Eq. (4.1) and (4.2) and solved for O_t:

$$O_t = \left[\frac{\Delta t - 2KX)}{2K(1-X)+\Delta t}\right] I_t + \left[\frac{\Delta t + 2KX)}{2K(1-X)+\Delta t}\right] I_{t-1} + \left[\frac{2K(1-X)-\Delta t}{2K(1-X)+\Delta t}\right] O_{t-1} \tag{4.3}$$

4.1.2 Variable Storage Coefficient Routing Method

The variable storage coefficient routing method (VSC) was developed by Williams (1969) and used in the HYMO (Williams and Hann, 1972). It takes into account the change in travel time with the change in the stage of flow during its calculation time step. Travel time (T_{tra}) is computed by dividing the volume of water in the channel by the flow rate based on the assumption of equal inflow and outflow rate. Consider an extremely short reach with small travel time and with approximately equal inflow and outflow rates, travel time as determined by the rating curves of the reach for a particular discharge rate can be written as (William, 1969):

$$T_{tra} = \frac{S_t}{O_t} = \frac{S_{t-1}}{O_{t-1}} \tag{4.4}$$

Compared with the storage-discharge relationship used in the Muskingum method, Eq. (4.4) represents the no wedge situation in the Muskingum method (i.e., X=0).

Substituting Eq. (4.4) into Eq. (4.1). Then,

$$\frac{I_{t-1}+I_{t+1}}{2} + \frac{S_{t-1}}{\left(\frac{\Delta t}{T_{tra}}\right)\left(\frac{S_{t-1}}{O_{t-1}}\right)} - \frac{O_{t-1}}{2} = \frac{S_t}{\left(\frac{\Delta t}{T_{tra}}\right)\left(\frac{S_t}{O_t}\right)} + \frac{O_t}{2} \tag{4.5}$$

Which can be simplified as

$$\begin{aligned} O_t &= \left(\frac{2\Delta t}{2T_{tra}+\Delta t}\right)\left(\frac{I_{t-1}+I_{t+1}}{2}\right) + \left(1 - \frac{2\Delta t}{2T_{tra}+\Delta t}\right)O_{t-1} \\ &= \left[\frac{\Delta t}{2T_{tra}+\Delta t}\right]I_t + \left[\frac{\Delta t}{2T_{tra}+\Delta t}\right]I_{t-1} + \left[\frac{2T_{tra}-\Delta t}{2T_{tra}+\Delta t}\right]O_{t-1} \end{aligned} \tag{4.6}$$

Where $\frac{2\Delta t}{2T_{tra}+\Delta t}$ is the storage coefficient. This equation is similar to Eq. (4.3), however, the storage coefficient varies with T_{tra} and varies from time step to time step. For natural channel reaches, since travel time varies with stage, the use of a constant travel time in flood-routing computations introduces error. The variable storage coefficient method as briefly described may improve the accuracy of routing calculations.

4.2 APSWM Reach Routing Method

Channel routing in APSWM is not the traditional hydrograph routing but the generation of the frequency distribution of peak discharges at the downstream end of a channel reach from the knowledge of the frequency distribution of peak discharges at the upstream end and the characteristics of the channel reach. Thus, APSWM only considers the attenuation effect, because hydrograph translation does not affect the frequency distribution of the peak outflows from a reach (Guo, 2006).

A channel reach affects inflow peakflow in a similar way as catchment transforming rainfall to runoff. When a channel reach is downstream of a catchment, the combined effect of the catchment and the channel reach to input rainfall hyetographs can be lumped together and represented by adding the wave travel time through the channel reach to the time of concentration of the catchment.

Based on the assumption of triangular hydrographs, at the downstream end of a channel reach, the functional expression of the peak flow can be obtained by revising Eq. (3.17):

$$Q_p = \frac{v_r}{t + t_c + K} \tag{4.7}$$

Where K is the wave travel time through a channel reach which equals the Muskingum K value of the channel reach.

Then, the probability distribution of the peak outflow from the reach can be expressed as follows.

For subcatchment and channel reach combinations with $f_c < \dfrac{S_{dd}}{t_c + K}$,

$$P[Q_p > q_p] = \begin{cases} \dfrac{2h\lambda}{2h\lambda + \zeta q_p} \exp\left[-\zeta S_{di} - \dfrac{\zeta(t_c+K)}{2h} q_p\right] & , \quad q < 2f_c h \\ \dfrac{2\lambda\zeta(1-h)(q_p - 2f_c h)}{(2h\lambda + \zeta q_p)(2\lambda + \zeta q_p + 2f_c\zeta - 2hf_c\zeta)} \exp\left\{-\dfrac{[\zeta S_{il} - \lambda(t_c+K) - f_c\zeta(t_c+K)]q_p - 2f_c\zeta h S_{di} + 2\lambda h S_{dd}}{q_p - 2f_c h}\right\} & \\ + \dfrac{2\lambda}{2h\lambda + \zeta q_p} \exp\left[-\zeta S_d - \dfrac{\zeta(t_c+K)}{2h} q_p\right] & , \quad 2f_c h \le q_p < \dfrac{2hS_{dd}}{t_c+K} \\ \dfrac{\lambda}{\lambda + \zeta(\frac{q_p}{2} + f_c - f_c h)} \exp\left[-\zeta S_d - \dfrac{\zeta(t_c+K)}{2h} q_p\right] & , \quad q_p \ge \dfrac{2hS_{dd}}{t_c+K} \end{cases}$$

(4.8)

For subcatchment and channel reach combinations with $f_c \geq \dfrac{S_{dd}}{t_c + K}$,

$$P[Q_p > q_p] = \begin{cases} \dfrac{2h\lambda}{2h\lambda + \zeta q_p} \exp\left[-\zeta S_{di} - \dfrac{\zeta(t_c+K)}{2h} q_p\right] & , \quad q < \dfrac{2hS_{dd}}{t_c} \\ \dfrac{2\lambda\zeta(1-h)(2f_c h - q_p)}{(2h\lambda + \zeta q_p)(2\lambda + \zeta q_p + 2f_c\zeta - 2hf_c\zeta)} \exp\left[-\dfrac{[\zeta S_{il} - \lambda(t_c+K) - f_c\zeta(t_c+K)]q_p - 2f_c\zeta h S_{di} + 2\lambda h S_{dd}}{q_p - 2f_c h}\right] & \\ + \dfrac{2\lambda}{2\lambda + \zeta(q_p + 2f_c - 2f_c h)} \exp\left[-\zeta S_d - \dfrac{\zeta(t_c+K)}{2} q_p\right] & , \quad \dfrac{2hS_{dd}}{t_c+K} \leq q_p < 2f_c h \\ \dfrac{\lambda}{\lambda + \zeta(\frac{q_p}{2} + f_c - f_c h)} \exp\left[-\zeta S_d - \dfrac{\zeta(t_c+K)}{2} q_p\right] & , \quad q_p \geq 2f_c h \end{cases} \tag{4.9}$$

4.3 Determination of Equivalent Routing Parameter Values in APSWM

In the case study, the calibrated OTTHYMO model of Ganaraska river employed the variable storage coefficient (VSC) method for reach routing, whereas APSWM requires the input of K values to generate the frequency distributions of outflows. As pervious discussed, the VSC method represents the situation of Muskingum routing when X=0. The value of K value is equal to the wave travel time through the reach. For the estimation of K, generally two methods are available.

(1) If the inflow and outflow hydrographs are available, K can be estimated as the elapsed time between the inflow hydrograph peak and the outflow hydrograph peak.

(2) For unknown outflow hydrographs, K can be estimated from channel characteristics.

Since the inflow and outflow hydrographs can be obtained for each reach by running the calibrated OTTHYMO, the first method will be used in the following case study to ensure equivalence between the APSWM model and the OTTHYMO model. However, K might be varied with flow rates and this variation is considered in the VSC method. In APSWM, a constant K is assumed and used for each reach, which is similar to the Muskingum method. In the following studies, the wave travel time obtained by analyzing the peak flows from the 10-year return period storm is used in APSWM.

4.4 Summary

APSWM considers the effect of reach routing by adding the wave travel time through the reach to the time of concentration of the catchment. The wave travel time is equal to the K value as used in the Muskingum routing method. In the OTTHYMO model of GRHS, variable storage coefficient routing method is employed. The storage-discharge relationship defined in VSC method is the same as the wedgeless situation in Muskingum routing. Because the inflow and outflow hydrographs are available after running OTTHYMO model, the wave travel time K can be estimated from the time interval between inflow and outflow peaks resulting from a 10- year return period storm. The effect of keeping K constant in APSWM may be revealed in the following comparison studies.

Chapter 5 Catchment Aggregation Strategy

5.1 Catchment Aggregation Strategy of OTTHYMO

In traditional rainfall-runoff modeling, the modeled catchment is usually divided into a number of sub-catchments according to the drainage system and the physical characteristics such as soil type or land use. Subcatchments might be connected by pipes or channel reaches, or they may drain into the same detention pond. The hydrograph from one sub-catchment is routed through reaches or ponds and subsequently superimpose with other hydrographs. Starting the calculation from the most upstream end and gradually moving downstream, the hydrograph at the point of interest is obtained.

5.2 Catchment Aggregation Strategy of APSWM

APSWM treats all the area upstream of a location of interest as one lumped catchment. The characteristics for this lumped catchment except time of concentration (t_c), such as S_{di}, S_{il}, f_c, h, S_{dp}, S_{il}, S_{iw}, S_{dd}, and S_d, are obtained by using an area-weighting technique. That is

$$M = \frac{\sum_i^n A_i M_i}{\sum_i^n A_i} \tag{5.1}$$

Where M represents the value of a parameter for the lumped catchment; M_i is the value of the same parameter of subcatchment i; and A_i is the area of subcatchment i; n is the total number of subcatchments in the lumped catchment.

For an aggregated catchment, if the components are in series, APSWM determines the hydrologically equivalent t_c as the sum of t_c's of the individual components (travel times through channel reaches or detention ponds). If the components are parallel to each other, APSWM determines the equivalent t_c as a weighted average of the individual component's t_c or travel times. The weight for an individual component is

$$w_i = \frac{E(V_{Ri})}{\bar{t} + t_{ci}} \tag{5.2}$$

Where $E(V_{Ri})$ is the expected value of runoff per rainfall event from the component i;

t_{ci} is the equivalent t_c of the component i; $\bar{t}$ is the average duration of rainfall events.

It is recognized that the APSWM lumping strategy might cause a loss of accuracy. For small drainage areas typical of stormwater management studies, this loss of accuracy may be insignificant (Guo, 2006). However, for larger watersheds, this loss of accuracy may be significant. This comparison study is conducted to assist in quantifying APSWM's loss of accuracy when catchment areas become larger and larger.

Chapter 6 Case Study and Analyses of Results

6.1 Description of Study Area and Available Data

The Ganaraska River watershed has a triangularly shaped basin with a total area of 277.9 km^2. The main stream originates in the Oak Ridges Moraine area at an elevation of 305 m above sea level. The river flows south-easterly for 42 km to Lake Ontario at Port Hope. The watershed is dominated by a mixture of woodland and agricultural land, and has not undergone significant changes in land use. The hydrologic soil group map provided by Ganaraska Region Conservation Authority (GRCA) indicated that most of the watershed is covered by sand or sandy loam which is classified as hydrologic soil groups (HSG) A, AB and B. One stream gage station (02HD012) is available which provides 32 years of observation of annual maximum instantaneous flows. Two Environment Canada weather stations within or adjacent to the watershed are available. They are located at Peterborough and Cobourg respectively. However, only the IDF curve at Peterborough is generated and updated by AES. The event-based rainfall statistics of Peterborough STP is available from Adams and Papa (2000).

The discretization and characterization of subcatchments were conducted by GRCA using ArcHydro GIS software. The watershed was discretized into 47 subcatchments based on stream orders and with some modifications to combine small subcatchments (See Fig. A-1 in Appendix A). The area of subcatchments ranges from 121 to 2609 ha. Weighted *CN* value for each subcatchment was evaluated by ArcHydro based on the land use, drainage area, and hydrologic soil group.

The OTTHYMO model for the watershed was developed and calibrated by Schaeffer & Associates Limited (2005). In this Ganaraska river OTTHYMO model, modified CN method was applied to calculate rainfall losses, and Nash instantaneous unit hydrograph (IUH) was used to transform effective hyetographs to hydrographs. Because this OTTHYMO model is developed for the purpose of floodline delineation, three historical events selected for calibration purposes are all heavy rainfall events. Three parameters, time to peak (t_p) and the number of reservoirs (n) in Nash IUH and CN^* in the loss model were determined through calibration. After calibration, n is set to be 1.4; t_p ranges from 0.6 hours to 4.64 hours; the CN^* ranges from 9 to 100, and the corresponding weighted CN ranges from 38 to 100. (See Appendix B: Input parameters of OTTHYMO and APSWM for GRHS)

6.2 Comparison Methodology

To achieve the objectives described in Chapter 1, a series of comparisons were made using APSWM and OTTHYMO models of the Ganaraska River watershed. Most runs were conducted using Toronto Pearson International Airport meteorological data. Because 33 years of continuous rainfall records are available at Toronto Pearson International Airport, the rainfall statistics of various IETDs and TRVs can be generated.

The first set of runs is designed to investigate the impact of IETDs and TRVs, and then to determine the appropriate rainfall inputs to APSWM. In this set, both OTTHYMO models and APSWM models were developed for simulating peak flow rates from 100% impervious areas (or CN=100) to avoid any additional discrepancies that may be caused by conversion between loss models.

The second set of runs is designed to investigate the performance of APSWM for individual subcatchments and to test the validity of the conversion method proposed in Chapter 3. Four typical subcatchments in the Ganaraska river watershed with different curve numbers were selected for comparison purposes.

The third set of runs is designed to quantify the discrepancy caused by APSWM's channel routing calculations. The results from three reaches with various wave travel times were reported.

The fourth set of comparisons is designed to examine the catchment aggregation strategy of APSWM by running the two models for the entire watershed. The peak flows of different return periods at four locations were compared. Finally, the two models were run using Peterborough meteorological data in order to compare with the results from frequency analyses using observed streamflow data.

6.3 Set #1 Comparison - Precipitation Input

Subcatchment #3 with CN=100 in Ganaraska river watershed is selected. It is denoted as Subcatchment A. Two more hypothetical impervious catchments with t_c's that are half and two times of t_c of Subatchment A are constructed as well. Input parameter values for these three subcatchments are presented in Table 6.1.

Table 6. 1 Basic Information of Three Individual Subcatchments in Set #1 Comparison Study

		Subcatchment A	Subcatchment A-1	Subcatchment A-2
Area (ha)		198.89	50	800
OTTHYMO	*CN*	100	100	100
	I_a (mm)	1.5	1.5	1.5
	Time to peak (hr) in Nash IUH	1.28	0.64	2.56
	Number of reservoirs in Nash IUH	1.4	1.4	1.4
APSWM	Impervious ratio (%)	100	100	100
	Impervious area depression (mm)	1.5	1.5	1.5
	Time of concentration(hr)	9.92	4.96	17.84

In the GRHS OTTHYMO model, two types of design storms - Chicago design storm and AES design storm were selected for comparisons. As the time of concentrations of above described subcatchments varies from 4.96 to 17.92 hours, 6-hour duration Chicago design storms are applied for the smallest catchment and the 24-hour duration Chicago design storms are applied for the largest one. Since AES design storms only have two durations, for this set of comparisons, the 12 hour duration and 30% distribution are appropriate. All the design storms were generated from the IDF Curves of Toronto's Pearson International Airport which are determined based on the 39- year historical rainfall records from 1950 to 1990.

In APSWM, a set of runs using rainfall inputs with IETDs of 6, 12 and 24 hours and TRVs of 0 mm and 1 mm are conducted for the three catchments. The input rainfall statistics corresponding to the respective IETD and TRV values are listed in Table 2.3.

The comparison results for this set are presented in Table 6.2. The corresponding comparison charts are presented in Fig. 6.1(a) through Fig. 6.1(c).

Table 6. 2 Comparison Results in Impervious Catchments

Discharges (cms)	OTTHYMO			APSWM				
	Chicago-24hr design storm	Chicago-6 hr design storm	AES-12 hr design storm	IETD= 24 hr TRV=0 mm	IETD=12 hr		IETD=6 hr	
					TRV= 1 mm	TRV= 0 mm	TRV= 1 mm	TRV= 0 mm
Subcatchment A t_c= 9.96 hr								
2 yr	2.78	-	3.19	3.84	3.71	3.21	3.53	2.70
5 yr	3.91	-	3.94	4.88	4.63	3.95	4.37	3.26
10 yr	4.19	-	4.52	5.70	5.36	4.53	5.02	3.70
25 yr	5.61	-	5.29	6.81	6.33	5.30	5.89	4.29
50 yr	6.27	-	5.89	7.67	7.08	5.90	6.56	4.74
100 yr	6.96	-	6.50	8.54	7.84	6.50	7.24	5.19
SubcatchmentA-1 t_c =4.98 hr								
2 yr	-	1.24	0.89	1.68	1.53	1.46	1.50	1.37
5 yr	-	1.69	1.24	2.20	1.96	1.83	1.88	1.68
10 yr	-	1.78	1.33	2.61	2.29	2.13	2.18	1.90
25 yr	-	2.39	1.79	3.18	2.74	2.52	2.60	2.27
50 yr	-	2.67	2.02	3.62	3.09	2.83	2.89	2.52
100 yr	-	2.90	2.24	4.07	3.44	3.14	3.21	2.78
Subcatchment A-2 t_c =17.92 hr								
2 yr	6.87	-	6.63	9.43	8.91	7.61	8.40	6.86
5 yr	9.59	-	9.36	11.73	10.95	9.24	10.28	8.24
10 yr	10.40	-	9.98	13.52	12.53	10.50	11.20	9.30
25 yr	13.77	-	13.48	15.94	14.65	12.18	13.59	10.70
50 yr	15.40	-	15.18	17.79	16.27	13.47	15.04	11.78
100 yr	17.13	-	16.87	19.67	17.91	14.76	16.50	12.86

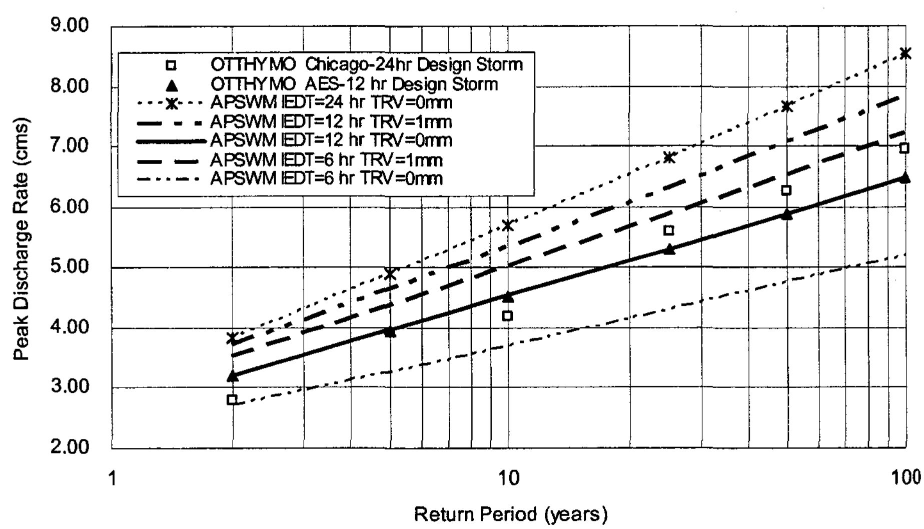


Fig. 6.1 (a) Peak discharge rates vs. return periods of Subcatchment A

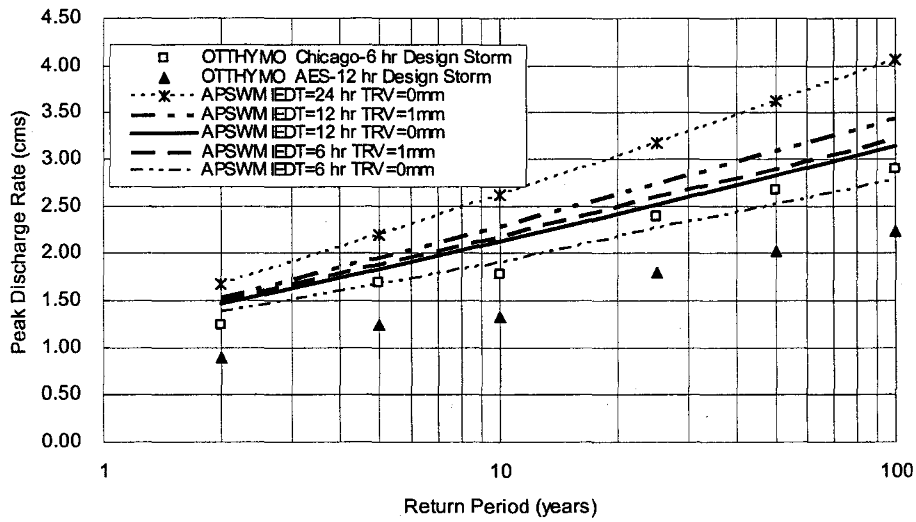


Fig. 6.1 (b) Peak discharge rates vs. return periods of Subcatchment A-1

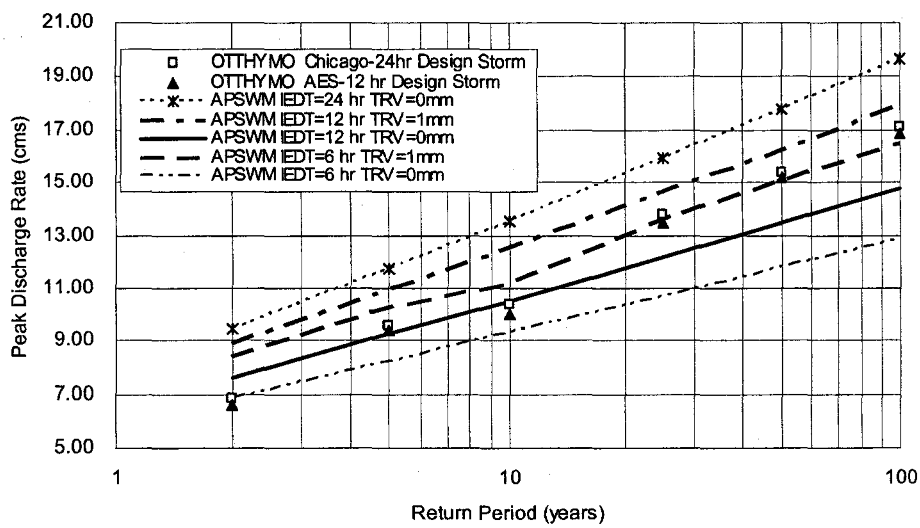


Fig. 6.1 (c) Peak discharge rates vs. return periods of Subcatchment A-2

Both the mean of rainfall event volume($\bar{v}$) and the mean of rainfall event duration ($\bar{t}$) increase with increasing IETD. According to Eq. (3.17), the increase of $\bar{v}$ leads to the increase of peakflow rates, whereas the increase of $\bar{t}$ leads to the decrease of peakflow rates. For this specific case, as illustrated by Fig. 6.1(a) through Fig. 6.1(c), peak flow rates increase as IETD increases after combining the effects from $\bar{v}$ and $\bar{t}$.

The effect from different TRVs is obvious. The peak flow rates generated from the precipitation statistics with rainfall events of volume < 1mm removed are 10% to 30% larger than those from the precipitation statistics with all events included.

Fig. 6.1(a) indicates that the peak discharge rates from APSWM with IETD=12hr and TRV=0 mm (drawn as a solid line) are very close to those from OTTHYMO in Subcatchment A with t_c = 9.96 hr. When t_c is reduced to 4.98hr in Subcatchment A-1 and increased to 17.97 hr in Subcatchment A-2, results simulated by APSWM and OTTHYMO are still fairly close to each other when the rainfall statistics of IETD=12 hr and TRV=0 mm are used.

As time of concentrations of most subcatchments of the Ganraska River watershed fall into the range of 4 hours to 18 hours, the rainfall input from IETD of 12 hours and TRV of 0 mm is appropriate for this case.

6.4 Set #2 Comparison – Rainfall-Runoff Transformation of Individual Subcatchments

Together with the impervious subcatchments introduced in the previous section, four typical subcatchments with different *CN*s are selected from the Ganaraska river watershed for this set of comparisons. Following the conversion methods described in Chapter 3, ultimate infiltration capacity (f_c) and initial losses (S_{il}) are converted from the modified cover number (CN*) and initial abstraction(I_a*), and time of concentration (t_c) is converted from time to peak (t_p) and the number of reservoirs (n) in Nash IUH. This conversion ensures that the subcatchment as modeled by OTTHYMO is hydrologically equivalent to the one as modeled by APSWM. The physical characteristics of the four test subcatchments and their input parameters to OTTHYMO and APSWM are summarized in Table 6.3.

Table 6. 3 Physical Characteristics of Four Individual Subcatchments and Their Input Parameters to OTTHYMO and APSWM

		Subcatchment A	Subcatchment B	Subcatchment C	Subcatchment D
Catchment ID in GRHS		#3	#47	#40	#22
Area (ha)		198.89	605.5	993.7	771.7
Slope(%)		1.23	1.8	1.74	1.65
Flow Length (m)		3511	6392	7331	8603
OTTHYMO	*CN**	100	48	64	90
	Ia in *CN** method (mm)	1.5	5	5	5
	CN	100	59	70	88
	Ia in *CN* method (mm)	1.5	15.98	8.69	2.71
	t_p (hr) in Nash IUH	1.28	1.74	2.14	2.12
	n in Nash IUH	1.4			
APSWM	Impervious ratios (%)	100	0	0	0
	S_{il} (mm)	1.5	15.98	8.69	2.71
	t_c (hr)	9.92	12.18	14.98	14.84
	f_c (mm/hr)	-	70.2	58	5.2

Two design storms - 24 hour Chicago design storm and 12 hour AES design storm developed from the IDFs of Toronto Pearson International Airport (Table 2.3) are used in OTTHYMO for comparisons. The rainfall statistics generated from all events with IETD=12 hr are used in APSWM.

Both modified CN method and SCS-CN method are run in OTTHYMO to quantify the differences resulting from the conversion from CN^* to CN and I_a^* to I_a. The results are summarized in Table 6.2 and plotted in Fig. 6.2(a) through Fig. 6.2(d).

Table 6. 4 Individual Subcatchment Comparison Results

Discharge Rate(cms)	OTTHYMO with Modified CN method		OTTHYMO with CN method	APSWM
	Chicago-24hr design storm	AES-12 hr design storm	AES-12 hr design storm	IETD=12 hr
Subcatchment A (#3) *CN*=100				
2 yr	2.78	3.19	2.52	3.21
5 yr	3.91	3.94	3.55	3.95
10 yr	4.19	4.52	3.78	4.53
25 yr	5.61	5.29	5.10	5.30
50 yr	6.27	5.89	5.75	5.90
100 yr	6.96	6.50	6.39	6.50
Subcatchment B(#47) *CN*=59				
2 yr	0.80	0.77	0.61	0.00
5 yr	1.60	1.53	1.49	0.00
10 yr	1.89	1.73	1.74	0.51
25 yr	3.22	3.05	3.34	2.63
50 yr	3.97	3.79	4.25	4.22
100 yr	4.82	4.58	5.22	5.82
Subcatchment C(#40) *CN*=70				
2 yr	1.94	1.92	1.93	0.00
5 yr	3.72	3.82	3.86	1.88
10 yr	4.36	4.31	4.36	4.01
25 yr	7.15	7.38	7.48	6.83
50 yr	8.66	9.02	9.16	8.96
100 yr	10.35	10.75	10.91	11.09
Catchment D (#22) *CN*=88				
2 yr	4.18	3.97	3.93	5.81
5 yr	6.96	6.63	6.49	7.89
10 yr	7.85	7.25	7.09	9.48
25 yr	11.48	10.88	10.63	11.58
50 yr	13.29	12.68	12.33	13.17
100 yr	15.21	14.49	14.18	14.76

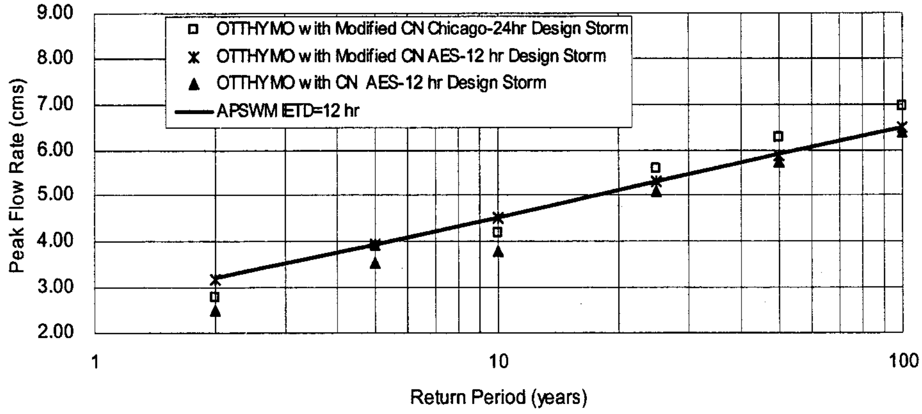


Fig. 6.2 (a) Peak discharge rates vs. return periods of Subcatchment A

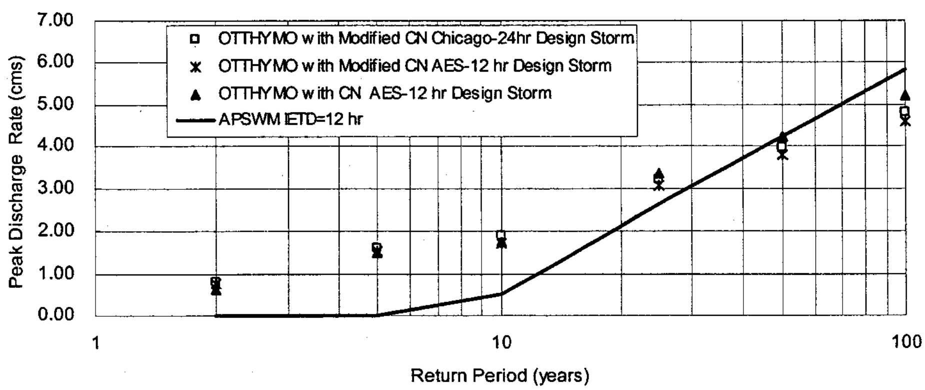


Fig. 6.2 (b) Peak discharge rates vs. return periods of Subcatchment B

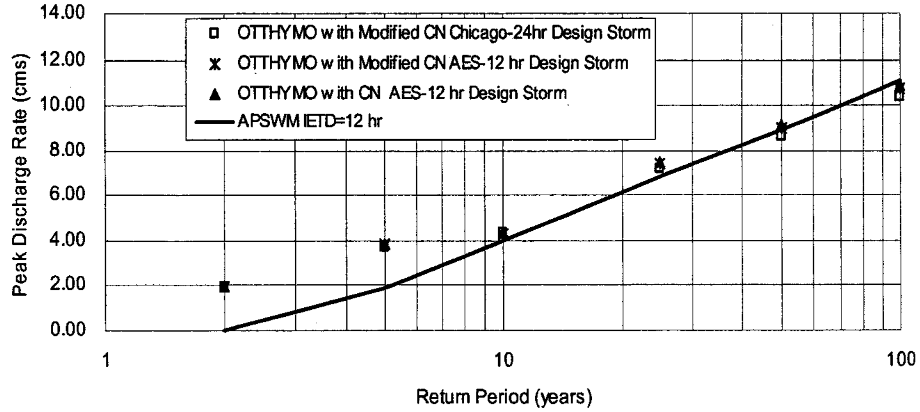


Fig. 6.2 (c) Peak discharge rates vs. return periods of Subcatchment C

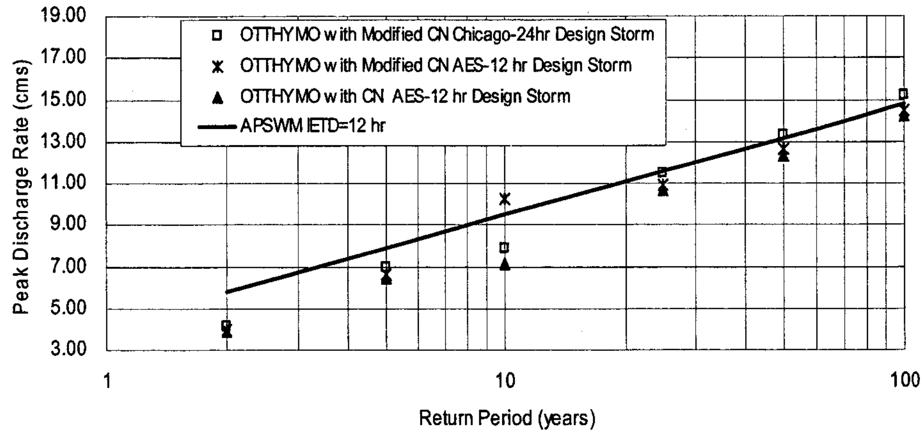


Fig. 6.2 (d) Peak discharge rates vs. return periods of Subcatchment D

Only minor differences are observed between results from using Modified CN method and CN method. This indicates that the process of conversion from CN^* to CN is acceptable. Fig. 6.2(a) and Fig. 6.2 (d) indicate that peak discharge rates of all return periods obtained from the two models for Subcatchment A and D are very close to each other. Both Subcatchment A and D represent low infiltration areas or areas with high imperviousness ratio. The close agreement for these two subcatchments further proves that APSWM might be used as an alternative to design storm models for urban areas.

For Subcatchment B and C, comparable results were obtained for larger return periods (Fig. 6.2(b) and 6.2(c)), while peak discharge rates from APSWM are obviously lower than those from OTTHYMO for lower return periods. Even zero flow rates were obtained for 2 and 5-year return periods. This might have been caused by the use of different loss methods in the two models. The modified curve number loss method in OTTHYMO assumes a fixed I_a at 5 mm, while in APSWM, the sum of depression storage and initial soil wetting infiltration depth (S_{il}), which has to be satisfied before runoff occurs, is quite larger than 5 mm for catchments with low curve numbers. Therefore, it is possible that close to zero peak discharge rates for low return periods may be predicted by APSWM for pervious catchments with high infiltration rates when baseflow is not considered.

It should be pointed out that because the Ganaraska River OTTHYMO model was developed and calibrated for the purpose of flood management, which focuses on the correct estimation of large flood flows. Only heavy storms were selected for calibration. Thus, the Ganaraska River OTTHYMO model may provide relatively more accurate estimations at 100, 50 and 25-year return periods than others. Overall, it can be seen that peak discharge results from APSWM and OTTHYMO are generally comparable and the discrepancies can be explained.

6.5 Set #3 Comparison –Reach Routing

Following the proposed method of determining the K and X for APSWM use in Chapter 4, the APSWM model of the entire watershed of Ganaraska River was developed. To examine the validity of APSWM's reach routing method, three reaches in the Ganaraska River Hydrology Study with wave travel times ranging from 0.75 hour to 3.34 hours are chosen for comparisons. The absolute attenuation and relative attenuation of peak discharges for all return periods are calculated in Table 6.5.

Relative attenuation is calculated as the ratio between the difference of inflow and outflow peaks and the inflow peak flow rate.

Relative attenuation in Reach-312 with K=3.34 hours computed by APSWM is very close to those from OTTHYMO, although peak inflows and outflows from the two models for return periods of 2 through 10-years differ significantly. Both models illustrate the same tendency that shorter reaches result in lower relative attenuation. However, the relative attenuations from the use of variable storage coefficient routing method in OTTHYMO decline faster with the decrease of wave travel time. Larger difference in relative attenuation exists for shorter wave travel times. Further research is needed to improve APSWM's reach routing method. Because the impact from reach routing decreases as the reach becomes shorter, the discrepancies from reach routing may be insignificant when larger catchment are considered.

Table 6. 5 Reach Routing Comparison Results

	APSWM				OTTHYMO-VSC method			
	Inflow peakflow rate(cms)	Outflow peakflow rate(cms)	Attenuation (cms)	Relative attenuation	Inflow peakflow rate(cms)	Outflow peakflow rate(cms)	Attenuation (cms)	Relative attenuation
Reach-312 K=3.34 hr								
2 yr	1.34	0.98	0.36	26.6%	30.83	24.79	6.04	20.0%
5 yr	2.58	1.93	0.65	25.3%	54.11	42.08	12.03	22.2%
10 yr	29.63	20.02	9.61	32.4%	59.54	46.12	13.42	22.0%
25 yr	68.54	46.30	22.24	32.4%	98.04	70.66	27.38	28.0%
50 yr	97.99	66.19	31.80	32.5%	111.39	80.80	30.59	27.9%
100 yr	127.43	86.00	41.43	32.5%	129.77	95.12	34.65	27.1%
Reach-323 K=1.42 hr								
2 yr	0	0	0	0.0%	13.74	11.88	1.86	13.5%
5 yr	0	0	0	0.0%	23.98	21.58	2.40	10.0%
10 yr	6.54	5.21	1.33	20.0%	26.49	23.79	2.70	10.2%
25 yr	31.39	24.96	6.43	20.4%	42.22	38.16	4.06	9.6%
50 yr	50.20	39.91	10.29	20.5%	50.6	45.63	4.97	9.8%
100 yr	69.01	54.86	14.15	20.5%	59.25	53.47	5.78	9.8%
Reach-309 K=0.75 hr								
2 yr	5.38	4.96	0.41	7.7%	3.83	3.71	0.12	3.1%
5 yr	7.50	6.93	0.57	7.6%	6.66	6.47	0.19	2.9%
10 yr	9.11	8.41	0.70	7.6%	7.35	7.14	0.21	2.9%
25 yr	11.24	10.38	0.86	7.7%	11.39	11.13	0.26	2.3%
50 yr	12.85	11.87	0.99	7.7%	13.4	13.15	0.25	1.9%
100 yr	14.47	13.36	1.11	7.7%	15.5	15.2	0.30	1.9%

6.6 Set #4 Comparison –Catchment Aggregation

6.6.1 Comparisons by Using Toronto Precipitation Data

The above comparisons show that, except for low return periods and catchments with highly permeable soils, the peak discharge rates simulated by APSWM are generally comparable with those from OTTHYMO at individual subcatchment and single reach scales. In this Chapter, the subcatchment aggregation strategy of APSWM will be examined by running two models for the entire watershed.

Two sets of design storms - 24 hour Chicago design storm and 12 hour AES design storm developed from the IDFs of Toronto Pearson International Airport are used in OTTHYMO for comparisons. The rainfall statistics generated from all events with 12 hour IETD are used in APSWM.

Results from four locations of interest with various drainage areas are reported for the purpose of examining the aggregation strategy of APSWM. The four locations are illustrated in Fig. A-2 of Appendix A.

(1) Junction 1 (J1) is the point where the tributary of Quays confluences with the Ganaraska River. The drainage area is 2128 ha, which is comprised of three subcatchments and two reaches. The three subcatchments are homogeneous in their soil and land use characteristics and the *CN*s of the three subcatchments are around 80. Considering that close agreement of peak flows for all return periods is obtained for individual subcatchments when *CN* is high, the comparison at this point is performed to test whether the same conclusion can be made between the two models with subcatchment aggregation.

(2) Junction 2 (J2) is the point where the tributary of North Ganaraska confluences with the Ganaraska River. The drainage area is 7048 ha and consists of nine subcatchments and four reaches. The curve numbers of these nine subcatchments vary from 45 to 91. The watershed characteristics of the North Ganaraska tributary are very similar to those of the entire watershed, where the upstream area has high infiltration losses and downstream area has low infiltration losses. Because the entire watershed is so large that the time of concentration can be longer than 12 hours or even 24 hours, the use of the design storm approach and the APSWM model both need to be questioned for such large watersheds. Thus, this junction with smaller drainage area is chosen in order to provide a reasonable watershed size to assess the aggregation strategies.

(3) Junction 3 (J3) is the point where the Dale gauge is located. The drainage area is 23872 ha. This point is chosen for further comparison with results from frequency analysis using observed streamflow records.

(4)Junction 4 (J4) is the outlet of the entire watershed. The drainage area is 27691 ha. This point is included to test the case of an extremely large watershed.

The comparison results are summarized in Table 6.6 and plotted in Fig. 6.3(a) through Fig. 6.3(d). The details about APSWM outputs are provided in Appendix C.

Table 6. 6 Comparison between APSWM and OTTHYMO Results for the Entire Watershed Using Toronto Airport Meteorological Data

Discharge (cms)		Peak discharge for different return periods (years)					
		2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
J1 Outlet of Tributary Quays (Drainage area=2128 ha)							
OTTHYMO	Chicago-24hr design storm	8.86	15.67	18.07	27.84	32.82	38.94
	AES-12 hr design storm	8.32	14.73	16.35	25.97	30.85	35.88
APSWM	IETD=12hr	10.97	16.67	20.98	26.69	31.01	35.34
J2 Outlet of Tributary North Ganaraska (Drainage area=7048.9 ha)							
OTTHYMO	Chicago-24hr design storm	15.85	28.73	32.98	49.67	58.22	67.64
	AES-12 hr design storm	14.81	27.06	29.87	46.71	51.10	64.00
APSWM	IETD=12hr	0.00	20.00	35.00	55.00	71.00	86.00
J3 Dale Gauge Station at main branch (Drainage area=23872ha)							
OTTHYMO	Chicago-24hr design storm	54.68	90.04	98.35	149.20	177.03	206.69
	AES-12 hr design storm	57.10	95.14	107.88	158.95	187.70	220.14
	AES -12 hr with an areal deduction factor of 0.9*	45.08	76.81	84.62	127.96	151.71	176.84
APSWM	IETD=12hr	10.00	42.20	86.40	145.10	189.50	233.90
	IETD=12hr with an areal deduction factor of 0.9*	8.49	24.67	64.86	117.91	158.16	198.34
J4 Outlet at Ontario Lake (Drainage area= 27691 ha)							
OTTHYMO	Chicago-24hr Design Storm	71.69	116.27	126.49	188.69	221.21	255.96
	AES-12 hr Design Storm	73.26	117.08	131.82	196.69	222.33	258.65
	AES -12 hr with an areal deduction factor of 0.9*	60.36	100.51	109.58	163.29	191.74	220.99
APSWM	IETD=12hr	17.66	69.19	126.58	202.48	259.88	317.29
	IETD=12hr with an areal deduction factor of 0.9*	14.80	45.99	97.92	166.61	218.59	270.55

*Note: OTTHYMO and APSWM runs with areal reduction factors are to account for the large drainage areas. Details about simulation runs considering areal reduction factors can be found in the next subsection.

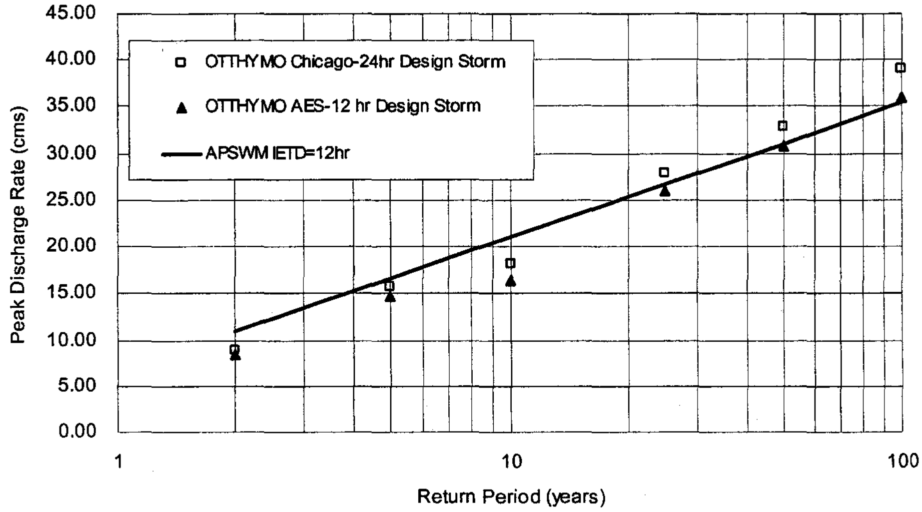


Fig. 6.3 (a) Peak discharge rates vs. return periods of J1

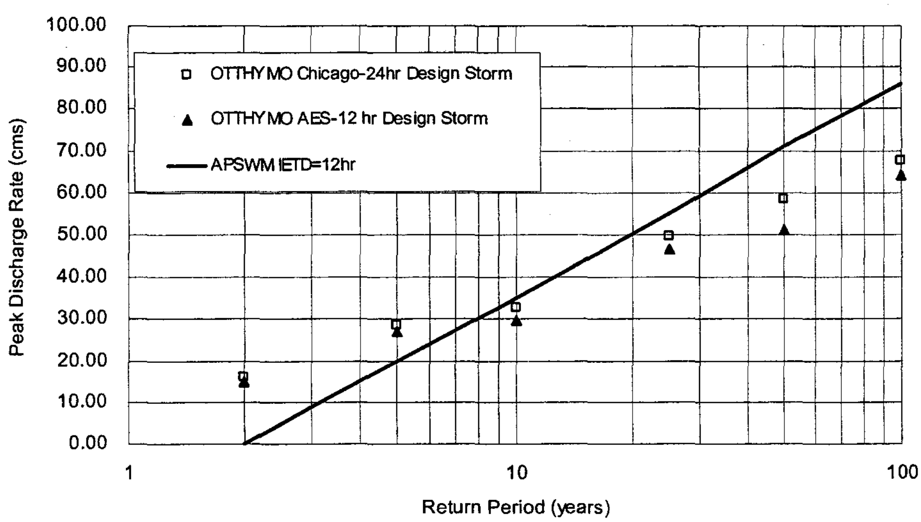


Fig. 6.3 (b) Peak discharge rates vs. return periods of J2

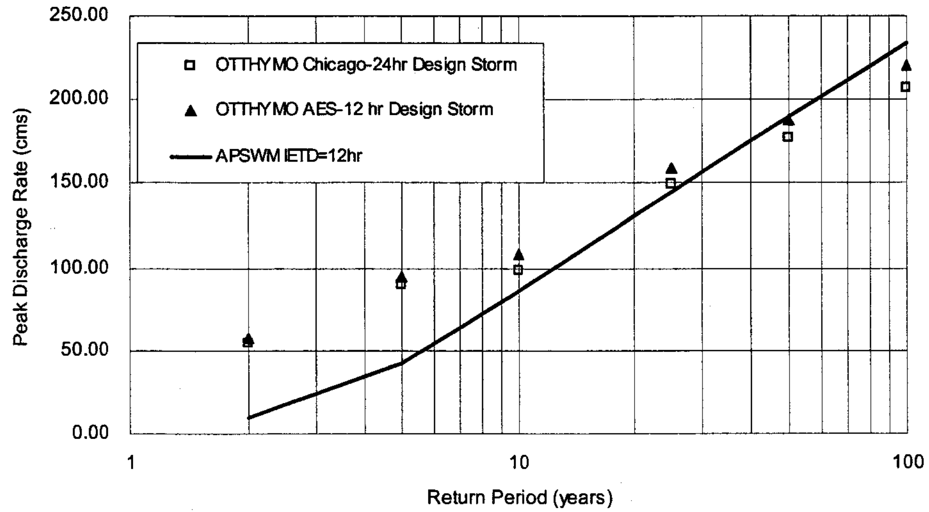


Fig. 6.3 (c) Peak discharge rates vs. return periods of J3

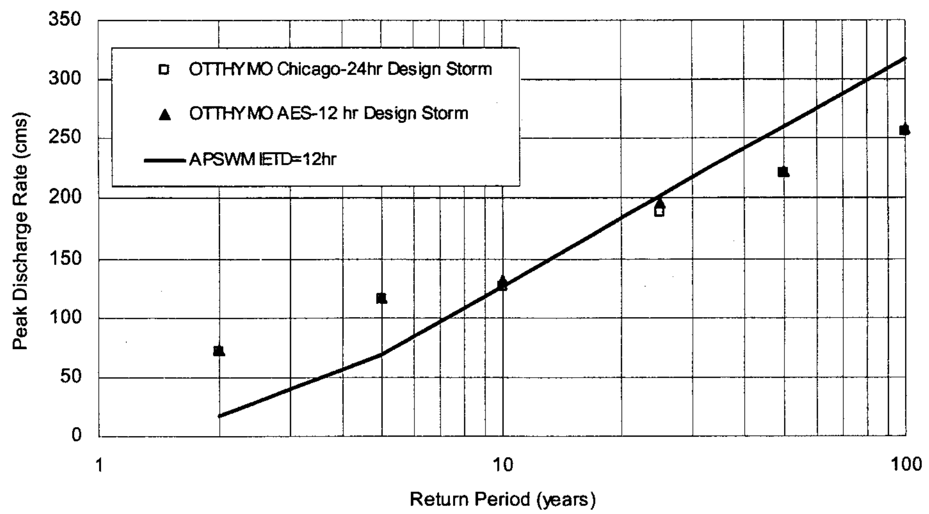


Fig. 6.3 (d) Peak discharge rates vs. return periods of J4

As expected and can be seen in Fig. 6.3(a), at Junction 1, the results from APSWM are quite close to those from OTTHYMO, the same as for individual subcatchments (Fig. 6.2(a) and Fig. 6.2 (d)). The close agreement indicates that the aggregation strategy used in APSWM does not increase discrepancy.

Reviewing the other three sets of comparisons shown in Fig. 6.3(b) through Fig. 6.3(d), peak discharges simulated by APSWM for high return periods are slightly higher and those for low return periods are slightly lower than those simulated by OTTHYMO. These comparison results are very similar to those for Subcatchment B and C in section 6.4. It is therefore concluded that the loss of accuracy due to the aggregation strategy used in APSWM is negligible and the differences resulting from reach routing are insignificant.

6.6.2 Comparison with Results from Streamflow Frequency Analysis

6.6.2.1 Frequency analyses

HYDAT is a compilation of streamflow data maintained by Environment Canada. From HYDAT, 32 years of observed annual maximum instantaneous flows from 1960 to 2001 (See Table 6.7) were obtained for one gauge (Ganrasaka River near Dale gauge station, #02HD012).

The corresponding watershed has not changed in its land use. No channel modifications have occurred either. Thus, the flow records provided by HYDAT can be directly used for frequency analysis.

Frequency analyses of streamflow records were performed using the CFA software which is developed by the Hydrology Division of the Water Resources Branch of Environment Canada. Four distributions, i.e., generalized extreme value distribution (GEV), three parameter lognormal distribution (3LN), Log pearson 3 distribution (LP3), and Wakeby distribution are provided by this software. The peak discharges for all the return periods of interest resulting from the use of three different distributions are given in Table 6.10.

Table 6. 7 Maximum Instantaneous Flow at the Gauge station (Ganrasak River near Dale Gauge Station #02HD012)

Year	Maximum instantaneous flows(cms)	Year	Maximum instantaneous flows(cms)
1960	84.7	1979	46.1
1961	33.1	1982	69.9
1962	29.7	1983	49.2
1963	68.8	1984	100
1965	77.9	1986	34.7
1966	47.6	1987	32.7
1967	40.8	1990	170
1968	40.5	1991	68.2
1969	47.3	1992	46.2
1970	19.6	1993	67.8
1971	30.3	1995	66.5
1972	53.2	1997	75.8
1974	52.4	1998	44.4
1975	87.8	1999	16.2
1977	62.9	2000	47.4
1978	46.4	2001	21.3

6.6.2.2 Peterborough Meteorological Data

To compare with the results from frequency analyses, both APSWM and OTTHYMO models of the entire watershed were run again by using meteorological data from the Peterborough station which is closer to the study area than Toronto Pearson International Airport.

Two sets of design storms –24 hour Chicago design storms and 12 hour AES design storms were obtained by using the Peterborough IDF curves (Table 2.4) which were determined based on 32 years of historical rainfall record from 1968 to 2001.

Rainfall Statistics with 12 hour IETD at Peterborough are listed in the following table.

Table 6. 8 Rainfall Statistics (IETD= 12hr) for Peterborough STP Station*

Average Annual Rainfall Volume (mm), $\bar{v}$	8.14
Average Rainfall Event Duration(hr), $\bar{t}$	9.24
Average Interevent Time(hr), $\bar{b}$	86.7
Average Annual Number of Rainfall Events, θ	61.3

*Source: Urban Stormwater Management Planning with Analytical Probabilistic Models (Adams and Papa, 2000)

The drainage area to the Dale gage station is 238 km^2, which is larger than 25 km^2. To compare with frequency analysis results, areal reduction in design storm modeling may need to be applied. Both IDF curves used for the generation of design storms and rainfall statistics used in APSWM represent point rainfall. Thus, another two runs of AES 12 hr design storm and rainfall statistics adjusted by an appropriate areal reduction factor are performed to quantify the differences from using point precipitation. According to the point rainfall depth adjustment curves recommended by the World Meteorological Organization (Fig. 2.3), a reduction factor of 0.9 is used to adjust IDF curves' rainfall depth for AES-12 hour design storm use. The rainfall depth before and after adjustment are listed in Table 6.9. Rainfall statistics input to APSWM is adjusted by multiplying average rainfall event volume by the same reduction factor of 0.9. The other two rainfall statistics remain unchanged. The average rainfall event volume with IETD of 12 hours for Peterborough STP Station is reduced from 8.14 mm to 7.36 mm after areal reduction.

Table 6. 9 Rainfall Depths for the Duration 12 Hours before and after Areal Reduction

Rainfall depth	Return Period Rainfall Amount(mm)					
(duration = 12 hour)	2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
Before adjustment	43.9	57.7	66.8	78.4	86.9	95.4
After adjustment	39.51	51.93	60.12	70.56	78.21	85.86

6.6.2.3 Results and Analyses

The comparison results are summarized in Table 6.9 and plotted in Fig. 6.4.

Table 6. 10 Comparison between APSWM , OTTHYMO and Frequency Analysis Results at Dale Gage Station (02HD012)

Discharges (cms)		Return period (years)					
		2 yr	5 yr	10 yr	25 yr	50 yr	100 yr
OTTHYMO	Chicago-24hr design storm	56.88	86.78	91.07	133.45	154.64	177.59
	AES-12 hr design storm	60.28	92.24	97.54	143.57	167.13	192.42
OTTHYMO(Areal reduction factor=0.9)	AES-12 hr design storm	47.96	73.29	77.23	113.86	131.26	150.11
APSWM	IETD=12 hr	9.20	32.52	75.38	132.07	174.98	217.87
APSWM (Areal reduction factor=0.9)	IETD=12 hr	7.82	16.01	54.19	105.24	142.88	182.52
Frequency Analysis	GEV distribution	49.50	73.70	91.40	110.00	136.00	157.00
	3LN distribution	49.90	74.80	92.20	109.00	132.00	150.00
	LP3 distribution	49.80	74.50	91.70	109.00	131.00	149.00

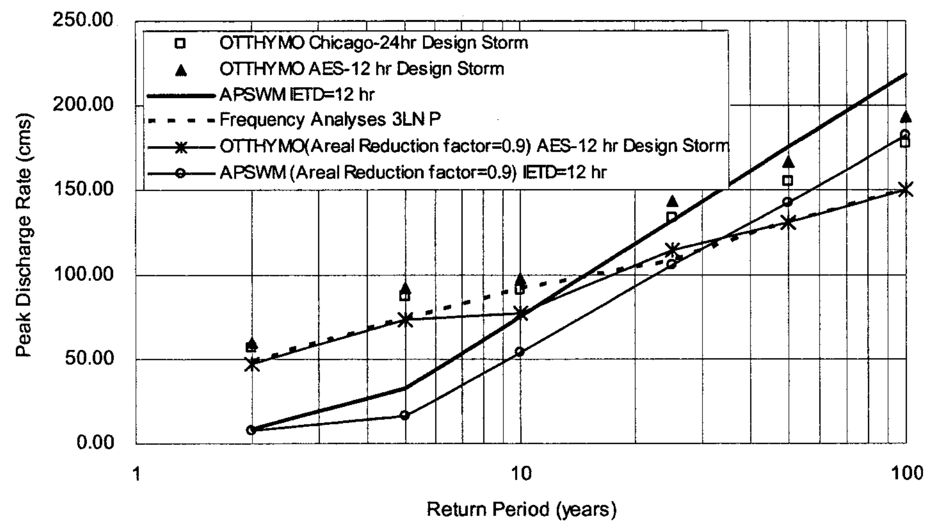


Fig. 6. 4 Peak discharge rates vs. return periods at Dale gage station

Compared with the results from frequency analysis, the discharge rates simulated by both APSWM and OTTHYMO are higher for return periods higher than 25 years. The peak flows simulated by APSWM are lower than those from frequency analysis for low return periods. The possible reasons contributing to these discrepancies are:

(1) The drainage area upstream of the gauge station exceeds the normal maximum drainage area where the design storm approach and APSWM may be applied. Application of areal reduction factor alone may not be enough to account for all the effects caused by large drainage areas.

(2) For low return periods, the peak flows simulated by APSWM are lower than those from frequency analyses. This is mainly because baseflows are not included in APSWM.

(3) Available precipitation data and streamflow data cover slightly different time periods. The Peterborough IDF curve was determined based on historical rainfall record from 1968 to 2001; while Dale gauge's 32 year observation of maximum instantaneous flows started from 1960 to 2001. Rainfall statistics of Peterborough used in APSWM are generated from rainfall records from 1964 to 1983.

(4) Streamflow records are maintained throughout the year. Annual maximum instantaneous flows may be caused partly by snowmelt events in some years, whereas both OTTHYMO and APSWM consider peak flows resulting from rainfall events only.

Despite the limitations of the APSWM model, it is encouraging to see that application of areal reduction factors to OTTHYMO and APSWM resulted in very similar degrees of reduction in peak discharges for different drainage areas. This can be seen in Tables 6.6 and 6.10.

Chapter 7 Conclusions and Recommendations

APSWM was developed as an alternative to design storm modeling for stormwater management purposes and it has been tested using several hypothetical cases and real catchments in urban areas. Close agreements were reported between results from APSWM, design storm-based models and continuous simulation models. The main objective of this project is to investigate APSWM's performance for larger rural areas. A series of comparisons were conducted between the design storm-based OTTHYMO model and APSWM for the Ganaraska River Hydrology Study. The following conclusions and recommendations can be made as a result of this study.

7.1 Conclusions

1. Generally, APSWM provided results comparable to those from the calibrated design storm-based OTTHYMO model and frequency analyses of observed streamflow data. For subcatchments with relatively higher *CN* values, the results are closer. For subcatchments with relatively low curve numbers, peak discharges of lower return periods simulated by APSWM are lower than those from OTTHYMO and frequency analyses of observed streamflow data. It was observed that despite totally different aggregation strategies employed in APSWM and OTTHYMO, the relative differences between the peak discharges simulated by the two approaches for the aggregated catchments are almost the same as those for individual subcatchments. It was therefore concluded that the loss of accuracy due to subcatchment aggregation in APSWM is insignificant for this case.

2. The time parameters, such as time of concentration (t_c), time to peak (t_p) and time lag (t_l), are important parameters to characterize a catchment's response to effective rainfall. Different models may utilize different time parameters and even the same parameter may have slightly different definitions and applications. Therefore, making sure that the time parameter values used in different models result in hydrologically equivalent subcatchment is a crucial prerequisite for any comparison studies. It was found that t_c defined in APSWM is two times of t_l in Nash IUH.

3. Some discrepancies from conversion from SCS-CN model to APSWM/Horton are inevitable. The discrepancies may be higher for more permeable

soils. *CN* represents the combined effect of land use and soil type, while infiltration rate in the Horton method is only dependent on soil types. The proposed method of conversion loses the land use information described by *CN*.

7.2 Recommendations

1. APSWM was originally developed for use in urban stormwater management planning and design. For which, the drainage area is generally not larger than 25 km^2, baseflows can be neglected and point precipitation is representative for the entire simulated area. However, to extend the application of APSWM to larger watershed areas, further research on the proper inclusion of baseflow and rainfall areal reduction should be conducted.

2. Comparison results reported herewith only apply to the specific case. Use of APSWM for rural areas with highly permeable soils should be made with caution. Before recommending APSWM for general use in rural area hydrologic studies, more extensive comparisons must be conducted. The appropriateness of the conversion method from SCS-CN to APSWM/Hoton parameters needs to be studied in more detail.

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APPENDIX A

Maps of the Study Watershed

Figure A-1: Ganaraska River Watershed Discretization

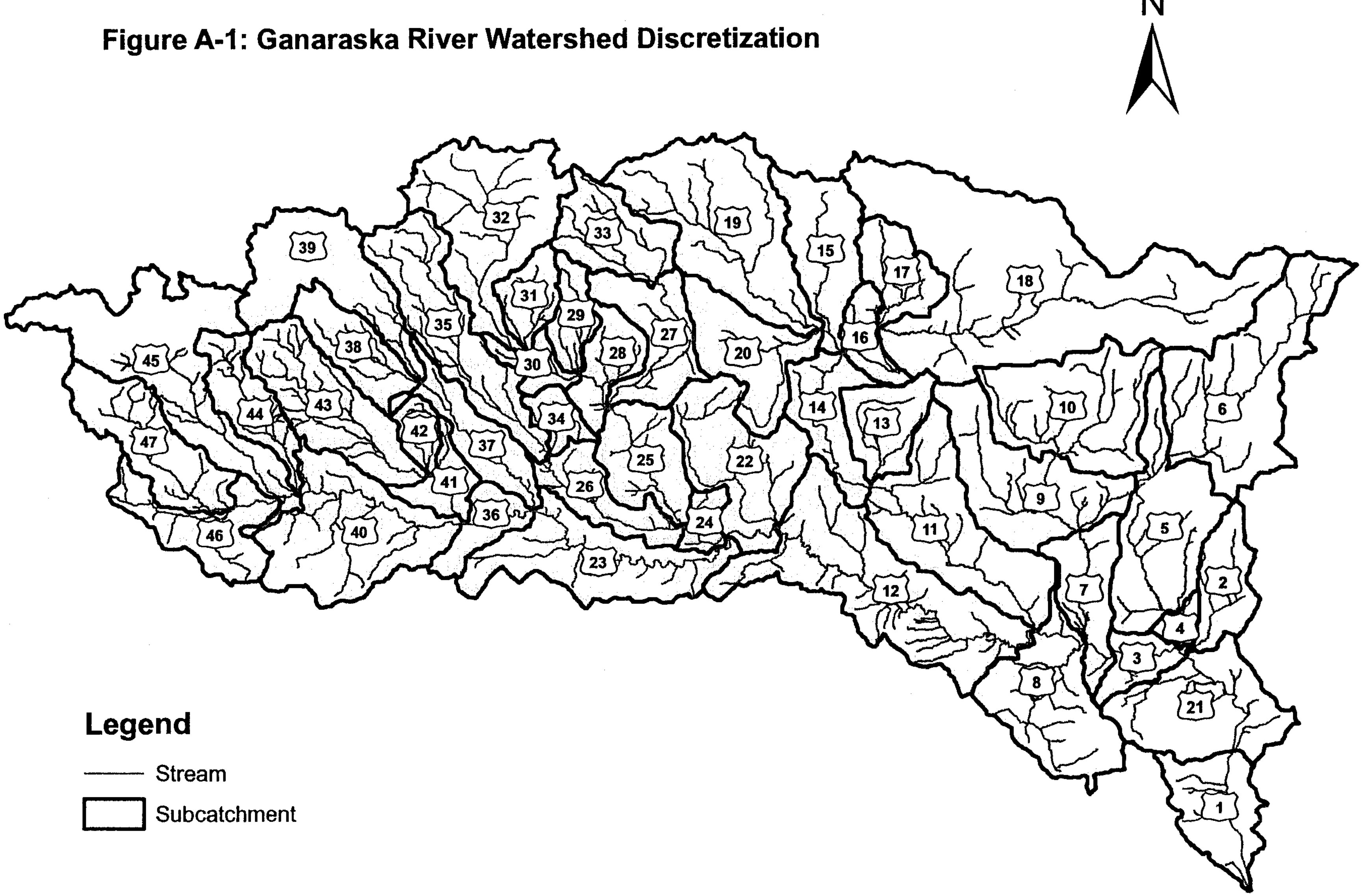
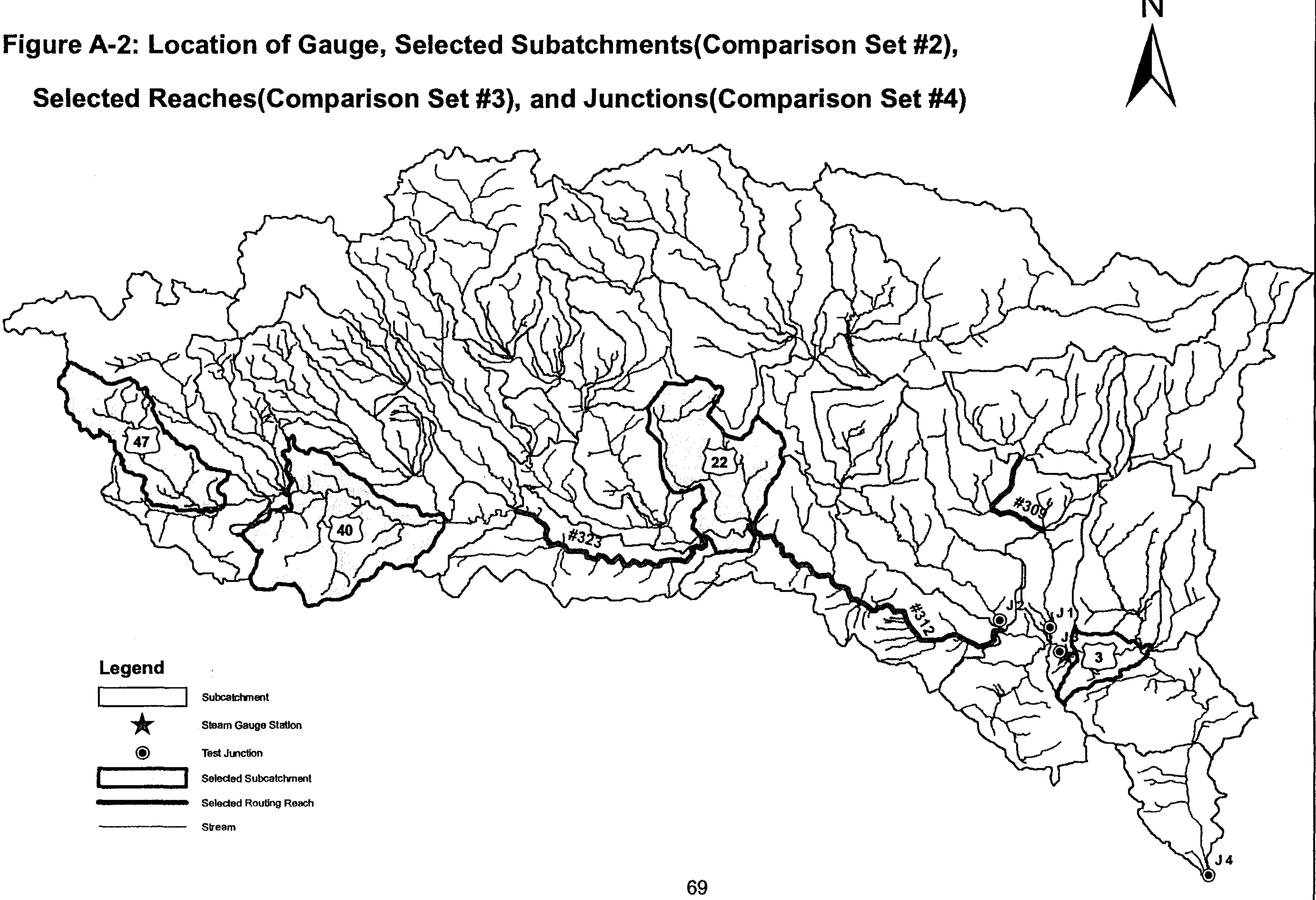


Figure A-2: Location of Gauge, Selected Subatchments(Comparison Set #2), Selected Reaches(Comparison Set #3), and Junctions(Comparison Set #4)



APPENDIX B

Input Parameters of OTTHYMO and APSWM for GRHS

Input Parameters of OTTHYMO and APSWM for GRHS

Subcatchment No	Area (ha)	OTTHYMO				APSWM	
		CN*	CN	I_a (mm)	t_p (hr)	f_c(mm/hr)	t_c(hr)
47	605.5	48	59	15.98	1.74	70.2	12.18
46	486.5	85	85	3.66	1.64	6.5	11.48
45	1190.5	19	39	32.35	2.64	76	18.48
44	360.3	23	41	28.79	1.26	76	8.82
43	703.8	45	57	15.48	1.5	72	10.50
42	134.9	90	88	2.71	0.6	5.2	4.20
41	201.2	95	92	1.83	1.21	3.5	8.47
40	993.7	64	70	8.69	2.14	58	14.98
39	736.2	9	32	44.02	1.8	76	12.60
38	363.2	46	57	15.04	1.04	72	7.28
37	375.4	95	92	1.83	1.59	3.5	11.13
36	227.6	82	83	4.27	1.26	12.7	8.82
35	773.9	47	58	14.62	1.64	71	11.48
34	141.9	78	80	5.13	0.71	22	4.97
33	461.6	10	32	42.62	1.1	76	7.70
32	988.9	10	32	42.62	2.01	76	14.07
31	227.1	56	64	11.21	0.91	64	6.37
30	141.3	70	74	7.05	1.05	44	7.35
29	201.4	94	91	2.00	0.76	3.9	5.32
28	280.4	90	88	2.71	0.81	5.2	5.67
27	474	75	78	5.82	1.15	30	8.05
26	353.7	81	82	4.48	1.38	15.8	9.66
25	453.3	82	83	4.27	1.47	12.7	10.29
24	121.2	100	95	1.02	0.96	2.17	6.72
23	820.4	88	87	3.08	3.29	5.6	23.03
22	771.7	90	88	2.71	2.12	5.2	14.84
21	735.5	100	95	1.02	2.8	2.17	19.60
20	542.8	81	82	4.48	1.86	15.8	13.02
19	1167.2	41	54	17.34	2.12	76	14.84
18	2609.3	73	76	6.30	4.64	37	32.48
17	327.2	64	70	8.69	1	57	7.00
16	188	75	78	5.82	0.81	30	5.67
15	526.9	47	58	14.62	1.64	71	11.48
14	381.4	94	91	2.00	1.7	3.9	11.90
13	350	94	91	2.00	1.17	3.9	8.19
12	1352.3	81	82	4.48	3.61	15.8	25.27

Catchment No	Area (ha)	OTTHYMO				APSWM	
		CN*	CN	I_a (mm)	t_p (hr)	f_c(mm/hr)	t_c(hr)
Continuous…							
11	956.1	73	76	6.30	2.46	37	17.22
10	898.8	85	85	3.66	2.03	6.5	14.21
9	837	79	81	4.91	1.84	19	12.88
8	753.7	100	95	1.02	2.07	2.17	14.49
7	392.3	86	85	3.46	1.53	6.5	10.71
6	1152.1	81	82	4.48	2.23	15.8	15.61
5	928.6	100	95	1.02	2.4	2.17	16.80
3	336.7	100	100	0.00	1.28	0	8.96
2	735.5	87	86	3.27	1.21	6.17	8.47
1	476.6	83	83	4.16	0.77	12.7	5.39

APPENDIX C

GRHS APSWM Output File using Toronto Pearson International Airport Precipitation Data

Project Name=Gan River
Created=2006-3-26 23:17:10
Modified=2006-12-20 20:47:22
End

Subcatchment Name=West Arm2
Area=605.5
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	206.45	0.000
5	519.26	0.000
10	11,282.48	0.511
25	58,067.45	2.623
50	93,454.11	4.222
100	128,831.12	5.820

Average Runoff Volume Per Rainfall Event=98.445
Average Annual (or Seasonal) Volume of Runoff=6369.414
End

Subcatchment Name=West Arm 1
Area=486.5
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	95,662.12	4.246
5	133,221.39	5.932
10	161,633.28	7.211
25	199,213.93	8.910
50	227,642.31	10.199
100	256,073.88	11.492

Average Runoff Volume Per Rainfall Event=3263.866
Average Annual (or Seasonal) Volume of Runoff=211172.16
End

Reach Name=104904
Subcatchments Discharging Directly Into This Reach:West Arm2, West Arm 1.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=1092
Average Imperviousness=0
Total Time of Concentration=11.772
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	3.013	2.942
10	6.041	5.905
25	10.048	9.821
50	13.081	12.784
100	16.115	15.748

Average Runoff Volume Per Rainfall Event=566.47
Average Annual (or Seasonal) Volume of Runoff=36650.623
End

Subcatchment Name=Carscadden
Area=1190.5
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	11.63	0.000
5	29.26	0.000
10	58.63	0.000
25	146.76	0.000
50	293.64	0.000
100	50,268.86	1.502

Average Runoff Volume Per Rainfall Event=25.597
Average Annual (or Seasonal) Volume of Runoff=1656.13
End

Reach Name=1052
Subcatchments Discharging Directly Into This Reach:Carscadden.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:104904.
APSWM Calculation Results:
Total Upstream Area=2282.5
Average Imperviousness=0
Total Time of Concentration=12.922
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	0.000	0.000
25	5.242	4.869
50	11.351	10.544
100	17.459	16.218

Average Runoff Volume Per Rainfall Event=215.692
Average Annual (or Seasonal) Volume of Runoff=13955.299
End

Subcatchment Name=Gan 11
Area=993.7
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	1,651.07	0.000
5	48,452.81	1.780
10	106,496.25	3.908
25	183,271.40	6.725
50	241,338.94	8.855
100	299,401.81	10.986

Average Runoff Volume Per Rainfall Event=461.762
Average Annual (or Seasonal) Volume of Runoff=29876.032
End

Subcatchment Name=lowry
Area=360.3
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	10.69	0.000
5	26.89	0.000
10	53.88	0.000
25	134.87	0.000
50	6,953.79	0.464
100	28,003.43	1.868

Average Runoff Volume Per Rainfall Event=11.804
Average Annual (or Seasonal) Volume of Runoff=763.69
End

Reach Name=1052.2
Subcatchments Discharging Directly Into This Reach:lowry.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:1052.
APSWM Calculation Results:
Total Upstream Area=2642.8
Average Imperviousness=0
Total Time of Concentration=13.277
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	0.000	0.000
25	4.338	4.126
50	11.069	10.523
100	17.797	16.921

Average Runoff Volume Per Rainfall Event=215.373
Average Annual (or Seasonal) Volume of Runoff=13934.646
End

Subcatchment Name=Burham2

Area=703.8
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	265.26	0.000
5	667.18	0.000
10	15,002.67	0.786
25	69,371.22	3.631
50	110,496.60	5.783
100	151,615.13	7.936

Average Runoff Volume Per Rainfall Event=118.089
Average Annual (or Seasonal) Volume of Runoff=7640.362
End

Subcatchment Name=Burham Tib
Area=134.9
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	29,974.78	3.173
5	40,401.51	4.343
10	48,293.14	5.240
25	58,714.66	6.445
50	66,596.28	7.367
100	74,481.32	8.297

Average Runoff Volume Per Rainfall Event=1228.502
Average Annual (or Seasonal) Volume of Runoff=79484.058
End

Reach Name=341
Subcatchments Discharging Directly Into This Reach:Burham2, Burham Tib.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=838.7
Average Imperviousness=0
Total Time of Concentration=5.587
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	5.458	4.501
25	13.085	10.797
50	18.859	15.558
100	24.635	20.323

Average Runoff Volume Per Rainfall Event=210.581
Average Annual (or Seasonal) Volume of Runoff=13624.561
End

Subcatchment Name=Burham 1
Area=201.2
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	52,026.66	2.900
5	67,561.52	3.796
10	79,333.51	4.484
25	94,873.73	5.400
50	106,625.94	6.098
100	118,384.03	6.801

Average Runoff Volume Per Rainfall Event=2817.863
Average Annual (or Seasonal) Volume of Runoff=182315.765
End

Reach Name=336
Subcatchments Discharging Directly Into This Reach:Burham 1, Gan 11.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:341, 1052.2.
APSWM Calculation Results:

Total Upstream Area=4676.4
Average Imperviousness=0
Total Time of Concentration=10.451
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	5.931	5.175
25	27.629	24.131
50	44.037	38.464
100	60.451	52.801

Average Runoff Volume Per Rainfall Event=782.935
Average Annual (or Seasonal) Volume of Runoff=50655.92
End

Subcatchment Name=Gan 10
Area=227.6
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	20,859.54	1.267
5	38,454.82	2.339
10	51,750.99	3.147
25	69,334.55	4.219
50	82,636.49	5.030
100	95,933.05	5.843

Average Runoff Volume Per Rainfall Event=440.199
Average Annual (or Seasonal) Volume of Runoff=28480.902
End

Subcatchment Name=Soper Trib
Area=363.2
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	153.62	0.000
5	386.38	0.000
10	9,334.24	0.702
25	37,397.49	2.809
50	58,618.57	4.404
100	79,841.41	5.999

Average Runoff Volume Per Rainfall Event=64.206
Average Annual (or Seasonal) Volume of Runoff=4154.113
End

Subcatchment Name=Soper 2
Area=736.2
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	6.66	0.000
5	16.75	0.000
10	33.56	0.000
25	84.01	0.000
50	168.09	0.000
100	26,300.74	1.150

Average Runoff Volume Per Rainfall Event=14.654
Average Annual (or Seasonal) Volume of Runoff=948.144
End

Reach Name=337
Subcatchments Discharging Directly Into This Reach:Soper Trib, Soper 2.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=1099.4
Average Imperviousness=0
Total Time of Concentration=10.403
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)

2	0.000	0.000
5	0.000	0.000
10	0.000	0.000
25	0.000	0.000
50	2.845	2.212
100	7.212	5.607

Average Runoff Volume Per Rainfall Event=45.01
Average Annual (or Seasonal) Volume of Runoff=2912.145
End

Subcatchment Name=Soper1
Area=375.4
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	97,071.61	4.244
5	126,056.64	5.547
10	148,020.88	6.540
25	177,015.90	7.860
50	198,943.23	8.865
100	220,881.53	9.873

Average Runoff Volume Per Rainfall Event=5257.584
Average Annual (or Seasonal) Volume of Runoff=340165.696
End

Reach Name=323
Subcatchments Discharging Directly Into This Reach:Soper1, Gan 10.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:337, 336.
APSWM Calculation Results:
Total Upstream Area=6378.8
Average Imperviousness=0
Total Time of Concentration=13.707
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	6.542	5.205
25	31.394	24.958
50	50.202	39.910
100	69.012	54.861

Average Runoff Volume Per Rainfall Event=1057.702
Average Annual (or Seasonal) Volume of Runoff=68433.349
End

Reach Name=330
Subcatchments Discharging Directly Into This Reach:L Gan T3, L Gan M5.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=1216
Average Imperviousness=0
Total Time of Concentration=8.175
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	0.000	0.000
25	0.000	0.000
50	0.383	0.345
100	5.669	5.109

Average Runoff Volume Per Rainfall Event=33.32
Average Annual (or Seasonal) Volume of Runoff=2155.834
End

Reach Name=328
Subcatchments Discharging Directly Into This Reach:L Gan M4, L Gan T2.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:330.
APSWM Calculation Results:
Total Upstream Area=1558.7
Average Imperviousness=0

Total Time of Concentration=5.92
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	0.000	0.000
25	2.868	2.632
50	11.982	10.993
100	21.100	19.355

Average Runoff Volume Per Rainfall Event=101.054
Average Annual (or Seasonal) Volume of Runoff=6538.168
End

Subcatchment Name=L Gan M5
Area=988.9
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	2.36	0.000
5	5.93	0.000
10	11.88	0.000
25	29.74	0.000
50	59.51	0.000
100	15,550.45	0.608

Average Runoff Volume Per Rainfall Event=15.527
Average Annual (or Seasonal) Volume of Runoff=1004.605
End

Subcatchment Name=L Gan T3
Area=227.1
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	201.50	0.000
5	3,520.05	0.301
10	16,785.65	1.435
25	34,329.95	2.935
50	47,597.87	4.070
100	60,867.42	5.206

Average Runoff Volume Per Rainfall Event=71.113
Average Annual (or Seasonal) Volume of Runoff=4601.007
End

Subcatchment Name=L Gan M4
Area=141.3
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	1,497.78	0.112
5	12,430.87	0.917
10	20,690.29	1.526
25	31,611.38	2.331
50	39,866.05	2.941
100	48,122.44	3.551

Average Runoff Volume Per Rainfall Event=104.626
Average Annual (or Seasonal) Volume of Runoff=6769.304
End

Subcatchment Name=L Gan T2
Area=201.4
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	50,285.55	4.224
5	65,815.91	5.608
10	77,593.78	6.673
25	93,153.79	8.100
50	104,921.15	9.190
100	116,687.45	10.290

Average Runoff Volume Per Rainfall Event=2534.486
Average Annual (or Seasonal) Volume of Runoff=163981.26
End

Subcatchment Name=L Gan M3
Area=280.4
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	62,304.88	5.110
5	83,977.64	6.959
10	100,381.00	8.374
25	122,042.92	10.264
50	138,425.47	11.706
100	154,815.14	13.158

Average Runoff Volume Per Rainfall Event=2553.535
Average Annual (or Seasonal) Volume of Runoff=165213.713
End

Subcatchment Name=Beatty 2
Area=461.6
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	1.10	0.000
5	2.77	0.000
10	5.55	0.000
25	13.88	0.000
50	27.78	0.000
100	7,258.66	0.516

Average Runoff Volume Per Rainfall Event=7.248
Average Annual (or Seasonal) Volume of Runoff=468.931
End

Subcatchment Name=Beatty 1
Area=474
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	25,934.57	1.730
5	62,457.40	4.173
10	90,175.30	6.027
25	126,788.30	8.477
50	154,484.33	10.332
100	182,182.73	12.188

Average Runoff Volume Per Rainfall Event=590.045
Average Annual (or Seasonal) Volume of Runoff=38175.912
End

Reach Name=333
Subcatchments Discharging Directly Into This Reach:Beatty 2.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=461.6
Average Imperviousness=0
Total Time of Concentration=14.7
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	0.000	0.000
25	0.000	0.000
50	0.000	0.000
100	0.516	0.272

Average Runoff Volume Per Rainfall Event=7.248
Average Annual (or Seasonal) Volume of Runoff=468.931
End

Reach Name=325

Subcatchments Discharging Directly Into This Reach:L Gan M3, Beatty 1.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:328, 333.
APSWM Calculation Results:
Total Upstream Area=2774.7
Average Imperviousness=0
Total Time of Concentration=7.631
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	0.000	0.000
25	18.606	14.987
50	33.079	26.651
100	47.555	38.316

Average Runoff Volume Per Rainfall Event=352.448
Average Annual (or Seasonal) Volume of Runoff=22803.358
End

Subcatchment Name=Cold Sp 2
Area=773.9
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	371.08	0.000
5	933.31	0.000
10	24,226.94	1.168
25	83,984.62	4.052
50	129,208.35	6.233
100	174,427.10	8.415

Average Runoff Volume Per Rainfall Event=146.143
Average Annual (or Seasonal) Volume of Runoff=9455.465
End

Subcatchment Name=Cold Sp T
Area=141.9
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	12,316.92	1.283
5	23,267.87	2.431
10	31,558.17	3.303
25	42,519.81	4.458
50	50,811.96	5.334
100	59,101.93	6.211

Average Runoff Volume Per Rainfall Event=258.648
Average Annual (or Seasonal) Volume of Runoff=16734.503
End

Reach Name=326
Subcatchments Discharging Directly Into This Reach:Cold Sp 2, Cold Sp T.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=915.8
Average Imperviousness=0
Total Time of Concentration=9.771
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	4.080	2.844
25	9.777	6.813
50	14.088	9.817
100	18.398	12.821

Average Runoff Volume Per Rainfall Event=230.106
Average Annual (or Seasonal) Volume of Runoff=14887.84
End

Subcatchment Name=L Gan M2
Area=453.3

Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	63,156.02	3.223
5	98,089.59	5.017
10	124,618.80	6.376
25	159,633.94	8.177
50	186,116.14	9.541
100	212,598.60	10.908

Average Runoff Volume Per Rainfall Event=1540.208
Average Annual (or Seasonal) Volume of Runoff=99651.446
End

Subcatchment Name=Cold Sp1
Area=353.7
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	42,413.05	2.323
5	69,714.27	3.820
10	90,380.33	4.957
25	117,699.33	6.460
50	138,361.03	7.600
100	159,026.47	8.741

Average Runoff Volume Per Rainfall Event=954.759
Average Annual (or Seasonal) Volume of Runoff=61772.911
End

Reach Name=324
Subcatchments Discharging Directly Into This Reach:Cold Sp1, L Gan M2.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:326, 325.
APSWM Calculation Results:
Total Upstream Area=4497.5
Average Imperviousness=0
Total Time of Concentration=10.393
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	11.257	10.534
25	30.895	28.897
50	45.751	42.791
100	60.608	56.684

Average Runoff Volume Per Rainfall Event=991.168
Average Annual (or Seasonal) Volume of Runoff=64128.587
End

Subcatchment Name=L Gan M1
Area=121.2
Imperviousness=1
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	47,858.42	2.566
5	57,220.88	3.190
10	64,305.95	3.674
25	73,667.18	4.328
50	80,749.43	4.831
100	87,830.92	5.341

Average Runoff Volume Per Rainfall Event=8551.722
Average Annual (or Seasonal) Volume of Runoff=553296.441
End

Reach Name=322
Subcatchments Discharging Directly Into This Reach:L Gan M1.
Junctions Discharging Directly Into This Reach:Osaca Gauge.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:324.
APSWM Calculation Results:
Total Upstream Area=11837.9

Average Imperviousness=0.01
Total Time of Concentration=9.91
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	2.085	1.736
5	3.643	3.017
10	36.688	30.562
25	97.366	81.090
50	143.290	119.330
100	189.218	157.581

Average Runoff Volume Per Rainfall Event=10907.076
Average Annual (or Seasonal) Volume of Runoff=705687.819
End

Subcatchment Name=Gan 7
Area=840.4
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	179,313.35	4.110
5	244,193.50	5.608
10	293,235.23	6.741
25	358,178.48	8.241
50	407,285.85	9.378
100	456,405.64	10.516

Average Runoff Volume Per Rainfall Event=6874.642
Average Annual (or Seasonal) Volume of Runoff=444789.36
End

Subcatchment Name=Gan 6
Area=771.7
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	171,471.74	5.923
5	231,118.21	8.005
10	276,262.55	9.584
25	335,879.19	11.676
50	380,966.24	13.264
100	426,072.91	14.856

Average Runoff Volume Per Rainfall Event=7027.685
Average Annual (or Seasonal) Volume of Runoff=454691.234
End

Reach Name=312
Subcatchments Discharging Directly Into This Reach:Gan 6.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:322.
APSWM Calculation Results:
Total Upstream Area=12609.6
Average Imperviousness=0.009
Total Time of Concentration=20.436
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	1.348	0.983
5	2.589	1.928
10	29.634	20.021
25	68.547	46.304
50	97.991	66.189
100	127.430	86.076

Average Runoff Volume Per Rainfall Event=11111.148
Average Annual (or Seasonal) Volume of Runoff=718891.253
End

Subcatchment Name=Garden Hill T
Area=542.8
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	65,088.50	2.675

5	106,985.88	4.407
10	138,700.71	5.716
25	180,625.38	7.449
50	212,333.52	8.761
100	244,047.40	10.073

Average Runoff Volume Per Rainfall Event=1465.206
Average Annual (or Seasonal) Volume of Runoff=94798.802
End

Subcatchment Name=Garden Hill 2
Area=1167.2
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	312.30	0.000
5	785.49	0.000
10	1,574.14	0.000
25	88,084.69	3.281
50	156,275.42	5.823
100	224,477.87	8.365

Average Runoff Volume Per Rainfall Event=148.897
Average Annual (or Seasonal) Volume of Runoff=9633.661
End

Subcatchment Name=Garden Hill 1
Area=526.9
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	251.92	0.000
5	633.63	0.000
10	16,356.09	0.783
25	57,074.86	2.730
50	87,865.53	4.202
100	118,652.66	5.675

Average Runoff Volume Per Rainfall Event=99.264
Average Annual (or Seasonal) Volume of Runoff=6422.378
End

Reach Name=315
Subcatchments Discharging Directly Into This Reach:Garden Hill T, Garden Hill 2, Garden Hill 1.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=2236.9
Average Imperviousness=0
Total Time of Concentration=13.392
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	0.000	0.000
10	5.201	5.072
25	12.479	12.166
50	17.985	17.533
100	23.493	22.902

Average Runoff Volume Per Rainfall Event=560.941
Average Annual (or Seasonal) Volume of Runoff=36292.873
End

Subcatchment Name=N Gan E2
Area=2609.3
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	85,175.01	1.442
5	286,669.98	4.865
10	438,999.38	7.460
25	640,577.01	10.882
50	793,003.99	13.473

100	945,506.59	16.064

Average Runoff Volume Per Rainfall Event=2501.776
Average Annual (or Seasonal) Volume of Runoff=161864.926
End

Subcatchment Name=N Gan ET
Area=327.2
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	551.37	0.000
5	16,439.46	1.278
10	35,546.33	2.765
25	60,820.03	4.731
50	79,937.25	6.219
100	99,055.34	7.707

Average Runoff Volume Per Rainfall Event=154.672
Average Annual (or Seasonal) Volume of Runoff=10007.286
End

Reach Name=316
Subcatchments Discharging Directly Into This Reach:N Gan E2, N Gan ET.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=2936.5
Average Imperviousness=0
Total Time of Concentration=29.49
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	1.400	1.375
5	5.715	5.619
10	8.982	8.832
25	13.298	13.075
50	16.563	16.285
100	19.829	19.496

Average Runoff Volume Per Rainfall Event=2576.548
Average Annual (or Seasonal) Volume of Runoff=166702.648
End

Subcatchment Name=N Gan E1
Area=188
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	10,286.29	0.953
5	24,772.13	2.309
10	35,765.73	3.334
25	50,287.34	4.692
50	61,272.27	5.720
100	72,258.13	6.750

Average Runoff Volume Per Rainfall Event=234.026
Average Annual (or Seasonal) Volume of Runoff=15141.501
End

Reach Name=314
Subcatchments Discharging Directly Into This Reach:N Gan E1.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:315, 316.
APSWM Calculation Results:
Total Upstream Area=5361.4
Average Imperviousness=0
Total Time of Concentration=25.099
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	7.591	6.741
10	15.348	13.632
25	25.597	22.742
50	33.350	29.629
100	41.103	36.516

Average Runoff Volume Per Rainfall Event=2755.974
Average Annual (or Seasonal) Volume of Runoff=178311.549
End

Subcatchment Name=N Gan 2
Area=381.4
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	95,227.95	3.950
5	124,638.47	5.200
10	146,942.74	6.151
25	176,409.42	7.415
50	198,693.77	8.375
100	220,976.14	9.340

Average Runoff Volume Per Rainfall Event=4799.668
Average Annual (or Seasonal) Volume of Runoff=310538.493
End

Subcatchment Name=N Gan T
Area=350
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	87,388.00	5.058
5	114,377.20	6.681
10	134,845.20	7.919
25	161,885.94	9.570
50	182,335.66	10.829
100	202,783.56	12.094

Average Runoff Volume Per Rainfall Event=4404.519
Average Annual (or Seasonal) Volume of Runoff=284972.398
End

Reach Name=311
Subcatchments Discharging Directly Into This Reach:N Gan T, N Gan 2.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:314.
APSWM Calculation Results:
Total Upstream Area=6092.8
Average Imperviousness=0
Total Time of Concentration=14.459
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.000	0.000
5	20.850	17.510
10	36.928	31.030
25	58.169	48.880
50	74.242	62.380
100	90.321	75.885

Average Runoff Volume Per Rainfall Event=3905.549
Average Annual (or Seasonal) Volume of Runoff=252688.992
End

Subcatchment Name=N Gan 1
Area=956.1
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	31,209.84	0.998
5	105,041.65	3.348
10	160,858.20	5.130
25	234,720.30	7.486
50	290,572.61	9.269
100	346,452.63	11.051

Average Runoff Volume Per Rainfall Event=916.701
Average Annual (or Seasonal) Volume of Runoff=59310.565
End

Reach Name=377

Subcatchments Discharging Directly Into This Reach:Quay 2.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:309.
APSWM Calculation Results:
Total Upstream Area=1735.8
Average Imperviousness=0
Total Time of Concentration=18.285
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	7.645	7.168
5	11.852	11.115
10	15.041	14.101
25	19.258	18.051
50	22.452	21.045
100	25.647	24.039

Average Runoff Volume Per Rainfall Event=5978.739
Average Annual (or Seasonal) Volume of Runoff=386824.391
End

Subcatchment Name=Gan 5
Area=1352.3
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	162,157.67	3.432
5	266,538.33	5.652
10	345,550.79	7.329
25	449,999.46	9.546
50	528,995.24	11.224
100	608,005.35	12.902

Average Runoff Volume Per Rainfall Event=3650.327
Average Annual (or Seasonal) Volume of Runoff=236176.161
End

Subcatchment Name=Quay 3
Area=898.8
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	176,734.04	5.376
5	246,124.12	7.497
10	298,614.58	9.107
25	368,044.16	11.239
50	420,565.08	12.854
100	473,091.89	14.472

Average Runoff Volume Per Rainfall Event=6029.935
Average Annual (or Seasonal) Volume of Runoff=390136.767
End

Reach Name=309
Subcatchments Discharging Directly Into This Reach:Quay 3.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=898.8
Average Imperviousness=0
Total Time of Concentration=18.7
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	5.376	4.963
5	7.497	6.925
10	9.107	8.411
25	11.239	10.378
50	12.854	11.868
100	14.472	13.361

Average Runoff Volume Per Rainfall Event=6029.935
Average Annual (or Seasonal) Volume of Runoff=390136.767
End

Subcatchment Name=Quay 2
Area=837

Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	84,261.99	3.521
5	148,981.59	6.228
10	197,887.16	8.276
25	262,551.43	10.984
50	311,452.68	13.033
100	360,358.05	15.084

Average Runoff Volume Per Rainfall Event=1801.932
Average Annual (or Seasonal) Volume of Runoff=116584.971
End

Subcatchment Name=Quay 1
Area=392.3
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	77,946.09	3.667
5	108,183.26	5.112
10	131,130.87	6.208
25	161,422.40	7.665
50	184,345.34	8.769
100	207,271.70	9.878

Average Runoff Volume Per Rainfall Event=2695.078
Average Annual (or Seasonal) Volume of Runoff=174371.567
End

Reach Name=J3-303
Subcatchments Discharging Directly Into This Reach:Quay 1, Gan 4.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:308, 377.
APSWM Calculation Results:
Total Upstream Area=23892.6
Average Imperviousness=0.036
Total Time of Concentration=17.282
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	10.883	10.004
5	46.442	42.019
10	95.500	86.386
25	160.325	145.091
50	209.402	189.492
100	258.465	233.902

Average Runoff Volume Per Rainfall Event=71227.632
Average Annual (or Seasonal) Volume of Runoff=4608427.772
End

Subcatchment Name=Gan 4
Area=753.7
Imperviousness=1
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	297,614.60	8.548
5	355,836.43	10.434
10	399,895.99	11.885
25	458,110.19	13.833
50	502,152.19	15.323
100	546,189.47	16.827

Average Runoff Volume Per Rainfall Event=53180.142
Average Annual (or Seasonal) Volume of Runoff=3440755.18
End

Subcatchment Name=Duck 3
Area=1152.1
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)

2	138,151.19	4.766
5	227,078.91	7.840
10	294,394.05	10.169
25	383,379.71	13.249
50	450,680.63	15.579
100	517,993.76	17.911

Average Runoff Volume Per Rainfall Event=3109.918
Average Annual (or Seasonal) Volume of Runoff=201211.68
End

Subcatchment Name=Gan 2
Area=189.8
Imperviousness=1
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	74,946.60	3.194
5	89,608.27	3.940
10	100,703.54	4.517
25	115,363.29	5.294
50	126,454.14	5.891
100	137,543.80	6.495

Average Runoff Volume Per Rainfall Event=13392.054
Average Annual (or Seasonal) Volume of Runoff=866465.879
End

Reach Name=305
Subcatchments Discharging Directly Into This Reach:Duck 3.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:None.
APSWM Calculation Results:
Total Upstream Area=1152.1
Average Imperviousness=0
Total Time of Concentration=17.6
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	6.401	5.742
5	9.246	8.289
10	11.415	10.227
25	14.301	12.803
50	16.498	14.763
100	18.704	16.730

Average Runoff Volume Per Rainfall Event=6139.327
Average Annual (or Seasonal) Volume of Runoff=397214.472
End

Subcatchment Name=Duck 1
Area=928.6
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	275,775.63	8.058
5	347,460.69	10.206
10	401,746.29	11.845
25	473,477.91	14.021
50	527,735.86	15.675
100	582,010.64	17.336

Average Runoff Volume Per Rainfall Event=20498.509
Average Annual (or Seasonal) Volume of Runoff=1326253.52
End

Subcatchment Name=Dale
Area=336.7
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	50,243.12	3.086
5	76,236.20	4.687
10	95,925.83	5.904
25	121,937.44	7.516
50	141,604.80	8.739

100	161,277.35	9.964

Average Runoff Volume Per Rainfall Event=1288.114
Average Annual (or Seasonal) Volume of Runoff=83341.005
End

Reach Name=321
Subcatchments Discharging Directly Into This Reach:Gan 2, Duck 1, Dale.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:J3-303, 305.
APSWM Calculation Results:
Total Upstream Area=26499.8
Average Imperviousness=0.04
Total Time of Concentration=16.375
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	13.981	13.482
5	62.264	59.772
10	116.350	111.685
25	187.764	180.285
50	241.804	232.171
100	295.843	284.065

Average Runoff Volume Per Rainfall Event=88310.091
Average Annual (or Seasonal) Volume of Runoff=5713662.855
End

Subcatchment Name=North of 401
Area=73.5
Imperviousness=0
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	21,828.03	0.554
5	27,502.00	0.702
10	31,798.79	0.814
25	37,476.44	0.962
50	41,771.04	1.075
100	46,066.96	1.189

Average Runoff Volume Per Rainfall Event=1622.486
Average Annual (or Seasonal) Volume of Runoff=104974.837
End

Reach Name=301
Subcatchments Discharging Directly Into This Reach:North of 401.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:321.
APSWM Calculation Results:
Total Upstream Area=26573.3
Average Imperviousness=0.04
Total Time of Concentration=16.767
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	13.540	13.317
5	60.220	59.052
10	112.075	109.817
25	180.613	177.008
50	232.494	227.830
100	284.374	278.674

Average Runoff Volume Per Rainfall Event=88667.808
Average Annual (or Seasonal) Volume of Runoff=5736807.152
End

Subcatchment Name=S of 401
Area=476.6
Imperviousness=0.5
APSWM Calculated Time of Concentration=N/A
APSWM Calculation Results:
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	114,864.93	5.718
5	146,678.42	7.729
10	172,641.76	9.263
25	208,351.65	11.304
50	235,843.95	12.856
100	263,534.15	14.414

Average Runoff Volume Per Rainfall Event=17889.851
Average Annual (or Seasonal) Volume of Runoff=1157473.339
End

Reach Name=308
Subcatchments Discharging Directly Into This Reach:Gan 5, N Gan 1.
Junctions Discharging Directly Into This Reach:None.
Ponds Discharging Directly Into This Reach:None.
Reaches Discharging Directly Into This Reach:312, 311.
APSWM Calculation Results:
Total Upstream Area=21010.8
Average Imperviousness=0.005
Total Time of Concentration=20.63
Table of Probabilistic Routing Results

Return Period (yrs)	Peak Inflow (c.m/s)	Peak Outflow (c.m/s)
2	0.939	0.917
5	21.801	20.719
10	56.221	53.548
25	101.760	96.874
50	136.208	129.666
100	170.663	162.457

Average Runoff Volume Per Rainfall Event=15567.949
Average Annual (or Seasonal) Volume of Runoff=1007246.271
End

Junction Name=J4-Peter street Gauge
Subcatchments Discharging Directly Into This Junction:S of 401.
Junctions Discharging Directly Into This Junction:None.
Ponds Discharging Directly Into This Junction:None.
Reaches Discharging Into This Junction:301.
APSWM Calculation Results:
Total Upstream Area=27049.9
Average Imperviousness=0.048
Total Time of Concentration=15.091
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	631,213.07	17.651
5	1,905,472.24	69.260
10	3,487,021.93	126.578
25	5,576,655.12	202.466
50	7,157,107.37	259.870
100	8,737,786.46	317.285

Average Runoff Volume Per Rainfall Event=105582.19
Average Annual (or Seasonal) Volume of Runoff=6831167.703
End

Junction Name=J2-2512
Subcatchments Discharging Directly Into This Junction:N Gan 1.
Junctions Discharging Directly Into This Junction:None.
Ponds Discharging Directly Into This Junction:None.
Reaches Discharging Into This Junction:311.
APSWM Calculation Results:
Total Upstream Area=7048.9
Average Imperviousness=0
Total Time of Concentration=14.935
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	22,556.48	0.833
5	567,013.52	20.801
10	978,588.72	35.923
25	1,523,032.33	55.923
50	1,934,942.91	71.054
100	2,346,802.98	86.184

Average Runoff Volume Per Rainfall Event=4767.337
Average Annual (or Seasonal) Volume of Runoff=308446.674
End

Junction Name=J1-2703
Subcatchments Discharging Directly Into This Junction:Quay 1.
Junctions Discharging Directly Into This Junction:None.
Ponds Discharging Directly Into This Junction:None.
Reaches Discharging Into This Junction:377.
APSWM Calculation Results:
Total Upstream Area=2128.1
Average Imperviousness=0
Total Time of Concentration=15.409
Table of Probabilistic Routing Results

Return Period(yrs)	Runoff Volume(c.m)	Peak Discharge(c.m/s)
2	317,134.19	10.972
5	481,774.08	16.668
10	606,054.51	20.984
25	770,511.56	26.690
50	894,806.99	31.013
100	1,019,151.84	35.340

Average Runoff Volume Per Rainfall Event=8132.155
Average Annual (or Seasonal) Volume of Runoff=526150.403
End