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A STANDARD HYDROGRAPH METHOD FOR THE PRELIMINARY ANALYSIS OF STORMWATER MANAGEMENT PROJECTS

by

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FOREWORD

This thesis is part of a comprehensive research program on urban hydrology conducted at the University of Ottawa. The purpose of the program is to develop a hierachical package of models for the three levels of drainage studies considered in many recent Canadian regulations:

1. Development of a standard hydrograph method for the preliminary analysis of stormwater management projects which is compatible with sophisticated computer models (this thesis).
2. Development of a conceptual hydrologic model for master drainage plans for use in both small and large areas.
3. Development of a model for the simultaneous routing of runoff in major and minor systems for the detailed design of drainage systems in new subdivisions.

While each of the studies is basically independent of the others, the common goal is to develop a package of methods which are compatible with each other and with previous research on the testing of the SWMM model with data from watersheds in Canada and the United States.

ABSTRACT

Current practices of runoff computations in urban drainage designs employs different levels of analyses. At the first level or preliminary stage, manual or desk-top methods are used and at the master plan and detailed design levels, complex computer models are used. Although the use of desk-top methods at the preliminary stage of the design seems appropriate, a situation is being created under which inconsistent and incompatible results are obtained when compared with the computer simulations used in the final stage of design.

A review of presently applied desk-top methods for drainage designs, including the Rational Method, the Modified Rational Method Hydrograph, and the SCS TR-55 Method shows that they have many limitations and weaknesses. Runoffs estimated by these various desk-top methods are inconsistent and are not compatible with the final design model.

In order that the urban runoff control criteria can be properly defined, a standard method for all runoff computations must be used. This research was initiated by the need for such a standard and desk-top method for the preliminary design of urban drainage systems and stormwater control facilities.

The first part of the research was carried out to investigate the possibilities of interfacing the commonly used manual Rational Method with the complex computer model SWMM for designing the pipe system. An extensive assessment of the Rational Method for peak discharge estimations using real and conceptual watersheds was carried out. It is concluded that the Rational Method gives results significantly different from the more complex computer model. Although a methodology is developed by which the Rational Method peak estimations can be adjusted, it is concluded that the Rational Method is basically an inadequate method to use for the present state-of-the-art of urban drainage design.

The second part of the research is therefore devoted to the development of an improved desk-top method entitled the Standard Hydrograph Method (SHM). The method developed is based on computer simulations using design storms. It is simpler to apply than the Rational Method but it provides both peak discharges and runoff hydrographs. The SHM procedure can be based on any design storm distributions. An example is given to develop a set of SHM relations using the computer model SWMM and Keifer and Chu's design storm distribution because they are common methods of design in Canada. As a further demonstration of the applicability of the SHM procedure, the SCS 24-hour Type II distribution design storms are also used.

A 'selective runoff control' criterion is proposed instead of the 'zero runoff increase' criterion. It is demonstrated, on the basis of a study conducted with a real rural watershed, that the latter is not only impractical but is also difficult to implement.

The SHM procedure is next extended to develop two desk-top methods for the estimation of detention storage for stormwater runoff control. The first method can be used for estimating volumes in conventional designs; the second method can be used for estimating volumes in a "dual storage" design which is presently being employed in new developments.

While most of the research is based on homogeneous watersheds, it is proposed that for the preliminary analysis of non-homogeneous watersheds desk-top programs for use on programmable calculators can be interfaced with SHM for routing hydrographs through channel and storage reservoirs.

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TABLE OF CONTENTS

Appendix page

LIST OF FIGURES

LIST OF TABLES

Chapter I

INTRODUCTION AND RESEARCH OBJECTIVES

1.1 INTRODUCTION

Despite the development of computer methods in the last decade, manual methods of runoff computations for the design of urban drainage systems are still being widely applied (Kibler.1982). The main reasons for the continuing use of manual, or desk-top, methods of computations are their ease of application and flexibility.

In a survey conducted in 1975 by Wisner and Clark (MacLaren Limited 1975) for Environment Canada, it has been found that out of the 40 municipalities surveyed across Canada, 32 are doing computations by desk-top methods while 5 are using both desk-top and computer methods (Montreal, Scarborough, Toronto, Hamilton and Winnipeg). In a recent survey conducted in 1980 at the University of Ottawa by the IMPSWM Research Program it has been found that the consultants and governmental agencies surveyed are still applying the manual Rational Method for the design of the storm sewer systems, including four who are using both the Rational Method and computer models. 80 percent of those who answered have also indicated that desk-top methods are of research priority in the preliminary analysis of drainage systems.

In some studies, such as those for preliminary designs and planning purposes, complex methods may not be warranted. Most computer models obviously simulate the rainfall-runoff process in a more rigorous manner, taking into account the rainfall pattern and duration, infiltration process, types of land surfaces, and overland flow and pipe routing. However, they normally require longer time for input preparation as well as higher cost of analyses. In many cases, proper training is needed for using the models correctly and in many engineering firms, such expertise may not be readily available.

Recognizing the capabilities of computer models and the apparent conveniences of desk-top methods, some municipalities have adopted a hierachical approach for the design of urban drainage systems, whereby desk-top methods are used for preliminary designs and more sophisticated models are used for final designs (Town of Markham 1978, Town of Oakville 1979). As a first step, desk-top computational methods facilitate the evaluation of different stormwater management alternatives. A final detailed analysis can be carried out with computer models once an alternative is selected. The Rational Method has been recommended for the preliminary sizing of the storm sewers. At the final design stage, detailed simulations with computer models, such as SWMM (Huber et al 1975) and ILLUDAS (Terstriep and Stall 1974), have been recommended, although the Rational Method has also been used in some cases.

The advantage of a hierachical approach is that models in the hierachy can and should be entirely complementary. By using a desk-top method, the hydrologist can analyze a drainage system quite easily and cheaply, and can get some feel for the response of the system to various alternatives. This step can point out shortcomings and provide valuable insights into more detailed analyses that may be required.

In general, methods which use hand computations and do not require complex input data and special training can be classified as desk-top methods. They are normally expressed in the form of simple equations, tables, diagrams or graphs that can be readily applied. Using desk-top programmable calculators, some computational procedures, such as hydrograph generation and routing through channels or reservoirs, can be applied as simple methods that only require a limited amount of input data (Croley 1978, 1980). Some examples of desk-top methods and computer models currently applied in Canada are listed as follows:-

1. Desk-top Methods

   Rational Method
   Modified Rational Method Hydrograph
   U.S. Soil Conservation Service TR-55 (SCS 1975)

2. Computer Methods

HYMO (Williams and Hann 1973)
SWMM (Huber et al 1975)
ILLUDAS (Terstriep and Stall 1974)
EXTRAN (Roesner et al 1982)

While any hierachical approach in modelling should clearly define the use of a correct method for each stage of the study, a situation exists under which results predicted with desk-top methods may not be compatible with computer models. Most methods or models predict runoff on the basis of:

- storm input
- antecedent moisture conditions
- estimation of rainfall losses
- model parameters and mathematical approach

Desk-top methods usually use a simpler form of input and require fewer parameters for runoff computations. The selection or determination of input parameters is often subjected to judgemental decisions or personal preferences because of their simplistic nature. For example, the Rational Method uses only a constant average rate of rainfall as an input for computing the peak rate of runoff while computer models normally use a rainfall hyetograph and can accept any format of storm input; the manual SCS TR-55 method uses the special SCS Type II 24-hour design storm distribution for estimating peak runoff rates.

Another example of the differences between desk-top and computerized methods is that in the Rational Method the peak runoff rate is determined in terms of a simple ratio of the rate of rainfall. This ratio accounts for all of the rainfall losses and is affected by the rainfall characteristics. Yet there are no clear quidelines in current practices for it to be properly determined. Rainfall losses, on the other hand, are calculated in detailed in computer models.

An interface of desk-top methods and computer models cannot be carried out successfully unless these differences are examined and the discrepancies between the results are eliminated. More important, input parameters must also be compatible and standardized before compatible results can be achieved.

Nonetheless, some desk-top methods are already limited in their nature. For instance, the Rational Method can only predict a peak flow value and not a runoff hydrograph. In other words, there is a need for a standard method that not only can be universally applied but can also predict both the peak discharge and the runoff hydrograph.

The use of computer models did not become widespread in Canada until the last five years and the hierachical approach has not been implemented until quite recently. As a result, very little research has been conducted in the examination of the interfaced use of desk-top methods and computer models. Previous studies (MacLaren Limited 1975, 1976) have compared the manual Rational Method with real measurements for which the method does not really apply; other studies (Burke and Gray 1979) have only shown the discrepancies without explaining the causes. The lack of assessment and investigations of the limitations of desk-top methods have been found even in recent publications (Kibler 1982). There is a great need for the present state-of-the-art to conduct research towards this goal.

1.2 SCOPE AND OBJECTIVES OF RESEARCH

The first objective of the present research is to review critically some of the desk-top methods currently used by engineers for runoff computations. Current desk-top methods for determining the runoff hydrograph and detention storage for the design of urban runoff control facilities are also reviewed.

The second objective is to conduct a comparison of the Rational Method, one of the most widely used desk-top methods in the design of urban drainage system, with SWMM, which is also one of the most widely used computer programs in Canada. The limitations and validity of the Rational Method for runoff quantity computations for sewer designs in small urban watersheds are systematically examined. The criteria for interfacing the Rational Method with SWMM are then reviewed. An improvement of the Rational Method is also proposed.

The third objective is to develop an improved desk-top method for the determination of peak runoff rates and runoff hydrographs for the design of drainage systems in small urban watersheds by means of computer modelled simulations. This is undertaken in order to achieve a consistent and standardized approach between a desk-top method and a computer model for runoff computations. In the present study, the derivation of the proposed "Standard Hydrograph Method - SHM" is based on SWMM.

The final objective is to examine the implications of current urban runoff control policies with particular reference to the "zero runoff increase" and "selective control" criteria. A desk-top procedure for determining detention storage requirements for runoff control is developed using SHM.

A channel and a reservoir routing procedure are suggested for use with pocket calculators for a more refined and simplified design of stormwater management and runoff control facilities.

Chapter II

REVIEW OF DESK-TOP METHODS FOR URBAN HYDROLOGY

2.1 INTRODUCTION

This chapter presents an overview of currently applied and recently introduced desk-top methods of runoff and detention storage computations for urban drainage designs, including the widely used Rational Method, the SCS TR-55 Method, the Modified Rational Method Hydrograph, and other currently available desk-top methods. The limitations of these methods are briefly discussed and special emphasis is given on studies which are carried out for the determination of the two important parameters of the Rational Method, namely, the inlet time and runoff coefficient. A critical review of the manual SCS method for determining infiltration rates using the CN number and proposed improvements are also presented.

2.2 RATIONAL METHOD

The basic concept of the Rational Method dates back to 1851 when Mulvaney, an Irish engineer, published a paper regarding the measurement of rainfall and flood discharges in a given watershed. In his paper, the underlying principles of the Rational Method, including the concept of the time of concentration, are believed to have been implied. The Rational Method was not used in the United States until it was introduced by Kuichling (1889). It was then subsequently introduced by Lloyd-Davies (1906) into England, where it has been often referred to as the Lloyd-Davies Method. Other forms of the Rational Method are known by the names of Caquot's Formula (Normand 1974, Desbordes 1976) in France and Imhoff's Formula in Germany.

The Rational Method has been mainly applied in urban watersheds for the design of pipe systems although it has also been applied in rural watersheds (Chow 1962, Ontario Ministry of Transportation 1981).

The classical definition of the Rational Method, its assumptions, limitations, and method of application are described in detailed in Appendix A.

In summary, the assumptions of the Rational Method are that: (i) for an excess uniform rainfall, the maximum flow is reached at a time that is equal to the time of concentration of flows in the watershed; (ii) the excess rainfall duration is equal to the time of concentration; (iii) the excess rainfall intensity is a constant percentage of the rainfall intensity; (iv) the flow determined is the maximum for the given excess rainfall intensity; and (v) for the estimation of peak flow from a non-homogeneous watershed, a weighted average runoff coefficient is used.

From a hydrologic viewpoint, these assumptions can be seriously challenged. In particular:

1. The runoff coefficient should vary with the time and rainfall characteristics.
2. The time of concentration assumes that the watershed is in saturation or at equilibrium level and runoff begins immediately; it is also affected by the rainfall characteristics.

2.2.1 Current Practices in Canada

Despite its limitations and required assumptions, the Rational Method is still used by many consultants and municipalities in Canada and the United States. A survey conducted by Damas and Smith (MacLaren Limited, 1975) of 36 municipalities in Ontario, has indicated that all of them are applying the Rational Method for storm sewer designs. Enquiries by J.F. MacLaren Limited (1975) and the University of Ottawa IMPSWM program in 1980 have given similar results. These surveys have also revealed other useful information such as the range of runoff coefficients and inlet times used, and the selection of the design frequencies pertinent to the current state of practices of the Rational Method.

Conducted in Ontario, the Damas and Smith Survey has indicated that among the consultants and municipaities surveyed, the range of inlet time used varied from a minimum of 5 minutes to a maximum of 20 minutes. The coefficients of runoff used for residential, commercial and industrial, school and institutional areas, are in the range of 0.4 to 0.49, 0.7 to 0.79, and 0.6 to 0.69, respectively.

Thirty-seven cities across Canada have participated in the survey conducted by James F. MacLaren Limited. Of the 37 cities who answered, only one has not used the Rational Method for runoff computations. Five cities (Montreal, Scarborough, Toronto, Hamilton and Winnipeg) have used both the manual Rational Method and the detailed computer models. The maximum inlet times used vary from 8 minutes (Toronto) to 40 minutes (St. John's). The minimum inlet times vary from 5 minutes in twelve different cities to 20 minutes in Valleyfield and Windsor. Runoff coefficients also vary widely from city to city as shown in Table 2.1. The design frequency for storm sewers of residential and commercial developments in most cities is 1 in 5 years.

It is evident from the surveys that the main discrepancies with the Rational Method are caused by the inconsistencies in determining C and t_i. They are determined in as many different ways as those using it and consequently have resulted in different values and hence designs that vary from city to city.

Table 2.1: Runoff Coefficients Used by Different Cities (MacLaren Limited 1975)

Land Use	Minimum C	Maximum C
Parks	.05 (Ottawa)	.35 (Guelph)
Residential	.20 (Montreal)	.80 (Ottawa, Pierrefonds)
Commercial	.30 (Brantford, Calgary)	.90 (Guelph, Winnipeg)
Industrial	.36 (Vancouver)	.95 (Guelph)

In the survey conducted by the IMPSWM Program at the University of Ottawa, it is observed that the Rational Method has been applied for the design of two systems - the minor (pipe) and the major (street) systems. The design frequency of the minor system ranges from 1 in 2 to 1 in 5 years and the design frequency of the major system ranges from 1 in 25 to 1 in 100 years.

Many methodologies have been proposed by various researchers for improving the accuracies of determining C and t_i. They will be reviewed in the next few sections.

2.2.2 Determination of Runoff Coefficient

Methodologies for determining the runoff coefficient are based mainly on interpreting the parameters affecting the rate of runoff including the percentage of imperviousness, soil characteristics, and rainfall and watershed characteristics. In urban watersheds, it is governed mainly by the percentage of imperviousness as compared with other parameters. In rural watersheds, which have high proportions of permeable surfaces for infiltration, the runoff coefficient is also sensitive to the soil type and the antecedent moisture condition of the soil. Rainfall with high rainfall intensities also contributes to the increase in the value of the runoff coefficient. The main problem in determining C is the lack of a general understanding in relating all of these parameters. Thus most relationships are put forth on the basis of interpreting only one or a few of the parameters.

2.2.2.1 Tabulated C Values

The simplest and most common way of selecting C is by relating it to the characteristics of the watershed. The coefficient is a fixed value which does not vary with time. Tables which give the average C for use in urban drainage designs for various types of surface characteristics and developments have been recommended in the ASCE Design Manual

No. 37 (ASCE, 1970) which can be regarded as one of the best references for general determination purposes (Table B.1, Appendix B). These coefficients are applicable for storms of 5 to 10-year frequencies. For less frequent, high intensity storms, higher coefficients are required (Schaake et al 1967).

A more elaborate procedure that has been applied in Erin and Niagara Counties in New York (1977) considers additional parameters - ground slope, soil type and return frequency (Table B.2). The runoff coefficient obtained from this table could reflect more closely the hydrologic cycle. C values according to soil type have also been proposed by Schwab et al (1971) for rural watersheds (Table B.3). All of these tables are given in Appendix B.

2.2.2.2 Correction of C for Storm Return Period

A simple ratio is used to adjust the runoff coefficient by multiplying the right hand side of the Rational Method formula by a frequency factor 'C_f' related to the return frequency of the storm (Denver Criteria: Wright-MacLaughlin 1969, American Iron and Steel Institute 1980). The equation for the Rational Method therefore becomes

$$Q = C_f C i A \qquad 2.1$$

where C is the runoff coefficient, i is the rainfall intensity in inches per hour, and A is the area of the watershed in acres. For metric units, a conversion factor of 0.000278 is needed to multiply the equation. In this case, the rainfall intensity will be in millimeters per hour and the area of the waterhsed will be in hectares.

For frequent storms with 2 to 10 years return frequency Cf is equal to 1.0, for less frequent storms with 1 in 25 and 1 in 100 years return frequencies Cf will be 1.1 and 1.25 respectively.

2.2.2.3 C in Terms of Rainfall Intensity

One of the first attempts to 'improve' the estimation of C in terms of rainfall intensity has been proposed by Gregory and Arnold (1932). As shown in Figure B.1, Appendix B, relationships between C and rainfall intensity has been made for various soil (packed or loose) and moisture conditions. In Figure B.2, curves dependent on land use parameters in terms of the type of vegetation, development, and surface cover as proposed by Ordon (1973) are shown. Rossmiller (1980) has also proposed an equation to determine the runoff coefficient C in terms of the rainfall intensity and duration:

$$SRO = P - F - I_a \tag{2.2}$$

$$P = i \times \text{Duration} \tag{2.3}$$

$$C = SRO/P \qquad 2.4$$

where SRO is surface runoff, P is precipitation, F is the total infiltration, Ia is initial abstraction which includes interception and depression storage, and i is the average rainfall intensity. Table B.4 gives examples of C determined for different rainfall intensities for a storm lasting 1.0 hour and a completely impervious watershed (Rossmiller 1980).

2.2.2.4 C in Terms of Imperviousness

Schaake, Geyer, and Knapp (1967) have observed that C is highly correlated with the imperviousness after analyzing measurements collected from 20 urban drainage areas in Baltimore. A linear equation, rel~~ating~~ C with the imperviousness (Imp) and slope (S) expressed in percentages, has been obtained:

$$C = 0.14 + 0.65\ (Imp) + 0.05\ (S) \qquad 2.5$$

Schaake et al have recommended that this equation should be used in conjunction with a special equation for calculating the time lag (t_l) for a main drainage channel of length L:

$$tl = 1.05L^{.24}/S^{.16}Imp^{.26} \qquad 2.6$$

An equation used in the STORM model (U.S. Army Corps of Engineers 1975) estimates the runoff rates by means of a volumetric runoff coefficient. It is obtained by considering the percentages of pervious and impervious areas:

$$C_{avg} = C_{perv}(1-I) + C_{imp}.I \tag{2.7}$$

where I is the percentage of imperviousness, C imp is the C value for the impervious area (0.9 for example) and C perv is the C value for the pervious area (0.2 for example). The use of volumetric runoff coefficients will be examined later in Chapter IV..

It is found that the values of C calculated by the second method are close to those calculated by the first one (J.F.MacLaren 1975). The effect of slope is therefore concluded to be negligible in this case.

Although C determined with the type of equations described above does not vary with the storm characteristics, it is felt that it eliminates the rather arbitrary judgement for determining C based on land use characteristics.

2.2.2.5 C in Terms of Rainfall Duration

This relationship has been first observed by Kuichling (1889) who has expressed the runoff coefficient (m) as being directly proportional to the storm duration 't'

$$m = at \tag{2.8}$$

where 'a' is an empirical constant. Horner (1910) has proposed that the relationships of C and storm duration can be developed for both pervious and impervious areas. Similar curves have been developed by Hoad (Fair and Geyer 1959) and Mitci (1974). Curves developed by Mitci have been found to be applicable for soil conditions in Montreal (Mitci 1974). The equations developed by Hoad and Mitci are listed in Table B.5 and all of the curves are plotted as shown in Figures B.3 and B.4.

2.2.2.6 Rossmiller (1980)

Rossmiller (1980) has proposed an equation for estimating C for the design of storm sewers in urban watersheds. This equation accounts for a number of the essential parameters, including land use, soil type, antecedent moisture conditions, recurrence interval, imperviousness, rainfall intensity, watershed slope, and surface roughness, respectively. Each of these parameters, acting in relation with the others, predicts the portion of rainfall which will appear as runoff by:

$$C = 7.2(10)^{-7}\ CN^{3}\ RI^{.05}\ ((.01CN)^{.6})^{-S.^{2}} (.001CN^{1.48})^{.15-I}((Imp + 1)/2)^{.7} \qquad 2.9$$

where CN is the SCS curve number, a dimensionless number between 0 and 100; RI is the return period in years; S is the average ground slope in percent; I is the rainfall intensity in inches per hour; and Imp is the imperviousness expressed in fractions. The SCS curve number is calculated with a weighted average equation based on the soil type and the ratio of imperviousness:

$$CN_{avg} = X_{perv}(1-Imp) + 98.(Imp) \qquad 2.10$$

where X represents the soil type of the pervious areas. Values of X suggested for each hydrologic soil group are shown in Table B.6.

One of the basic parameters governing Rossmiller's equation is the Curve Number (CN). However, the validity of the CN has often been questioned especially concerning its relationship with the antecedent moisture condition (AMC); the SCS method for determining the rate of infiltration has also been criticized. A review of these criticisms will be presented in a later Chapter. Based on these criticisms, it is concluded that Rossmiller's equation should be used with reservations.

2.2.3 Determination of Inlet Time

The time of concentration in the Rational Method is defined as the time required for the excess runoff to travel from the hydraulically most remote point of the watershed to the point of outflow. In an urban drainage system, it is traditionally visualized as composed of two parts - the inlet time 't_i' of flow over land surfaces and the travel time of flow 't_i' through the pipe or channel system (see Appendix A). Since the velocity of overland flow is much smaller than the velocity of flow in the pipe system, the inlet time is relatively long as compared with the travel time when the size of the watershed is small. The estimation of inlet time is also complex. Under urban conditions, the overland flow often needs to travel over a mixture of permeable, grassed (backyards or frontyards) and impermeable, paved (street) surfaces until it reaches the first inlet of the storm sewer system. As shown in Figure 2.1, the inlet time for such a non-homogeneous surface is highly variable and can be represented by any value within the shaded area of Figure 2.1. It is not surprising that most methods for determining the inlet time are not for a mixture but only for either impervious or pervious land surface with identifiable characteristics.

The inlet time depends on many factors such as the length and depth of flow, surface roughness, and rainfall characteristics. Judgemental decisions have often resulted in wrong inlet times thus leading to inaccurate estimations of the rainfall intensities, and hence the peak runoff rates.

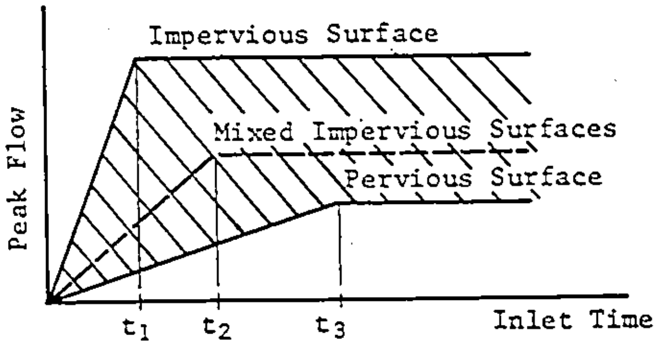


Figure 2.1: The Determination of Inlet Time

Determination of inlet time is also affected by the location of the first upstream pipe section which dictates the length of overland flow. In most urbanized watersheds or residential developments, the 'inlet area' tributary to this pipe section is usually quite small and less than 1 acre. However, in most rural watersheds, the 'inlet area' for which flows travel overland can be quite large. Hence the inlet time for urban areas can be less than 10 to 20 minutes (ASCE Design Manual No.37, 1970), while the inlet time for rural areas can be longer than 60 minutes (Chow, 1962). The type of land use and size of the 'inlet area' for which the formulas apply must therefore be clearly defined.

Since the time of concentration is approximately equal to the inlet time in urban watersheds because of the relatively short travel time of flow in the pipe system, most methods are concerned with determining the inlet time, or more specifically, with the overland flow time, which may be estimated experimentally or theoretically by using a uniform flow approximation.

2.2.3.1 t_i in Terms of Drainage Area

This is the simplest form of equation since it is based on a single parameter, the drainage area, which is raised to some power and multiplied by a constant. This type of equation is empirically determined and some of them have been listed by Rossmiller (1980). Two examples are given below:

$$t_i = 0.9A^{0.6} \tag{2.11}$$

$$t_i = 1.45A^{.045} \tag{2.12}$$

where t_i is in hours and A is the drainage area in square miles. This type of equation is particularly suitable for large watersheds with relatively long times of concentration. Rossmiller has advised that they should be used only in rural watersheds.

2.2.3.2 t_i in Terms of Length and Slope

Kirpich's Formula (1940) is the most used relation in this category.

$$t_i = 0.00078(L/S^{.5})^{.77} \tag{2.13}$$

where t is in minutes, L is the length in feet, and S is the slope in feet per foot, the difference in elevation in feet between the most remote point and the outlet divided by the horizontal distance between them in feet. The equation is based on a study conducted by Ramser (1927) who has collected data from six small watersheds located in Jackson, Tennessee. These watersheds are mainly farmland with steep slopes and relatively dry soils. Rossmiller (1980) has therefore recommended that its use be limited to rural watersheds. He has also recommended that correction factors of 0.2 to 0.4 should be used if the equation is applied in urban watersheds.

2.2.3.3 t_i in Terms of Length, Slope and Roughness or Runoff Coefficient

1. Kerby (1959)

This empirical formula calculates the overland flow time based on the surface roughness, the length of flow in feet and the ground slope

$$t_i = 0.83(nL/S^{.5})^{.467} \tag{2.14}$$

where n is the roughness coefficient obtained from Table B.7, L is the length in feet of overland flow, and S is the slope in feet per foot. It is based on experimental curves derived by Hathaway (1945) who has investigated the drainage characteristics of military airfield runways. L must be less than 1200 ft.

2. Seelye (1968)

$$t_i = 0.16(L/C)^{.5}S^{-.333} \tag{2.15}$$

where L is the length of overland flow in feet, C is the Rational Formula runoff coefficient C, and S is the slope in feet per foot.

2.2.3.4 Kinematic Wave Equation

The Kinematic Wave Equation gives a theoretical interpretation of the overland flow characteristics. It is formulated on the basis of the continuity and momentum equations commonly referred to as the St. Venant or shallow-water wave equations.

Using Manning's formula for normal flow condition, the solution of the equations can be expressed as (Overton and Meadows, 1971):

$$t_i = (0.928/i^{.4})(nL/\sqrt{S})^{.6} \qquad 2.16$$

where i is the depth of runoff, n is the Manning roughness coefficient, L is the length of flow, and S is the ground slope. Consideration of different configurations, including single plane surface, cascade of planes, V-shaped watershed, and converging surface with 100% impervious surfaces has been made by Overton and Meadows (1976). Ragan and Duru (1972) have constructed a nomograph based on Equation 2.16 and evaluated its performance. By comparing the computed times of concentration with those obtained experimentally by Izzard (1946) and the U.S. Army Corps of Engineers (1954), they have found that the values estimated are close.

2.2.3.5 Experimental Determination (Izzard)

Izzard (1946) has conducted a series of experiments to examine the hydraulics of overland flow on paved and turf-surfaces normally found on airport runways. As a result of these tests, it is possible to compute the hydrograph of runoff from a plane surface resulting from different rainfall intensities. Equations that describe the time necessary for the equilibrium condition to occur, when runoff occurs overland, are obtained:

$$t_i = 41\ bL^{1/3}\ i^{-2/3} \qquad 2.17$$

and

$$b = (0.0007i + C_r)S^{-1/3} \qquad 2.18$$

where L is the length of overland flow in feet, i is the intensity of rainfall in inches per hour, S is the slope of the surface and C_r is a resistance coefficient of different types of flow surfaces evaluated experimentally (Table B.8). This equation considers the same parameters as used in the Kinematic Wave Equation. Both equations can be considered to include most of the parameters affecting the time of travel over land surfaces and therefore are more complete than the other equations.

Since the data used by Izzard are obtained from experiments by using a plot measuring 6 ft. wide and maximum of 72 ft. long, the equation is valid only for iL less than 500 (where i is in inches per hour and L is in feet) and very small areas.

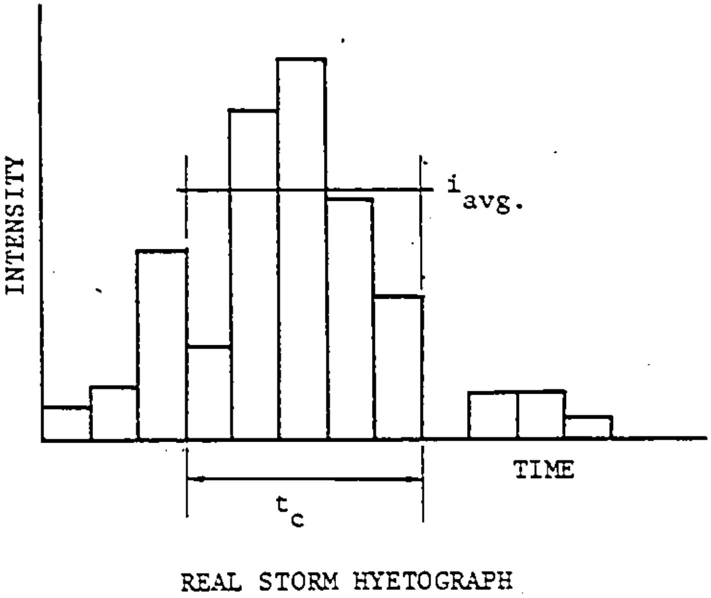
The computer model ILLUDAS (Terstriep and Stall 1974) uses a simplified form of Izzard's equation to determine the time for flow to contribute from the most distant point on the grassed area to the inlet. This is visualized as the distance from the backyard property lines to the street gutter. Izzard's equation is simplified by assuming an average constant rate of rainfall intensity (i=1.0 in/hr.) for all storms and a C_r value for grassed areas consisting of unclipped bluegrass turf (0.046). By substituting these assumptions into Izzard's equation, the equation used in ILLUDAS is

$$t_i = 1.02L^{.4}/S^{.333} \qquad 2.19$$

where t_i is the travel time over grassed area, L is the length, and S is the slope of the grassed area.

2.2.4 Previous Assessment of the Rational Method

The assessment of the Rational Method in the past has mainly been limited to comparing the peak flows calculated by the Rational Method with the peak runoff rates measured from real storm events. Since the Rational Method uses only a 'block rainfall' obtained from the Intensity-Duration-Frequency curve, some assumptions must be used to circumvent the inability of the method to account for the variation of rainfall with time in real storm events. For example, the procedure indicated in Figure 2.2 is used in several studies. According to this method, the average rainfall intensity used by the Rational Method is estimated from a real



1. t_c determined at outlet

2. Maximum average intensity found for t_c from the real storm hyetograph.

3. C and A determined from watershed characteristics

4. $Q_p = C\ i_{avg}\ A$

Figure 2.2: Estimation of Peak Rainfall Intensity From Recorded Rainfall For RM Peak Flow Computation (MacLaren Limited 1975).

storm event by calculating the average rainfall intensity for a duration of the real storm that is equal to t_c and that comprises the peak intensity.

Watkins (1962) has used this method to calculate the peak discharges using 283 storm events observed on 12 catchments and compared them with observed values. The mean absolute error is found to be between 10% and 20% on seven catchments and between 20% and 26% on three. For two of the remaining catchments, mean errors of 47% and 160% are found.

The same procedure is used in a study in the review of Canadian urban drainage practices and design methods by Wisner and Clarke (MacLaren Limited 1975). The Rational Method is tested against measurements for four small catchments located separately in Chicago, Baltimore and Kingston (Ontario). Results show that for the Calvin Park watershed, the Rational Method computed peak flows are 68% higher than measurements. The Rational Method is also tested for two larger areas selected from the publication of Watkins (1962). One watershed is a 611-acre residential development while the other is a 5,270-acre rural watershed. The peak flow values calculated with the Rational Method are found to be two to three times larger than the measured ones. Although no explanation is given for the cause of these differences, it is concluded that the Rational Method is not appropriate for the simulation of runoff from real storm event.

Comparisons with measured flows by other authors have further indicated a wide variation of conclusions. They have been summarized by Colyer and Pethick (1976) in a literature review of storm drainage design methods. Jens and McPherson (1964) have reported applications in Baltimore, St. Louis, Los Angeles and Oxhey and found a mean absolute error in the prediction of peak discharge of 31.5%. Swinnerton et al (1972) have tested the method on 12 storms recorded on motorways and obtained an average absolute error of 83.3%. Chow and Yen (1975) have used the Rational Method to calculate the peak discharge for four storms in Oakdale, Chicago. They have concluded that the Rational Method could be used for quick calculation of peak discharge if high accuracy is not required. However, based on all these studies, Colyer and Pethick (1976) have concluded that the Rational Method is adequate for its respective level of sophistication although improved accuracy can be further achieved.

2.3 DESK-TOP SYNTHETIC HYDROGRAPH METHODS

One of the major limitations of the Rational Method is that it only predicts the peak runoff rate which is a single point on the runoff hydrograph. The development of hydrograph methods such as the unit hydrograph by Sherman (1932) and the synthetic unit hydrograph by Snyder (1938), and the synthetic hydrograph, which is obtained by routing the instantaneous unit hydrograph defined with the time-area method, by Clark (1945), have significantly improved means of estimating runoff.

A special type of synthetic hydrograph methods which assumes that the runoff hydrograph can be approximated by a simple shape has also been used. One such type of simple methods is the Modified Rational Method Hydrograph (Williams 1950) which assumes a triangular runoff hydrograph for a lumped watershed. The use of a triangular-shaped hydrograph for estimating the runoff hydrograph has also been incorporated in the Inlet Method (Kaltenbach 1963) and the Linearized Subhydrograph Methods developed by Chien and Saigal (1974) and Sarikelle et al (1978).

Attempts have also been made to use tabular or graphical methods, based on computer simulations: the SCS TR-55 Method (Soil Conservation Service 1975). The simple dimensionless inlet hydrograph model developed by Ragan et al (1975) is based on the computer model Maryland Linked System Design Method (MLSDM)..

In the next few sections, some of the linearized hydrograph and dimensionless hydrograph methods for synthesizing the runoff hydrograph are reviewed.

2.3.1 Modified Rational Method Hydrograph

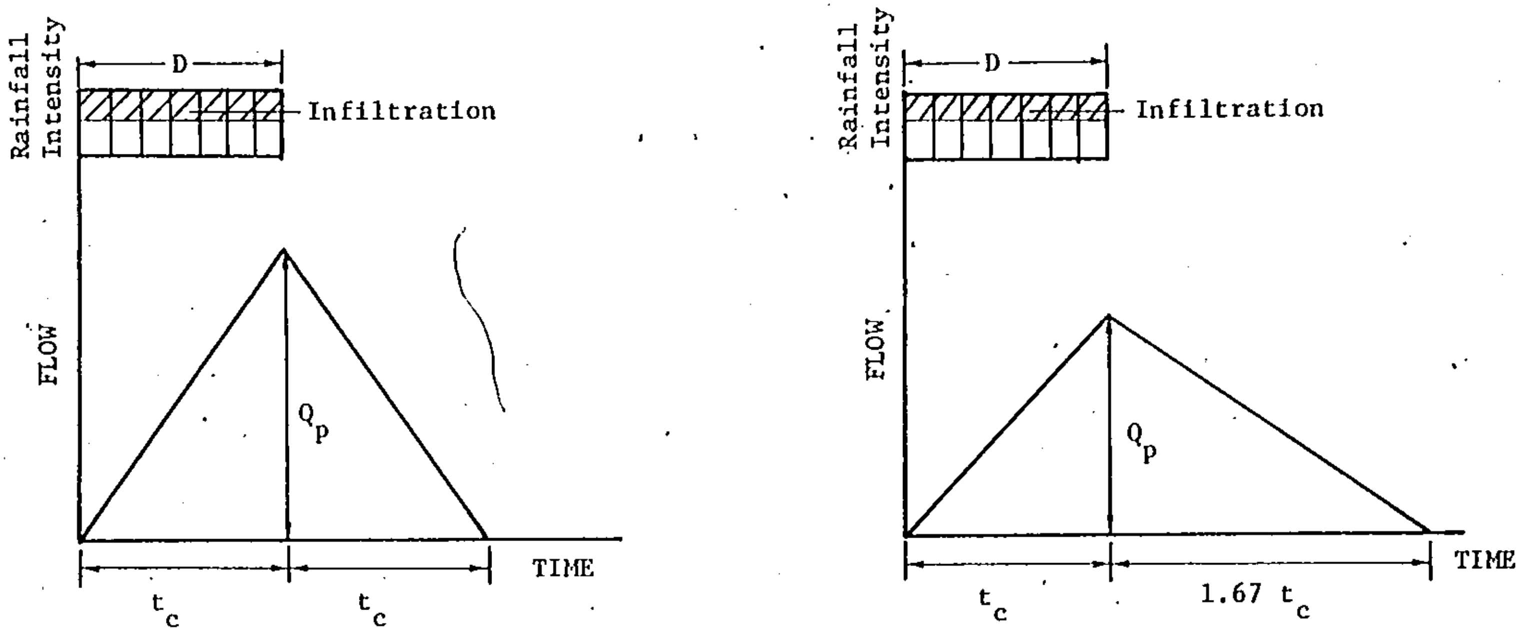
The basic Rational Method can only be used to estimate peak flows. However, the peak flow computed by the Rational Method can be assumed to be the peak of an equilateral triangular hydrograph with a time base equal to 2 t_c (Figure 2.3a, Wanielista 1978). The hydrograph is caused by a rainfall of constant intensity with a duration equal to t_c; the volume of effective runoff is then equal to the area of the triangular hydrograph. It can be calculated by

$$V = CiAD \tag{2.20}$$

where V is the volume of runoff, C is the runoff coefficient, i is the constant rainfall intensity, A is the area of the watershed, and D is the duration of rainfall which is assumed to be equal to t_c.

For closer approximation of the shape of an actual hydrograph, the recession limb of the Modified Rational Method hydrograph can be extended to 1.67 t_c (Figure 2.3b). Since the same runoff volume must be maintained, the peak of this hydrograph will be reduced and equal to 0.75 CiA.

There are still some differences between the shape of the Modified Rational Method hydrograph and the actual runoff hydrograph. In general, the shape of a real hydrograph is described in terms of the peak (Q_p), the time to peak (t_p), the time lag (t_l), the time of concentration (t_c) and



(a) (b)

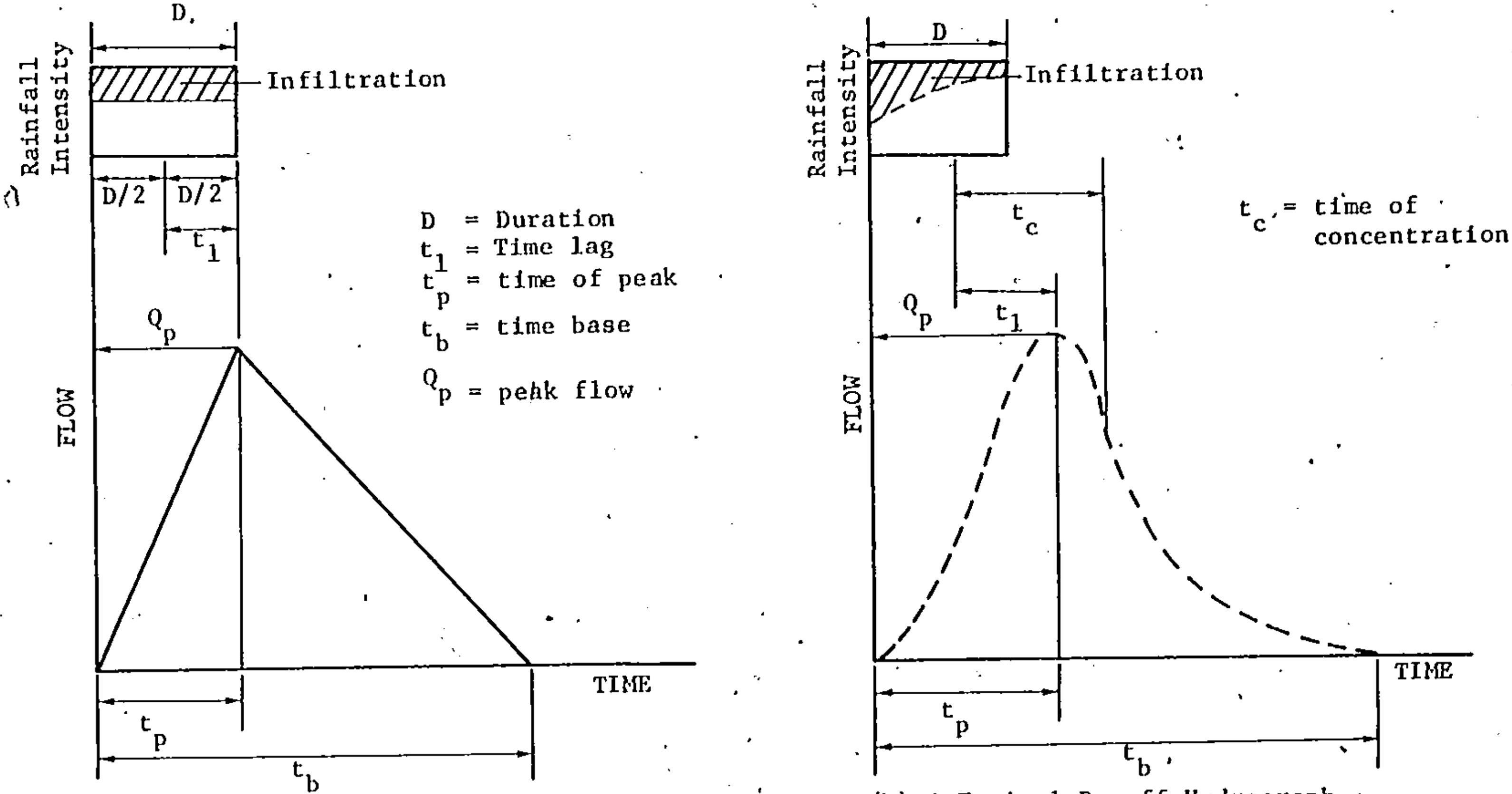
Figure 2.3: The Modified Rational Method Hydrograph.
(Wanielista 1978)

the time base (t_b) (Figure 2.4b) for a storm of duration D. It can be seen from this Figure that for an actual hydrograph, the time of concentration is defined as the time from the centroid of the excess rainfall to the inflection point on the falling limb of the hydrograph where the recession curve begins. For the Modified Rational Method hydrograph, the time of concentration is assumed to coincide with the time to peak. It can also be seen that the hydrograph derived with the Modified Rational Method has a shorter time lag which is equal to D/2.

The use of the Modified Rational Method Hydrograph for determining detention storage volumes will be described in a later section.

2.3.2 Linearized Subhydrograph Method

The linearized subhydrograph method developed by Chien and Saigal (1974) and Sarikelle, Chien, and French (1978) is a simple method of determining the synthetic hydrograph based on the principles of the Modified Rational Method hydrograph. This method also considers the rainfall duration and a rainfall profile. For the development of the linearized subhydrographs, three cases of subhydrograph as illustrated in Figure 2.5 are established with respect to the relative conditions of the rainfall duration and the time of concentration.

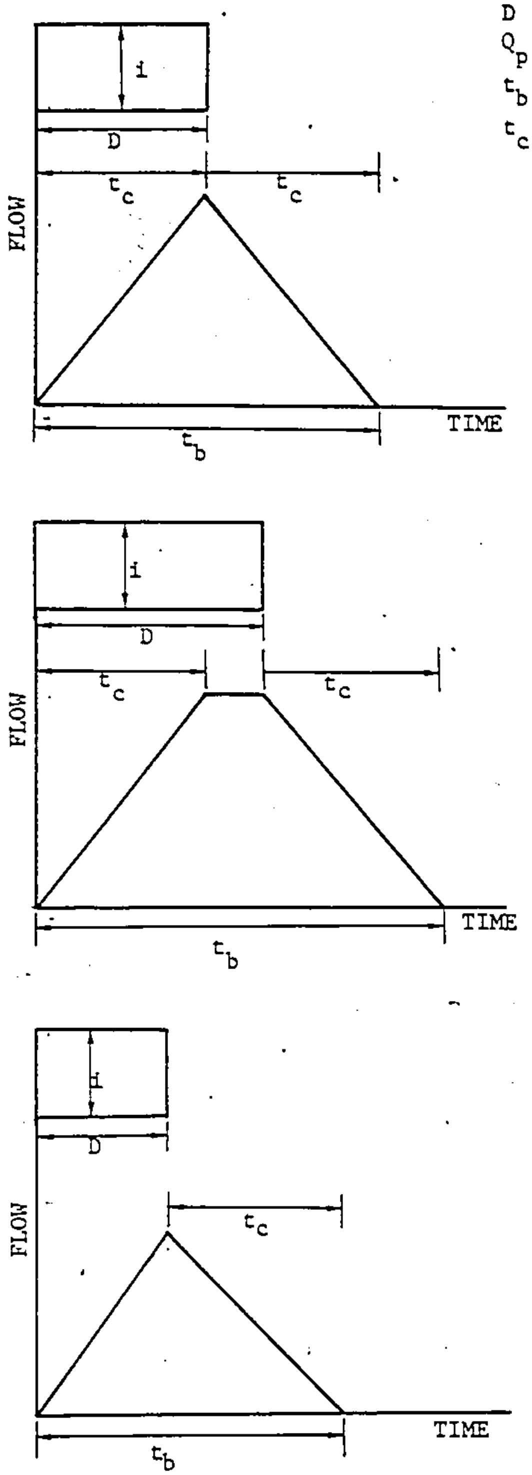


(a) The Modified Rational Method Hydrograph

(b) A Typical Runoff Hydrograph

Figure 2.4: Comparison of Modified Rational Method Hydrograph and A Typical Runoff Hydrograph

C = runoff coefficient
i = rainfall intensity
D = storm duration
Q_p = peak flow, A = area
t_b = time base, V = runoff volume
t_c = time of concentration



Case 1. $D = t_c$
$Q_p = C\ i\ A$
$t_b = 2\ t_c$
$V = C\ i\ A\ D$

Case 2. $D > t_c$
$Q_p = C\ i\ A$
$t_b = D + t_c$
$V = C\ i\ A\ D$

Case 3. $D < t_c$
$Q_p = C\ i\ A \frac{2D}{D + t_c}$
$t_b = D + t_c$
$V = C\ i\ A\ D$

Figure 2.5: The Linearized Subhydrograph Method (Chien and Saigal 1974).

In the first case, the storm duration is equal to the time of concentration of the subwatershed. The peak runoff occurs when flows all of the tributary watershed contribute to the outlet flow; the peak rate of runoff is computed by the Rational Formula and the time base of the subhydrograph is $2t_c$. The volume of runoff resulting from the storm is $CiAt_c$. In the second case, the storm duration is longer than the time of concentration of the subwatershed. Thus, the peak rate of runoff is reached before the end of the storm so that all of the subwatershed is contributing at the peak rate for the part of the runoff period equal to the storm duration less the time of concentration. After this period, the runoff will recede to zero in a time period equal to t_c. The form of the subhydrograph, therefore, is a trapezoid with the peak runoff rate the same as in the first case but the time base of the subhydrograph is longer and equal to $D + t_c$. The volume of runoff is CiAD which is larger than the first case. In the third case, the storm duration is less than the time of concentration of the subwatershed. Thus, the entire watershed is not contributing to flow and the peak rate of runoff is reduced by a proportion of $2D/(D + t_c)$. The volume of runoff is less than both of the previous cases.

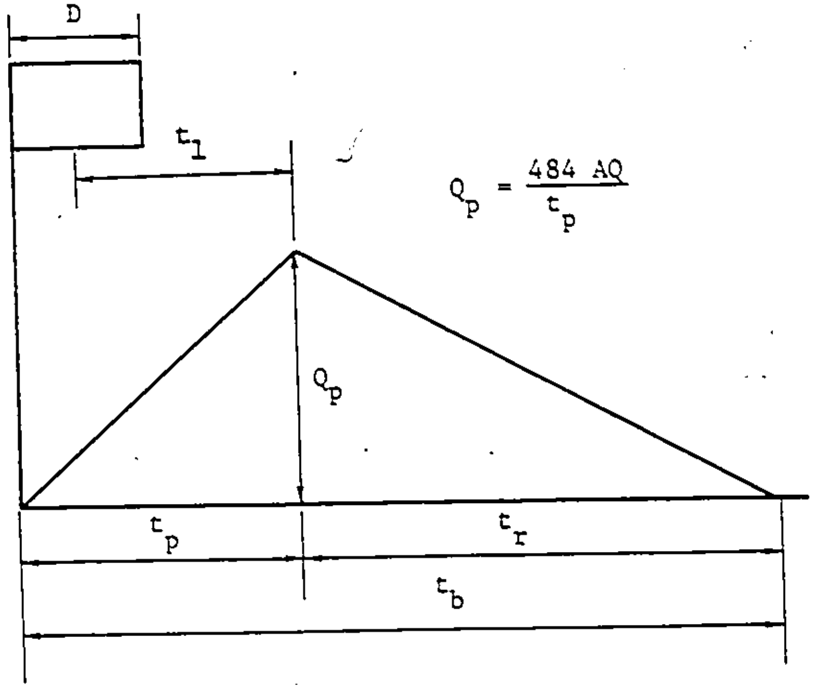
Chien and Saigal have suggested three different equations including Kerby's formula and the Kinematic Wave equation for estimating the time of concentration (see Section 2.2.3.4). The Kinematic Wave equation has been recommended. The Kinematic Wave equation has also been used by Sarikelle et al for estimating the time of concentration.

The weighted runoff coefficient is estimated with Hoad's relations for impervious and pervious areas as shown in Figure B.3, Appendix B (see also Section 2.2.2.5). A simplified time-offset method is adopted for routing, whereby individual subhydrographs are linearly transposed, or lagged, to the point under consideration by the average travel time.

2.3.3 Use of SCS Unit Hydrograph Method for Peak Flows

The U.S. Soil Conservation Service (1957) hydrograph method is developed with the simple assumption that a hydrograph can be represented in the form of a triangle as shown in Figure 2.6. For each increment of excess rainfall, the length of the recession limb of the runoff hydrograph is approximated by $1.67t_p$ and t_p is approximated by the sum of $D/2$ and $0.6t_c$. This rainfall increment is limited not to be greater than $0.25t_p$. The peak (Q_p) of the incremental hydrograph is determined by the following equation:

$$Q_p = 484\ AQ/t_p \tag{2.21}$$



D = duration of excess rainfall

t_l = time lag of the watershed (the time from the center of mass of the excess rainfall to the peak of runoff)

t_p = time to peak (hrs)

t_r = time to recede (hrs)

t_b = time base (hrs)

Q_p = peak runoff rate (cfs)

A = area (sq. miles)   Q = inches of runoff

$t_r = 1.67\ t_p$

$t_p = \frac{D}{2} + t_l$

Figure 2.6: The S.C.S. Unit Hydrograph Method (S.C.S. 1957).

where A is the area of the watershed in square miles, Q is the runoff volume in inches, and Q_p and t_p are in cubic feet per second and hours, respectively. The constant 484 has been found to vary from 300 in very flat swampy areas to 600 in steep terrain (American Iron and Steel Institute 1980) when it is calibrated with measurements.

Incremental runoff hydrographs are first produced for the runoff excess determined during each time step. These hydrographs are then superimposed and a total runoff hydrograph is produced. A more detailed discussion of the SCS-CN procedure for determining the runoff excess will be given in the next Section.

It is noted that Equation 2.21 applies only for a unit rainfall and it cannot be used to estimate the peak runoff of any given amount of runoff volume Q resulting from any given rainfall pattern.

2.3.4 SCS TR-55 Method

A more sophisticated method currently used is the SCS TR-55 Method (SCS 1975) developed by the U.S. Department of Agriculture. The TR-55 Method employs several tables, graphs, and charts for estimating runoff volumes, peak discharges, runoff hydrographs, and volumes for stormwater detention storages. The tabular method, which makes use of Table 5-3 of the manual, is used for computing peak discharges from urban areas; it permits the development of composite hydrographs at any point within the watershed by dividing the watershed into subareas and calculating the time of concentration for each subarea and the travel time through each reach. This method is especially applicable for use in non-homogeneous watersheds such as partly urbanized areas; it can also be used to determine the effects of structures, including channel modifications, at different locations in a watershed. The ordinates of the hydrographs are expressed in terms of cubic feet per second per square mile per inch of runoff. The graphical method, which uses Figure 5.2 of the manual, is used to estimate the peak discharge from urban areas (cfs per square mile per inch of runoff) using only the time of concentration and is applicable to a watershed where runoff characteristics are uniform and channel routing is not required. Finally, the chart method, which uses the charts of Appendices D and E of the manual, is also used to estimate the peak discharge from agricultural areas (cfs. per inch of runoff) for a given size of area for various CN values and ground slopes. Adjustment factors for percent impervious areas and modified hydraulic length are required for applications in urbanized watersheds.

All of the TR-55 methods are developed based on computer runs of the TR-20 model (SCS 1965) using the standard SCS Type-II 24-hour design storms. The methods are most useful for planning purposes; however, it has been suggested that they can be used for design if the limitations of the methods are recognized and not misinterpreted. A major constraint of the TR-55 methods is that it is limited to the use of the SCS Type-II 24-hour design storms. Both the peak discharge and the shape of the hydrograph may therefore only be used for this type of storms.

Considerations regarding the application of SCS-CN method for computing infiltration have often been examined. The infiltration is determined with relationships based on the SCS CN value which are as follows:

$$Q = \frac{(P-I_a)^2}{P-I_a+S} \tag{2.22}$$

$$I_a = 0.2S \tag{2.23}$$

$$S = (1000/CN) - 10 \tag{2.24}$$

where Q is the total depth of runoff in inches, P is the total rainfall precipitation in inches, I_a is the initial abstraction, S is the potential abstraction, and CN is the runoff curve number which is related to the soil and cover conditions of the watershed. According to SCS, CN can be linked to the moisture condition of the soil with an Antecedent Moisture Condition (AMC) level. The level of AMC can be separated into three discrete classes: AMC I (dry), II, and III (saturation). It is found that in some applications, especially those with low rainfall volumes, over-estimated rainfall losses and under-estimated peak discharges may result.

The CN method is found to give very low volumes of runoff for AMC I with a 60 minute duration storm when compared with that modelled by ILLUDAS (Burke and Gray 1979). After analyzing runoff volumes of 585 storm events collected from 36 watersheds in the Tennessee Valley, Bales and Betson (1981) have found that the CN values determined based on measurements are comparatively higher than those suggested by the SCS on the basis of the soil type and land use of the watersheds. In the later case, CN determined by assuming both the AMC II and AMC III conditions are found to have underpredicted the runoff volumes. Studies conducted by Altman et al (1980) involving six watersheds in Texas have shown similar results.

The use of Curve Numbers for calculating infiltration rates have therefore been questioned by many researchers including Aron et al (1977), Hawkins (1978, 1980), Rallison and Cronshey (1979), Wisner and P'ng (1980). Some of the questions often asked are: (i) why is the initial abstraction equal to 0.2S; (ii) when is storm intensity more important than AMC; and (iii) what probability levels are associated with the envelopes in defining AMC I and AMC II.

Golding (1979) has found that the initial abstraction value which is equal to 0.2S is even larger than the total recorded rainfall in many cases. For areas with low CN, the SCS equation may therefore underestimate the amount of runoff volume and runoff rates significantly. He has therefore suggested that the initial abstraction should be varied on the basis of CN ranging from 0.075S to 0.2S. Aron et al (1977) have recommended that a value of 0.1S or lower should be used. The use of separate simulation of runoff from impervious and pervious areas has also been recommended (Wisner and P'ng 1980).

The exceedence probability of the antecedent moisture condition has been recently investigated in a study carried out jointly by the Department of Civil Engineering, University of Ottawa, and the Institute of Rural Engineering, Ecole Polytechnique School of Lausanne (EPFL) for a watershed located near Geneva, Switzerland. By relating CN with the Antecedent Precipitation Index (API) using rainfall and runoff data collected, the exceedence probability of CN can be defined. The study will be presented in a later Chapter and in Appendix C.

Criticisms regarding the application of the SCS TR-55 methods for runoff computations have also been recently discussed by McCuen (1981) and Wisner et al (1981). They are summarized as follows:

1. Runoff from Observed Storms

The complete set of data from a real watershed, the Gray Haven watershed, Baltimore, has been used by Wisner et al for analysis. The mean CN determined with the measured storms and runoff volumes on the watershed is higher than the computed average CN according to the TR-55 manual. Thus, it is concluded that the method may lead to underestimated total runoff volumes for this watershed.

2. Peak Discharge Estimation

Wisner et al have compared the graphical and chart methods for determining the peak discharge in terms of the time of concentration and the area of the watershed, respectively, for data representing hypothetical watersheds ranging in sizes from 10 to 160 acres. It is concluded that different results are obtained by the two methods; the chart method giving peak discharges in terms of area seems to have significantly underestimated peak discharges.

McCuen has indicated that the time of concentration used for determining the peak discharge should be calculated only with the graph provided in the manual. When it is applied in urban watersheds, it should be properly adjusted for impervious area and hydraulic length using the Tables given in the manual.

3. Channel Routing

The TR-55 tabular method is a method for estimating the total runoff hydrograph for any point in a watershed. The TR-20 program, which uses the Convex routing method, is used to calculate the routed runoff hydrographs through stream channels. Wisner et al have found that the tabular method underestimates the routed flows as compared with the Convex method which is carried out manually and the computer program EXTRAN, which uses the full dynamic equations for routing. It is therefore concluded that the Convex method, which is as simple to use as the TR-55 tables, is more accurate. It does not have any limitations regarding the storm profile. The Convex routing method can be further simplified by the use of programmable calculators.

4. Other Comments

One of the main criticisms of Wisner et al is that the SCS TR-55 manual does not convey all the facts and theoretical basis pertaining to the derivation of the various diagrams and tables. Potential users may therefore follow a passive and noncritical approach in their applications. McCuen points out that in a recent study, approximately 50 percent of the users have misapplied the TR-55 procedures.

2.3.5 Dimensionless Hydrograph Method

The dimensionless hydrograph is a hydrograph for which the ordinates are expressed in terms of a relatively simple relationship by the ratio of discharge to peak discharge (Q/Q_p) versus the ratio of time to time of peak (t/t_p):

$$Q/Q_p = f(t/t_p) \tag{2.25}$$

where Q is the runoff rate at times other than the time of peak. The time of peak (t_p) is defined as the time when runoff starts to occur until the time when the runoff hydrograph reaches its peak discharge rate (Q_p). Hence, at the peak of the dimensionless hydrograph, both Q/Q_p and t/t_p are equal to unity (i.e. $Q(t_p)=Q_p$).

Use of the dimensionless hydrograph for synthesizing runoff has been adopted by Izzard (1946) and the Soil Conservation Service (1972). Some of the more recent attempts are by Wu (1963), Ragan et al (1975), and Stephenson (1981).

The dimensionless unit hydrograph developed by Izzard (1946) has been mainly determined from experimental data (Section 2.2.3.5), while that developed by SCS has been based on the analysis of a large number of unit hydrographs from real watersheds varying widely in sizes and geographical locations. The SCS dimensionless unit hydrograph can be considered as an improvement of the simple triangular shaped SCS hydrograph described in Section 2.3.3.

Wu (1963) has shown that a dimensionless unit hydrograph can be synthesized by means of a two-parameter gamma function expressed in the following form:

$$Q(t)/Q_p = (t/t_p)^{n-1}\left[e^{(1-n)}\right]^{(t/t_p-1)} \qquad 2.26$$

where Q(t) is the discharge rate at time t, Q_p is the peak discharge at time of peak t_p, n is a parameter accounting for the storage and routing effects of the watershed. This equation has been applied by Williams and Hann (1973) for generating the rising limb and the peak portion of the synthetic unit hydrograph used in the HYMO model. A comparison of the dimensionless unit hydrographs derived by Izzard, the Soil Conservation Service, and Williams is shown in Figure 2.7. It is noted that Izzard's dimensionless unit hydrograph model is mainly for sheet flows over plane surfaces.

The dimensionless hydrograph developed by Ragan et al (1975) is distinctly different. While the previous ones have been mainly synthesized as dimensionless unit hydrographs (for a unit rainfall), the latter has been synthesized based on a complete rainfall event. Therefore, its shape incorporates the shape of the input rainfall hyetograph. It has been derived with experimental simulations with conceptual watersheds using a computer model, the Maryland Linked System Design Model (MLSDM), and with design storms which have been distributed with Keifer and Chu's

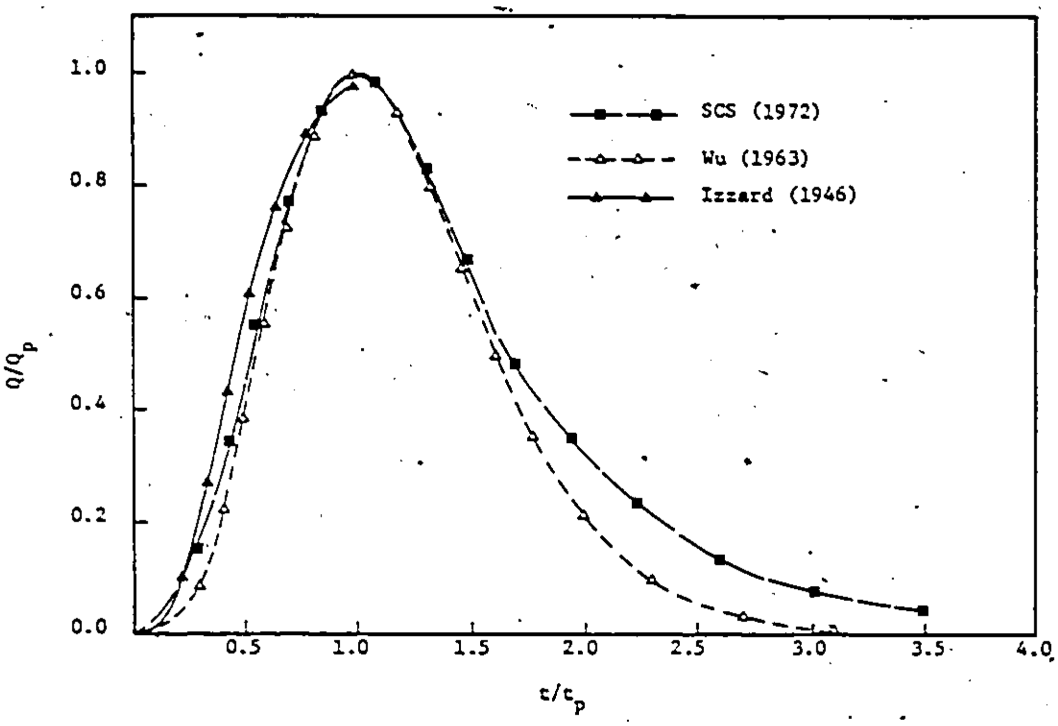


Figure 2.7: A Comparison of Dimensionless Unit Hydrographs

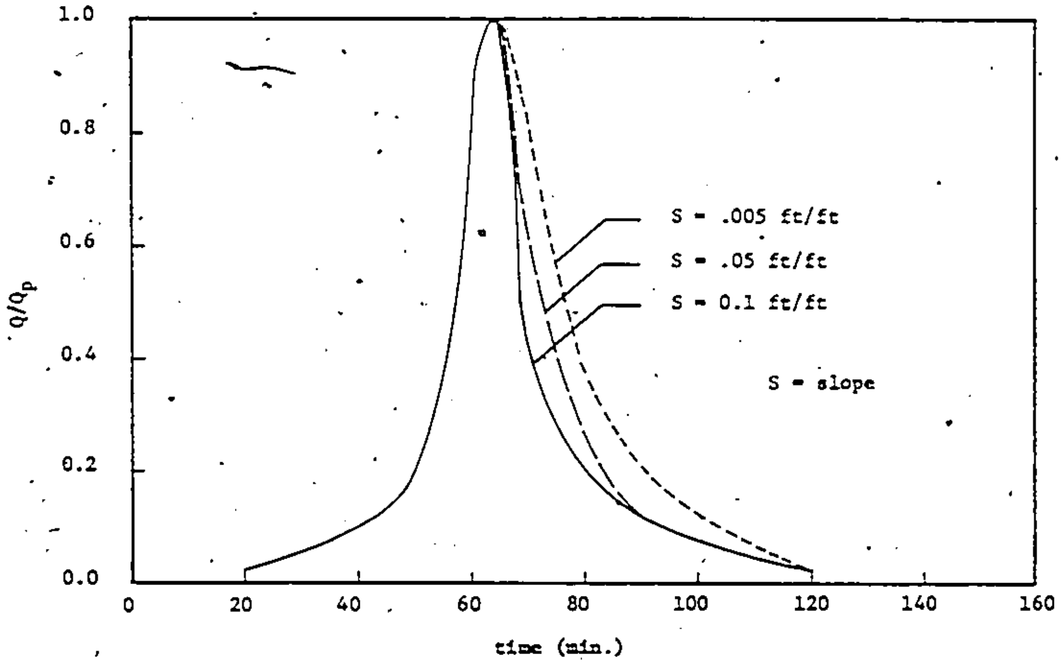


Figure 2.8: Dimensionless Hydrograph Developed with MLSDM by Ragan et al (1975)

method (1957). This approach is quite similar to the SCS TR-55 method described in the last Section, for which simulations have been carried out using the SCS TR-20 computer model for design storms which have been distributed with the SCS Type-II 24-hour design storm distribution.

The MLSDM has been used to conduct a series of 100 numerical experiments on a small hypothetical watershed which drains into an inlet located at the end of a curb and gutter channel. The length of the watershed has been varied between 200 feet and 600 feet. Different average ground slopes have been considered and different percentages of imperviousness have been employed. The rainfall input consists of design storms with 160-minute duration for different return periods. Equivalent lengths of overland flow are used for each experiment.

By analyzing all of the computer simulations, Ragan et al have found that the method is able to construct a common dimensionless hydrograph. The dominant parameters affecting the shape of the runoff hydrograph have been found to be slope and percentage of imperviousness as compared to depression storage, length, and width of the watershed. The time to peak of the runoff hydrograph has been found to be insensitive to most of the parameters such as the imperviousness, the rainfall characteristics, and the length of travel (which is relatively short); it has varied mainly between 61 min. and 72 min. and is very close to the time to peak intensity of the design storm which has the peak intensity at 60 minutes. As shown in Figure 2.8, the rising side of the dimensionless hydrograph is represented by a single curve and the recession side of the dimensionlss hydrograph is represented by three slightly shifted curves corresponding to various slopes. It is noted that the time axis is expressed in terms of the true time and not the ratio of t/t_p.

A number of tests have been carried out by Ragan et al (1975) to test the sensitivity of peak flows to lower initial and final infiltration rates of 0.7 in/hr. and 0.3 in/hr., respectively, using storms with low return periods (less than 1 in 10 years return period). It has been concluded that for the imperviousness ratios of 30%, 50%, and 70%, the increases in peak flows were only 7%, 5%, and 3%, respectively. Thus it has been concluded that the assumed values of infiltration rates used originally to develop the dimensionless hydrograph were appropriate. However, no tests have been conducted for 1 in 2 year storms. Ragan et al have suggested that changes in the infiltration rates for these more frequent storms may cause changes in the shape of the dimensionless hydrograph.

By means of a model that is based on the solutions of the Kinematic wave equations, Stephenson (1981) has also been able to develop a dimensionless hydrograph for a complete excess rainfall hyetograph that is triangularly shaped.

While the traditional unit hydrograph approach is based on a unit rainfall, the dimensionless hydrographs developed by Ragan et al, Stephenson, and the SCS TR-55 method use a complete design storm event. They have therefore adopted a simplified procedure for defining the runoff hydrograph. For the determination of the runoff hydrograph, they do not directly use the convolution procedure, which means a procedure of calculating the incremental hydrograph for the runoff excess during each rainfall increment and superimposing them into the total runoff hydrograph. In this case, the runoff hydrograph can be easily derived by just knowing the peak discharge rate (Q_p) and the time of peak (t_p) of the watershed.

2.4 DESK-TOP METHODS FOR SIZING DETENTION

Various methods, both structural and nonstructural, can be used for the control of runoff from urban developments (Soil Conservation Service 1975). One of the simplest methods for the minimization of runoff increases is by means of detention storage. In an urban system, it can be distributed into different elements of the development such as parks, parking lots, roofs and sections of the sewer network in the form of underground on-line storage (Poertner 1974). A combination of these storage facilities serves as an effective means for the control of increased runoff after urbanization. Park storage, in particular, has been successfully included for the control of urban runoff in new developments located in Toronto, Canada (Wisner et al 1981).

The "dual storage" concept, which is a combination of park storage and underground storage in oversized storm sewers, is also applied in some new developments. The concept is based on the recognition that urban drainage systems are composed of two interconnected units:

1. the "minor" system - consisting of storm sewers for conveying flows during minor storms with high return frequencies such as 1 in 2 and 1 in 5 years,
2. the "major" system - consisting of streets and channels for conveying flows during major storms with low return frequencies such as 1 in 25 and 1 in 100 years.

The "dual storage" concept reduces runoff by storing flows in the "minor" system by means of oversized sewers and storing flows in the "major" system by means of depressions in parks. The detailed design and analytic aspects of the "dual storage" and "dual drainage" concepts have been studied by Kassem (1982).

2.4.1 Determination of Detention Storage

Basically, the procedures required for the computation of detention storage involve the prediction of the runoff hydrograph and the calculation of the volume of detention storage required to reduce the peak runoff rate to a smaller value. The inflow hydrograph is routed through a detention storage by means of the reservoir routing procedure in order to obtain the outflow hydrograph. This lengthy reservoir routing procedure is best suited to computer analysis. In the planning stage, where only an indication of whether or not a detention facility is feasible, complete analysis may not be necessary. This suggests that a simple method which uses only assumed and simple shapes of inflow and outflow hydrographs may be employed.

A multitude of simple methods are available for the planning design of detention facilities. According to Donahue et al (1981), some of the simple methods currently used for planning purposes include ILLUDAS (Terstriep and Stall, 1974), Abt and Grigg (1978), Wycoff and Singh (1976), and the SCS TR-55 Method (Soil Conservation Service, 1975). Other approximate methods that are available include the Modified Rational Method Hydrograph (Poertner, 1974) and Bouthillier and Peterson (1978). A review of the assumptions and procedures that form the basis of some of these methods is provided in the next few sections.

2.4.2 Modified Rational Method Hydrograph

A triangular shaped Modified Rational Method hydrograph that is described in Section 2.3.1 will be resulted if the duration of the rainfall is equal to the time of concentration. If the rainfall duration is longer than the time of concentration, a trapezoidal hydrograph instead will be resulted (see Section 2.3.2). The peak of this trapezoidal hydrograph is calculated by CiA. However, this peak is less than the peak of the triangular hydrograph because the average intensity of rainfall is less when the duration of the rainfall increases. Thus, for any rainfall I.D.F. curve, one can draw a family of trapezoidal hydrographs as shown in Figure 2.9, with each hydrograph corresponding to a longer duration of rainfall (Poertner 1974, Wanielista 1978).

The rising and falling limbs of the trapezoidal hydrograph, in each case, are equal to the time of concentration of the watershed. The falling limb begins at a time D which is the duration of the storm. When such a family of hydrographs is developed, the maximum detention volume required can be computed for a given outflow rate through an iterative procedure. The storage volume is the maximum area below one of the trapezoidal hydrographs and above the plotted allowable outflow rate. It is assumed that the maximum area and therefore the largest volume of runoff is the most critical size for the design of the detention facility.

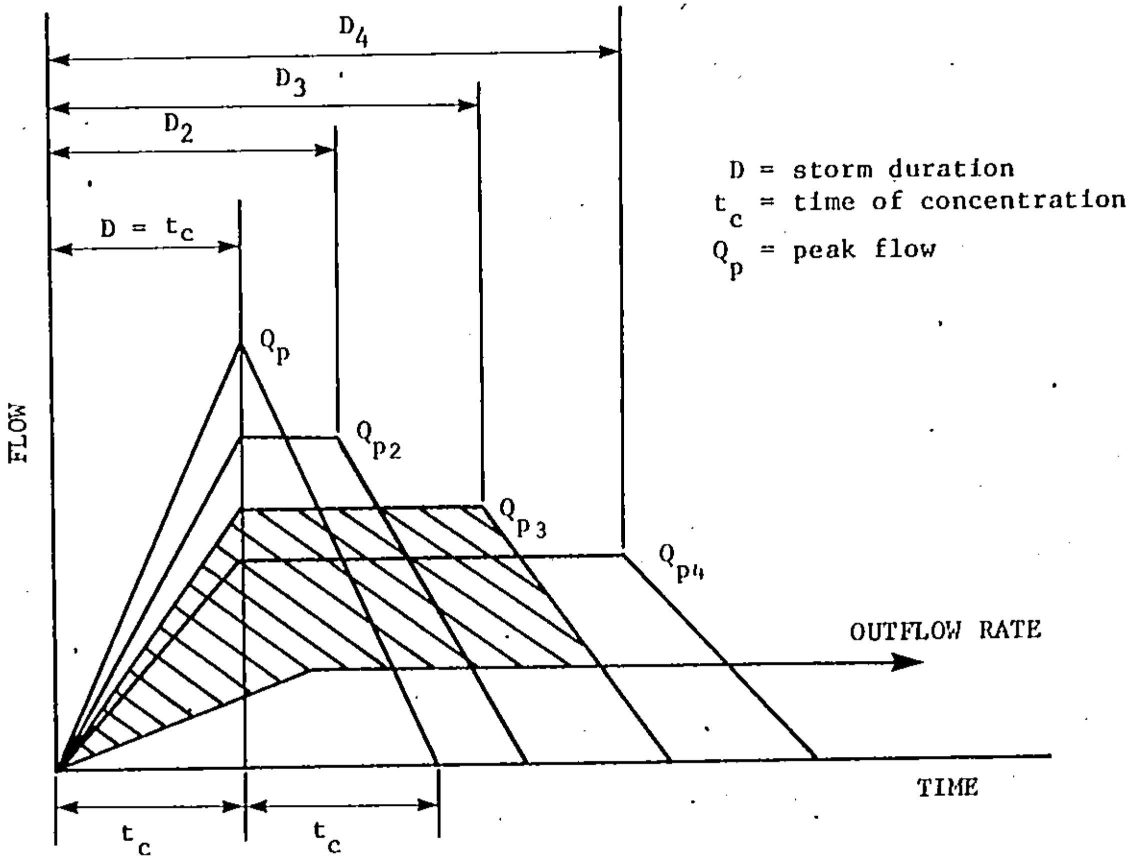


Figure 2.9: Family of Hydrographs Formed With The Modified Rational Method Hydrograph.

2.4.3 U.S. Soil Conservation Service TR-55

Chapter 7 of the Technical Release No. -55, or TR-55 (SCS, 1975), provides a simple method for "analyzing the effects of storage reservoir on peak discharges". The computer program SCS TR-20 (SCS, 1965) is used first to evaluate the average storage and routing effects using many structures. On the basis of these results, two simple relationships for estimating the average storage requirements are formulated.

The accuracy of these two types of curves depends on the relationship between the available storage, the inflow hydrograph, and the volume of inflow. When only a small volume of detention storage is available, the shape of the outflow hydrograph is very sensitive to the rate of rise of the inflow hydrograph. Conversely, when a large volume is available for detention storage, the shape of the inflow hydrograph has less effect on the outflow hydrograph which, in this case, is controlled by the hydraulics of the outlet device. Hence, for peak outflow rates through pipe spillways that are less than 300 cfs per square mile (csm) of drainage area and weir spillways that are less than 150 csm, the detention storage in watershed inches is estimated using the volume of runoff in inches for various release rates. For peak outflow rates through pipe spillways that are greater than 300 csm, the ratio of the volume of storage to the volume of runoff (V_s/V_r) is given as a function of the ratio of the peak rate of outflow to the peak rate of inflow (Q_0/Q_i).

The TR-55 method has been recommended by Donahue et al (1981) even though it has consistently given significantly higher results (approximately 70% to 80% higher) when compared with other models, including ILLUDAS (Terstriep and Stall 1974), Abt and Grigg (1978), and Wycoff and Singh (1976)! His recommendation is based on close comparisons obtained with a design model developed by Bondelid and McCuen (1979). It is however felt that the comparisons are close because both methods use the SCS TR-20 program for reservoir routing; the validity of his recommendation is therefore doubtful.

Although their comparisons have not revealed the accuracy of the TR-55 method, Donahue et al have felt that the main objective of the study is to identify the relative differences between methods so that regulatory agencies or practitioners can decide whether or not a method is conservative when it is being used in the planning of stormwater management detention storages.

2.4.4 Wycoff and Singh (1976)

An idealized model, which is based on triangular inflow and outflow hydrographs, is developed as an approximate method for the hydrologic design of small flood detention reservoirs. As shown in Figure 2.10, the ratio of storage volume (V_s) to runoff volume (V_r) can be expressed as a direct function of the ratio of peak outflow rate (αQ_p) to peak inflow rate (Q_p) and is independent of all other parameters:

$$V_s/V_r = 1 - \alpha Q_p/Q_p \tag{2.27}$$

Equation 2.27 can be used to obtain an estimate of the volume of storage required for a certain controlled peak outflow rate for a known inflow hydrograph that is triangular. However, if the inflow and outflow hydrographs are curvilinear as shown in Figure 2.11, a more generalized model is needed. Considering the relationship established by Equation 2.27, Wycoff and Singh have proposed a model with the following relationship:

$$\frac{V_s}{V_r} = C\{1 - \frac{\alpha Q_p}{Q_p}\}^x \{\frac{t_b}{t_p}\}^y \tag{2.28}$$

where C, x, and y are constants, tb is the time base of the inflow hydrograph which is defined as the time from the beginning of rise to a point on the recession limb where the flow rate is equal to 5% of the peak flow (see Figure 2.11), and tp is the time to the peak of the inflow hydrograph. The constants C, x, and y are derived empirically. A number of selected inflow hydrographs are routed through various

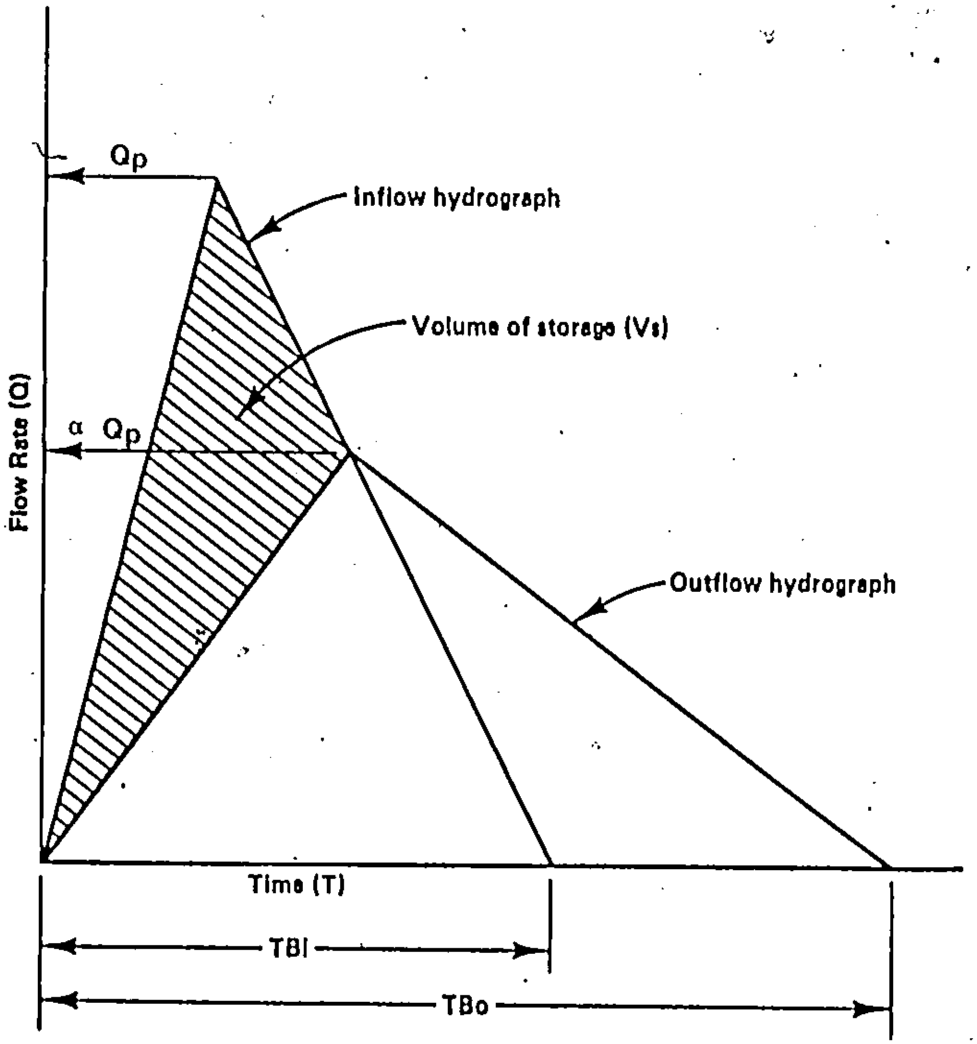


Figure 2.10: Triangular Inflow-Outflow Hydrographs.

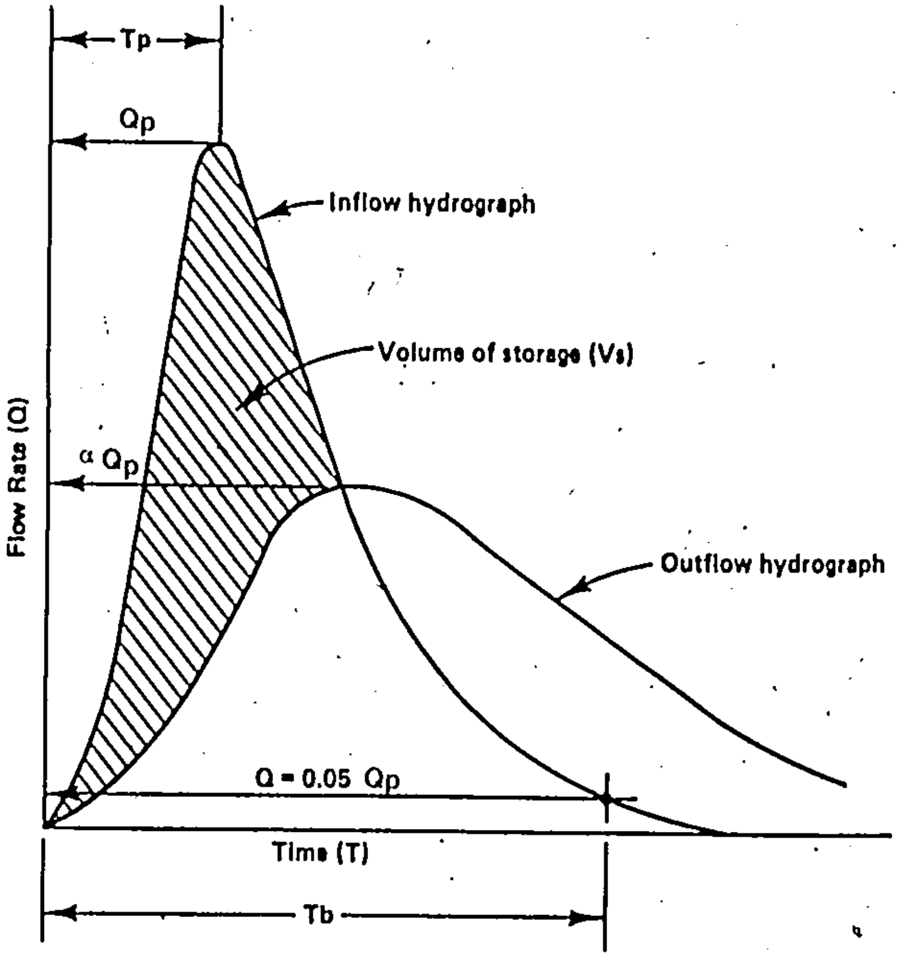


Figure 2.11: Curvilinear Inflow-Outflow Hydrographs.

(Wycoff and Singh 1976)

reservoir and spillway combinations using the HYMO model. All of the parameters (V_s, V_r, Q_o, Q_i, t_b, and t_p) are measured and values of V_s/V_r, $(1- \alpha Q_p/Q_p)$, and t_b/t_p are computed for each routing. The following relationship is obtained using a regression analysis of all the data:

$$V_s = \frac{1.291 V_r (1-\alpha)^{.753}}{(t_b/t_p)^{.411}} \qquad 2.29$$

where α is the percentage of release rate. Since the equation is derived empirically, it is applicable mainly when the parameters Q_o/Q_i vary from 0.15 to 0.89 and t_b/t_p vary from 2.5 to 4.9, respectively.

By verifying with independent flood routing data of an urban stormwater management master plan developed in Florida, it has been concluded that the model is accurate enough for most preliminary design purposes.

2.4.5 Bouthillier and Peterson (1978)

The method has been developed based on four shapes of inflow hydrographs as shown in Figure 2.12. The peak of the inflow hydrograph has been estimated in terms of the total rainfall precipitation (P) and time base (t_b) for 100% runoff. The time base of the inflow hydrograph has been assumed to be equal to the storm duration, which varies from 1 hour to 10 hours; the total rainfall versus rainfall dura-

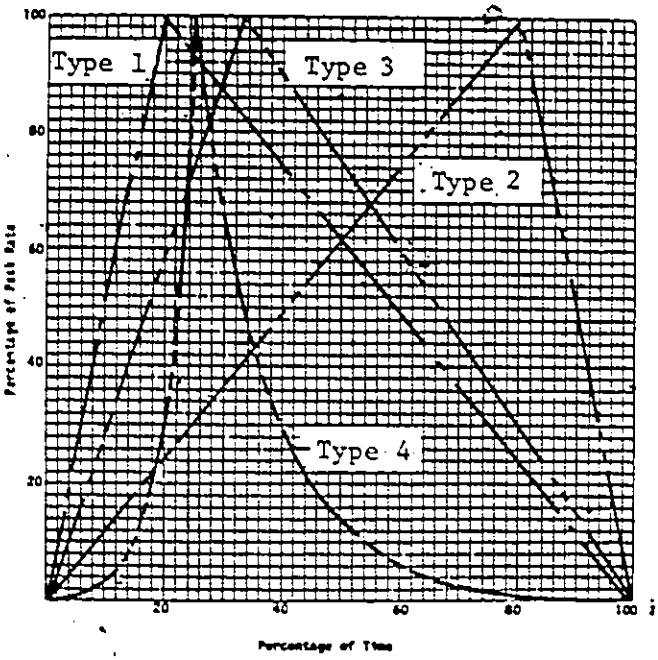


Figure 2.12: Inflow-Hydrographs Assumed by Bouthillier and Peterson (1978).

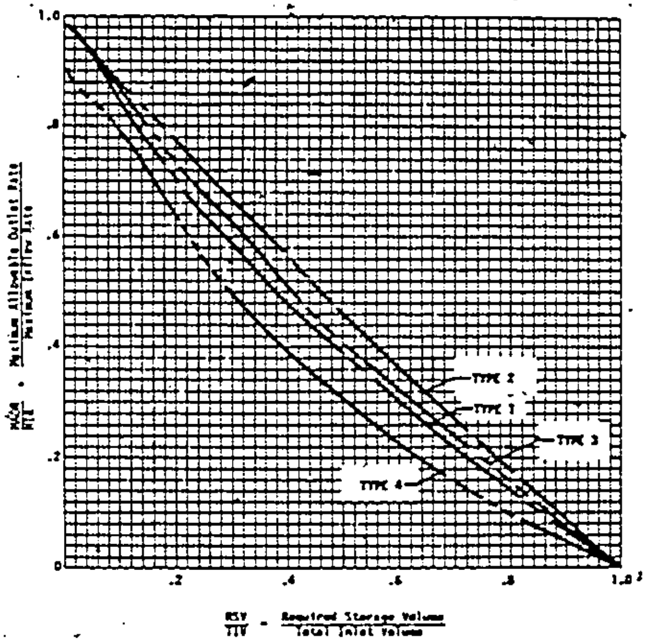


Figure 2.13: Composite Plot of Ratio of Maximum Allowable Outlet Rate to Maximum Inflow Rate versus Ratio of Required Storage Volume to Total Inlet Volume (Bouthillier and Peterson 1978).

tion has been determined with the I.D.F. curve for the Edmonton area. Hence, the peak runoff rate (Q_p) per unit area in inches per hour has been derived by means of the following equations:

1. Triangular shaped inflow hydrograph (Types 1,2, and 3)

$$Q_p = P \times 2.0/t_b \tag{2.30}$$

2. "Cusp" shaped inflow hydrograph (Type 4)

$$Q_p = P \times 6.3/t_b \tag{2.31}$$

Computer simulations have been carried out to route the various types of hydrographs for different storm return frequencies through a reservoir having a fixed physical characteristic and a bottom pipe outlet. The ratios of required storage volume to total inflow volume and peak outflow rate to inflow rate have been determined for each case and the relationships as plotted in Figure 2.13 have been obtained. It can be observed that the Type 2 hydrograph, which has a delayed peak, is more conservative since it requires more storage volume as compared with other types.

2.4.6 Abt and Grigg (1978)

A desk-top method capable of sizing one or a series of detention reservoirs has been developed by Abt and Grigg. The inflow and outflow hydrographs have been assumed to be triangular and trapezoidal, respectively. The rising limbs of the inflow and outflow hydrographs have been assumed to coincide until the limiting outflow rate has been reached (Figure 2.14). The duration of the recession limb of the inflow hydrograph can be modified by a ratio m (i.e. duration $m*t_c$). For a single detention reservoir, the volume of storage, V_s, is given by:

$$V_s = 0.5(t_c + mt_c)Q_p(1-\alpha)^2 \frac{60}{43560} \qquad 2.32$$

where Q_p is the peak inflow rate, is the percentage of release rate (Q(out)/Q(in)), t_c is the time of concentration, and m is the ratio of recession duration to time of peak which is assumed to be equal to the time of concentration. A general simplified equation denoting the storage volume for any particular reservoir in a series is derived:

$$V_s = Kt_cQ_p(1-\alpha)^2 \left(\sum_{i=0}^{n-1} \alpha^i\right)^2 \frac{60}{43560} \qquad 2.33$$

where n is the sequential reservoir position from the top of the watershed and k is a reduction coefficient dependent upon the sequential reservoir position, the inflow-outflow control strategy and the recession duration.

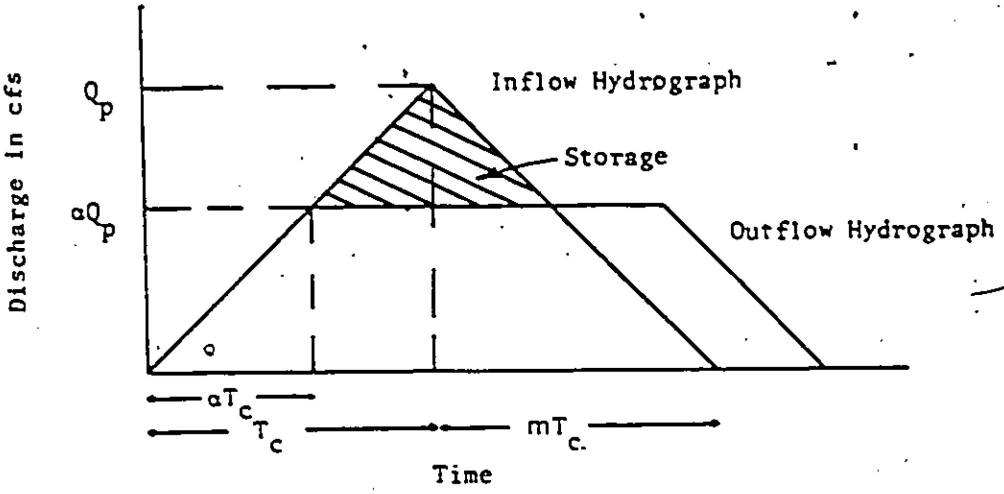


Figure 2.14: An Inflow-Outflow Relationship (Abt and Grigg 1978).

The storage estimation equation for a series of detention reservoirs is compared with the HEC-1 model. Several recession durations namely, t_c, $2t_c$, and $3t_c$, of the storage estimation equation are used. It is found that the mean ratios of HEC-1 storage to the storage estimated with the equation are 1.89, 1.05, and 0.83, respectively. It is concluded that the use of recession durations of $2t_c$ and $3t_c$ are better than a duration of t_c.

2.5 SUMMARY AND RESEARCH NEEDS

2.5.1 The Rational Method

The previous review has considered only the most used attempts to improve the methodologies for estimating C and ti. Needless to say, because of their multiplicity, the practitioners are confused and there has been a lack of studies to recommend the most suitable one for use. Even at this stage very few methods have been verified or generalized. In many instances, the equations vary widely in predicted values and are only valid for specific conditions (Burke and Gray 1979). Another difficulty is that most of these methodologies or equations do not account for all of the parameters that govern the values of C and t_i.

For example, the tabulated C values have been criticized because actually C does not remain constant during the storm event. It is expected that as the storm progresses, C will increase due to the increase in saturation level of the soil and the decrease in infiltration rates. C is also affected by the rainfall intensity. A high rainfall intensity will result in a high value of C because after the infiltration rate has been reduced to a minimum value and the depression storage is satisfied, any increase in the rainfall rate will be reflected in the rate of runoff.

Since the rainfall intensities are higher for storms with low frequencies, adjustments of C for various frequencies have been suggested. These ratios however have been criticized because it appears that they have been chosen arbitrarily. Studies carried out by Schaake et al (1967) using data measured in Baltimore have indicated that statistical analysis of both runoff and rainfall data are better procedures for determining these ratios. However, they have found that the average increase of C for storms with return frequencies of 1 in 1 to 1 in 10 years amounts to approximately 10%, which is close to the Cf value suggested.

Other methods have expressed C in terms of the ratio of imperviousness, rainfall intensity, or rainfall duration.

Methods that express C in terms of the rainfall intensity have been proposed by Gregory and Arnold (1932) and Ordon (1973). It is found that in these methods, the variation of C with time, which is caused by the decrease in infiltration rates from the beginning of the storm, has not been considered. The curves proposed by Ordon (1973) do not consider the antecedent moisture condition of the soil. This type of method is also difficult to apply because according to the Rational Method, the average rainfall intensity decreases as the storm duration (which is assumed equal to the time of concentration) increases. It means that the runoff coefficient determined with these methods will decrease when the area of the watershed and the time of concentration increase, which is not the case.

Curves that consider the variation of C in terms of the rainfall duration have been proposed by Horner (1910), Hoad (Fair and Geyer 1959), and Mitci (1974). However, before applying any one of these relationships, one should be quite sure of the author's definition of the time which has been variously labelled as the 'time from beginning of storm' (Horner) and the 'duration of rainfall' (Hoad). The difficulty of these methods lies in the problem of quantifying 't'. As noted in the ASCE Design Manual No.37 (ASCE 1970), it would be incorrect to assume that the start of the rainfall and the beginning of the time of concentration of flow to coincide, for this would result in selecting too low a value for C. One should consider the antecedent moisture condition of the soil prior to the design storm and should adjust accordingly the value of C at the beginning of the design storm, that is, at the start of the time of concentration. McKinney's method (1967) is an example in which the time of concentration is assumed to start some time after the beginning of the storm and thus C is greater than zero at the start of the time of concentration (Figure B.5).

Similar difficulties can be observed for the determination of the inlet time.

One can see that some of the equations for calculating the inlet time are developed empirically on the basis of measurements from natural watersheds (Kirpich 1940, Seelye 1968). Since this type of formulas are developed empirically, they are often not replicable when applied in other watersheds. Very often, these formulas have been applied without consideration of their particular basis. It is observed that the Kinematic Wave equation is the only equation that is theoretically based. Izzard's formula can be considered to be more appropriate as compared with the other equations because it has been developed with experimental data which are obtained under tightly monitored laboratory conditions.

Equations proposed by Kirpich (1940), Kerby (1959), and Seelye (1968) have related the inlet time with the watershed length, slope, or simply with the drainage area. These equations do not account for the variations in the rainfall intensity which usually lead to changes in the depth of overland flow and hence the travel time. Two of the equations that have accounted for the rainfall intensity and most of the parameters that affect the travel time are the Kinematic Wave equation and the one by Izzard. However, an iterative procedure (Ragan and Duru 1972) is necessary for solving these types of equations when applied with the Intensity- Duration-Frequency (I.D.F.) curves of the Rational Method. Burke and Gray (1979) have found that by using several sets of I.D.F. curves, the inlet time estimated according to Izzard's formula gives unreasonable results. Tables 2.2 and 2.3 show comparisons of the various parameters used by different methods for estimating C and t_i . Tables 2.4 to 2.6 show examples of the values of C and t_i calculated according to the various methods (attempts are being made to ensure that they are being calculated with similar data). One can observe the wide variation of the predicted values from the comparisons.

It is not surprising that despite all these methods and equations, most governmental or regulatory agencies have just selected arbitraily one of these methods as a rule-of-thumb procedure. Presently, C is usually determined from tabulated values of the type described in Section 2.2.2.1 (Town of Oakville 1979) and flat inlet times (between 5 to 15 minutes), are simply assumed instead of being calculated (ASCE Design Manual No.37 1970). The determination of C and t_i based on 'engineering judgement' have sometimes led to

Table 2.2: Comparison of Parameters Used for Determining C by Different Methods

Type	Parameters Considered				Rainfall Intensity	Return Period	Storm Duration
	Imperv.	Soil Type	Land use	Slope			
(a) Fixed C							
ASCE Manual			x			x	
Erie & Niagara Counties		x	x	x		x	
Weighted average, Eqn.	x	x					
Schaake et al	x			x			
Rossmiller	x	x		x	x	x	
(b) Variable C							
Gregory & Arnold		+AMC			x		
Ordon			x		x		
Horner	x						x
Hoad	x						x
Mitci	x						x

Table 2.3: Comparison of Parameters Used for Determining t_i by Different Methods

Type	Parameters Considered Length	Slope	Surface Retardation	Runoff Coefficient	Rainfall Intensity
Izzard	x	x	x		x
Kinematic Wave	x	x	x		x
Kirpich	x	x			
Kerby	x	x	x		
Seelye	x	x		x	

Table 2.4: Values of C Computed by Different Methods

Type	Runoff Coefficient	Remarks
(a) Fixed C		
ASCE Manual	0.3-0.5	single family areas
Erie & Naigara Counties	0.3	medium density residential
Weighted average, Eqn.	0.41	C pervious = 0.2, C impervious = 0.9
Schaake et al	0.39	Imp = 0.3, S = 1.0
Rossmiller	0.74	X = 74, RI = 5, S = 1, I = 2, Imp = 0.3
(b) Variable C		
Ordon	0.66-0.72	Urban residential, storm duration 10-20 min. rainfall intensity 3-4 in/hr.
Horner	0.35-0.45	30% imperviousness, storm duration 10-20 min,
Hoad	0.25-0.85	30% imperviousness, storm duration 10-20 min.
Mitci	0.33-0.45	30% imperviousness, storm duration 10-20 min.

Table 2.5: Values of t_i Computed by Different Methods, 30% Imperviousness

Storm Return Frequencies	Time in Minutes									
	Izzard		Kinematic		Kirpich		Kerby		Seelye	
	Perv.	Imperv.	Perv.	Imperv.	Perv.	Imperv.	Perv.	Imperv.	Perv.	Imperv.
1:5	221	2.8	12.6	1.5	1.7	0.9	5.8	1.8	16	5.5
1:25	149	2.5	10.3	1.2	1.7	0.9	5.8	1.8	16	5.5
1:100	103	2.5	9.2	1.1	1.7	0.9	5.8	1.8	16	5.5

Note: Two flow paths are assumed,

(a) length from backyard to frontyard over grassed (pervious) surface is 150 ft., slope 2%, and roughness coefficient of 0.06

(b) length to first storm sewer inlet over paved street (impervious) surface is 50 ft., slope 1%, and roughness coefficient of 0.013

Table 2.6: Values of t_i Computed by Different Methods, 70% Imperviousness

Storm Return Frequencies	Time in Minutes									
	Izzard		Kinematic		Kirpich		Kerby		Seelye	
	Perv.	Imperv.	Perv.	Imperv.	Perv.	Imperv.	Perv.	Imperv.	Perv.	Imperv.
1:5	11.1	4.3	5.1	2.2	0.72	2.2	3.5	3.0	9.3	9.5
1:25	7.4	3.8	4.2	1.9	0.72	2.2	3.5	3.0	9.3	9.5
1:100	5.1	3.8	4.1	1.8	0.72	2.2	3.5	3.0	9.3	9.5

Note: Two flow paths are assumed,

(a) length from backyard to frontyard over grassed (pervious) surface is 50 ft., slope 2%, and roughness coefficient of 0.06

(b) length to first storm sewer inlet over paved street (impervious) surface is 150 ft., slope 1%, and roughness coefficient of 0.013

inadequate designs. A Rational Method designed system therefore may not always provide the level of protection suggested by the design criteria. There is a need at present to develop a more consistent method of determining C and t_i.

Due to the wide variation in the conclusions of the assessment of the Rational Method, the practitioners still do not know what steps are necessary to improve the Rational Method. Indeed most of the studies for assessing the performance of the Rational Method have not indicated the assumptions applied and the cause of errors in their comparisons. For instance, these errors may be related to the physical characteristics of the watershed, the choice of the Rational Method parameters and storm input. These studies, including those carried out for improving the methods of estimating C and t_i, have not answered questions regarding, for example, the maximum size of watershed for which the Rational Method applies and the assumption of the Rational Method in routing flows through a pipe system.

A further deficiency of all these studies is the lack of assessment under design conditions for which the Rational Method is normally applied. In fact, the Rational Method is a design method which cannot be used to reconstruct real storm events. Therefore, a comparison of the Rational Method with measurements is clearly inappropriate (although the performance of the final design should always be checked with real storm events).

The recognition of the need for interfacing simple methods with computer models and the recognition of the limitations in all of the previous studies have prompted the present research to explore alternative means of assessing the performance of the Rational Method in estimating flows for sewer designs.

2.5.2 Desk-Top Hydrograph Methods

The review of desk-top hydrograph methods have indicated that there are two main types of methodologies, namely the triangular hydrograph method and the dimensionless hydrograph method. A comparison of these methods is illustrated in Table 2.7.

It is concluded that the Modified Rational Method Hydrograph is inadequate because it does not represent correctly the shape of the actual runoff hydrograph. Its peak and time base are determined with the basic Rational Method thus leading to inconsistencies and errors. Studies have been conducted by Rao (1975) and Kao (1975) in comparing the Modified Rational Method hydrograph with unit hydrograph methods for use in the determination of detention volume. Rao has found that as the area of the watershed becomes larger, the unit hydrograph method tends to be closer to re-

TABLE 2.7

COMPARISON OF DESK-TOP HYDROGRAPH METHODS

METHOD	STORM DISTRIBUTION INPUT	SHAPE OF HYDROGRAPH ASSUMED	ROUTING OF SUB-HYDROGRAPHS TO DESIGN POINT	PEAK DISCHARGE ESTIMATION	RUNOFF VOLUME ESTIMATION	COMPLEXITY
Modified Rational Method Hydrograph	uniform intensity (from IDF curve)	triangular	"lumped" model	Rational Method	Rational Method (C i A D)	low
Linearized Sub-hydrograph Method	any storm	triangular and trapezoidal depending on storm duration	time-offset	Rational Method	Rational Method (C i A D)	high
SCS Unit Hydrograph Method	any storm	triangular	-	484 AQ/t_p	SCS - CN procedure	high
SCS TR-55 Method	SCS 24-hour Type II	tabulated discharge values for SCS 24-hour Type I design storm	Convex Method	tabulated and graphical values for SCS Type I storms	SCS - CN procedure	low
Dimensionless Hydrographs:						
SCS, Williams (HYMO) Unit Hydrograph	any storm	SCS: regression analysis of measurements Williams: two gamma parameter equations	-	Williams: BA B/t_p	SCS - CN procedure	high
Ragan et al	Keifer and Chu	modelled by HLSDM based on the shape of storm and watershed characteristics	continuity and momentum	modelled by HLSDM	Horton's infiltration	low

Q = runoff volume, A = area of watershed, t_p = time of peak, B = a watershed parameter ality than the Modified Rational Method hydrograph. Kao has concluded that since the Modified Rational Method hydrograph assumes an average rainfall intensity and an uniform rainfall distribution, the method can be used without resulting in great errors only for small watersheds where uniform rainfall distribution in both time and space are possibly justified.

The Modified Rational Method hydrograph assumes that the storm duration is exactly equal to the time of concentration of the watershed and it cannot simulate runoff hydrographs resulting from non-uniform or real storm events.

The latter limitation has been considered by the Linearized Subhydrograph method which is capable of determining hydrographs caused by any storm event. However, it is too complex to use because the time of concentration and the duration of the rainfall increment have to be compared in each time step in order to determine the subhydrograph. The linearized subhydrograph is still based on the Rational Method for estimating the peak of the runoff hydrograph thus inheriting its weaknesses.

The linearized subhydrograph method has also been criticized for the rigid shapes of the hydrographs that it has assumed. Walesh (1975) has suggested that the shape of the linearized subhydrograph for the third case of the method be modified to a trapezoid with a peak of 0.75 CiA. Considering the differences between the shape of the triangular Rational Method hydrograph and the typical runoff hydrograph, Shen (1975) has suggested that the development of a more physically sound hydrograph method by using a dimensionless hydrograph, which modifies the linearized subhydrographs into curvilinear subhydrographs, may be appropriate.

The use of synthetic unit hydrograph methods such as the ones proposed by SCS (1972) and Wu (1963) are time consuming to apply since a lengthy convolution procedure is required to obtain the complete runoff hydrograph caused by a non-uniform storm. In fact, they are more suitable for use with the computer and not for use as simplified methods. Wu's formula, for instance, is used in the computer model HYMO (Williams and Hann 1973).

The SCS TR-55 method provides a comparatively simple approach for estimating the hydrograph by means of tabulated discharge values which are expressed in terms of the runoff rate per unit area per unit depth of runoff. A convolution procedure is not directly used since the tabulated values are derived based on the SCS 24-hour Type II distribution design storms. However, the method has several limitations as described in detail in Section 2.3.4.

The computer simulated dimensionless hydrograph developed by Ragan et al (1975) has offered the advantages of a simple method but has retained the advantages of a physically based design model. It is as simple to apply as the SCS TR-55 method. It is however felt that the method is for very small 'inlet' areas where the attenuation effects of routing on peak flows are not significant. The extension of this method to larger watersheds has to be investigated.

A criticism of the dimensionless hydrograph is that it is limited by the shape and the distribution of the rainfall hyetograph. Hence, it is not applicable for all storm events. However, this type of dimensionless hydrograph can significantly simplify the determination of the runoff hydrographs for complete storm profiles.

It can be observed that all of the methods described above have not been compared with a computer model with the exception of the method developed by Ragan et al. Based on the foregoing, it is concluded that there is a need to develop a new improved method similar in principle to the SCS TR-55 method and the dimensionless hydrograph method but is more applicable for use under Canadian conditions. The method should be developed based on computer modelled simulations and can be used to develop peak discharge and hydrograph relations.

2.5.3 Desk-Top Methods for Sizing Detention

Table 2.8 compares some of the characteristics of the desk-top methods for sizing the detention storages. A numerical example is given to compare the storage detention volumes calculated by each method.

The methods give different estimates mainly because of the procedures used for determining the peak runoff (inflow) rate and the shapes of the inflow and outflow hydrographs assumed. Some of the methods assume linear triangular or trapezoidal hydrographs while the others are based on curvilinear hydrographs. The use of curvilinear hydrograph is however considered to be a closer representation of the actual hydrograph as compared with a triangular hydrograph.

It is concluded that the Modified Rational Method hydrograph has several limitations:

1. The Modified Rational Method hydrograph is formulated on the basis of a constant and average design rainfall intensity with a duration equal to or larger than the time of concentration and it cannot account for storms which are not uniform. The shape of the modified Rational Method hydrograph is also not an accurate representation of the actual hydrograph (see Section 2.3.1).

TABLE 2.8

COMPARISON OF DESK-TOP METHODS FOR SIZING DETENTION STORAGE

METHOD	SHAPE OF INFLOW HYDROGRAPH ASSUMED	SHAPE OF OUTFLOW HYDROGRAPH ASSUMED	METHOD OR EQUATION FOR SIZING STORAGE	REMARKS	DETENTION NUMERICAL EXAMPLE 120 ac, I=25%, Q_p=255 cfs, Q_o=25%Q_p t_p=1.2, t_b=3.5 hrs. V_r=1.1 in., P=2.5 in.	
Modified Rational Method Hydrograph	trapezoid	trapezoid	iterative procedure	inherits the weaknesses of the Rational Method; results are inconsistent when compared with SWMM	100,000 cu.ft. (2,830 cu.m.)	(C = 0.375)
SCS TR-55 Method	not given	not given	two graphical methods: (i) V_s v's V_r ($Q < .47$ cfs/ac.) (ii) V_s/V_r v's Q_o/Q_i ($Q_o > .47$ cfs/ac.)	based on SCS Type II distribution design storms and CN for runoff volume	216,000 cu.ft. (6,120 cu.m.)	(CN = 84, Q = 1.1")
Wycoff and Singh	curvilinear triangle	curvilinear triangle	$V = \frac{1.29R(1-\alpha)^{.753}}{(t_b/t_p)^{.411}}$ (α is % reduction, R is runoff depth)	verified with HYMO model for reservoir routing	320,000 cu.ft. (9,068 cu.m.)	–
Bouthillier and Peterson	triangle	routed by a model	graphical methods	has many limitations	273,000 cu.ft. (7,740 cu.m.)	(Type 4, D = 4 hr., P = 2.5, C_v = .44, V_s/V_r = .57)
Abt and Grigg	triangle	trapezoid	$V = (\frac{1+H}{2})\, Q_p\, t_p (1-\alpha)^2 (\frac{60}{43560})$ (H is t(fall)/t(rise)) (V is in ac-ft., α is % reduction)	assumes that the rising side of both inflow and outflow hydrographs coincide	345,000 cu.ft. (9,780 cu.m.)	(hydrograph approximated, V without approximation is 930,000 cu.ft., or 26,350 cu.m.)
SWMM	–	–	–	–	220,000 cu.ft. (6,230 cu.m.)	–

2. The detention volume determined is not related to one given inflow hydrograph but a family of unrelated trapezoidal inflow hydrographs; one cannot specify a given inflow hydrograph when the Modified Rationnal Method hydrograph is used.

Rao (1975) has concluded that the size of the detention storage determined with the modified Rational Method hydrograph is about 20% to 40% higher than that calculated with the unit hydrograph method.

In a recent study conducted by Wisner and Cheung (1980), it has been found that the storage volumes calculated with the Modified Rational Method hydrograph are inconsistent with that modelled by SWMM. A series of comparisons has been carried out using hypothetical watersheds with the characteristics of residential developments and design storms with different return periods. Table 2.9 illustrates the results of the study.

The graphical methods given by the SCS TR-55 method is limited to only a single type of storm distribution, namely the SCS Type II distribution design storm. The shapes of the inflow and outflow hydrographs used by SCS for deriving the graphs are not known because they are not mentioned in the manual. Therefore, it is not clear how applicable the graphs are.

TABLE 2.9
COMPARISON OF THE DETENTION STORAGES DETERMINED WITH THE MODIFIED RATIONAL METHOD HYDROGRAPH AND SWMM
(Wisner and Cheung 1980)

1:2 Year Frequency

Area (ac.)	(ha.)	Modified Rational Method Hydrograph (cu.ft.)	(cu.m.)	SWMM (cu.ft.)	(cu.m.)
20	(8.1)	15,000	(425)	10,000	(283)
60	(24.3)	49,000	(1389)	31,000	(878)
80	(32.4)	66,000	(1870)	41,000	(1162)
120	(48.6)	96,000	(2720)	60,000	(1700)

1:25 Year Frequency

Area (ac.)	(ha.)	Modified Rational Method Hydrograph (cu.ft.)	(cu.m.)	SWMM (cu.ft.)	(cu.m.)
20	(8.1)	18,000	(510)	19,000	(538)
60	(24.3)	56,000	(1589)	57,000	(1615)
80	(32.4)	78,000	(2210)	76,000	(2154)
120	(48.6)	117,000	(3315)	120,000	(3400)

1:100 Year Frequency

Area (ac.)	(ha.)	Modified Rational Method Hydrograph (cu.ft.)	(cu.m.)	SWMM (cu.ft.)	(cu.m.)
20	(8.1)	24,000	(680)	35,000	(992)
60	(24.3)	74,000	(2097)	110,000	(3117)
80	(32.4)	100,000	(2834)	140,000	(3967)
120	(48.6)	155,000	(4392)	220,000	(6234)

The method developed by Bouthillier and Peterson (1978) is found to have several limitations. First, it is concluded that the method is quite similar to the Modified Rational Method Hydrograph for estimating the inflow hydrograph thus inheriting similar weaknesses: (a) A triangular shaped hydrograph has been assumed with its peak discharge rate estimated by means of a "block" rainfall of constant intensity and a runoff coefficient (volumetric); thus, it does not consider other storm profiles; (b) The time base of the runoff hydrograph has been assumed to be equal to the storm duration while ordinarily the time base of the runoff hydrograph would be somewhat longer than the rainfall period. Second, for less than 100% runoff, Bouthillier and Peterson have suggested a volumetric runoff coefficient be used for estimating the proportion of runoff. However, no information has been given on how this coefficient can be determined and how it can be related with the imperviousness percent and types of land use of the watershed. Third, the peak runoff rate has been determined as a direct multiplication of the watershed area. Hence, any peak reduction due to increased overland and channel routing attenuation when the area of the watershed increases have not been considered. Consequently, it should be applied only in very small watersheds for which routing effects are negligible.

The Abt and Grigg method has assumed rigid shapes of inflow and outflow hydrographs. A comparison of three standard shapes of outflow hydrographs in a study conducted by Boyd (1981) has indicated that the hydrograph shapes used by Abt and Grigg will underestimate the volume of detention. For an inflow hydrograph that has a different shape, it must first be approximated with a triangle before the equation can be applied. It is found that without this procedure, results obtained can be over-estimated by as much as two to three times, which is found in the numerical example given in Table 2.8. It is mainly caused by the use of only Qp and tp as the main parameters in the equation; all other methods have considered the runoff volume as a parameter which indirectly limits the shape of the hydrograph.

It is felt that the method proposed by Wycoff and Singh has less limitations. Both of the inflow and outflow hydrograph shapes have been assumed to be curvilinear which are close to most actual hydrograph shapes. It is not based on other procedures such as the Rational Method or the SCS method and is verified with the HYMO model for reservoir routing.

Because of the various limitations of the available approximate methods for sizing detention, there is a need to develop an improved desk-top method for sizing detention storage. Furthermore, few of these methods are properly tested and compared with the more detailed computer modelled simulations; and, all of these methods consider only the conventional method of detention using storage ponds and did not consider current methods of urban runoff control using the "dual" storage system.

It is the intent of the present study to develop a new desk-top method for detention storage based on a computer simulated inflow hydrograph. Taking a simplified approach, it is concluded from Wycoff and Singh's study that a common outflow hydrograph that is curvilinear in shape can be assumed if the outflow hydraulics are not considered and free outfall conditions exist.

Chapter III

METHODOLOGY

3.1 INTRODUCTION

One of the solutions for obtaining compatible runoff computations from both desk-top and detailed computer methods in the preliminary and final stages of design is to employ only a single methodology using a:

- Standard Computer Model

and a

- Standard Storm Input

If a standard desk-top method is developed on the basis of the standard computer model and the standard storm input, a desk-top method that gives compatible runoff computations as compared with the detailed design methods can be obtained.

This chapter presents the selection of the proposed standard computer model and the standard storm input for the assessment of the Rational Method and the development of a standard desk-top method for urban drainage designs. In the first part of this Chapter, a methodology of using "lumped" conceptual watersheds for runoff computations with the computer model SWMM is developed and presented. A methodology of selecting the appropriate time step, the duration, and the type of design storm for runoff computations is also presented. The second part of this chapter describes the methodology used in selecting the test watersheds and the criteria used in establishing the conceptual watersheds for analyses in this study.

3.2 SWMM AS A STANDARD MODEL FOR URBAN DRAINAGE DESIGNS.

Among the computer urban drainage models used in Canada, such as SWMM, ILLUDAS, STORM, and HVM (Perks 1978, McPherson 1979), SWMM is one that has been most widely used for design. It has been applied in cities such as Edmonton, Winnipeg, and Toronto (Marsalek, 1976).

The SWMM model (Huber et al 1975) is a comprehensive model that can be used for the analysis of urban runoff; one of its capabilities is to analyze pipe systems in urban developments. Although HYMO has been recently used for the analysis of urban watersheds, it cannot analyze a pipe system and it is found that some major modifications of its input parameters are necessary before good results can be obtained (Wisner and Cheung, 1979, MacLaren Limited: MTRCA Study 1979). HYMO is often used for planning studies in large, rural watersheds.

ILLUDAS has not been tested and verified for Canadian conditions in as much detail as SWMM. It has also been shown by Wenzel and Terstriep (1976) to be very sensitive to different soil types and antecedent moisture conditions.

Two major studies, sponsored by Environment Canada, have been conducted for the assessment and testing of SWMM (MacLaren Limited 1975, 1976). In the first study, simulations obtained with SWMM have been compared with measurements from four small residential watersheds. The watersheds encompassed ranged from an imperviousness of 27% to 68% and their sizes ranged from 13 to 90 acres. Of the four watersheds tested, three are located in the United States and one is located in Ontario, which is the Calvin Park watershed of Kingston. Results of the comparisons with the Calvin Park watershed have indicated that SWMM simulations are within 20% of the measured peaks, times to peak, and runoff volumes. These results are considered acceptable for the modelling of urban drainage systems. By calibrating SWMM through the modifications of model parameters, it has been found that the results can be further improved.

In the second study, an in-depth review and assessment of SWMM has led to additional improvements of the model for application under Canadian conditions. A hypothetical watershed has been used to analyze the sensitivity of SWMM to imperviousness, width of overland flow, infiltration rates, detention depth, roughness coefficients, and ground slope.

The results of the sensitivity tests have shown that the percentage of imperviousness has the most significant effect on surface runoff among all of the parameters tested. The remaining parameters are ranked in the order of their decreasing influence as indicated above. The SWMM default values for these parameters have appeared to be reasonable estimates when no actual site measurements exist.

SWMM simulations have been then verified on three residential developments in Winnipeg and Toronto, with sizes ranging from 48 acres to 2,330 acres. A total of 15 storm measurements have been simulated. It has been concluded on the basis of the comparisons that SWMM has simulated the flows with satisfactory accuracy and flexibility for a wide range of conditions. The use of SWMM for runoff simulations has been, therefore, recommended for studies related to most aspects of storm water management.

Many other studies have also been carried out for the assessment, calibration, and verification of SWMM using Canadian data (Dillon 1979, Gore and Storrie 1980, Ng and Marsalek 1981). Other studies that have not been conducted with Canadian data are those by Heeps and Mein (1974), and Singh (1979). In all of these studies, SWMM has been shown to give close comparisons with measurements.

The routing of overland hydrographs through the pipe system by SWMM has also been tested in an independent study by the author (Cheung and Kassem 1979) with the EXTRAN routing model (Roesner et al 1982) which uses the full dynamic routing equations. It has been found that when the pipes are not surcharged, the hydrographs simulated by SWMM are same as those simulated by EXTRAN.

Based on the foregoing, it is concluded that SWMM has several advantages as compared with other models. It provides a consistent and accurate description of a variety of urban drainage conditions. Its simulations have been found to compare closely with measurements for Canadian watersheds. SWMM is well documented and is not a proprietary model. It is therefore readily available and widely applied in Canada. It is concluded that the SWMM-RUNOFF model can be regarded as a standard model for most urban drainage designs in Canada.

3.3 SELECTION OF STANDARD STORM INPUT FOR URBAN DRAINAGE DESIGNS

The shape, intensities, and duration of the rainfall pattern are important because they affect the rate of runoff. Both real and design storms can be used. However only design storms are used in this study.

Most urban drainage systems are traditionally designed on the basis of the design storm which is assigned a return period or a probable frequency of occurrence. Many cities such as Chicago (Keifer and Chu 1957), Winnipeg (MacLaren

Limited 1978), Edmonton (MacLaren limited 1979), and Oakville (Town of Oakville 1979) have emphasized the use of design storms with, perhaps, the eventual validation of the design by using a real storm event.

The design rainfall can be in general defined in two forms:

1. The Intensity-Duration-Frequency (I.D.F.) relationships, which are obtained through statistical analysis of real storm data, are defined in the form of a uniform intensity for a given duration and return frequency. They are applied in the Rational Method for computing peak runoff rates. Figure 3.1 illustrates a set of I.D.F. curves with various return frequencies that is used in the Toronto area. Other curves that are used in the City of Edmonton and the City of Ottawa, respectively, are shown in Figures 3.2 and 3.3.
2. The use of design storm hyetographs which are defined from I.D.F. curves. The shape, or distribution, of the design storm hyetograph can be derived in several ways.

Some of the methods commonly used for obtaining the design storm distributions are those proposed by Keifer and Chu (1957) for short duration design storms (e.g. 3 hours), U.S. Soil Conservation Service Method (SCS 1975, Hann 1976)

FIGURE 3.1

Intensity-Duration-Frequency Curve for the Toronto-Bloor Gauge Station

Based on recording rain gauge data for
the period 1940-1955, 1957
1959-1965
1967-1971

INTENSITY-DURATION-FREQUENCY CURVE FOR 1/2, 1/5, 1/10, 1/25 AND 1/100 YEAR FREQUENCIES

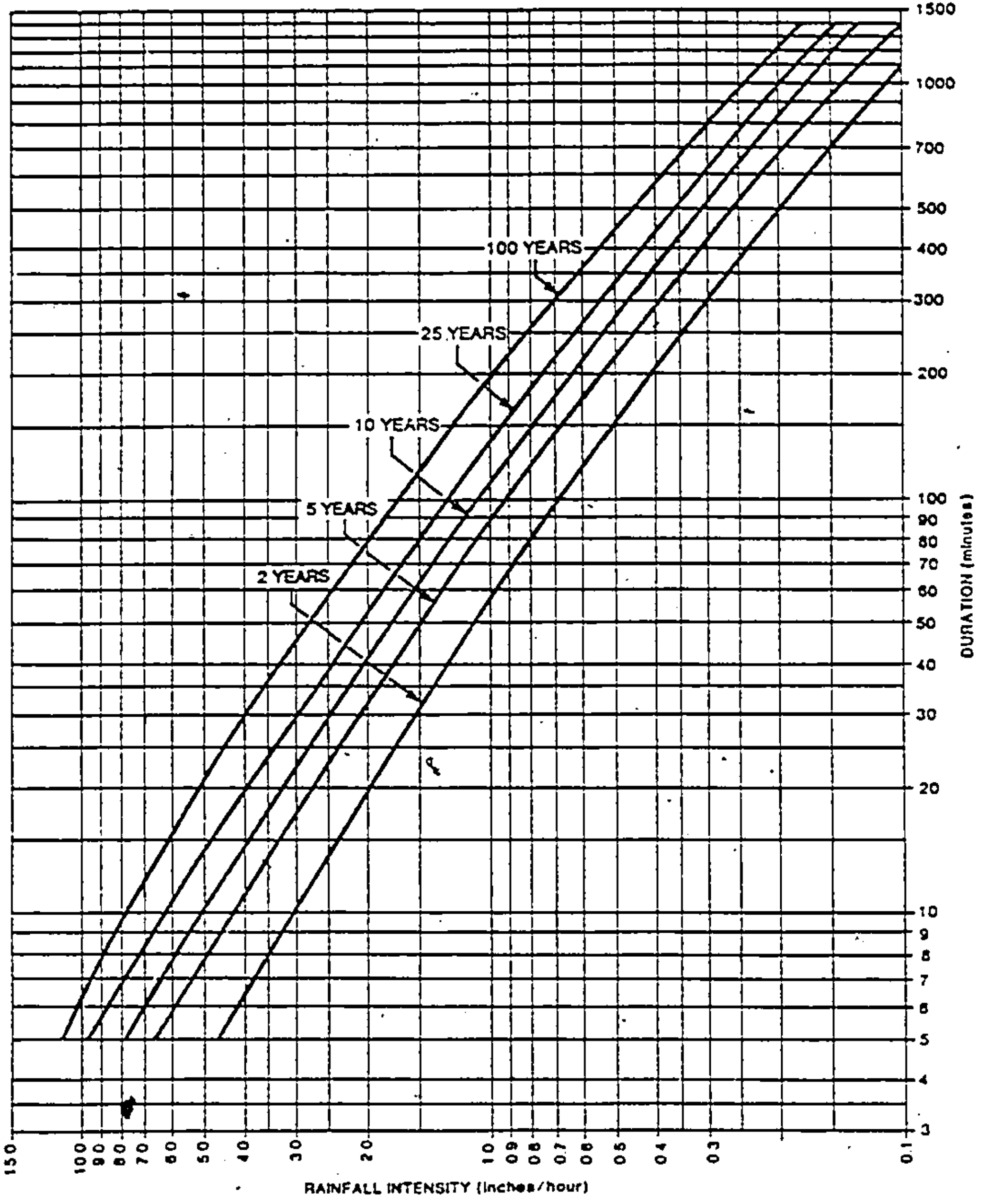


Figure 3.2: Intensity-Duration-Frequency Curve for the City of Edmonton

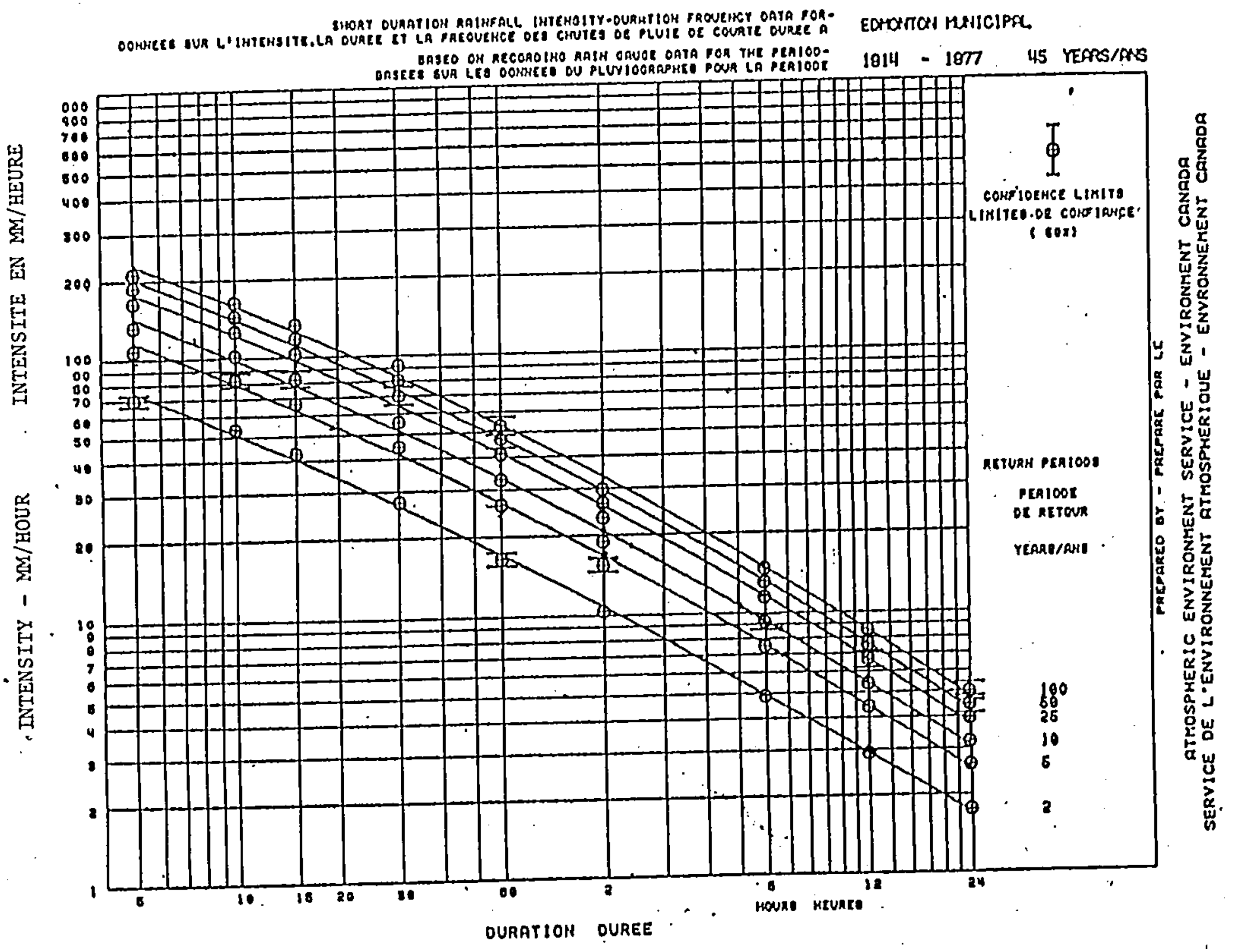
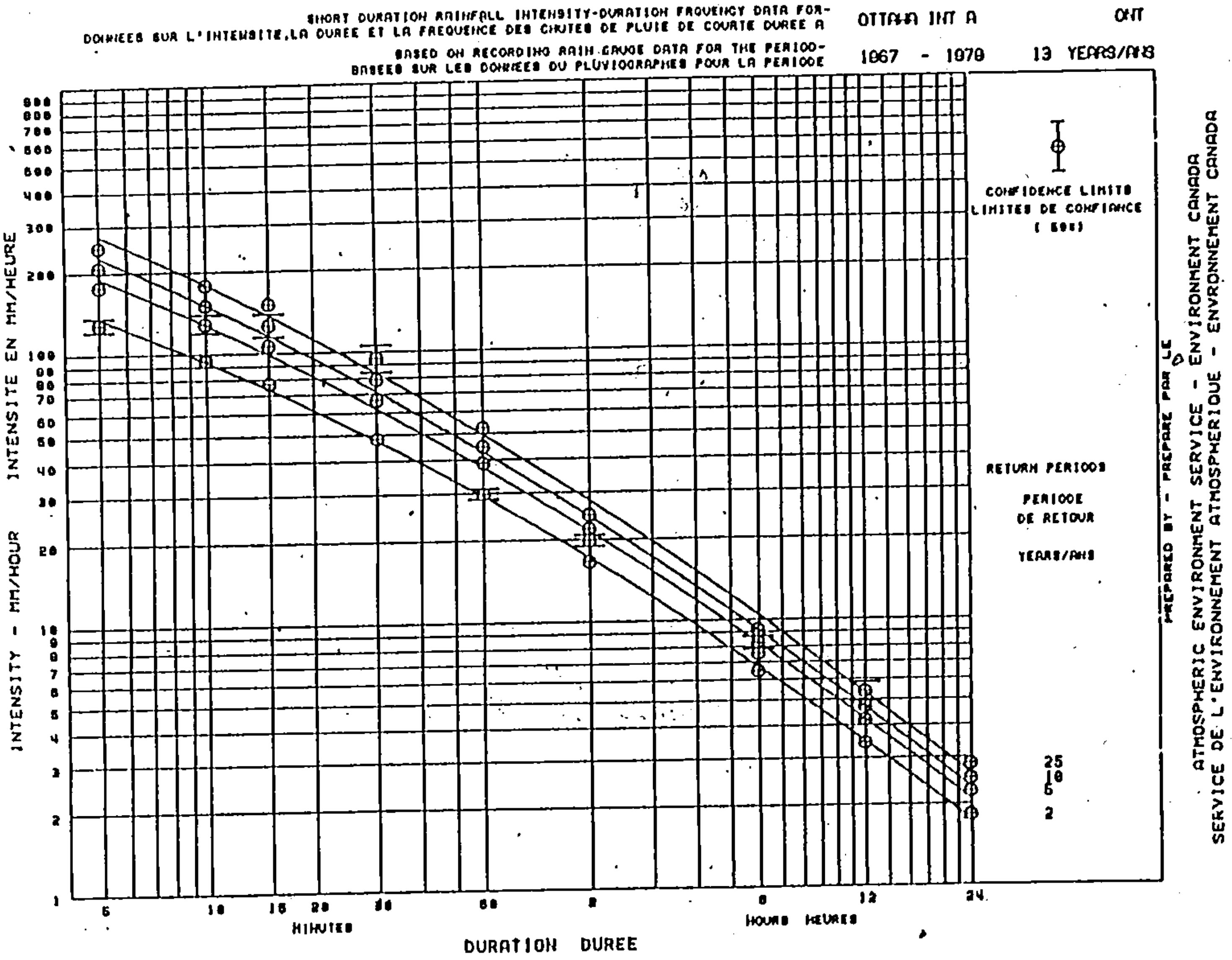


Figure 3.3: Intensity-Duration-Frequency Curve for the City of Ottawa



for long duration 24-hour Type II design storms, and Huff (1967) for 1-hour duration design storms as used in the ILLUDAS model. Hogg (1980) has also proposed a number of distribution curves after examining the time distribution of actual storms collected across Canada by the Atmospheric and Environmental Service. Design storms obtained with these various methods of distribution from the same I.D.F. curve are shown in Figure 3.4. It is observed that the peak intensity of the storm distributed with Keifer and Chu's method is higher than all of the other storm profiles.

The design storm concept has been criticized by various researchers (Marsalek 1978, Patry and McPherson 1979) for a number of reasons. It attempts to represent real storms with a single standard pattern although in reality actual storm patterns vary widely. The return period associated with a design storm is also imprecise because the frequency curve from which it is based is derived from different storms in a time sequence other than the actual occurrence. It is also suggested that the modelling of design storms should be closely related with the antecedent moisture condition such that the correct frequency of runoff can be obtained. The relation of the antecedent moisture condition to design storms will be further discussed in a later section.

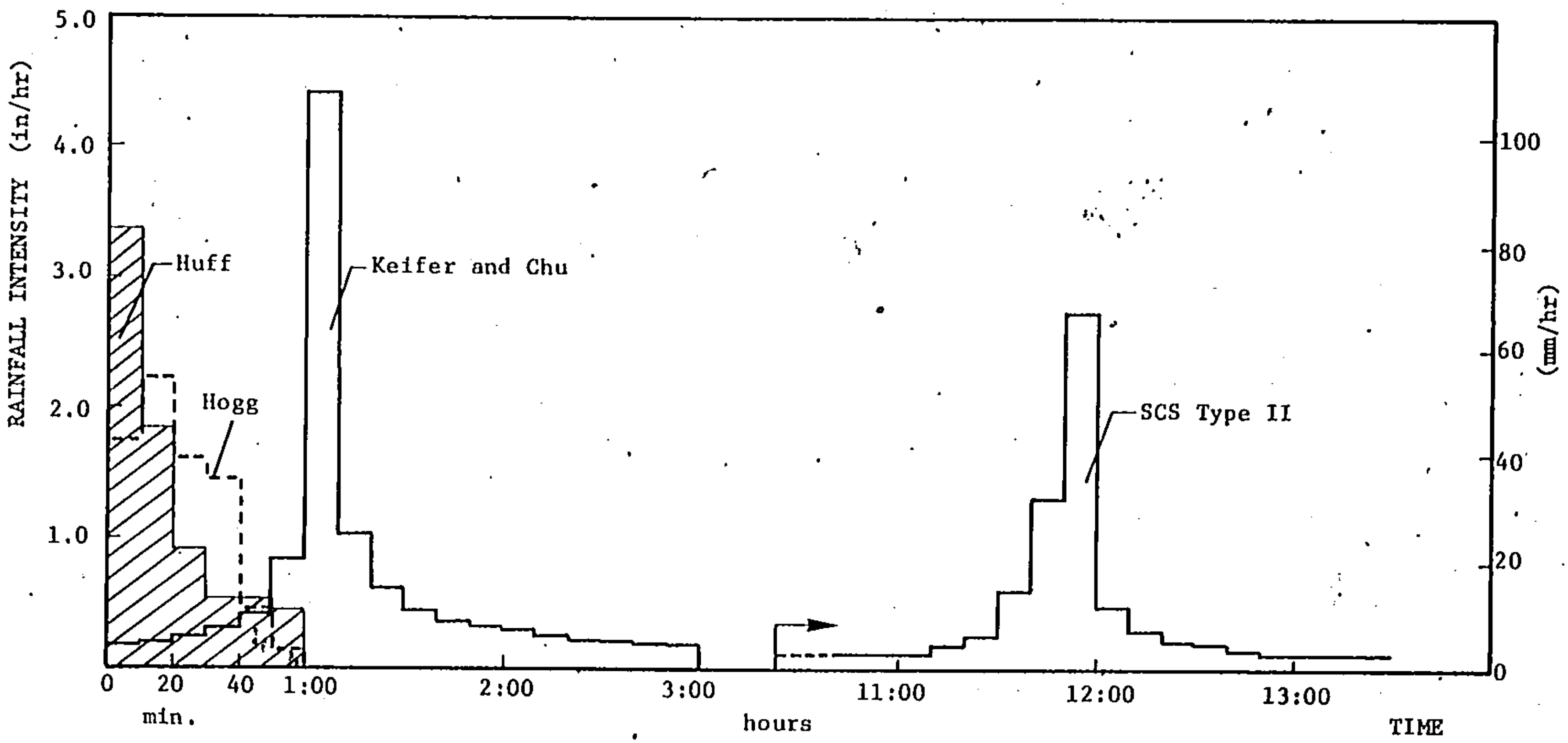


Figure 3.4: Comparison of Design Storms Distributed with Different Methods

Marsalek (1978) has compared flows simulated with design storms distributed with Keifer and Chu's method with actual storms for a watershed located in southern Ontario. He has found that the actual storms have produced runoff peaks appreciably smaller than those obtained with the synthetic Keifer and Chu's design storms. Similar conclusions have been made even when the design storms have been discretized with two different time steps including 1-minute and 5-minute intervals.

Alternatives of using a single design storm, however have been suggested. For instance, it is possible to use a series of design storms of different duration which are comparable to real storms (Urbonas, 1979) in order to obtain better results of runoff estimations.

Other studies have concluded that design storms have given reasonable estimates of runoff rates as compared with real stroms. In a study conducted by MacLaren Limited (1978) for the City of Winnipeg, design storms obtained with Keifer and Chu's method (1957) are compared with real storms. By means of SWMM, flows are simulated for both real and design storms and plotted against return frequencies. Flows for both types of storms of equivalent frequency are found to be quite close. It is concluded that the design storms do represent a realistic event and are suitable for use. In a simliar study conducted for the City of Edmonton (MacLaren Limited 1979), the same conclusions are obtained.

In a more recent study conducted by Wisner and Gupta (1981), who have compared design storms obtained with Keifer and Chu's method (1957) with a series of real storms, it is also concluded that close agreement of runoff values are obtained when the design storms are discretized with time steps varying between 5 and 10 minutes (Figure 3.5). Four different watersheds located separately in the city of Winnipeg, Edmonton, and Toronto and in South Africa have been used.

Because of the conflicting conclusions about Keifer and Chu's design storm method, it is not clear yet whether this type of design storms is conservative or not. However, most municipalities have continued to use them for design.

The selection of the discretization time step of design storms is also an important aspect of the modelling procedure. It may be pointed out that the choice of discretization time step of the design storm significantly affects the peak intensity and runoff rates (Wisner and Gupta 1981). The procedure used for the discretization procedure with Keifer and Chu's method (1957) is illustrated in Figure 3.7. In general, the first step involved in the derivation of Keifer and Chu's design storm is to define the duration (say 3 hours) and the time of occurrence of the peak intensity of the design storm (say at one-third of the storm duration). The next step is then to use the two equations as shown in

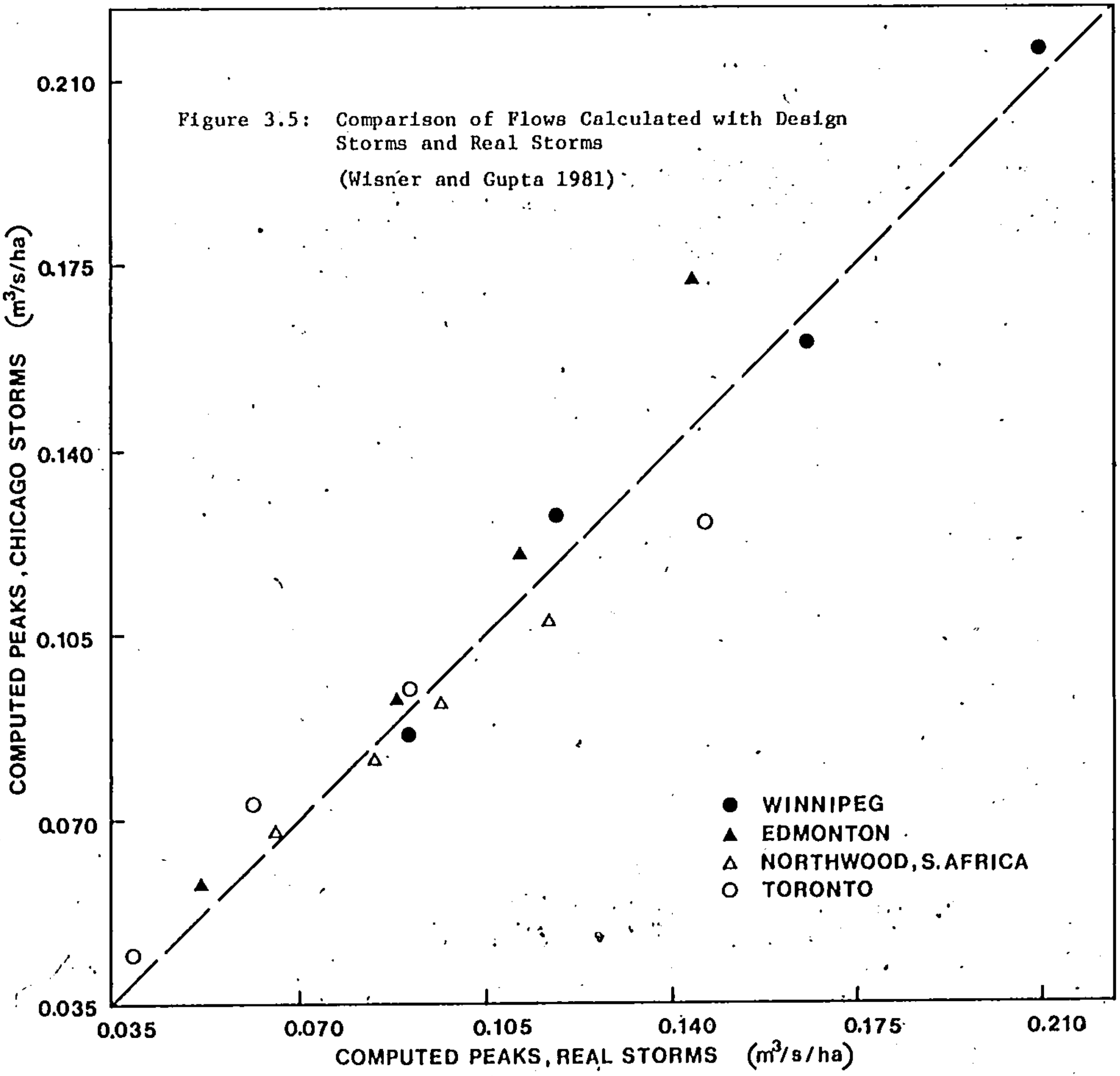


Figure 3.5: Comparison of Flows Calculated with Design Storms and Real Storms

(Wisner and Gupta 1981)

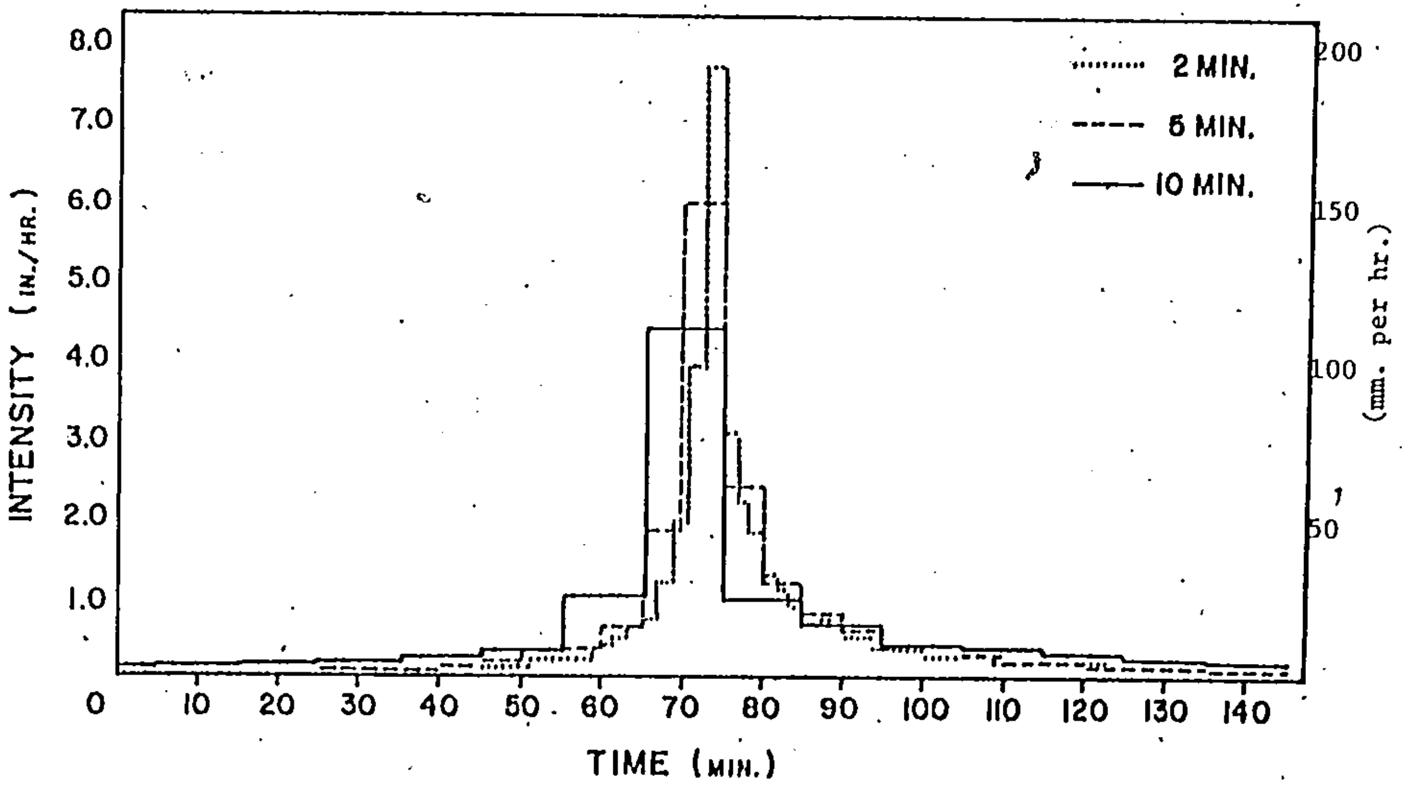


Figure 3.6: Comparison of Peak Intensities Obtained with 1-minute, 5-minute, and 10-minute Discretization Time Steps.

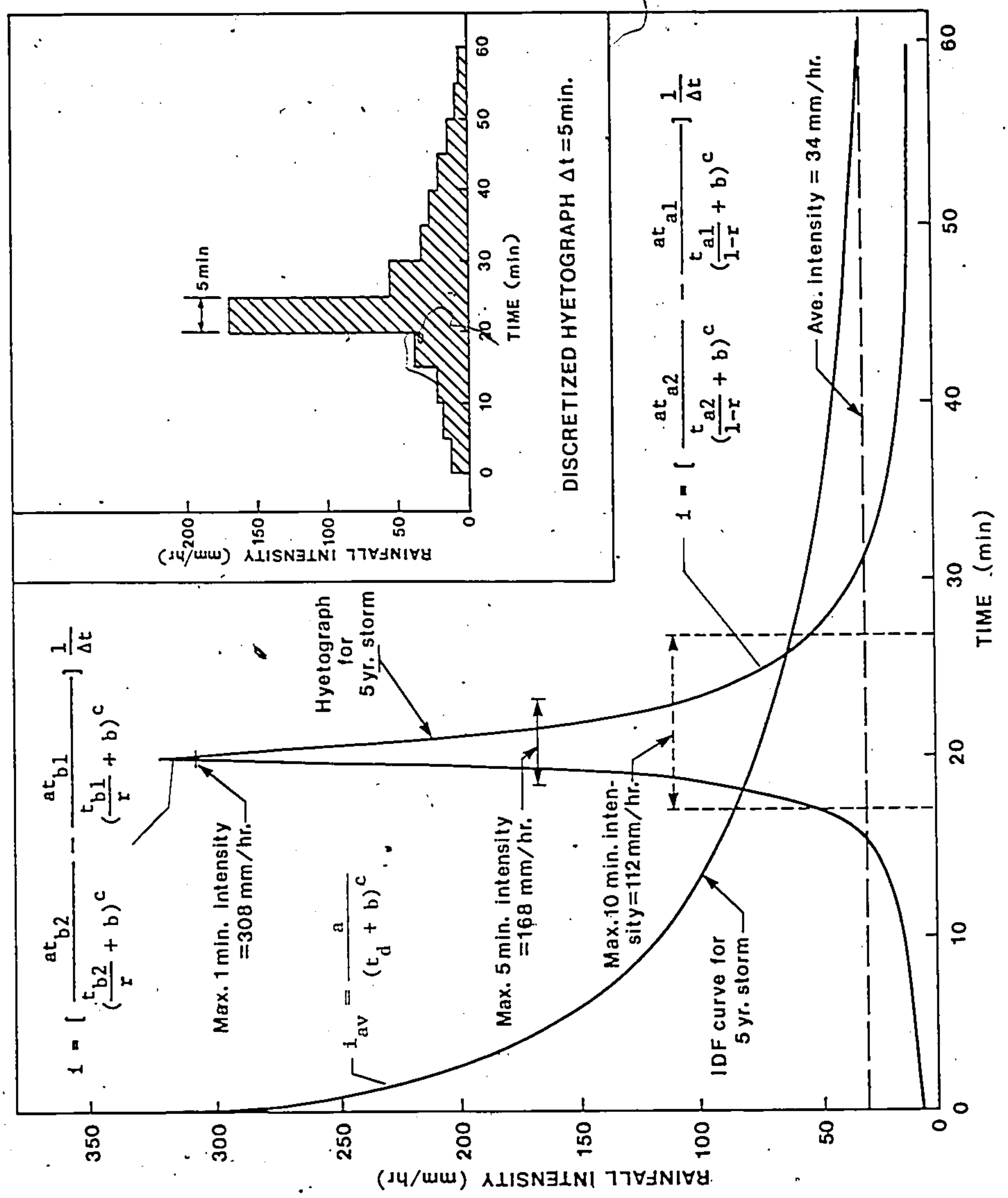


Figure 3.7: Keifer and Chu's Design Storm Method (Keifer and Chu 1957)

Figure 3.7 to calculate the rainfall intensities of the design storm hyetograph for the duration before and after the peak for a given time step of discretization. It may be pointed out that increasing the time step of discretization would have the effect of reducing the intensities of the storm profile (Figure 3.7). The resulting change in peak intensity values associated with the choice of discretization time step is obvious. As shown in Figure 3.6, for a design storm profile discretized with 1 minute, 5 minute and 10 minute time steps using Keifer and Chu's method (1957) the changes in the rainfall intensities are significant. These in turn can lead to significant changes in peak flows. Table 3.1 shows that the peak flows generated are smaller when longer time steps are used to disretize the design storms for simulations with the SWMM model. It is also observed that the Keifer and Chu's method of storm distribution are in general 'peakier' than the SCS Type-II 24-hour design storms; hence runoff rates simulated with the former type of design storms will be more critical (Wisner et al 1980).

Peak flows are also associated with the time of concentration which is the time it takes for all runoff to concentrate at the outlet of the watershed for a storm of uniform intensity. The discretization time step of a design storm during which the rainfall intensity is uniform should there-

Table 3.1: SENSITIVITY OF SWMM (RUNOFF) TO PEAKINESS OF A CHICAGO STORM (Wisner and Gupta 1981)

RETURN PERIOD (YR.)	TIME STEP OF STORM DIS-DISCRETIZATION (MIN.)	PEAK FLOW (mm/hr.)		
		AREA (ha)		
		4.9	25.9	38.9
1	1	34.5	27.7	23.9
	5	25.7	23.4	21.1
	10	20.1	19.1	18.3
2	1	39.9	32.5	28.5
	5	30.2	27.7	24.9
	10	23.6	22.4	21.6
5	1	51.1	42.9	37.9
	5	38.9	36.6	33.5
	10	29.7	28.7	28.2

fore be selected in relation to the time of concentration of the watershed. For urban watersheds, the time of concentration is relatively short because of the high percentage of impervious surface and the presence of a pipe system that induces swift conveyance of runoff. Design storms discretized with small time steps such as 10 minutes are therefore considered adequate to simulate correctly peak runoff rates. Espey et al (1977), for example, have concluded that a short-duration ten-minute unit hydrograph can be used to describe the runoff process of small, quick-responding urban watersheds. Eagleson (1962) has applied unit hydrograph methods to study rainfall-runoff relationships in urban watersheds and derived 10-minute unit hydrographs representative of the watershed response. Viessman (1966, 1968) has also used very short one-minute duration unit hydrographs to analyze the rainfall-runoff process in urban watersheds.

It is concluded that the main objective of selecting the time step is to select one that gives adequate but not over-conservative peak runoff rates for urban runoff computations.

The duration of design storms is the next important parameter to be considered. Long duration storms which are usually associated with low rainfall intensities, such as the 24-hour SCS Type II design storms, are likely to produce high runoff volumes but smaller peak runoff rates (Wisner et al, June 1980) during the long duration of rainfall. They are therefore appropriate for analyzing runoff from rural areas which have high percentages of pervious areas available for infiltration, and are therefore sensitive to rainfall losses. For urban areas, where runoff are mainly from impervious surfaces and the time of concentration is shorter, short duration and high intensity rainfall will be more critical (Espey et al 1977). As compared in Figure 3.4, the peak intensity of Keifer and Chu's design storm is higher than that of the SCS Type II design storm. The association of high intensities with the short duration of Keifer and Chu's design storms is particularly suitable for the analysis of runoff from urban areas. Analyses of real storms by Price and Howard (1978) have shown that most high intensity storms are associated with short durations.

It is decided from the foregoing discussion that Keifer and Chu's design storm method (1957) is appropriate for this study, which is primarily concerned with the analysis of critical runoff from small urban watersheds of less than one square mile. Due to its widespread use, it can be considered as a standard method for discretizing design storms. Since I.D.F. curves of various cities are quite similar (see Figures 3.1 to 3.3), it is expected that the shapes of design storm hyetographs will be quite similar too. The development of the "SHM" on the basis of a standard shape of design storm hyetograph would facilitate its future application. The design storms used in this study by SWMM for the assessment of the Rational Method and the derivation of the "SHM" are discretized with 10-minute time steps, which are considered adequate for all analyses as discussed above.

There is a major difference between the storm input data required by the Rational Method and hydrograph methods. The Rational Method requires, as an input, a uniform storm profile which describes the rainfall intensity in terms of a maximum average value (i.e. a "block-rainfall" with a constant rate), while hydrograph methods, or computer models, can accept as input any storm profiles. Since the Rational Method rainfall intensities are determined from I.D.F. curves, it is felt that for consistency of input, the design storms used in SWMM simulations should also be from the same curves.

The I.D.F. curves shown in Figure 3.1 are used to develop the design storm profiles for the different return frequencies throughout this entire study.

3.4 SELECTION OF TEST WATERSHEDS

Most of the methods for assessing and testing runoff computation methods use real watersheds. In the present study, several types of watersheds are used, namely real, conceptual, and "lumped" conceptual watersheds. Conceptual watersheds are used because real watersheds do not always provide the flexibility when a particular parameter such as slope, percentage of imperviousness, and overland surface resistance, or the sensitivity to storm intensities and flow routing, are to be assessed. Typically, a conceptual watershed can be used to test the sensitivity of a method, or a model, to a particular parameter. The characteristics of the watershed can be freely specified by altering the percentage of imperviousness, size, configuration of the watershed, and drainage pattern. Some examples of studies which apply conceptual watersheds for the study of models are those by Machmeier and Larson (1968), Ragan et al (1975), Marsalek (1978), and Kibler and Aron (1980).

It should be pointed out that although conceptual watersheds are convenient to use, caution must be taken in specifying their characteristics in order to ensure that they do represent actual cases. In this study, they have been determined by examining several existing developments located in Ontario.

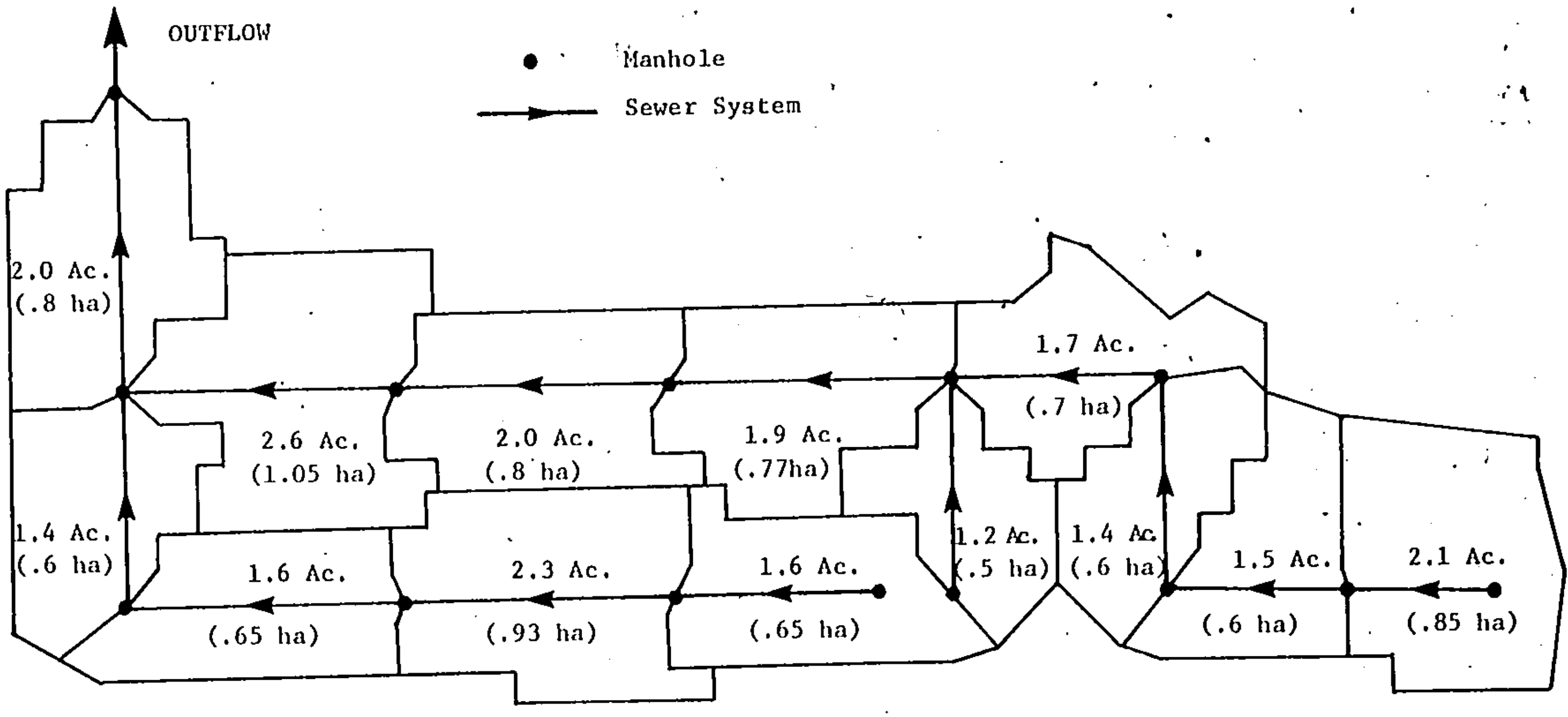
The watersheds used for the assessment of the Rational Method and the development of SHM are described in the following two sections.

3.4.1 The Rational Method

The comparison of the Rational Method with SWMM is carried out with both real and conceptual watersheds. Two separate stages of testings are performed in this study using small and large watersheds, respectively.

In the first stage, the Rational Method and SWMM are compared using two very small, real watersheds of approximately 23 acres each as shown in Figures 3.8 and 3.9. These two watersheds (Watersheds A and B) are obtained after examining different residential subdivisions located in the Toronto area; they both have imperviousness of 30%. Detail flow computations are carried out by discretizing the watershed, in each case, into smaller sub-areas with sizes ranging from 1.2 acres to 2.6 acres for test Watershed A and 0.4 acres to 2.6 acres for test Watershed B. Simulation of the overland flow hydrographs and routing through the pipe system are carried out entirely by the SWMM model. The pipe capacities are designed not to allow any surcharged flows. The SWMM routing routine has been shown to give very close results as compared with EXTRAN for small watersheds and unsurcharged pipe systems (Section 3.2). The total peak flow and hydrograph at each pipe length are simulated.

Peak flow values are then computed by the Rational Method and compared with SWMM simulations. In addition to 30%, the characteristics of Watersheds A and B are varied by



TOTAL AREA = 23.3 acres
(9.4 ha.)

Figure 3.8: Schematic of Test Watershed A

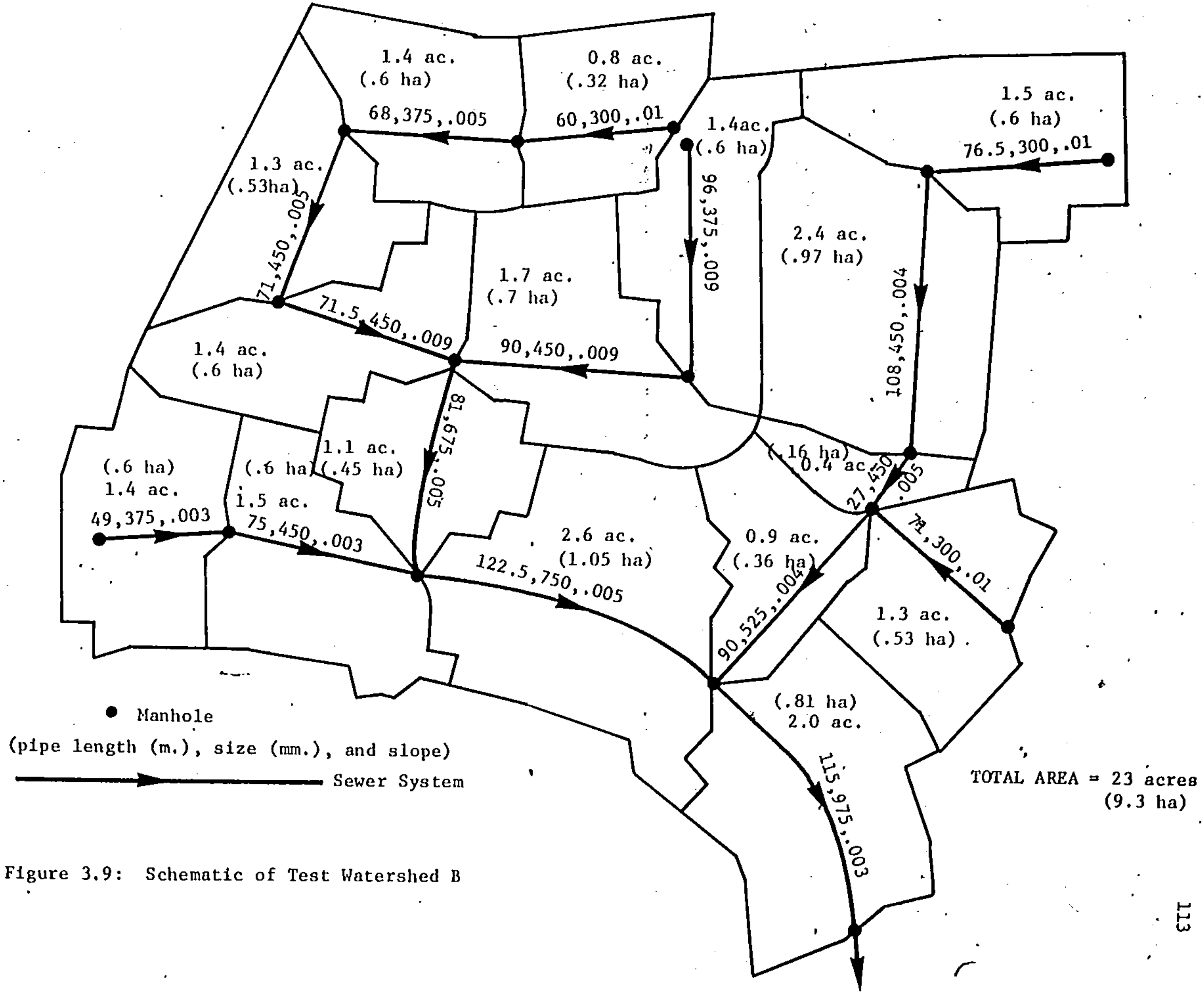


Figure 3.9: Schematic of Test Watershed B

using other imperviousness ratios in order to test the performance of the Rational Method under different conditions. At this stage of the study, the consistency of the Rational Method in estimating peak flows with SWMM is examined. Flows calculated with the Rational Method are adjusted if they are not the same as SWMM and procedures for adjusting the Rational Method flows are investigated and tested to see if they also apply in larger watersheds.

In this study, the definition of "small" watershed is based on the "inlet time" (Section 2.2.3) used by the Rational Method. When the area of the watershed is small, the time of concentration is relatively short and the time of flow travelling in the sewer is negligible (Muzik 1981). Therefore, the inlet time of flows which travel overland constitutes a large percentage of the total time of concentration and has a significant influence in determining the rainfall intensity and hence the peak runoff rate (Appendix A). "Small" watersheds are therefore defined as those for which the "inlet time" is a dominant factor on the peak runoff determination. The effect of t_i reduces when the area of the watershed and the travel time of flow in the pipe system increases.

In the second stage of the assessment, the Rational Method is tested for larger watersheds. In this study, "large" watersheds are defined as those for which the inlet time is not a dominant factor in the determination of the total time of concentration (i.e. the average rainfall intensity.) As the area of the watershed increases, flows travel more in the pipe system. The effect of the inlet time becomes less significant since it will no longer be a substantial percentage of the time of concentration.

An additional effect, which is negligible for small watersheds, is the effect of routing attenuation on peak flows in larger watersheds. Experiments in this stage of the study are therefore also conducted to investigate the performance of the Rational Method in flow routing.

Conceptual watersheds are used namely, Watersheds C and D (Figures 3.10 and 3.11). The conceptual watersheds are mainly composed of twenty identical real sub-watersheds connected together by means of a long trunk sewer with inlets spaced at equal distances apart. These real sub-watersheds are each made up of the real Watershed B used in the first stage of analysis. Since Watershed B has an area of 23 acres, the total area of Watershed C and Watershed D are each equal to 460 acres, respectively.

Inflows at each inlet are therefore the same and are obtained through detailed SWMM simulations carried out previously in the first stage for Watershed B. The total flow or area that contributes increases along the trunk sewer in the downstream direction. The distances between the inlets

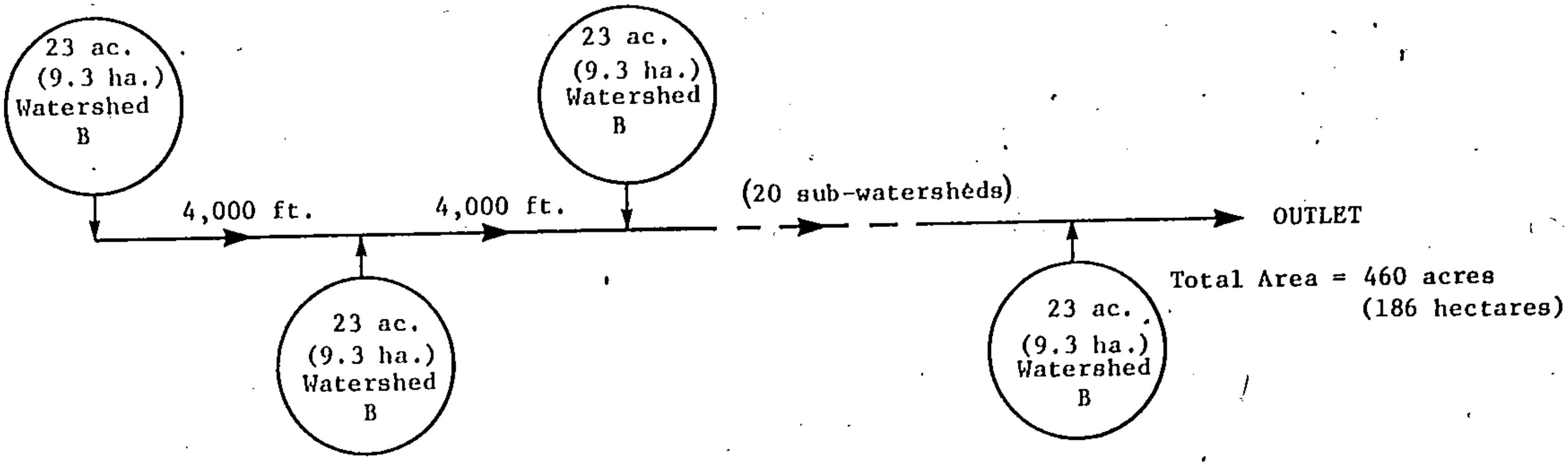


Figure 3.10 : Schematic of Test Watershed C.

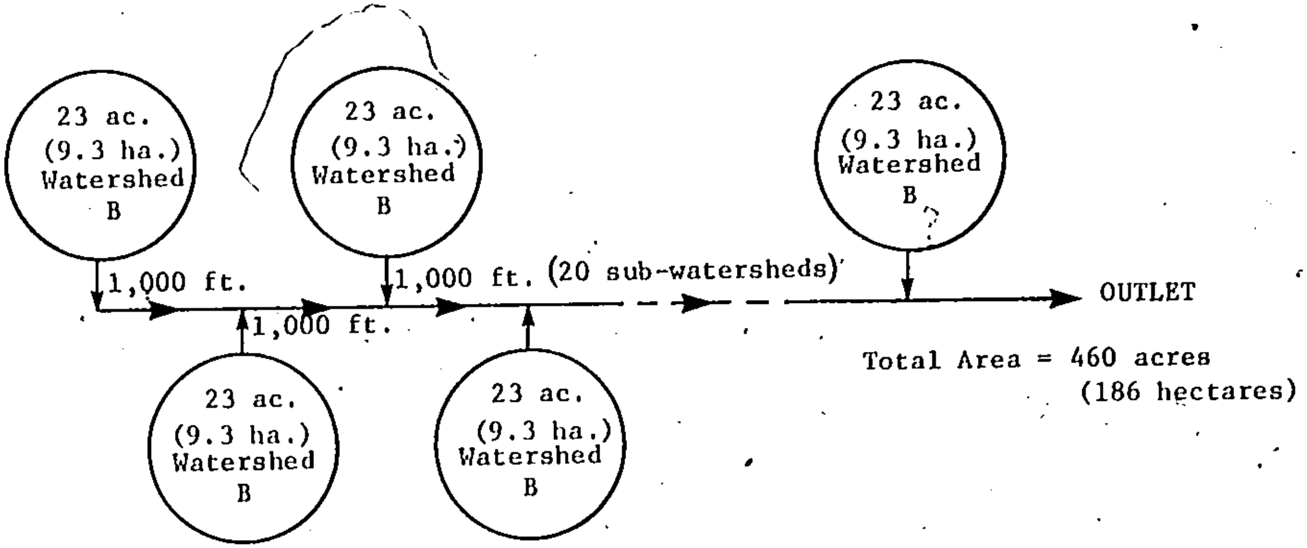


Figure 3.11: Schematic of Test Watershed D.

are selected after examining several real subdivisions of urban developments the Toronto area. The sensitivity of the Rational Method to the reduction of the distances between the inlets are tested by means of Watershed D.

Imperviousness ratios of 30%, 70%, and 100% are used for analysis.

For the large watersheds, the hydrographs are routed in the main sewer by EXTRAN (Roesner et al 1982) which uses the full dynamic wave equations for routing in a pipe network. It is selected because it is capable of accepting inflow hydrographs at any point in a pipe system and, therefore, is quite suitable for the tests in this study.

3.4.2 Standard Hydrograph Method (SHM)

The development of "SHM" is based on SWMM simulations using standardized, "lumped" conceptual watersheds with different sizes of 10 ac., 20 ac., 40 ac., 60 ac., 80 ac., 120 ac., and 160 ac., respectively.

The "lumped" conceptual watersheds are standardized in such a way as to represent the typical chracteristics of urban developments in the southern Ontario region. The use of "lumped" watersheds will be presented in the next section. Percentages of imperviousness used for simulations are 25% and 35%, which are representative of those found in residential developments. For comparison purposes, a percentage imperviousness of 70%, found mostly in commercial developments is also included.

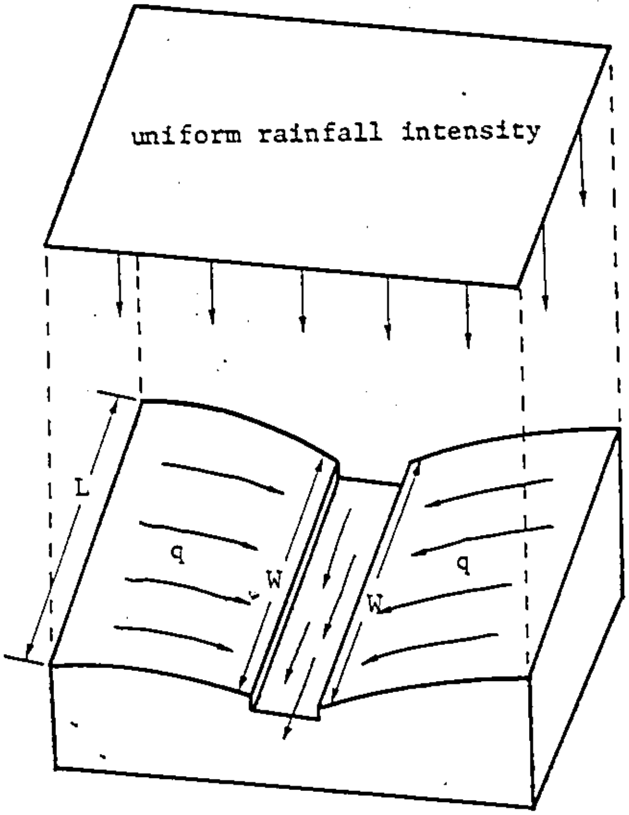
The SHM is tested at the end of the study with real watersheds.

3.5 A STANDARD "LUMPED" MODEL

All SWMM simulations for the development of "SHM" are conducted with a simplified approach using a "lumped" model. The idea behind this approach is to model the conceptual watershed regardless of its size as one single area such that its peak outflow and runoff hydrograph are equivalent to those obtained through detailed simulation using small discretized sub-areas. This approach is appropriate since only values at a few critical points of the watershed are required. The methodologies of using "lumped" as compared with detailed SWMM simulations and the development of equivalent watershed parameters namely, "width" of the watershed and "length" of pipe routing, are presented next.

The initial step of applying SWMM for the analysis of a drainage system normally involves preparing a schematized network of hydraulic elements of subcatchments and pipes. The overall watershed is subdivided into discrete subcatchments and pipe network, whereby each element is conceptually represented in the model in terms of its size, slope, and other parameters. The number of subcatchments and pipe sections that can be used for running a SWMM simulation largely depends on the information required from the simulation. If flow information describing the conditions in each of the pipe sections is of interest, then it is necessary to use a fine discretization to define the subcatchments tributary to each of the pipe sections modelled. On the other hand, if the objective is merely to simulate the outflow from the entire catchment, it may be sufficient to adopt a coarse discretization through the use of fewer and larger subcatchments and pipe sections. The latter approach reduces considerably the effort and costs of setting up and running the SWMM simulations, but it also means that a certain degree of accuracy will be lost by omitting internal details.

In the RUNOFF Block, each subcatchment is considered as a plane surface; the excess runoff depth (rainfall minus infiltration and initial abstractions) is assumed to be evenly distributed over the flow plane and is used to calculate the overland flow rate per unit width of the subcatchment. The width of the subcatchment (hydraulic width) is defined as the dimension of the flow plane across which the overland sheet-flow occurs (Figure 3.12). It indicates the ease with which the overland flow can be discharged from the subcatchment and is a function of the density of streets and pipes or channels. For a subcatchment where two surfaces are contributing flow along a distance L, the total width of over-



W = 2L when both sides are draining

Figure 3.12: Idealized Subcatchment-Gutter Arrangement Illustrating the Subcatchment Width for the SWMM Model

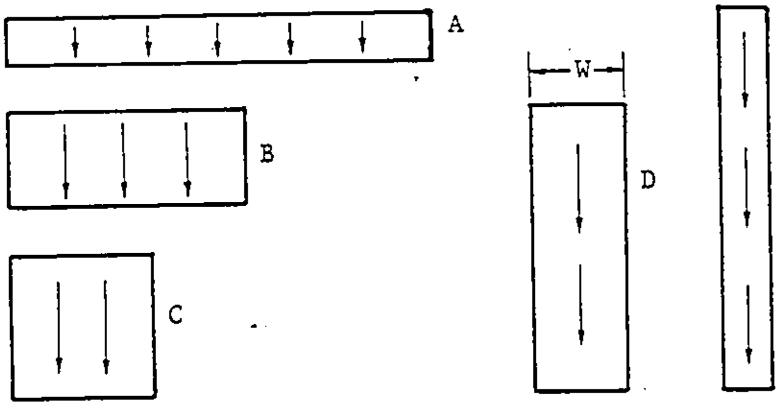


Figure 3.13: Different Subcatchment Shapes to Illustrate the Effect of the Subcatchment Width

land flow is twice the length of the drainage gutter; overland flow is perpendicular to gutter flow. As shown in Figure 3.13, a small width will result in a delayed and an attenuated peak hydrograph and a large width will result in the rapid discharge of flow and an advanced and larger peak hydrograph. Thus, when a group of sub-catchments are "lumped" into a single catchment, the total hydraulic width can be based on the sum of the hydraulic widths of the individual sub-catchments. The resulting "lumped" overland hydrograph will be very similar to the summing up of individual subcatchment hydrographs.

A similar analogy applies for flow routing through the pipe system which is computed by SWMM using the continuity and Manning's equations. The pipe system is composed of individual sections, each having a specific length, size, roughness and slope. For coarse discretization, an equivalent routing effect must be provided for the "lumped" overland hydrograph to produce a "lumped" outflow hydrograph that is similar to the detailed one. This is carried out by replacing the sewer system with an "equivalent" length of pipe made up of the averages of individual pipe sizes, slope and roughness. It may be anticipated that as fewer subcatchments are employed, more small pipes are neglected with a consequent reduction of pipe storage in routing computations, thus leading to a shorter time of peak of the routed hydrograph and higher peak flows. Such peak increases can be eliminated by increasing the equivalent pipe length or by reducing the width of the "lumped" catchment.

In general, two methods of "lumping" for simplified simulation are possible. The first method involves altering the equivalent length of the pipe to obtain an accurate simulation. The width of the lumped watershed in this case is equal to the sum of the widths of individual subcatchments. The second method involves altering the width of the lumped watershed and keeping the equivalent length of pipe of the lumped watershed constant.

The use of lumped SWMM analysis has been studied in detailed by Proctor and Redfern and James F. MacLaren Limited (1976). More recent studies have been conducted by Purenne et al (1979) and Ahmad (1980). In the study conducted by Proctor and Redfern and J.F.MacLaren Ltd., both methods of lumping have been examined by using one hypothetical and two real watersheds. Three levels of discretization have been tested by subdividing the hypothetical area into 37, 5, and 1 subcatchments. The "width" of the subcatchment and "equivalent" length of the lumped watershed have been then determined corresponding to each of the "lumping" procedures.

For the hypothetical area, results of the simulations for a 2-hour duration design storm using the first method of "lumping" are very similar to those using detailed simulation. A slight increase in the peak flow have been observed when the number of subcatchments descreases. For the same data, but with a longer 4-hour duration design storm and lower rainfall intensities, less differences in simulated peak discharges have been observed. Use of a storm with longer and lower intensity thus resulted in diminished effects of infiltration and steady state conditions whereby the outflow rate is proportional to the rainfall rate.

For the second method of "lumping", the width of the lumped watershed has been reduced to compensate for the complete omission of the pipe system. Several simulations have been carried out with the 2-hour design storm. It has been observed that the total subcatchment "width" has to be reduced by 30% in order to obtain peak flows close to those obtained through detailed simulation.

Tests with real watersheds have also indicated that the use of aggregated watershed widths and equivalent pipe lengths have given results in good agreement with measurements.

Ahmad (1980) has developed a lumping procedure similar to the first method for the modelling of existing storm trunk sewers in the city of Edmonton. The methodology of lumping has been tested on two small residential watersheds of 39 acres and 33 acres, respectively. Using a design storm discretized at 5 minute intervals, he has found that the peak flow rate with a single lumped catchment is 20% higher than that obtained from detailed simulations. A better agreement (within 10%) has been obtained when the design storm is discretized at 15 minutes interval. (A design storm discretized at 5 minutes interval has higher rainfall intensities as compared with one discretized at 15 minutes as described in Section 3.3).

3.5.1 Determination of Equivalent "Width" and "Length"

For the development of the SHM procedure, a separate analysis is carried out to determine the equivalent "width" and "length" of the lumped conceptual watershed. An approach parallel to the first method of lumping is used.

3.5.1.1 Average Catchment "Width"

In order to determine the average "width" of the lumped hypothetical watersheds, two residential developments located in Toronto with imperviousness ratios of 30% are examined. For each development, the total drainage width (which is equal to 2L when both sides of the gutter contribute inflows, Figure 3.12) as the area accumulates is calculated. Values of L determined are plotted and shown in Figure 3.14. It is found that the average drainage width (W) per acre in these residential developments varies between 280 ft. (=2L)

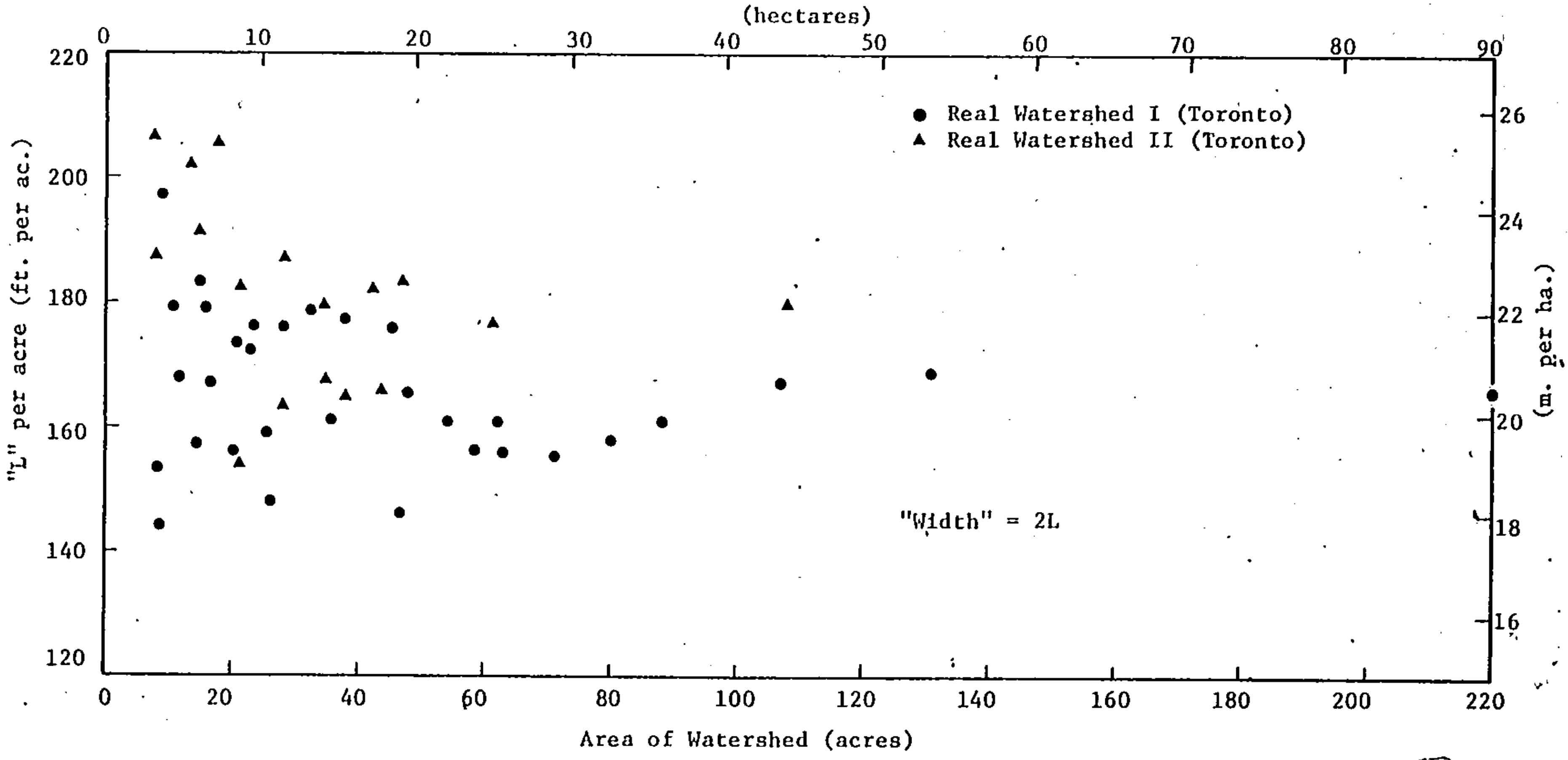


Figure 3.14: Average Subcatchment Widths for Residential Developments

per acre and 460 ft. (=2L) per acre. Since the lumped watersheds used for this study are hypothetical, it is considered appropriate to use an average "width" parameter in order to simplify the procedures to determine the total "width" of the lumped hypothetical watersheds. An average width of 350 ft. per acre is therefore adopted. The total width of the lumped watershed is then equal to the sum of the individual subcatchment widths expressed in feet per unit area. Hence, for a lumped watershed, the aggregated "width" is 350 feet per acre when a gutter has flows contributing along both sides.

It is noted that this average "width" ratio may vary for other conditions or other types of developments. For example, in commercial areas, the "width" ratios may increase because of higher percentages of imperviousness. The sensitivity of peak flows to the variation of "width" will be discussed further in Section 3.5.1.3. The "width" ratio adopted for this study is considered only as an average value.

3.5.1.2 Average Equivalent Pipe "Length"

It is observed that when a watershed is "lumped", an equivalent pipe length has to be used to compensate for the correct effect of routing in the pipe system. An average ratio of equivalent pipe length per acre similar to the one used by "width" is adopted. To determine the average equivalent pipe "length" per acre for routing, a sensitivity analysis using a real watershed (Figure 3.15) is conducted. SWMM simulations are carried out using a fixed "width", while pipe "lengths" are systematically varied. The "width" of the watershed is determined by summing up the "widths" of the sub-watersheds. The results of the sensitivity analysis are shown in Figure 3.16.

The average "length" of the equivalent pipe required to produce the same peak runoff rate as in detailed simulations is approximately 20 ft. per acre. Figure 3.17 shows a comparison of the hydrographs obtained with detailed and the lumped procedure (by means of the average equivalent pipe length determined). It is also observed that the peak flow is not sensitive to changes in the pipe "length", particularly at the higher range of values. In the range of 1,000 to 3,000 ft. (300% variation), the change (reduction) in peak flow is within 23%.

3.5.1.3 Sensitivity of "Width"

Since the "width" of the subcatchment may vary slightly from one residential development to another, a sensitivity analysis is carried out to determine the discrepancies caused by averaging the "width". The real watershed (Figure 3.15) is lumped and the total catchment "width" calculated

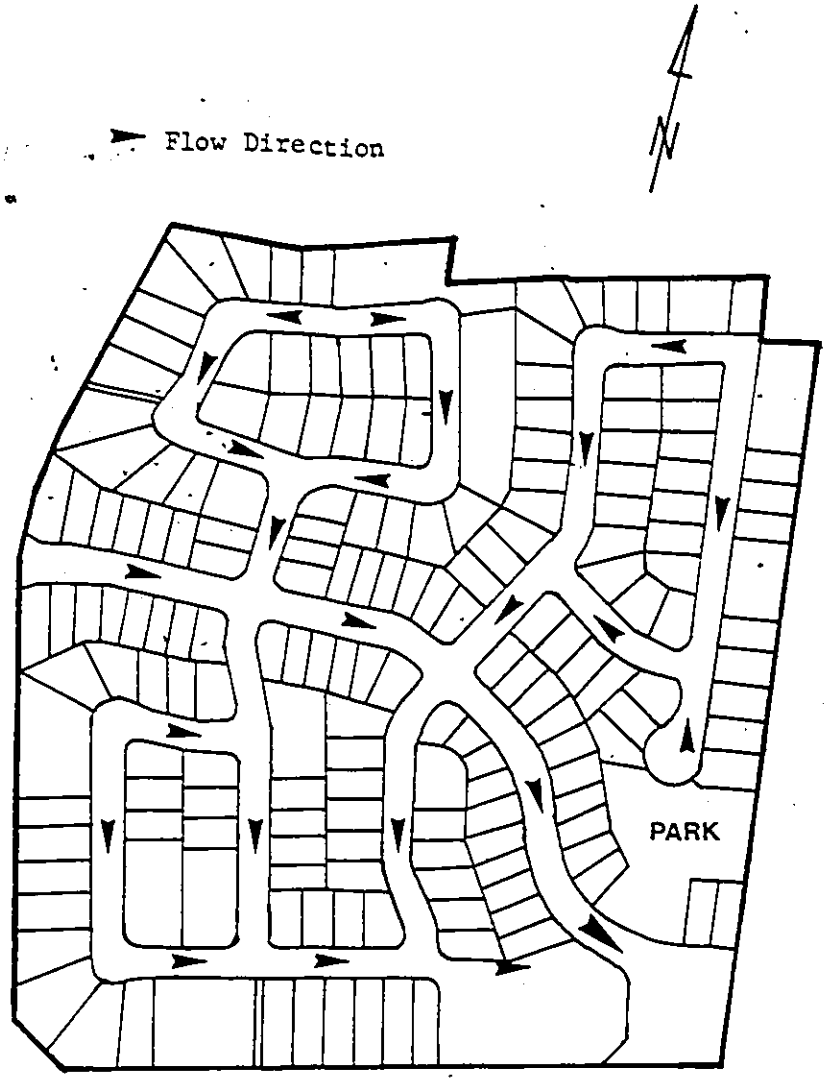


Figure 3.15: Real Watershed Used for the Determination of the Equivalent Pipe Length

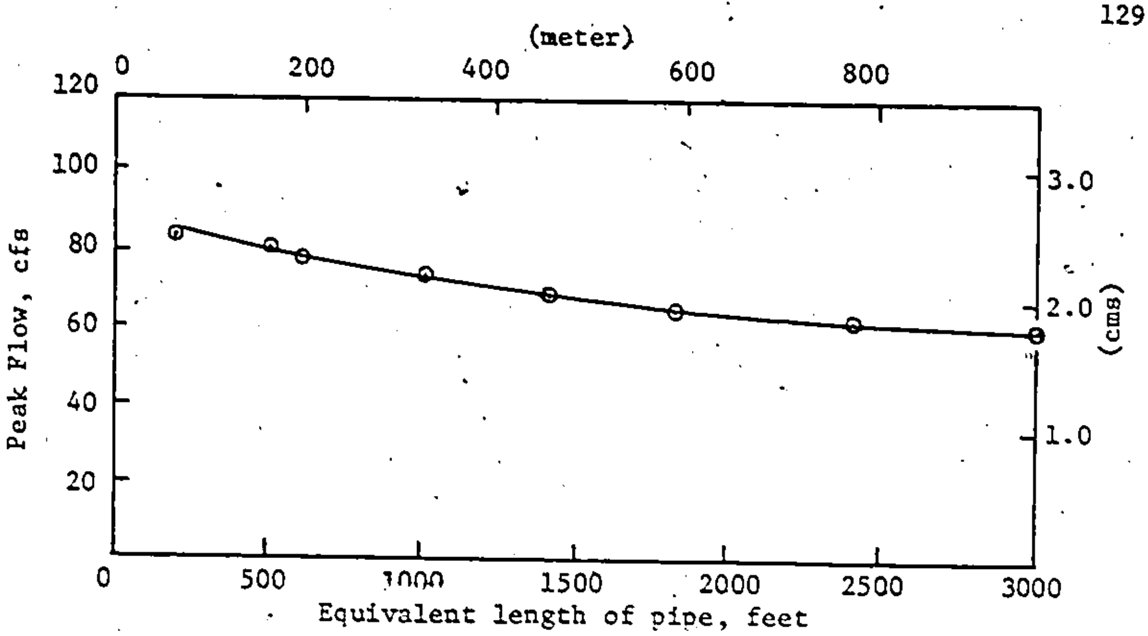


Figure 3.16: Sensitivity of Peak Runoff to Length of Gutter with SWMM RUNOFF Block Routing (Overland Flow Width is the Sum of the Widths of Individual Sub-watersheds).

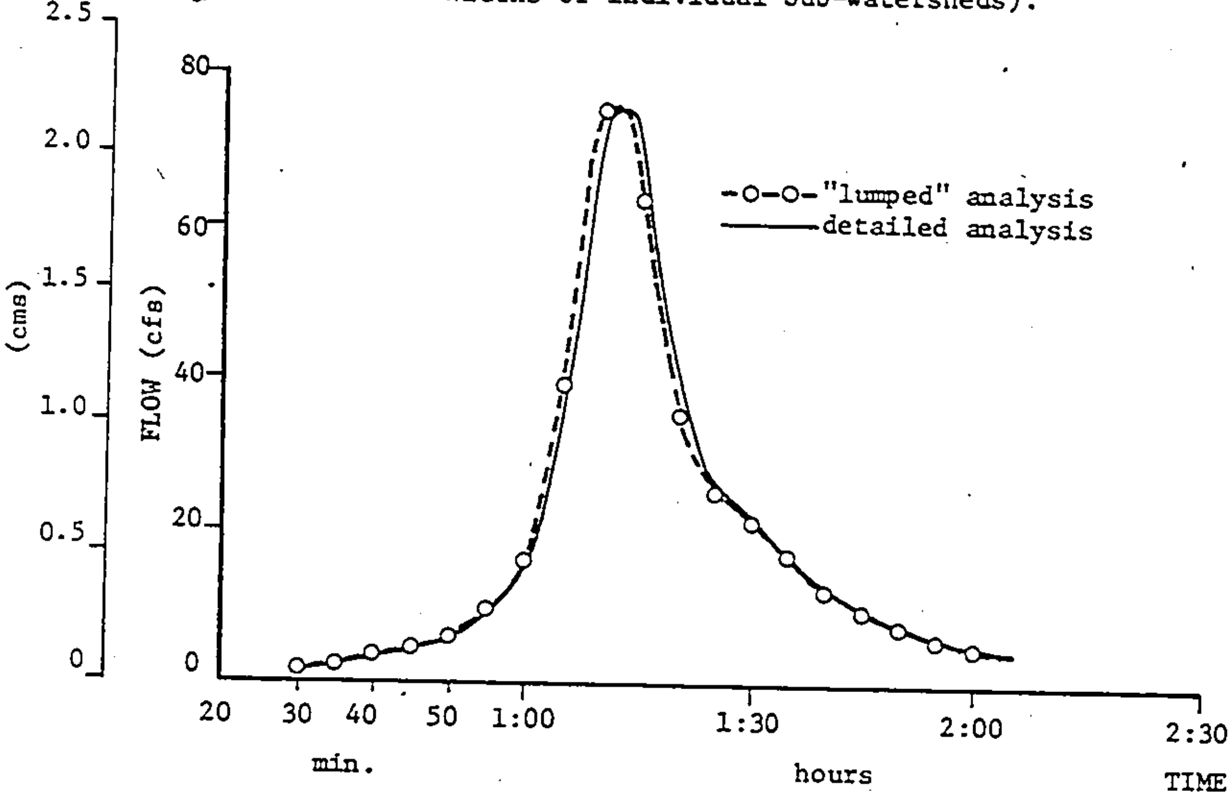


Figure 3.17: Comparison of "Lumped" and Detailed Analysis by SWMM

is 18,500 ft. SWMM simulations are then carried out by varying the overland "width" but keeping the equivalent pipe "length" constant. Figure 3.18 summarizes the results of the simulations. It can also be seen that the peak flows are not sensitive to changes in "width" particularly at a higher range of "width" values between 10,000 and 30,000 ft. (300% variation). For this range, the change (increase) in peak flow is only 10%. Using the average "width" ratio of 350 ft. per acre, the peak flow determined (77 cfs) is very close to that obtained through detailed simulation.

3.5.1.4 Summary

On the basis of this analysis, it is concluded appropriate to use average values of 350 ft. per acre and 20 ft. per acre to calculate the total catchment "width" and equivalent pipe "length", respectively, of the lumped watersheds. It is concluded that the discrepancies of flows simulated, resulting from variations in "width" and "length", are relatively small.

The watershed formulated is in effect a "standard" watershed for which the physical characteristics are generalized by means of an average watershed 'width' and an average equivalent pipe 'length' found typically in residential developments.

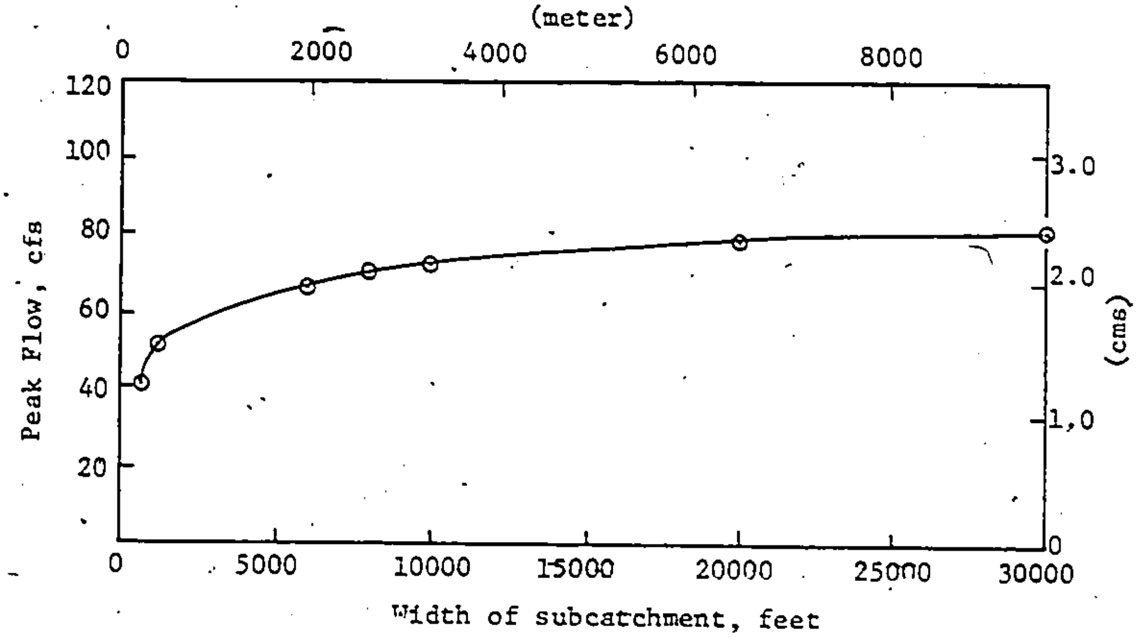


Figure 3.18: Sensitivity of Peak Runoff to Width of Sub-catchment with SWMM RUNOFF Block Routing.

3.6 SUMMARY

In summary, the previous sections have provided a description of the overall methodology undertaken in this research: (i) an investigation of the methodology for the selection of the standard computer model including a comparison of the routing routines of SWMM and EXTRAN have been carried out; (ii) the examination of the methodologies for the selection of the standard design storm including the proper type of distribution, storm discretization time step and duration for urban drainage designs have been conducted; and (iii) a methodology for the use of a standard, "lumped" watershed has also been proposed.

Chapter IV

ASSESSMENT OF RATIONAL METHOD WITH SWMM FOR URBAN DRAINAGE DESIGNS

4.1 STEPS OF ASSESSMENT

The assessment of the Rational Method is carried out systematically by conducting numerical experiments. In each numerical experiment, the standard model, SWMM, is used to generate the design flows using design storms distributed with Keifer and Chu's Method (see Section 3.3) and a specific watershed configuration (Section 3.4.1). The percentage of imperviousness is also varied accordingly for each watershed for sensitivity analysis. With the same Intensity-Duration-Frequency (I.D.F.) curves, by which the design storms used by SWMM are derived, flows are also calculated with the Rational Method. Flows estimated with the Rational Method are then compared with SWMM simulated flows and the design storm, watershed size, or watershed configuration is systematically varied for the next numerical experiment. Any discrepancies in the performance of the Rational Method as compared with SWMM are identified and analyzed.

The numerical experiments are carried out with different watershed sizes in two separate stages as outlined below:

1. "Small" Urban Watersheds

Two small, real watersheds are used for analysis (Section 3.4.1). In this study, "small" watersheds are defined as those for which the inlet time is a large percentage of the total time of concentration of the drainage area and therefore is a dominant factor in peak flow computations. In this range of watershed size, the effects of routing of flows in the pipe system are negligible.

The assessment of the Rational Method for the computation of peak runoff in "small" urban watersheds are basically carried out in two steps.

In the preliminary step of the analysis, flows calculated with the Rational Method using conventional methods of estimating C and t_i are compared with those calculated with SWMM in order to see whether any differences exist. Conventional methods are used in order to evaluate the Rational Method under a condition for which it is commonly applied.

Conventionally, fixed values of C have been employed (Section 2.5.1). Since many of the proposed functions for estimating C in terms of the rainfall intensity, the rainfall duration, or the Curve Number are inappropriate to use because of their unclear assumptions and limitations (Section 2.2.2), the weighted average equation (Equation 2.7) is therefore selected for calculating C. A value of 0.2 is used for C pervious and a value of 0.9 is used for C impervious in the weighted average equation.

For the determination of the inlet time, fixed values of inlet time have also been conventionally employed. For instance, fixed inlet times of 5 minutes have been used by the Town of Scarborough, 10 minutes have been used by the Town of Markham, and 20 minutes have been used by the Town of Nepean. It is decided to use an inlet time of 10 minutes as an initial value. This value of t_i will be examined as more tests are carried out.

In the next step of the analysis, the Rational Method is adjusted by modifying C and t_i until the peak flows calculated with the Rational Method are equal to that simulated with the SWMM model. Two approaches are used in modifying the Rational Method parameters C and t_i :

1. The first approach is to vary the inlet time t_i for the first upstream sub-area of the watershed but keep C unchanged until the peak flow rate calculated is equal to SWMM. This t_i is then used for calculating the total time of concentration and peak flows as computations proceed downstream towards the outlet.
2. The second approach is to calculate C on the basis of the total runoff volume (Q) simulated by the SWMM model. Hence, if the total rainfall precipitation is P, a runoff coefficient can be defined:

$$C_V = Q/P \qquad 4.1$$

where C_v is the volumetric runoff coefficient, which will be hereafter referred to as C(SWMM).

Using C(SWMM), the inlet time t_i is again varied until the peak flow rates calculated with the Rational Method are equal to SWMM.

The chief aim of the second approach is to see whether by adjusting both the inlet time and the runoff coefficient, the results of the Rational Method can be improved and to what extent t_i has to be adjusted, if necessary, as compared with the first approach.

It is noted that C(SWMM) is a runoff coefficient derived through the modelling of a design storm obtained according to Keifer and Chu's Method (1957) and to an infiltration profile according to Horton's equation. It therefore essentially represents the combined relations of imperviousness, rainfall profile, rainfall intensities, rainfall duration, depression storage, and infiltration rates. C(SWMM) is different from C(RM) because the former is expressed in terms of volumes while the latter is expressed in terms of rates.

The volumetric runoff coefficient is employed because it has been used in some methods as a simple procedure for generating hourly runoff volumes (i.e. inches per hour). It can therefore be viewed as an indicator of the average losses as used in the Rational Method. Some examples by which the volumetric runoff coefficient is applied for runoff computations are the Hydroscience Model (Hydroscience 1979) and the STORM Model (U.S. Army Corps of Engineers 1975). They have both used the following equation to calculate the hourly runoff rates:

$$V_c = 0.9I + 0.15\ (1.0-I) \qquad 4.2$$

where I is the ratio of imperviousness (Section 2.2.2.4).

Similar equations for calculating the volumetric runoff coefficient have also been proposed by Viessman and Miller (1972) and Desbordes (1975).

The numerical experiments are conducted using design storms (Section 3.3) with different return frequencies, including 1 in 2, 1 in 5, 1 in 25 and 1 in 100 years. The real watersheds are modified with different land uses and percentages of imperviousness, including 10%, 30%, 50%, 70%, and 90%, respectively. Thus the Rational Method is assessed for a broad range of characteristics.

2. "Large" Urban Watersheds

After the Rational Method peak flows are adjusted to be equal to SWMM for the "small" watersheds, the same method of flow adjustment is extended to larger watersheds. The criterion to be tested is whether since the Rational Method peak flows are equal to SWMM for each sub-watershed (which is Watershed B), as obtained in the previous tests, similar flows can also be obtained when the area of the watershed increases.

As described earlier (Section 3.4.1), the "large" watersheds in this study are defined as those for which the inlet times are not dominant factors in the determination of the times of concentration and the peak runoff rates of the watersheds. These watersheds are also defined as those for which the effect of routing attenuation is not negligible.

The normal procedures for the computation of runoff involve two separate stages of analyses, namely the transformation of rainfall into overland runoff and the routing of flows through channels. These procedures are not so obvious with those used by the Rational Method in which these two different stages are simplified into a single procedure. For the routing of flows, the Rational Method substitutes by considering different rainfalls of decreasing intensities (obtained from the I.D.F. curve) as the time of concentration, that is, the area of the watershed, increases. Since the Rational Method peak flows are equal to the SWMM peak flows for each small sub-watershed (as adjusted earlier), any differencs in the results because of this simplified procedure will reflect the effect of routing.

4.2 DISCUSSION OF RESULTS FOR "SMALL" WATERSHEDS

The analyses indicate that by using the conventional method of applying the Rational Method, peak flows calculated by the Rational Method are quite different from those calculated by SWMM. Without adjusting C and t_i, it is observed that for a percentage imperviousness of 30%, the Rational Method peak flows are higher than SWMM for the 1 in 2 years design storm but equal to SWMM for the 1 in 5 years design storm and lower than SWMM for the less frequent, 1 in 25 and 1 in 100 years design storms. Figure 4.1 shows the differences between the peak flows for Test Watershed B. Similar discrepancies are observed for other percentages of imperviousness ratios. The discrepancies observed for the imperviousness ratio of 70% are shown in Figure 4.2 and that for Test Watershed A (for the 1 in 5 year design storm and and 30 % imperviousness ratio) are shown in Figure 4.3.

4.2.1 t_i Varied and C Constant

Following this approach, by which t_i is varied and C is unchanged, the Rational Method peak flows are adjusted to be equal to SWMM. It is found that by selecting an appropriate inlet time for the 'inlet area' (see Section 2.2.3), peak flows calculated with the Rational Method can be made equal to SWMM. Equal flows will be maintained when this t_i is used for calculating the total time of concentration

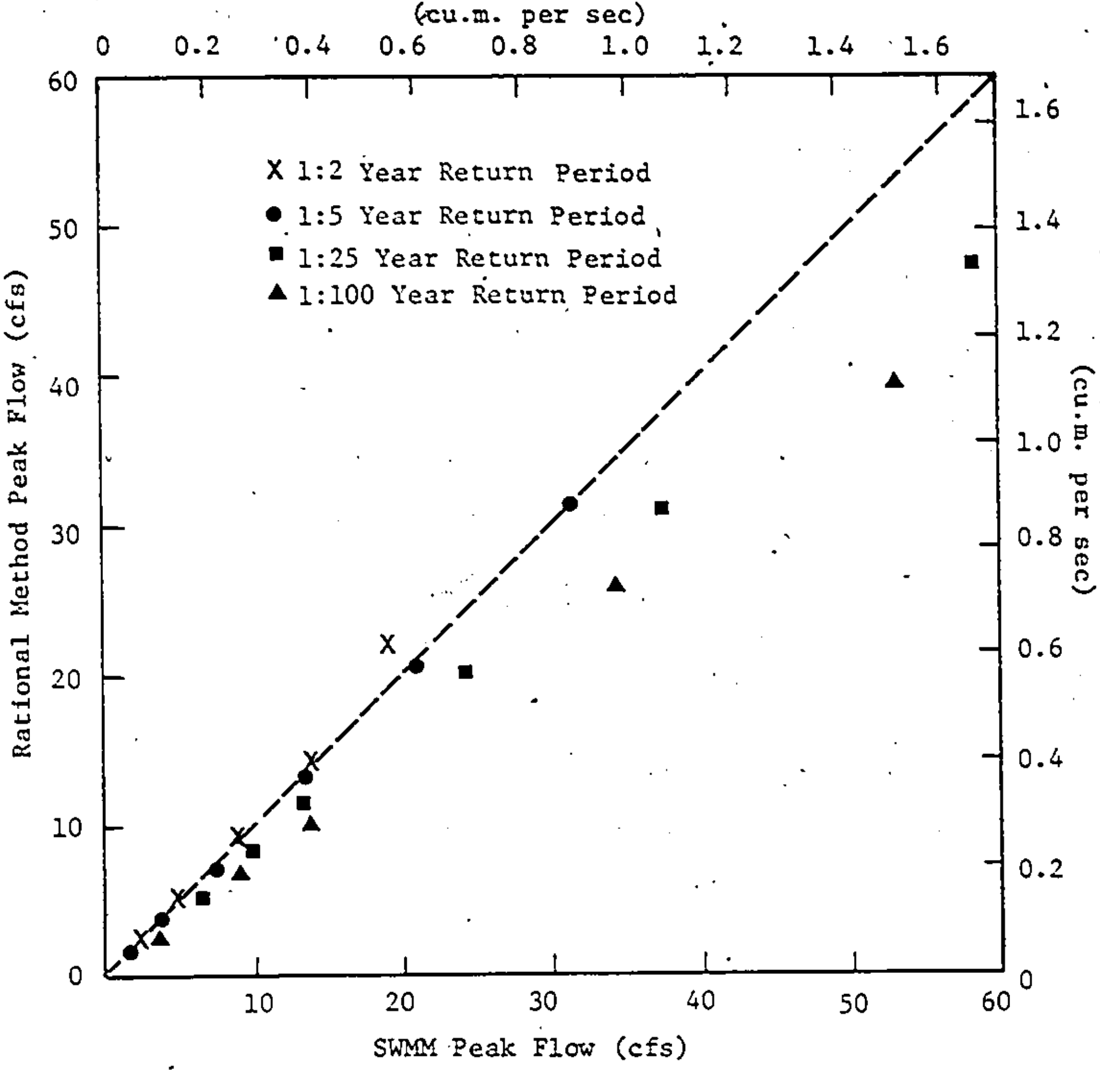


Figure 4.1: Comparison of the Rational Method and SWMM Flows Calculated for "Small" Watershed B with Imperviousness Ratio of 30%

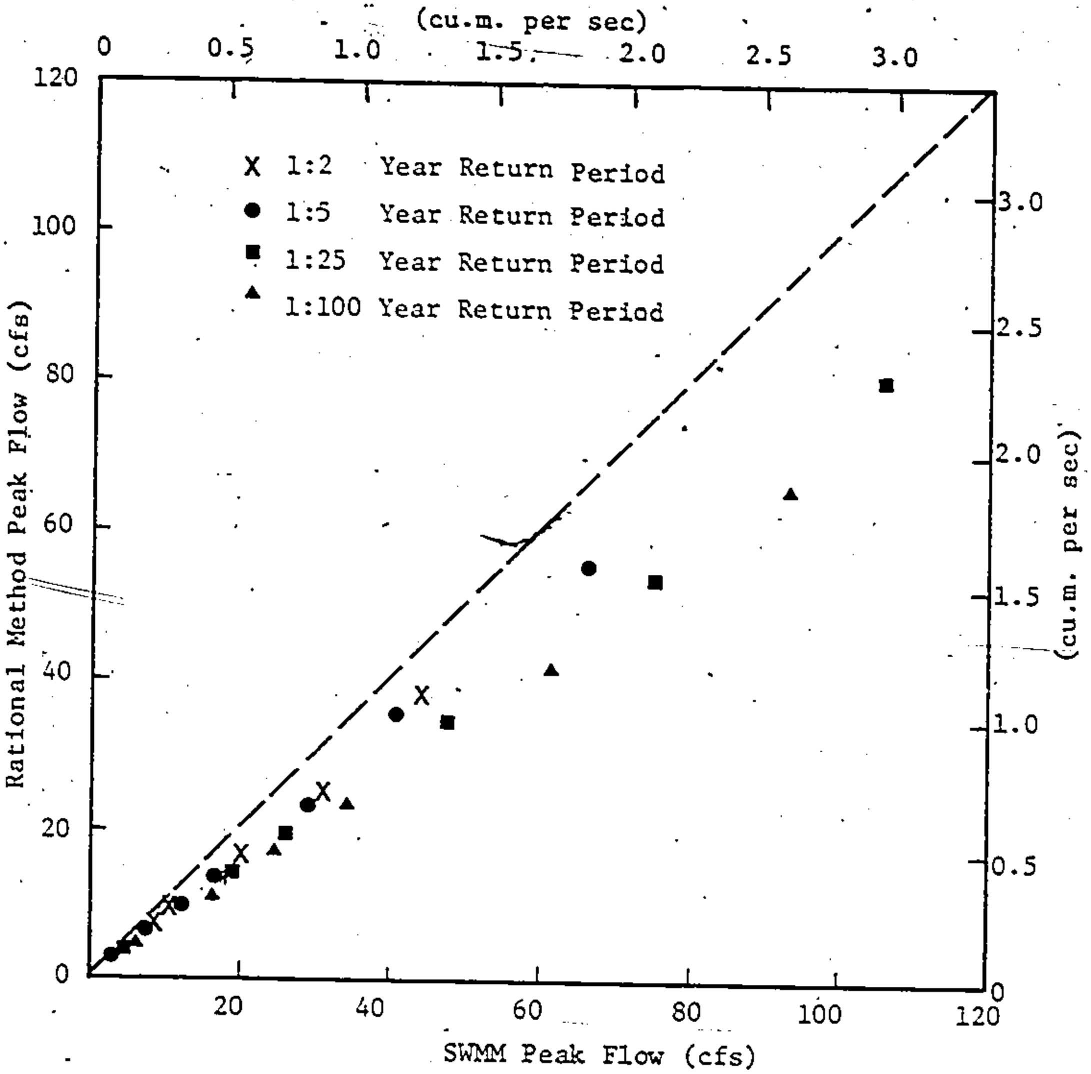


Figure 4.2: Comparison of the Rational Method and SWMM Flows Calculated for "Small" Watershed B with Imperviousness Ratio of 70%

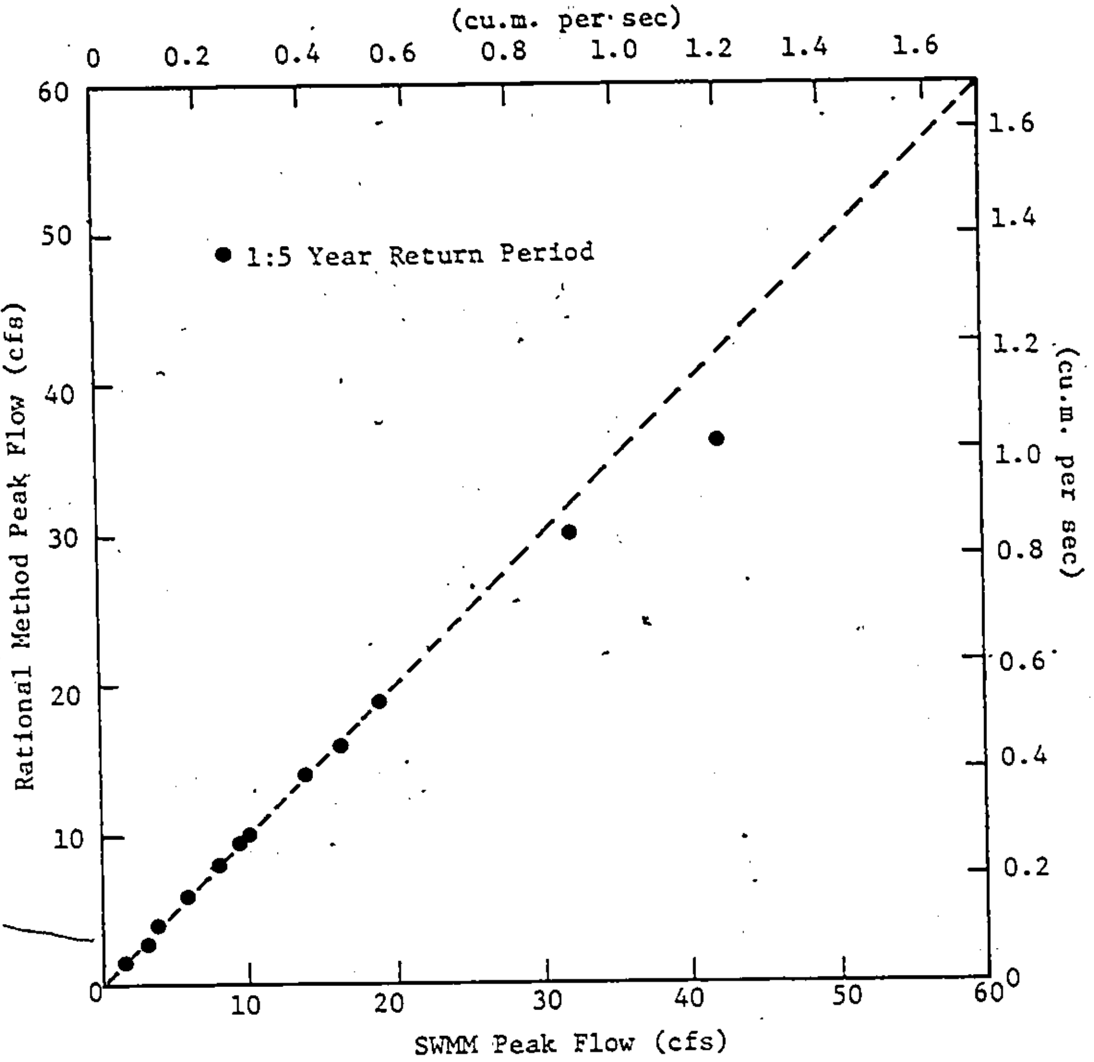


Figure 4.3: Comparison of the Rational Method and SWMM Flows Calculated for "Small" Watershed A with Imperviousness Ratio of 30%

(i.e. $t_p = t_i + t_t$, see Appendix A) and the estimation of flows downstream. This method of adjustment is still valid even when the configuration of the drainage system is changed since similar adjustments are required for both Watersheds A and B. The mean t_i values used for adjusting the Rational Method peak flows for different percentages of imperviousness and various design storm return frequencies for both test watersheds are plotted as shown in Figure 4.4. The following important points can be observed:

1. t_i decreases as the imperviousness ratio increases - reflecting that a higher percentage of paved surfaces have resulted in the ease of conveyance and a shorter time of concentration;
2. t_i decreases as the storm frequency decreases - reflecting that higher intensity and total rainfall have resulted in the earlier saturation of the soil, higher amount of runoff, and therefore a shorter inlet time;
3. The total time of concentration calculated is relatively short. The average total time of concentration for both watersheds A and B is approximately 15 minutes (with a maximum of 20 minutes) for an area of 23 acres. Some of the typical values of t_c and fitted t_i are shown in Table 4.1. It shows that t_i is a large percentage of the total time of concentration in "small" watersheds.

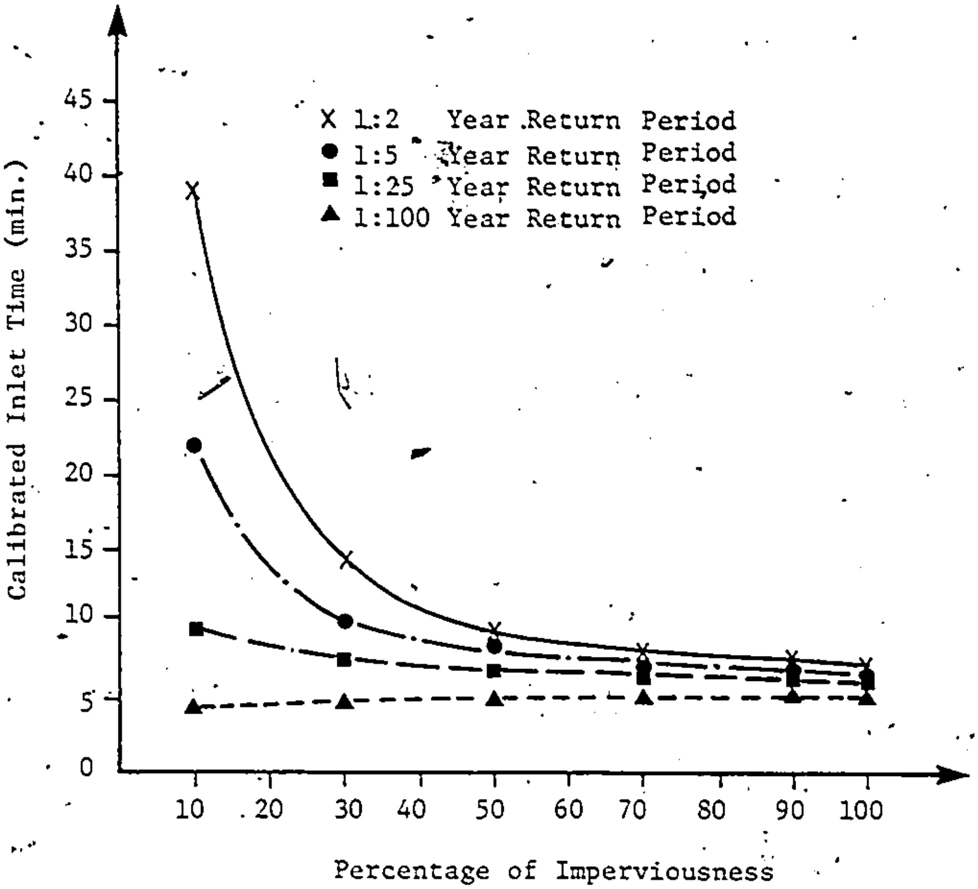


Figure 4.4: Calibrated Inlet Time versus Percentage of Imperviousness for Different Design Rainfall Periods Using C(RM)

Table 4.1

Comparison of Inlet Time and Time of Concentration for "Small" Watersheds

Design Storm	Percentages of Imperviousness					
	30%			70%		
	t_i (min)	t_c (min)	t_i/t_c	t_i (min)	t_c (min)	t_i/t_c
1:2	14.0	20.0	.70	8.5	12.0	.71
1:5	10.0	16.0	.63	7.5	11.0	.68
1:25	7.5	12.0	.63	6.5	9.5	.68
1:100	5.0	10.0	.50	5.5	8.5	.65

4.2.2 t_i and C Varied

With the second approach, the volumetric runoff coefficient is first calculated with SWMM: C(SWMM) = Q/P. The volumetric runoff coefficients, C(SWMM), determined for each return period and different imperviousness ratios are plotted as shown in Figure 4.5. It is noticed that the volumetric runoff coefficients C(SWMM) vary significantly with the percentage of imperviousness and and the design storm return period. A plot of C(RM) calculated with the weighted average equation (Eqn. 2.7) is also shown in the same figure. It shows that the curve for C(RM) is situated near to the average of C(SWMM) when the imperviousness is less than 50% but is less than C(SWMM) when the percentage of imperviousness is higher.

By using C(SWMM) instead of C(RM), the inlet time t_i is again adjusted until the Rational Method peak flows are equal to SWMM. The t_i used for each case is plotted against the percentage of imperviousness for each design storm return frequency as shown in Figure 4.6. Trends different from the first approach are observed. Although t_i decreases with increasing percentages of imperviousness, the overall change in t_i for the 1 in 2 years design storm is less than 2 minutes when the percentage of imperviousness increases from 10% to 100%, while the overall change in t_i is 4 minutes for the 1 in 100 years design storm.

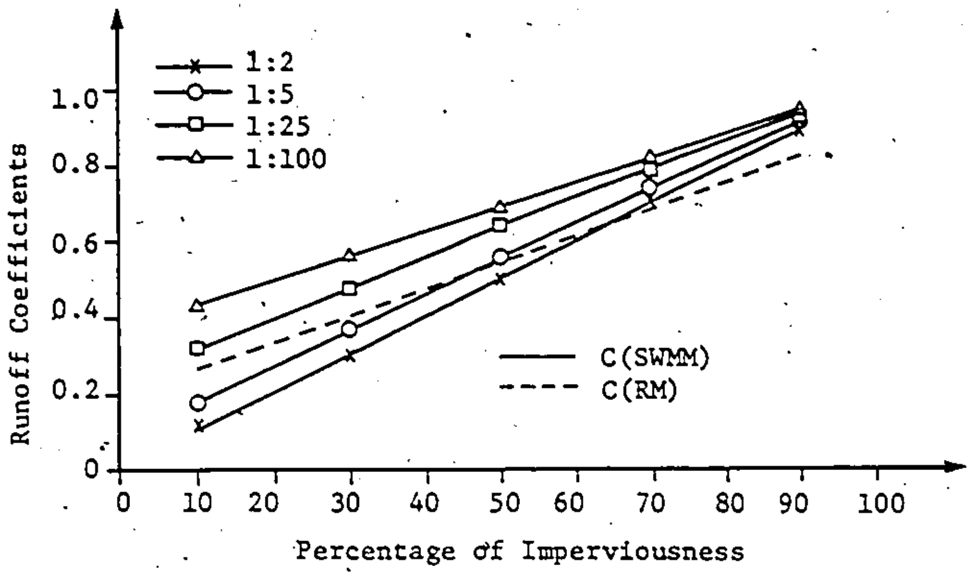


Figure 4.5: Volumetric Runoff Coefficients Determined by SWMM for Different Design Storm Return Periods and Percentages of Imperviousness

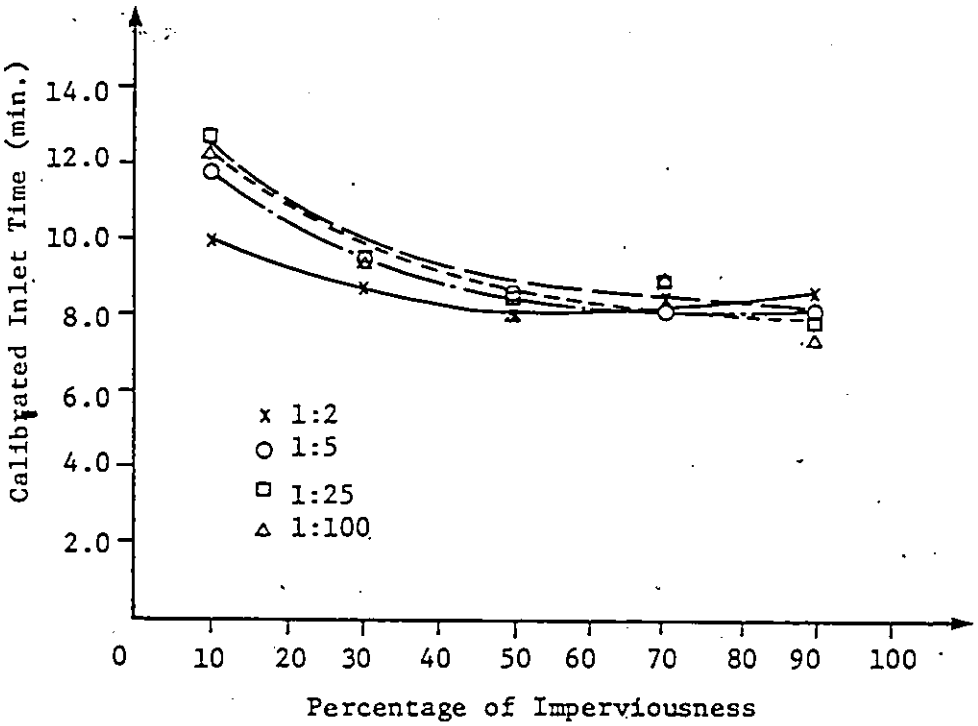


Figure 4.6: Calibrated Inlet Time versus Percentage of Imperviousness for Different Design Rainfall Periods Using C(SWMM)

Instead of a decrease in the inlet time when major storms (1 in 25 and 1 in 100 years storms) occur, it is observed that the inlet time increases for the low percentages of imperviousness. The t_i for the 1 in 5, 25, and 100 years storm are, for instance, longer than the t_i for the 1 in 2 years storm. Thus the inlet time cannot be consistently related with the return period of the design storm as in the first approach. It is therefore concluded that the first approach is more appropriate for 'adjusting' the Rational Method peak flow estimations and will be used for the analysis of the Rational Method for "large" watersheds.

4.3 DISCUSSION OF RESULTS FOR "LARGE" WATERSHEDS

Numerical experiments are carried out with three imperviousness ratios, namely 30%, 70%, and 100%, and design storms with four different return periods, namely the 1 in 2, 1 in 5, 1 in 25, and 1 in 100 years for Test Watershed C (see Section 3.4.1). For Test Watershed D, only the 30% imperviousness ratio is analyzed.

The discrepancies between the Rational Method peak flows (Q(RM)) and the SWMM peak flows (Q(SWMM)) are expressed in terms of the ratios of Q(SWMM) to Q(RM). They are plotted against the time of concentration as shown in Figures 4.7 to 4.9. Q(RM) is equal to Q(SWMM) for each sub-watershed; hence the ratios of Q(SWMM) to Q(RM) all

RATIO OF Q(SWMM) TO Q(RM) AGAINST THE TIME OF CONCENTRATION FOR THE IMPERVIOUSNESS RATIO OF 30 %

(Test Watershed C)

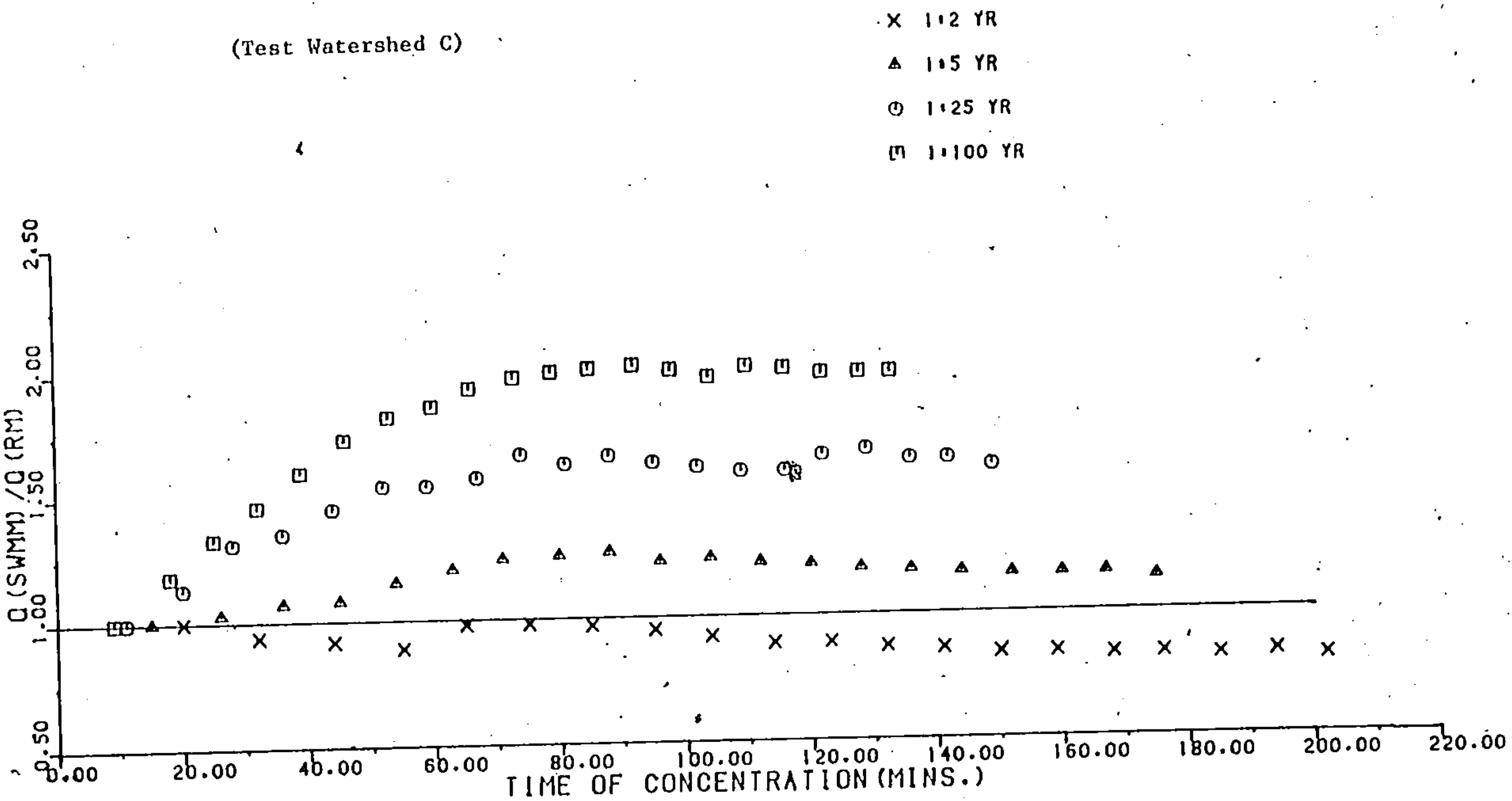


FIGURE 4.7

RATIO OF Q(SWMM) TO Q(RM) AGAINST THE TIME OF CONCENTRATION
FOR THE IMPERVIOUSNESS RATIO OF 70 %

(Test Watershed C)

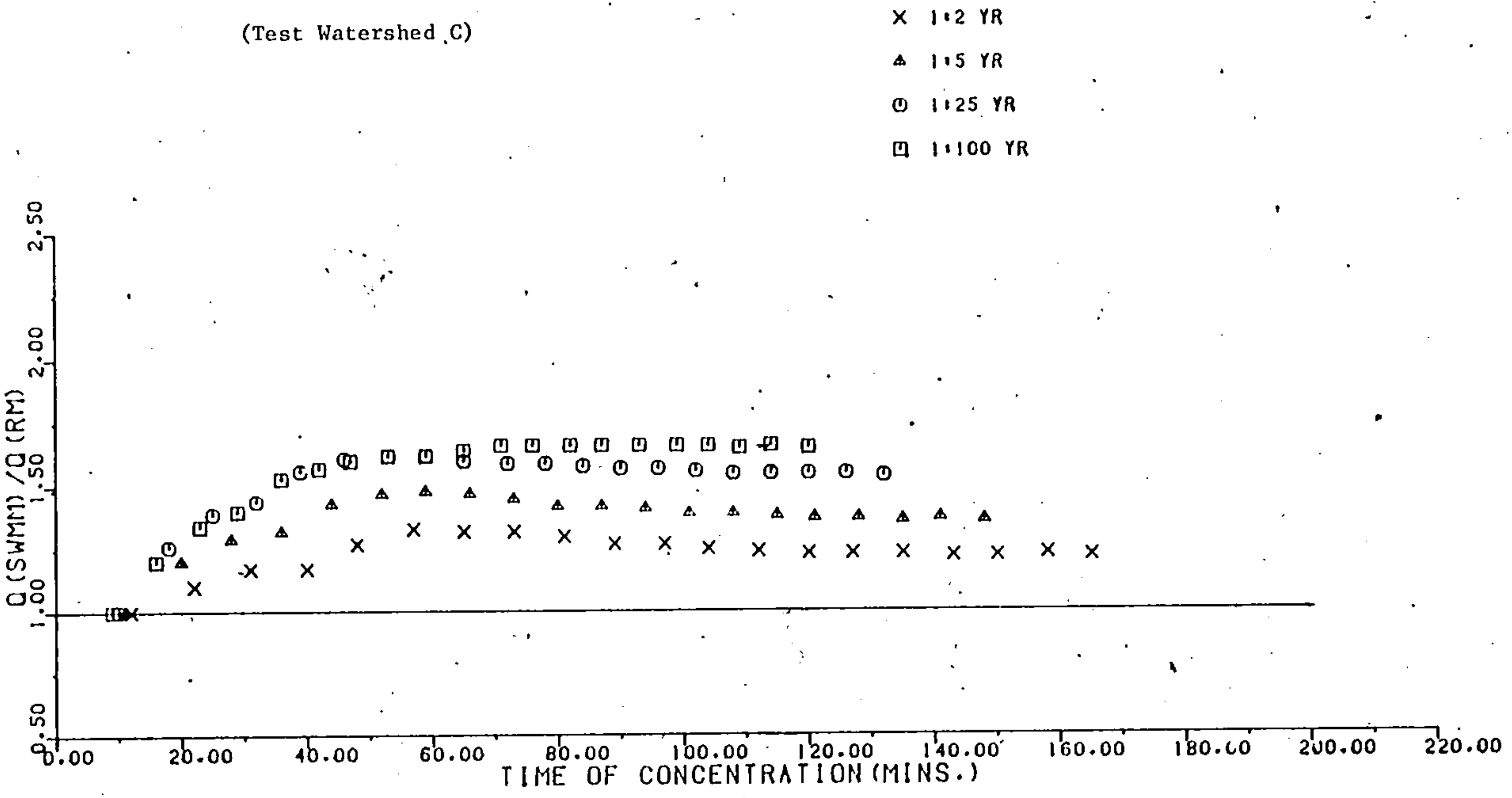
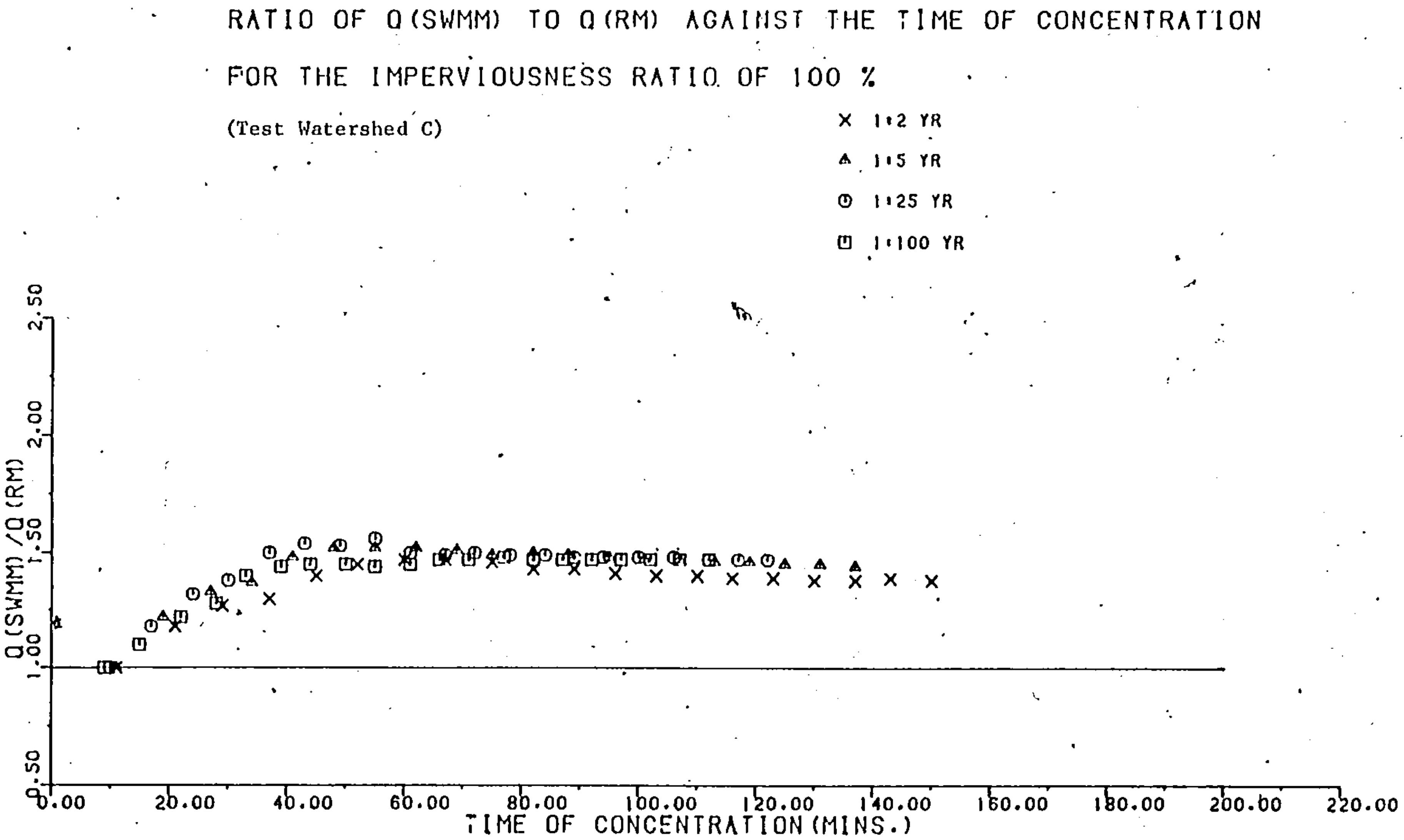


FIGURE 4.8


RATIO OF Q(SWMM) TO Q(RM) AGAINST THE TIME OF CONCENTRATION

FOR THE IMPERVIOUSNESS RATIO OF 100 %

(Test Watershed C)

FIGURE 4.9

start at unity. The following important points are interpreted from the results:

1. The Rational Method peak flows are systematically lower than the SWMM flows (with the exception of the 1 in 2 years storm with the 30% imperviousness ratio) when the area of the watershed increases (i.e. when the time of concentration increases). It is concluded that for the "large" watersheds, the Rational Method cannot give satisfactory results even when the 'correct' inlet time is used. It is concluded that other parameters such as the routing effect and the runoff coefficient (which varies with the storm duration, Section 2.2.) must be considered for "large" watersheds. Their effects are being reflected in the ratios of Q(SWMM) and Q(RM).
2. It is apparent that the Rational Method has resulted in higher peak attenuation as the drainage area increases. Since the routing procedure by EXTRAN has been shown to give good estimations when it is compared with other routing methods (Wisner and Kassem, 1980), it can be concluded that provided that there is no surcharge, the Rational Method underestimates the peak flow values as the effect of routing becomes significant.

The under-estimation of the Rational Method flows may be investigated further by referring to Figure 4.10 which compares the plots of peak flow (Q_p) per unit area (A) versus area for both the Rational Method and SWMM. The ratio of Q_p per unit area reflects the amount of attenuation of the peak runoff rate. As the area of the watershed increases, the ratio of Q_p per unit area decreases because of additional attentuation and the offset in time of additional inflows. It can be seen that the peak flow per unit area for the Rational Method is much lower than that simulated by the SWMM runoff and EXTRAN routing models.

As mentioned earlier, the routing of flows by the Rational Method is substituted by means of a simplified procedure. It can be seen that the routing effects are accounted for in the Rational Method mainly by the shape of the I.D.F. curve:

$$Q_p = CiA \qquad 4.3$$

$$Q_p/AC = i = f(t_c) = f(I.D.F.) \qquad 4.4$$

The assumption of the Rational Method in the use of the I.D.F. curve has caused the errors to occur because in effect the shape of the curve of Q_p/A for

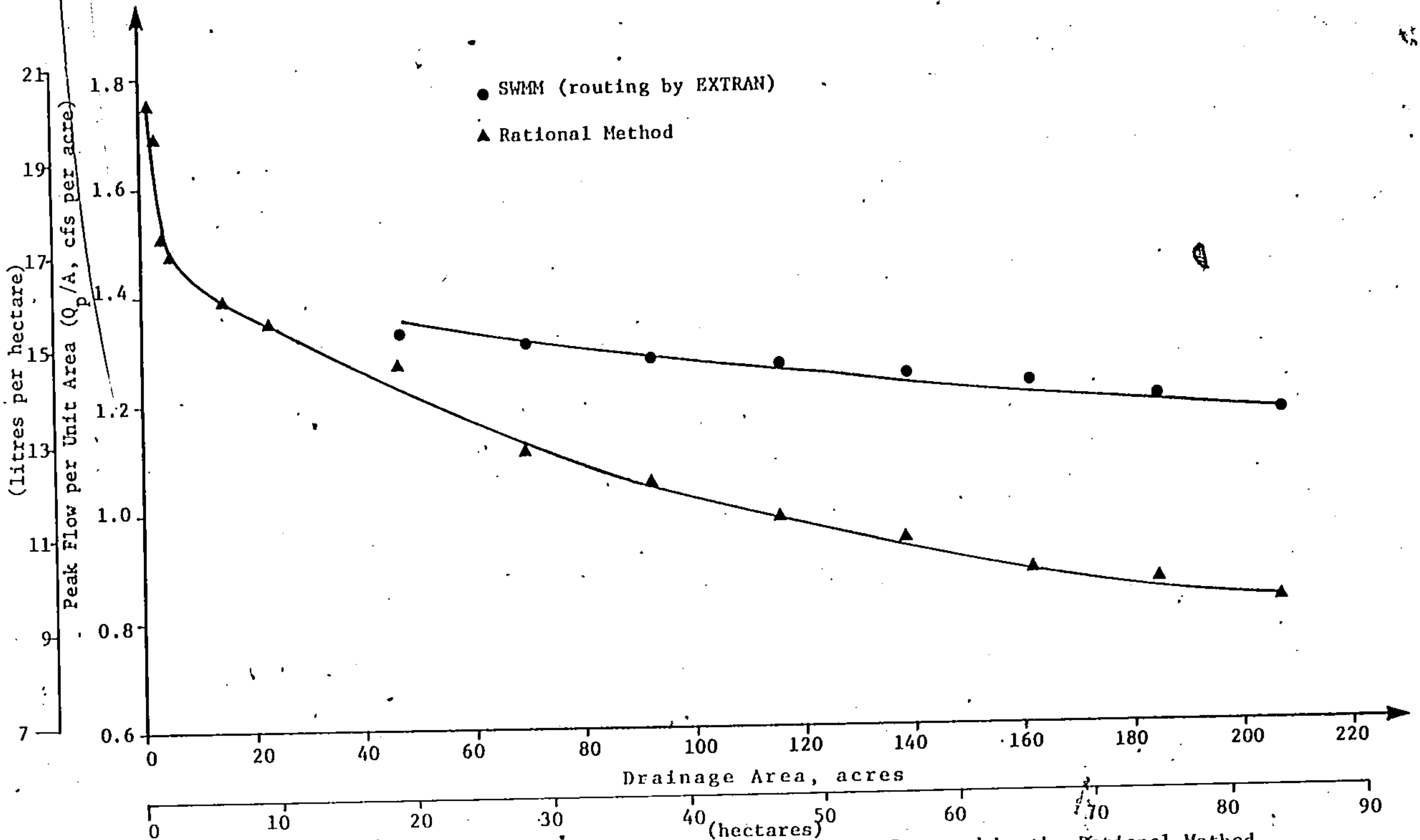
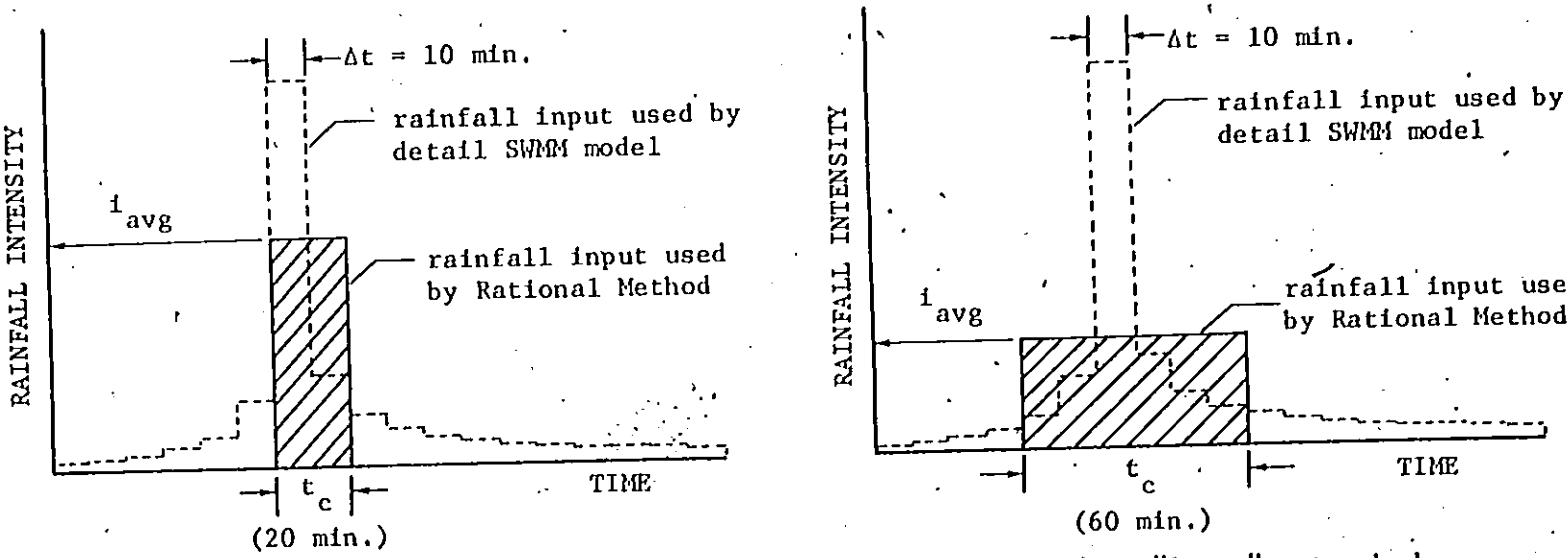


Figure 4.10: Peak Flow per Unit Area versus Drainage Area Computed by the Rational Method and SWMM

the Rational Method follows the shape of the I.D.F. curve. Hence, the Rational Method does not 'route' by means of the flow velocity and flow continuity but it only 'routes' by means of the shape of the I.D.F. curve, which is incorrect.

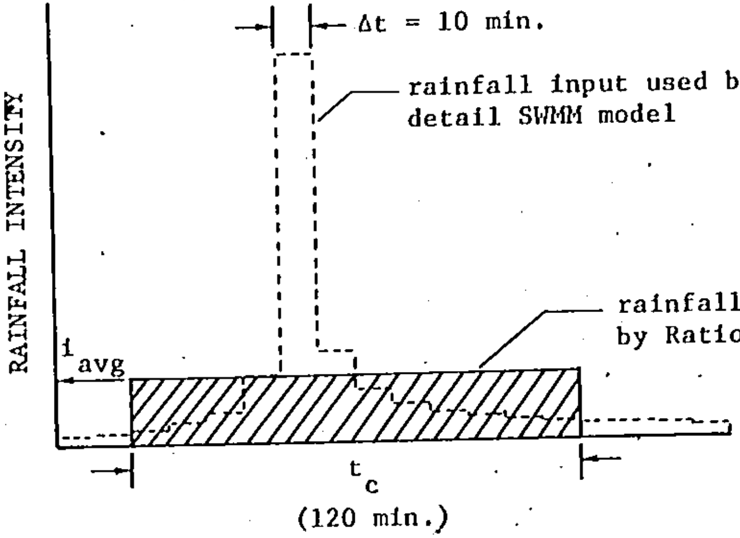
Consequently, it is concluded that the use of I.D.F. curves in conjunction with the time of concentration in the Rational Method will always lead to significant errors for flow computations in "large" watersheds.

3. It is also noticed that for "large" watersheds for which the time of concentration is long and t_i is insignificant, the relationship between the time of concentration and the discretization time step of the design storm may also have to be carefully considered (see Section 3.3, Selection of Standard Storm Input). If the time of concentration is relatively close to the discretization time step of the design storm, the rainfall intensities during this period of time can be considered to be relatively uniform. However, if the time of concentration is much longer than the discretization time step of the design storm, the rainfall intensities during this period of time will actually vary significantly in accordance with the time distribution of the rainfall hyetograph (Figure 4.11).



(a) low t_c, "small" watershed

(b) long t_c, "large" watershed



(c) long t_c, "large" watershed

Figure 4.11: Comparison of Rainfall Input Used by the Rational Method and Detail Design Model

Since the Rational Method uses only a uniform "block" of average rainfall intensity, the rainfall input used by the Rational Method, as the drainage area increases, will not be compatible with the storm profile used by the SWMM model. In this case, the "block" rainfall used by the Rational Method has a very low average intensity as compared with the 'peaky' intensities of the design storm hyetograph used by SWMM. It can therefore be concluded that when non-uniform storms are applied, and when the routing of flows are important, the Rational Method does not give satisfactory results as compared with the more detailed SWMM model which can accept complete storm profiles as input.

It may be possible to see whether the use of a block uniform rainfall in SWMM with the same average intensity as in the Rational Method will make the Rational Method flows closer to SWMM. (The duration of this block rainfall is equal to the time of concentration determined in the Rational Method (Figure 4.11).)

However, this will result in flows that are less critical than those simulated with the 'peakier', non-uniform design storms (Section 3.3, Selection of Standard Storm Input). In fact, the peak intensities of non-uniform storms are much higher than the uniform storms since the latter are expressed in average values. As an illustration, Table 4.2 compares the peak intensities based on the same I.D.F. curves for the uniform rainfall used by the Rational Method and the non-uniform rainfall used by SWMM when the time of concentration of the watershed is 1 hour and 2 hours, respectively (this corresponds approximately to a watershed with sizes in the range of 160 acres and 350 acres or larger, respectively, depending on the storm return frequency).

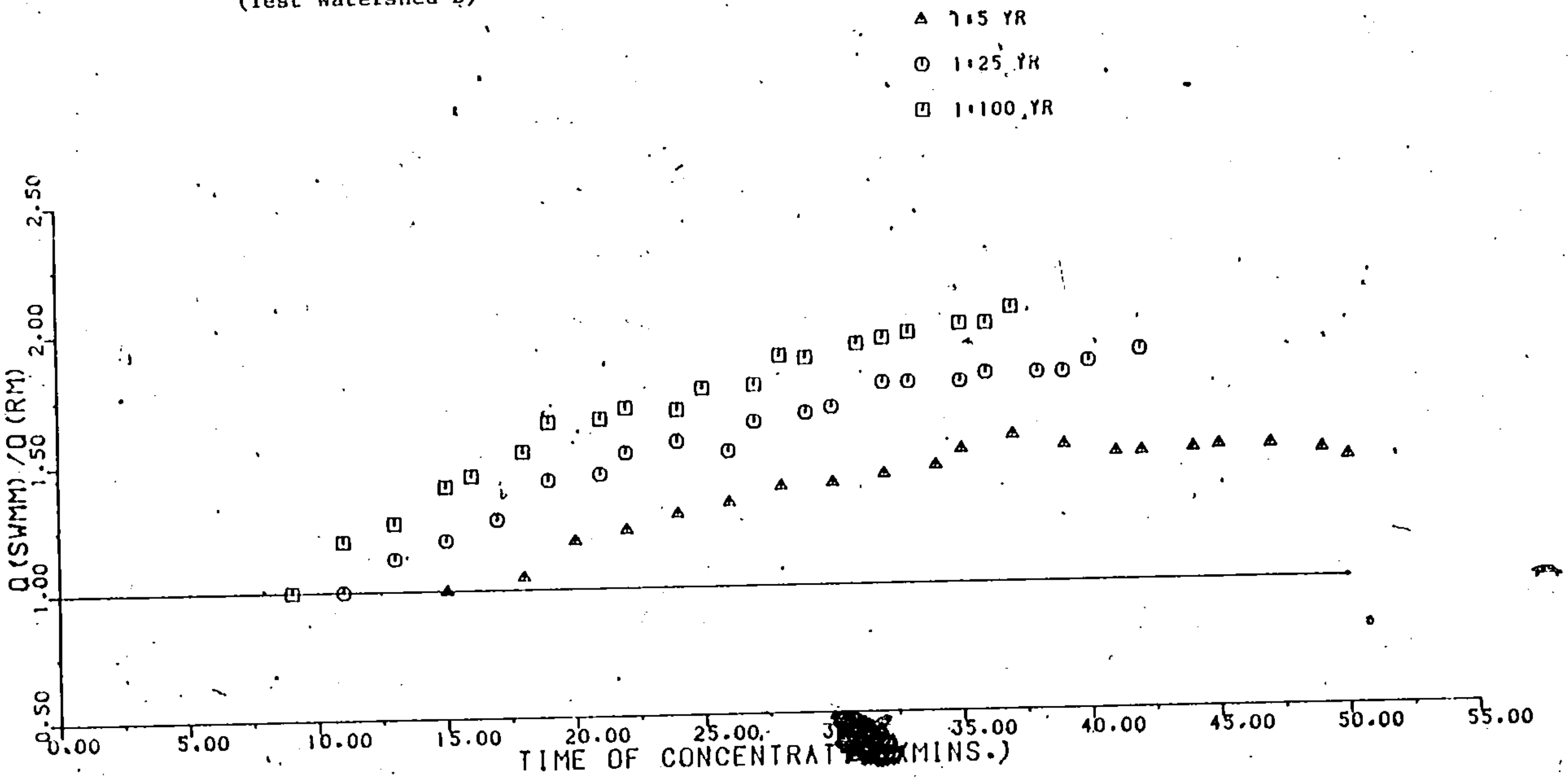
It is also noted that for urban runoff designs, non-uniform storms should be used since they are more critical and they correspond more closely to an actual storm profile.

4. A different configuration of watershed is tested by means of Test Watershed D (Figure 3.11, Section 3.4.1). The distances between the inlet locations are reduced by four times. This is not a typical system since the actual distance between real watersheds with sizes of 23 acres will not be as low. However, the same discrepancies between the Rational Method and the SWMM model are observed (Figure 4.12).
5. The above analyses have shown the extent of discrepancies that can be expected from the Rational Method

Table 4.2

Comparison of Peak Rainfall Intensities Used by the Rational Method and SWMM

Design Storm	Block Rainfall Used in Rational Method		Design Rainfall Used in SWMM
	D = 2 hours (in/hr) mm/hr	D = 1 hour (in/hr) mm/hr	D = 3 hours (Δt = 10 min) (in/hr) mm/hr
1:2	.56 (14.2)	.92 (23.4)	3.07 (78.0)
1:5	.79 (20.1)	1.28 (32.5)	4.34 (110.2)
1:25	1.09 (27.7)	1.79 (45.5)	6.30 (160.0)
1:100	1.46 (37.1)	2.44 (62.0)	7.76 (197.1)



RATIO OF Q(SWMM) TO Q(RM) AGAINST THE TIME OF CONCENTRATION

FOR THE IMPERVIOUSNESS RATIO OF 30 %

(Test Watershed D)

FIGURE 4.12

when it is used to estimate peak flows from "large" watersheds. Adjustments of the flows estimated by the Rational Method are required for watersheds with a time of concentration longer than 20 minutes and it would consist of applying the correction coefficients similar to Figures 4.7 to 4.9.

The survey of current practices (IMPSWM Program 1980, Section 2.2.1) in urban drainage designs has indicated that the Rational Method has been used for determining flows in the major system for rainfall with low frequencies such as the 1 in 100 years, yet the study carried out here shows that by applying the Rational Method with low frequency, major storm events will result in significant errors.

4.4 SUMMARY AND CONCLUSIONS

The assessments conducted above have enabled the validity of the Rational Method in estimating the design peak flows for both "small" and "large" urban watersheds in relation to the design storm return frequencies and the imperviousness ratios to be extensively investigated. The causes of the discrepancies have been examined and the methodologies for improving the accuracy of the Rational Method have been proposed. Basically, improvements are required for the Rational Method before compatible flows with SWMM simulations can be obtained. They can be summarized as follows:

1. The adjusted t_i values shown in Figure 4.4 are reasonable estimates for which the Rational Method can be improved for calculating more compatible peak flows for "small" urban watersheds. Since they are obtained based on comparisons with SWMM simulations, they incorporate indirectly the various effects of the watershed characteristics, rainfall characteristics, and antecedent moisture conditions (the initial value of infiltration rate used by SWMM). They can be applied for various design storm frequencies and imperviousness ratios.

   For practical applications, it is concluded that a conservative value of inlet time for runoff computations is 10 minutes for the design storm frequencies of 1 in 2 and 1 in 5 years, and 5 minutes for the design storm frequencies of 1 in 25 and 1 in 100 years, respectively.

2. For larger watersheds for which t_i is not a significant factor, flows determined with the Rational Method are systematically lower than SWMM even when the 'correct' inlet time is used. These discrepancies are caused when the routing effect is significant and when the time of concentration is relatively longer than the discretization time step of the design storm; another parameter the effects of which

increase with the storm duration is the runoff coefficient. All of these effects are being reflected in the ratios of Q(SWMM) to Q(RM), which can be viewed as 'correction factors' for the Rational Method flows. Hence, if the Rational Method is applied in "large" watersheds, 'correction factors' similar to that shown in Figure 4.7 to 4.9 are needed to adjust the Rational Method flows to be equal to SWMM flows. It is observed that these factors have maximum values and they are shown in the following table:

Maximum Ratios of Q(SWMM) to Q(RM)

Percentage of Imperviousness	Design Storm Return Frequency (yr) 1:2	1:5	1:25	1:100
30 %	0.9	1.2	1.6	2.0
70 %	1.3	1.45	1.6	1.65
100 %	1.5	1.5	1.5	1.5

In light of these limitations, it is concluded that the Rational Method can give results compatible with a detailed hydrograph model only for relatively "small" watersheds. The use of correction coefficients makes it less attractive to be used. These correction coefficients are limited for homogeneous watersheds and may differ from waterhsed to watershed. (However, the correction coefficients obtained in this study have demonstrated the extent of discrepancies expected from the Rational Method.) They must also be related with the travel time of flow in the pipe system. Since the travel time of flow varies according to the characteristics of the pipe size and slope, the correction coefficients may not apply for a pipe system with very large sizes or flat invert slopes. The improvement parameters will also change when a different design storm distribution is used.

The Rational Method is also limited to estimating only peak flows instead of full runoff hydrographs which are required for the analysis of runoff control facilities. An improved desk-top method of runoff computations must therefore be explored.

Chapter V

DEVELOPMENT OF STANDARD HYDROGRAPH METHOD (SHM)

5.1 INTRODUCTION

In light of the limitations and discrepancies in the presently applied Rational Method and hydrograph methods, such as the SCS TR-55 procedures, there is a need for a more consistent desk-top method; a desk-top method that predicts not only the peak discharge rates and runoff hydrographs but also gives results that are compatible with the more sophisticated computer models used in the final stage of design. Pursuant to this objective, it is attempted in the present study to develop an improved methodology for the preliminary analysis and design of urban drainage systems with the following characteristics:

1. The derivation of runoff volumes, peak discharge rates, and hydrographs for applications in small urban watersheds utilizing a "standard" design storm that is applied by many cities and regulatory agencies for the analysis of urban drainage systems.
2. The derivation of the "SHM" based on simulations with a standard computer model that has been widely applied and well tested in Canada.

5.2 THE PEAK RUNOFF RELATIONSHIP

Early attempts have been made by researchers to express the peak runoff in terms of a single variable, the area of the watershed. These expressions have normally been derived empirically and do not take into account the time or frequency factor. Subsequently, the Rational Method which considers the rainfall characteristics has been proposed by Kuichling (1889). Similar types of formulas have also been proposed by Fuller (1917) and Pettis (1934).

Other attempts have related the runoff rate Q(t) in terms of the area A of the watershed and the depth of runoff Q at time t. For example, the two-parameter gamma function has been applied by Dooge (1959) and Nash (1960) for synthesizing the unit hydrograph.

$$\frac{Q(t)}{AQ} = \frac{1}{t_p} \cdot \frac{(n-1)X^{n-1}\ e^{-X}}{\Gamma(n)} \qquad 5.1$$

in which

$$X = t(n-1)/t_p \qquad 5.2$$

where t_p is the time of peak, n is the number of linear reservoirs depicting the amount of flow attenuation and storage by the watershed, and $\Gamma(n)$ is the gamma function equal to (n-1)!. Wu (1963) has shown that the U.S. Soil Conservation Service method (Section 2.3.3) for computing the peak runoff rate from a watershed is also based on this function. He has shown that Equation 5.1 can be rewritten as:

$$\frac{Q_p}{AQ} = \frac{1}{t_p} f(n,t_p) \qquad 5.3$$

$$f(n,t_p) = 0.756 \qquad 5.4$$

$$Q_p = \frac{640 \times 0.756 \times A \times Q}{t_p} = \frac{484AQ}{t_p} \qquad 5.5$$

Equation 5.5 is the formula used by the U.S. Soil Conservation Service method.

A similar relationship which defines the peak runoff rate from a watershed in terms of the runoff depth and watershed area has also been applied for a variable storm profile by Young and Prudhoe (1973). These authors have derived it with average annual rainfall and large watersheds with relatively impermeable soil cover (clays or boulder clay) for which the runoff is equal to the rainfall amount.

For any given storm it has been possible to derive the runoff relationships by assuming conceptually that the watershed resembles a single reservoir. The inflow and outflow rates of the watershed are given by the reservoir equation:

$$i - q = \frac{dS}{dt} \qquad 5.6$$

where i is the inflow rate, t is the time, and S is the quantity of water stored which is a function of the rate of outflow q. The solution of Equation 5.6 has been derived by Younge and Prudhoe (1973):

$$q = \frac{1}{\mu} \int_o^t \frac{\mu i dt}{f'(q)} \qquad 5.7$$

where $$\mu = \text{Exp}\int \frac{dt}{f'(q)} \qquad 5.8$$

$$f'(q) = \frac{d(f(q))}{dS} \qquad 5.9$$

where μ is an integrating factor. The instantaneous inflow rate i can be expressed in terms of the mean annual rainfall $\bar{i}$ and a deviation i(t) from the mean

$$i = \bar{i} + i(t) \qquad 5.10$$

Hence by substituting Equation 5.7 into Equation 5.10

$$q = \frac{1}{\mu}\int_o^t \frac{\mu \bar{i} dt}{f'(q)} + \frac{1}{\mu}\int_o^t \frac{\mu i(t) dt}{f'(q)} \qquad 5.11$$

Most hydrographs that result from a single rainfall event rise steadily to peak, that is, they are continuous and monotonic functions. This is also true of the storage-discharge relationships. Thus, the reservoir characteristics

S=f(q) can be represented by a power series in terms of the runoff or flow rate (q), and q can be expressed in terms of the time (t), that is, Q(t):

$$S = f(q) = k_1q + k_2q^2 + k_3q^3 + \ldots + k_jq^j = \sum_{j=1}^{n} k_jq^j \qquad 5.12$$

$$Q(t) = f(t) = b_1t + b_2t^2 + b_3t^3 + \ldots + b_jt^j = \sum_{j=1}^{n} b_jt^j \qquad 5.13$$

By differentiating both power series and by further algebraic manipulation (Young and Prudhoe 1973), Equation 5.11 can be expressed as:

$$Q(t) = \frac{\bar{i}t}{k_1} + \bar{i}\,[g_2t^2 + g_3t^3 + \ldots] + \frac{1}{\mu}\int_o^t \frac{\mu i(t)dt}{f'(q)} \qquad 5.14$$

where the coefficients g are the appropriate combinations of b_j and k_j of the two power series. Also the coefficients g and their associated term in Equation 5.14 are such that their contribution to q is very small. Since by definition $\int_0^t i(t)\,dt$ is equal to zero and the function $\mu/f'(q)$ tends to be very small, Equation 5.14 can be expressed as

$$Q(t) = \frac{\bar{i}t}{k_1} + \varepsilon \qquad 5.15$$

where ε is a small quantity and is negligible (Young and Prudhoe 1973), and k_1 is the first coefficient in the power series of S. For a single storm event, putting k_1 equal to k and $\bar{i}$ equal to Q/t, where t is the storm duration and Q is the total excess rainfall volume:

$$Q(t) = \frac{Q}{k} + \varepsilon \qquad 5.16$$

In Equation 5.16,, the excess rainfall Q is expressed as a volume, but it can also be expressed in its most usual form in terms of a linear measure (i.e. depth). k is a parameter with the dimension of time; it defines the response time of runoff of the watershed to rainfall precipitation and can be expressed in terms of the watershed and storm characteristics. Thus a linear equation is formulated:

$$Q(t) = \frac{AQ}{T} + \varepsilon \qquad 5.17$$

where Q(t) is the rate of runoff, Q is the total depth of excess runoff, and T is the response time of the watershed.

5.2.1 Proposed Formula

When Q(t) is equal to the peak flow Q_p and assuming that ε is negligible, Equation 5.17 can be expressed as:

$$\frac{Q_p}{AQ} = \frac{1}{T} \qquad 5.18$$

T is the response time of the watershed and therefore is a function of the watershed characteristics namely, the watershed length (L), slope (SLP), and imperviousness (I), and the rainfall characteristics (ST):

$$T = f(L,SLP,I,ST) \qquad 5.19$$

therefore,

$$\frac{Q_p}{AQ} = f\ (L,SLP,I,ST) \qquad 5.20$$

By expressing the term Q_p/AQ in terms of a peak factor P, a simple formula can be obtained:

$$Q_p = A.Q.P. \tag{5.21}$$

where Q_p is the peak runoff rate with the dimension of L^3/T, A is the area of the watershed with the dimension of L^2, Q is the depth of runoff with the dimension of L, and P is the peak factor with the dimension of 1/T.

5.2.2 Peak Factor P

The peak factor is the ratio of the peak runoff rate to the area of the watershed and the the depth of runoff. It decreases when the area of the watershed increases and therefore the routing effects of flows increase (or vice versa). The peak factor is used to define the peak rate of runoff and is a function of the watershed parameters for a given design storm and runoff characteristics.

5.2.3 Runoff Quantity Q

For a given design storm and imperviousness ratio, the total depth of runoff Q divided by the total depth of rainfall PRECIP results in a volumetric runoff coefficient R (as described before in Figure 4.5):

$$R = Q/PRECIP \tag{5.22}$$

In the present study, SWMM which uses the Horton's infiltration equation has been used to calculate the total runoff depths for the selected design storms.

5.2.4 Rainfall Peakiness Factor S

As described in Section 3.3, the characteristics of a design storm vary considerably when different methods and time steps are used to distribute the storm. For example, for the same discretization time step (10 min.) as shown in Figure 3.4, the peak intensity of the storm distributed with Keifer and Chu's method (1957) is much higher as compared with the SCS Type II distribution. Similarly, if a storm is distributed with different time steps but the same method, the peak intensity of the storm will also change correspondingly (Figure 3.6).

For urban areas, the response of runoff to rainfall is reflected relatively fast due to the high drainage density and ease of conveyance over paved surfaces. The peak rainfall intensity which is much higher than the intensities during other time steps (see Figure 5.1) therefore significantly dictates the peak runoff rate. Consequently, the peak runoff rates resulted from design storms distributed with Keifer and Chu's method are larger than those distributed with the SCS Type II Method. The same occur when a design storm is discretized with a shorter time step (thus re-

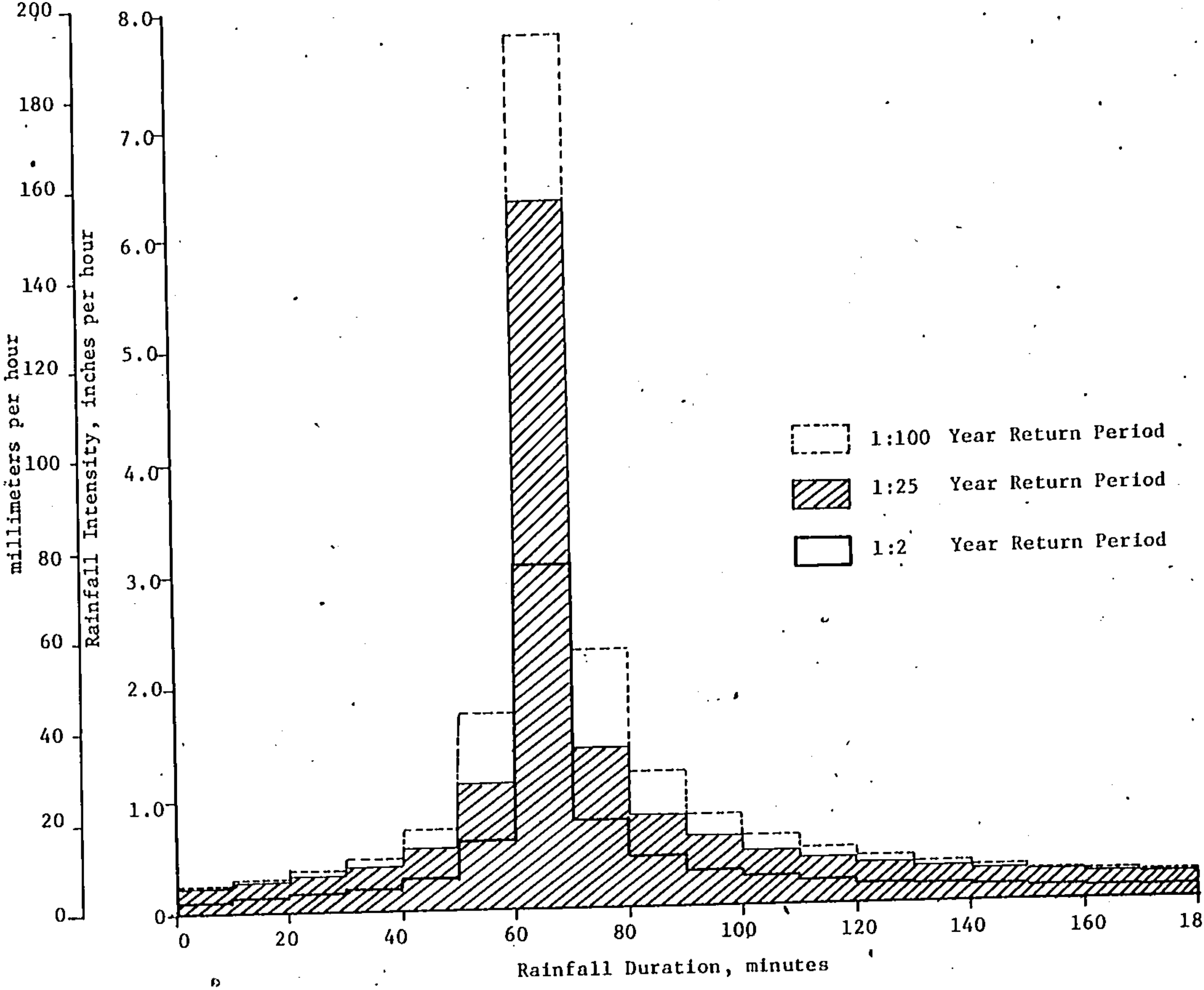


Figure 5.1: Design Storm Profiles Distributed with Keifer and Chu's Method (1957)

sulting in high rainfall intensities) or a longer time step (thus resulting in low rainfall intensities).

The characteristics of a given design storm can therefore be represented by means of a peakiness factor S. This factor can be used to relate the rainfall characteristics with the peak discharge rate. For instance, if the peak discharge caused by a design storm distributed with the Keifer and Chu's design storm method is Q(KC) and the peak discharge caused by a design storm distributed with the SCS 24-hour Type II method is Q(SCS), the rainfall peakiness factor can be used to modify the Q(SCS) to be equal to Q(KC). In this case, Q(SCS) will be less than Q(KC) because the rainfall intensities of the SCS design storms are less. Hence, if Q(KC) is used as a basis, the rainfall peakiness factor S for design storms distributed with Keifer and Chu's method is equal to unity. For other design storms, the S values will vary. S can be included in Equation 5.21 as follows:

$$Q_p = A.Q.P.S. \qquad 5.23$$

It is noted, however, that S does not have any physical meaning. It is only a ratio for defining peak runoff rates for other design storms in terms of the peak runoff rates calulated for the Keifer and Chu's design storms. S is also valid only for adjusting the peak runoff rate and not valid for adjusting the ordinates of the runoff hydrograph because the shape of the runoff hydrograph is also a function of the shape of the design storm hyetograph.

5.3 DETERMINATION OF RUNOFF HYDROGRAPHS FOR URBAN WATERSHEDS

The determination of the runoff hydrograph can be carried out by means of synthetic unit hydrographs. All unit hydrographs are based on a net unit rainfall of uniform intensity for a specific duration. For the Instantaneous Unit Hydrograph (I.U.H.), the duration of the unit rainfall is assumed to be zero. In order to derive a synthetic hydrograph for non-uniform storm patterns with varied rainfall intensities using the I.U.H., an incremental hydrograph is generated for the excess rainfall during each time step and then superimposed with the incremental hydrographs obtained for other time steps (Figure 5.2). This process is represented by the "convolution integral" which is also known as the "Duhamel integral":

$$Q(t) = \int_{o}^{D} U(0,t-\tau)I(\tau)dt \qquad 5.24$$

where Q(t) is the ordinate of the runoff hydrograph resulting from the complex storm, I(τ) is the effective rainfall intensity, D is the rainfall duration, t is the length of

If the runoff hydrograph resulting from a non-uniform rainfall instead of a uniform rainfall is divided by the total runoff volume, a hydrograph resulting from one unit of excess rainfall is obtained. This hydrograph is however different from the unit hydrograph because its shape incorporates both the profile of the effective rainfall hyetograph and the watershed characteristics.

This type of hydrograph has been used in the SCS TR-55 method (SCS 1975) and by Ragan et al (1975) wherein it has been employed as a simplified method for deriving the runoff hydrographs for a fixed profile of rainfall. For the SCS TR-55 Method, the SCS 24-hour Type II distribution is used. The use of this type of hydrographs can be demonstrated by means of Figure 5.2. If the effective rainfall intensities are all increased, for example by the same percentage (k%), thus maintaining the same profile of effective rainfall, the co-ordinates of the incremental hydrographs will also be increased by k% and the total runoff hydrograph co-ordinates will also be increased proportionally. Equation 5.25 can therefore be expressed as follows:

$$Q'(t) = kQ(t) = \sum_{i=1}^{n} U(0,t-(i-1)\Delta t)kI_i\Delta t$$

$$= k\sum_{i=1}^{n} U(0,t-(i-1)\Delta t)I_i\Delta t \qquad 5.27$$

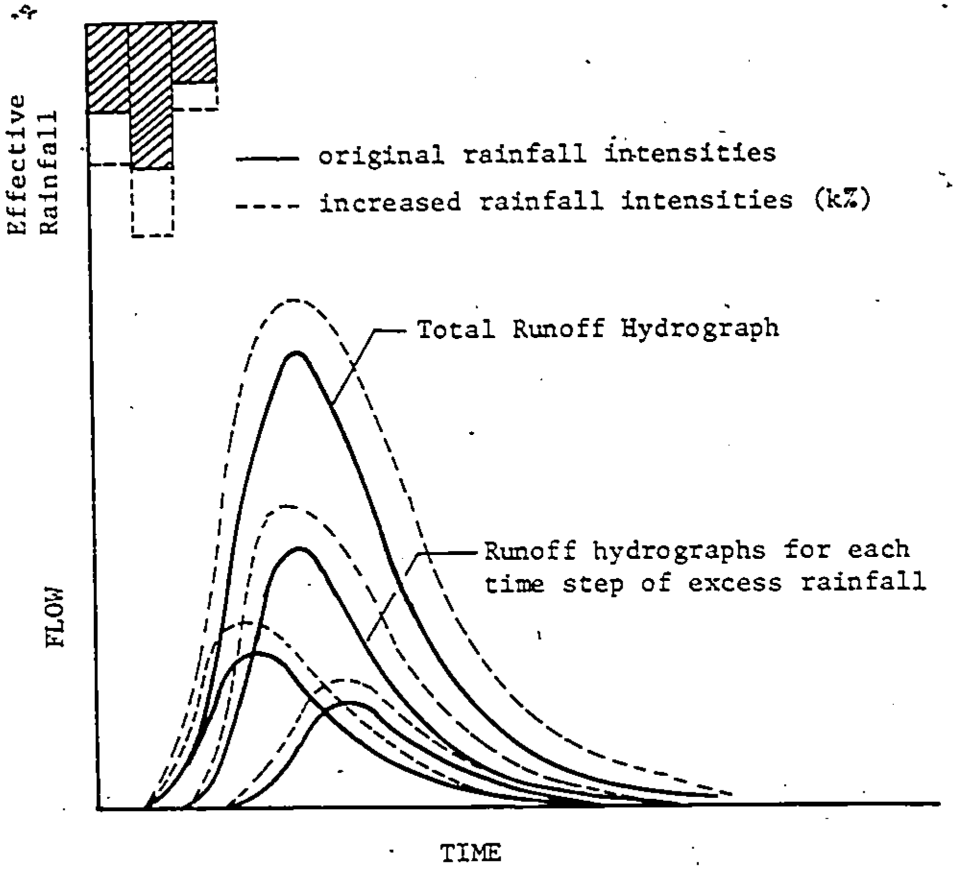


Figure 5.2 : Combining Hydrographs

each rainfall increment, and $U(0,t-\tau)$ is the kernel function of the unit hydrograph. The summation form (Chow, 1964) of the convolution integral is:

$$Q(t) = \sum_{i=1}^{n} U(0,t-(i-1)\Delta t)I_i\Delta t \tag{5.25}$$

where n is the number of different intensity blocks, t is the time from the beginning of the effective rainfall, and t is the discrete rainfall time steps.

The application of synthetic unit hydrographs in urban watersheds has been demonstrated by Eagleson (1962) and by Espey et al (1977). Eagleson has studied the hydrographs of measured storm sewer outflow from urban watersheds up to 7.5 sq.miles in size and correlated them with the properties of sewers and drainage characteristics in order to derive synthetic unit hydrographs for other unmeasured sewered areas. Using Snyder's formula, the peak Q_p of the synthetic unit hydrograph per unit area A is expressed by

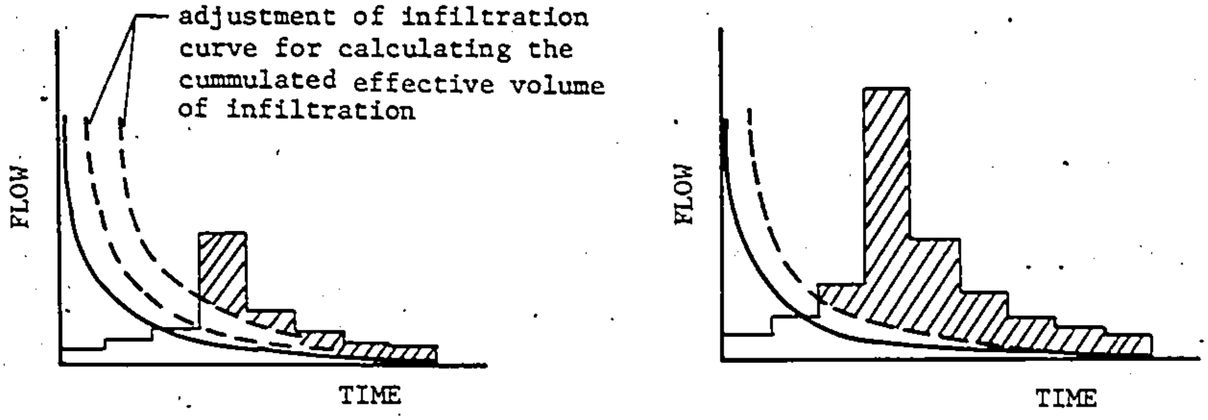
$$Q_p/A = Kt_p \tag{5.26}$$

where K is Snyder's unit hydrograph peak discharge coefficient and tp is the lag time measured from the beginning of rainfall to the centre of mass of the unit hydrograph. K is obtained through regression analysis of measured data and t_p is calculated by dividing the mean travel length with the flow velocity in the storm sewer that is estimated with Manning's equation.

where k is the percentage increase, or decrease, in the effective rainfall intensities.

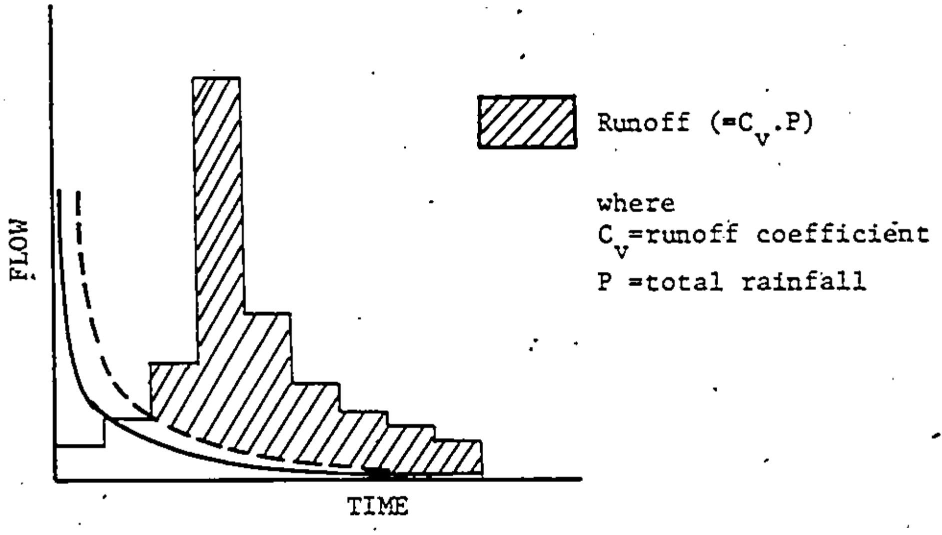
All design storm methods distribute the rainfall with identical shapes. If a total rainfall quantity is distributed by means of, for example, the Keifer and Chu's method (1957) or the SCS 24-hour Type II method (SCS 1975), the same shape of rainfall hyetograph will result. The SCS method expresses the rainfall intensity at each time step of the design storm as a percentage of the total rainfall amount; thus the rainfall intensities are always in proportion to the total rainfall quantity and the shapes of the design storms obtained are indentical with each other. The Keifer and Chu's design storm method is also similar in principle (Figure 5.1).

However, it is noticed that although the rainfall profiles are similar in shape, the effective rainfall profiles may still vary because of the infiltration that occurs in the pervious part of the watershed. For a frequent storm such as one with a 1 in 1 or 1 in 2 year return period, the effective rainfall pattern can differ from the less frequent storms (Figure 5.3). Hence, the shape of the runoff hydrograph can change accordingly. However, as the return period of the storm decreases (i.e. for larger storm events) the



(a) low return period

(b) medium return period



(c) high return period

Figure 5.3: Effective Design Storm Profiles for Different Return Periods

soil is saturated near the beginning of the storm due to higher rainfall intensities and volumes, with the result that the pervious surfaces will act mainly as impervious surfaces and produce immediate runoff. In this range of frequencies, the pattern of effective rainfall will be similar with the result that similar shapes of runoff hydrographs are produced. It may therefore be possible to assume that for major storms distributed with similar patterns, similar shapes of runoff hydrographs are produced from the pervious surfaces and can be represented approximately with a general hydrograph similar, in principle, to the unit hydrograph.

The assumption of a generalized hydrograph for a complete storm is particularly appropriate for urban areas where runoff is mainly from the impervious areas. Since infiltration only occurs in pervious area, the shape of the complete runoff hydrograph will be mainly dictated by the runoff from the impervious area. Table 5.1 illustrates the percentages of contribution of runoff volume and peak discharge from the pervious and impervious areas for various return periods and imperviousness ratios of 30% and 70%, respectively.

It is noticed that many urban drainage designs are carried out with design storms distributed with a fixed method (Section 3.3). The use of this type of simplified method

TABLE 5.1

CONTRIBUTION OF RUNOFF FROM PERVIOUS AND IMPERVIOUS AREAS

Percentage of Contribution

Return Period of Design Storm	30% imperviousness				70% imperviousness			
	TOTAL RUNOFF		TOTAL PEAK		TOTAL RUNOFF		TOTAL PEAK	
	Pervious	Impervious	Pervious	Impervious	Pervious	Impervious	Pervious	Impervious
1:2	0	100	0	100	0	100	0	100
1:5	16	84	7	93	3	97	1	99
1:25	37	63	20	80	10	90	4	96
1:100	46	54	29	71	14	86	7	93

for determining the runoff hydrograph can therefore be extended to the present study.

For the determination of the runoff hydrograph, the dimensionless hydrograph which expresses Q/Q_p versus t/t_p is used (Section 2.5.2).

For the dimensionless hydrograph, the watershed characteristics are implicit in the parameter t_p, time to peak. tp is defined as the time from the beginning of the rainfall excess to the peak of the runoff hydrograph. By knowing Qp (peak discharge) and t_p (Table 5.1), it is possible to obtain the complete runoff hydrograph. Q_p can be calculated with Equation 5.24 (as given in the last section) and t_p is a function of the rainfall and watershed characteristics.

5.4 APPLICATION OF SHM FOR NON-HOMOGENEOUS WATERSHEDS

Further to the SHM procedures, a more refined procedure is needed for the analysis of non-homogeneous watersheds. A watershed containing various land use characteristics can first be discretized into sub-watersheds of homogeneous characteristics and analyzed with the SHM procedure. Peak flows and runoff hydrographs estimated with SHM for each sub-watershed can then be combined by means of channel routing and, if necessary, by means of routing through detention reservoirs when stormwater management facilities exist.

In general, there are two main types of routing techniques, namely hydraulic and hydrologic routing methods. Hydraulic routing is based on unsteady flow solutions and is generally carried out by means of the detail, dynamic equations of momentum and continuity that are known as the St. Venant equations. Hydrologic routing is based on the simpler, steady flow equations. It uses the storage-continuity equation where storage is expressed in terms of inflow, outflow, or both.

Examples of hydrologic routing methods for channels consist of the modified Puls Method, the Muskingum Method, the Variable Storage Coefficient (VSC) or Variable Travel Time (VTT) Method, and the Convex Method. Some of these routing methods have been incorporated in computer models presently used. The modified Puls Method, for example, has been employed by the ILLUDAS (Terstriep and Stall 1974) computer model and the Variable Storage Coefficient Method has been employed by the HYMO model (Williams and Hann 1973) for channel routing; the Convex Method has been employed by the Soil Conservation Service in the SCS TR-55 Manual for its tabular method of routing the runoff hydrographs resulting from the SCS 24-hour Type II distribution design storm (SCS 1975).

A detailed review and comparison of the various types of simple hydrologic routing methods with worked examples have been conducted by Wisner and Kassem (1980). A comparison of the Convex routing method with the EXTRAN routing model (which uses the complete dynamic wave solutions) indicates that the peak runoff rates estimated are reasonably close and within 1% to 5%, while the other methods have higher discrepancies. The time to peak of the routed hydrograph using the Convex Method is in general close (within 1% to 2%) to that of the EXTRAN model. However, these comparisons are made by means of routing flows generated from a hypothetical watershed of 0.4 square mile through a rectangular channel with a fixed slope of 0.002 and Manning's n of 0.013.

As a simplified approach, it is concluded that the Convex Method of channel routing can be used for preliminary designs. In a separate research (Wisner, Cheung and Liang 1981), a program for use with pocket calculators for the Convex Routing Method has been written and presented. It is felt that the program can be applied here as a simple procedure for the routing of the runoff hydrograph determined with the SHM.

In the same study, the Storage-Indication Method for the routing of hydrographs through reservoirs with orifice and/or overflow weir has also been presented. It can be applied here as a simple procedure for the analysis of routing through detention reservoirs in this study.

These two programs are suggested for use in conjunction with SHM for the computation of runoff from non-homogeneous watersheds (although other simplified routing methods may still be employed).

Further studies have been carried out by Liang (1982) concerning the use of these programs.

5.5 DERIVATION OF SHM PARAMETERS

The SHM provides a general methodology by which simple, standard relationships can be derived for determining the peak runoff rate and runoff hydrographs for a given basin or region. For a municipality which employs a given set of I.D.F. curve, it is possible to derive the peak factors P and dimensionless hydrographs for a specific computer model and design storm distributions for the SHM. Once these data are derived, they can be applied easily to determine the peak runoff rates and runoff hydrographs.

In this section, an example of the steps required for deriving the parameters of the SHM peak discharge equation and the SHM dimensionless hydrograph will be described. The Keifer and Chu's design storm distribution method (1957) and the SCS Type II distribution method (1973) are used; other design storm distributions can also be considered but these design storm methods are selected because they have been used by most practitioners and municipalities for urban drainage designs (Chapter III). The computer model SWMM is used for runoff simulations.

5.5.1 SHM Peak Discharge Equation

The first step involved in deriving the parameters of the SHM peak discharge equation is to obtain the I.D.F. curves for the region considered. In this work, the I.D.F. curves for the Toronto region are used (Figure 3.1). The next step is to determine the design storm hyetographs using the I.D.F. curves for the Keifer and Chu's and SCS 24-hour Type II design storm distributions (see Section 3.3).

The parameters of the SHM peak discharge equation (Equation 5.24) namely, P, Q, and S are then derived by means of using the SWMM model, the design storms, and "lumped" conceptual watersheds. "Lumped" conceptual watersheds with sizes of 10 ac., 20 ac., 40 ac., 60 ac., 80 ac., 120 ac., and 160 ac., respectively as described in Section 3.4.2 are used; they all have similar physical characteristics of an average catchment 'width' of 350 ft. per acre and an equivalent pipe 'length' of 20 ft. per acre (see Section 3.5). Different land use characteristics, namely the 25% and 35% imperviousness ratios which are found mostly in residential developments and the 50% and 70% imperviousness ratios which are found mostly in commercial developments, respectively, are considered. An average ground slope of 1.0 percent is assumed.

5.5.1.1 Derivation of Q and R

The depths of runoff in inches for each design storm return period and imperviousness ratios are obtained from the SWMM simulations. The total runoff depths simulated are plotted against the recurrence intervals in years as shown in Figure 5.4. By dividing the depth of effective runoff (Q) with the total depth of rainfall (PRECIP), the runoff factors (R) calculated for the different design storm return periods and imperviousness ratios are shown in Figure 5.5.

Plots of the rainfall and runoff depths that are simulated by SWMM using the SCS 24-hour Type II design storm distribution are also given in Figure 5.6; plots of the runoff factors (R) versus the return periods for the 25% and 35% imperviousness ratios are shown in Figure 5.5.

5.5.1.2 Derivation of P

By means of the peak runoff rates Q_p simulated by SWMM and the known watershed area A and runoff depths Q, the peak factors P can next be derived (P is equal to Q_p/AQ).

Figures 5.8 and 5.9 show plots of the derived peak factors (in cfs per acre per inch of runoff) against the area of the watershed for different design storm return periods. Figure 5.8 is for an imperviousness ratio of 25% and Figure 5.9 is for an imperviousness ratio of 35%. Peak factors determined for the imperviousness ratios of 50% and 70% are also shown in Figures 5.10 and 5.11, respectively.

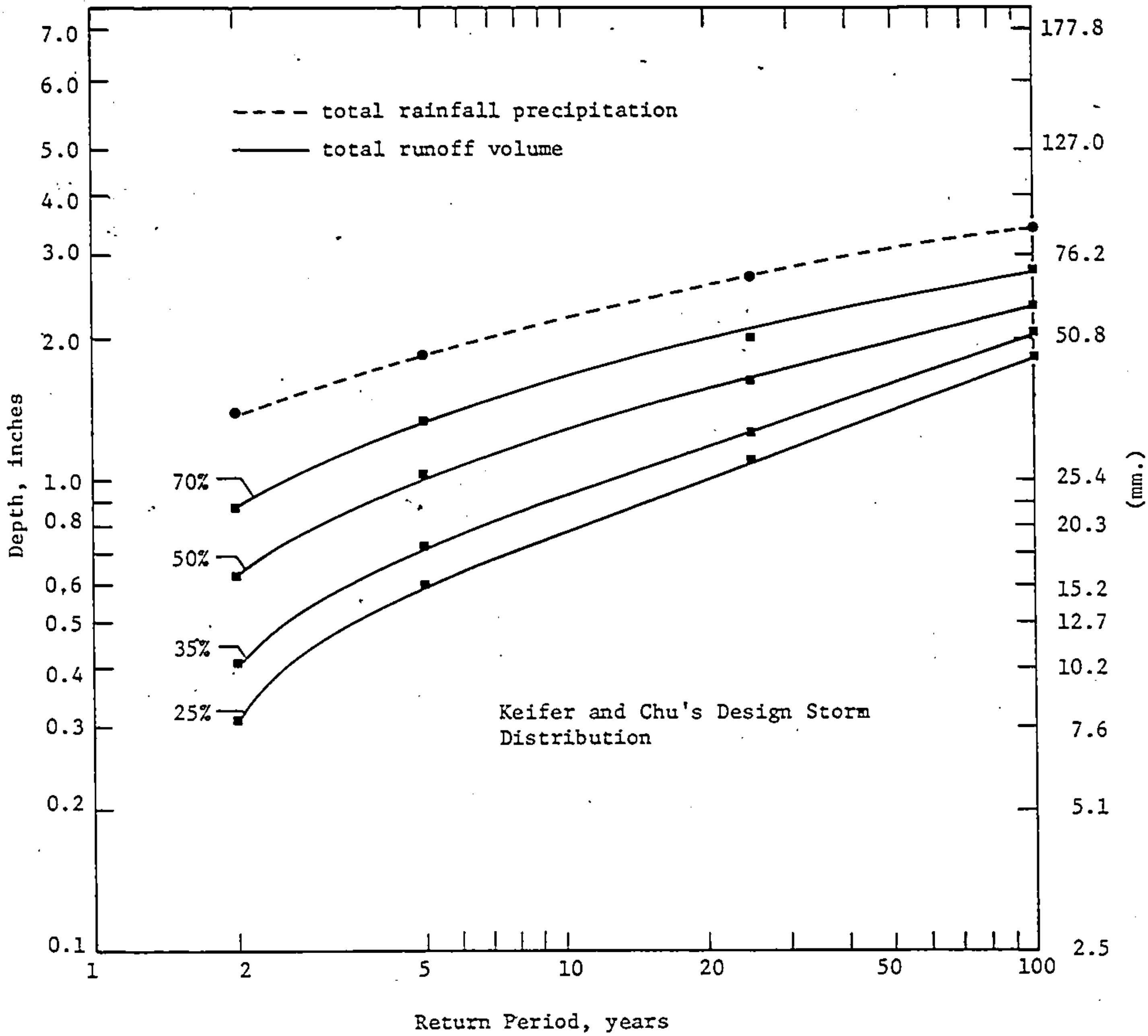


Figure 5.4: Total Rainfall Precipitation and Runoff Volume versus Return Periods for Different Percentages of Imperviousness

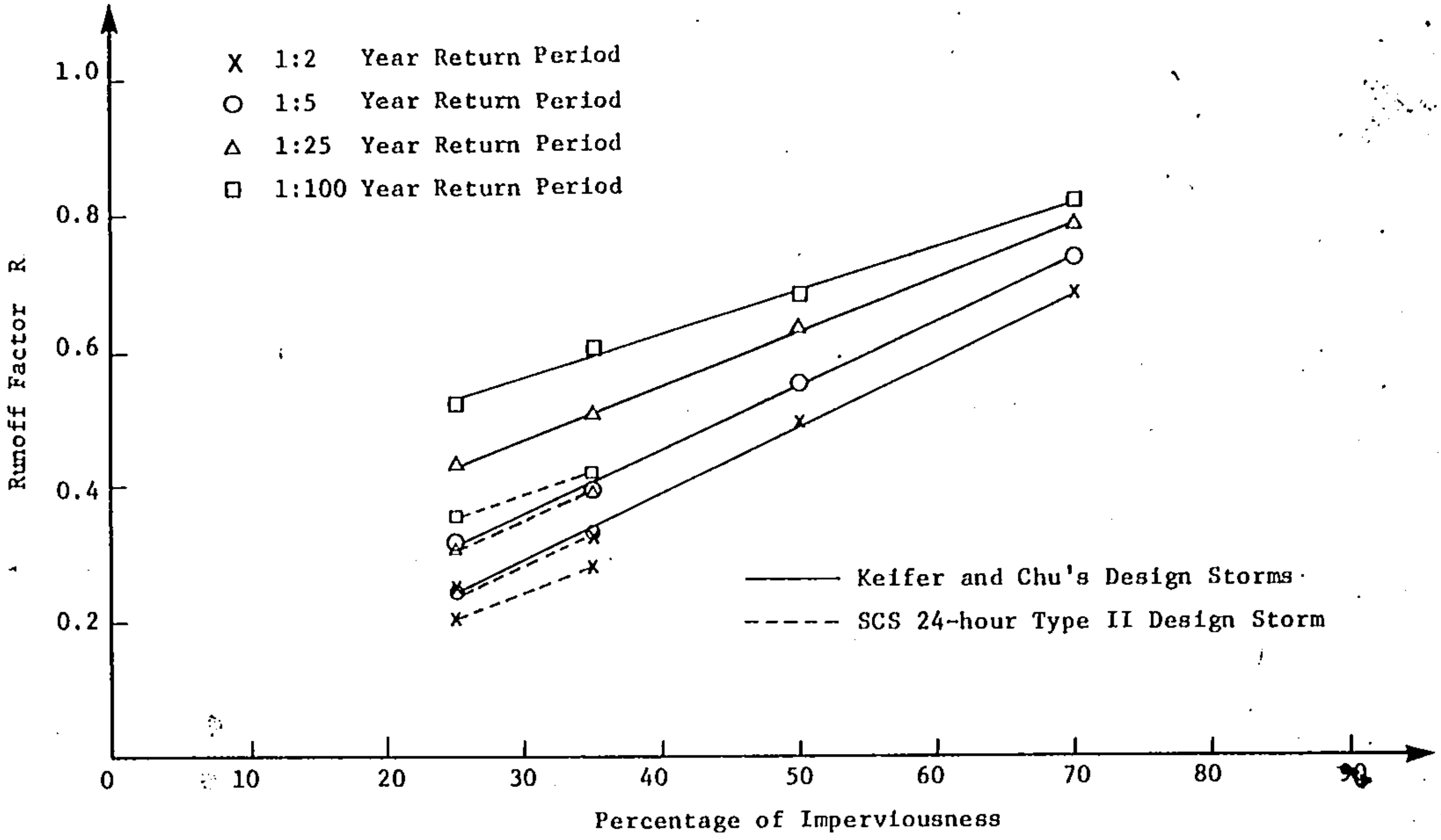


Figure 5.5: Runoff Factor R Simulated by SWMM

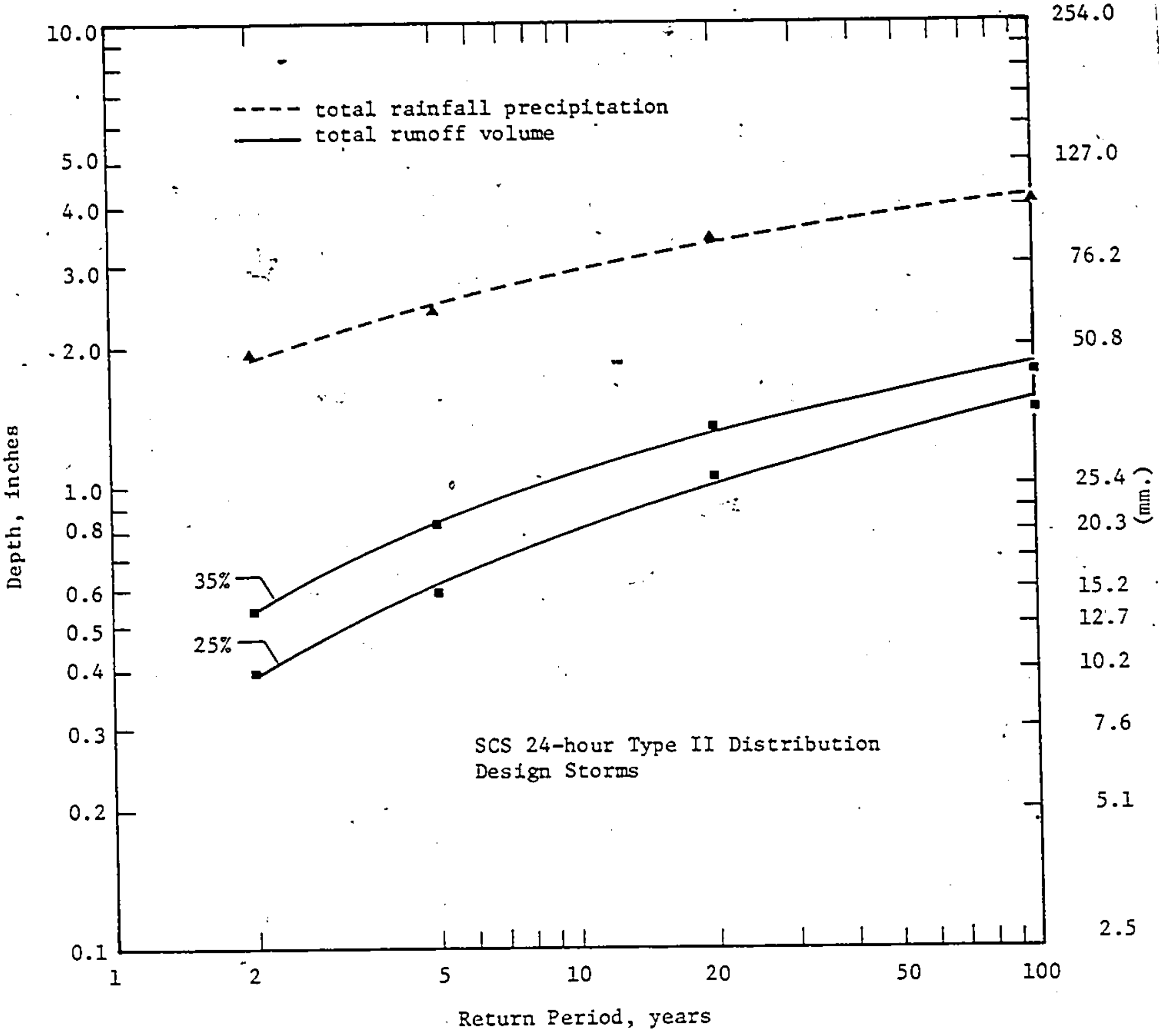


Figure 5.6: Total Rainfall Precipitation and Runoff Volume versus Return Periods for Different Percentages of Imperviousness

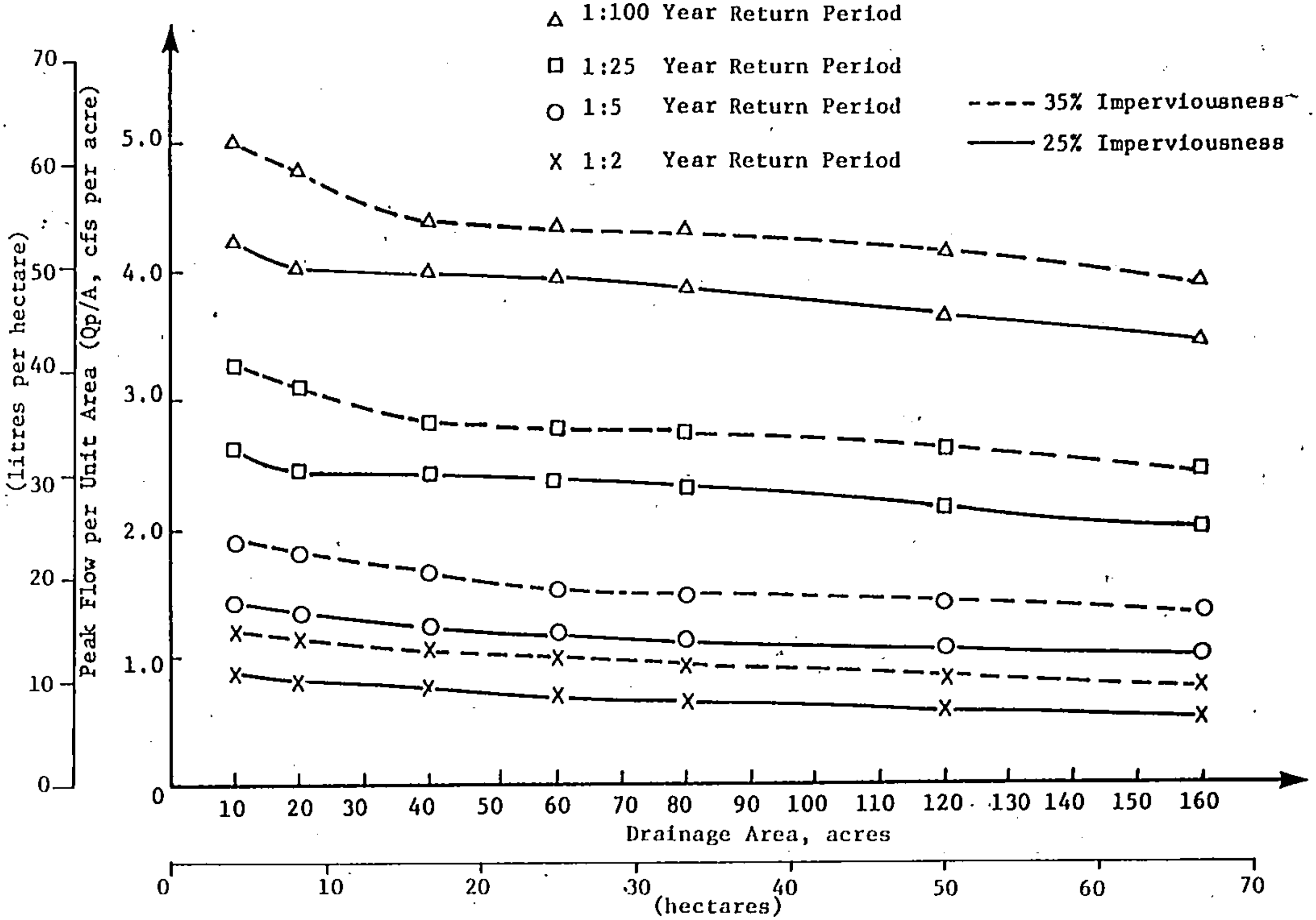


Figure 5.7: Peak Flow per Unit Area versus Drainage Area Computed by SWMM for Different Return Periods and Percentages of Imperviousness

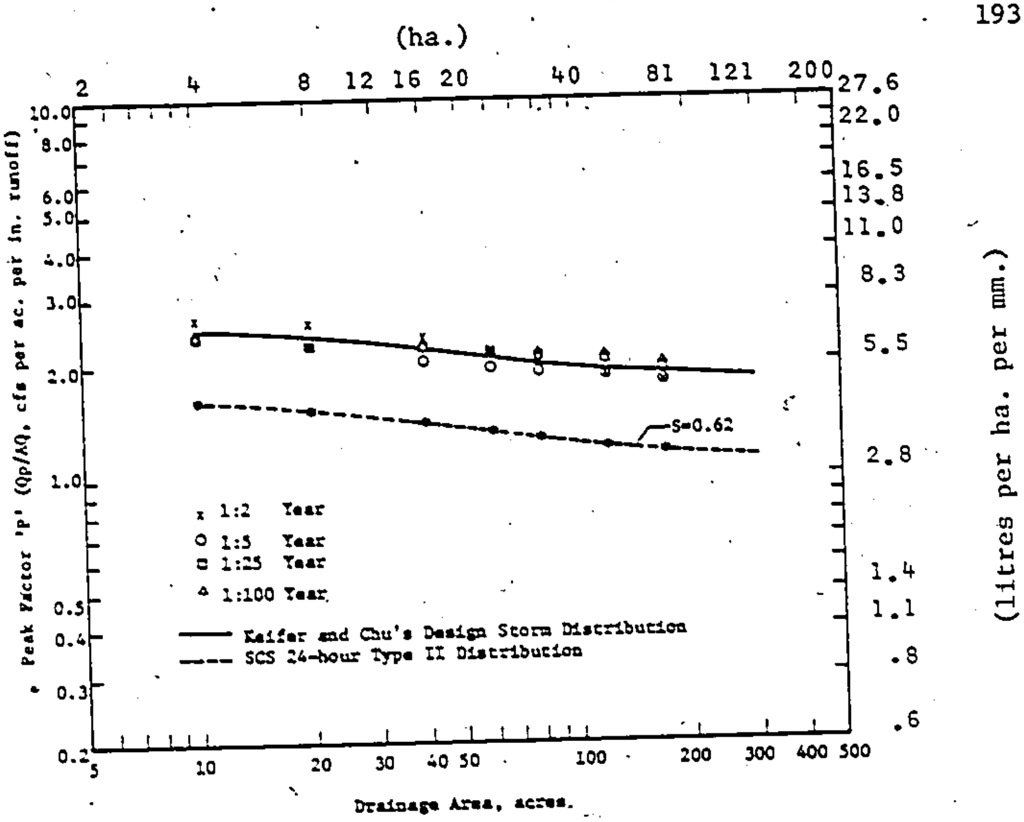


Figure 5.8: Peak Factor P Against Drainage Area for the Percentage Imperviousness of 25%

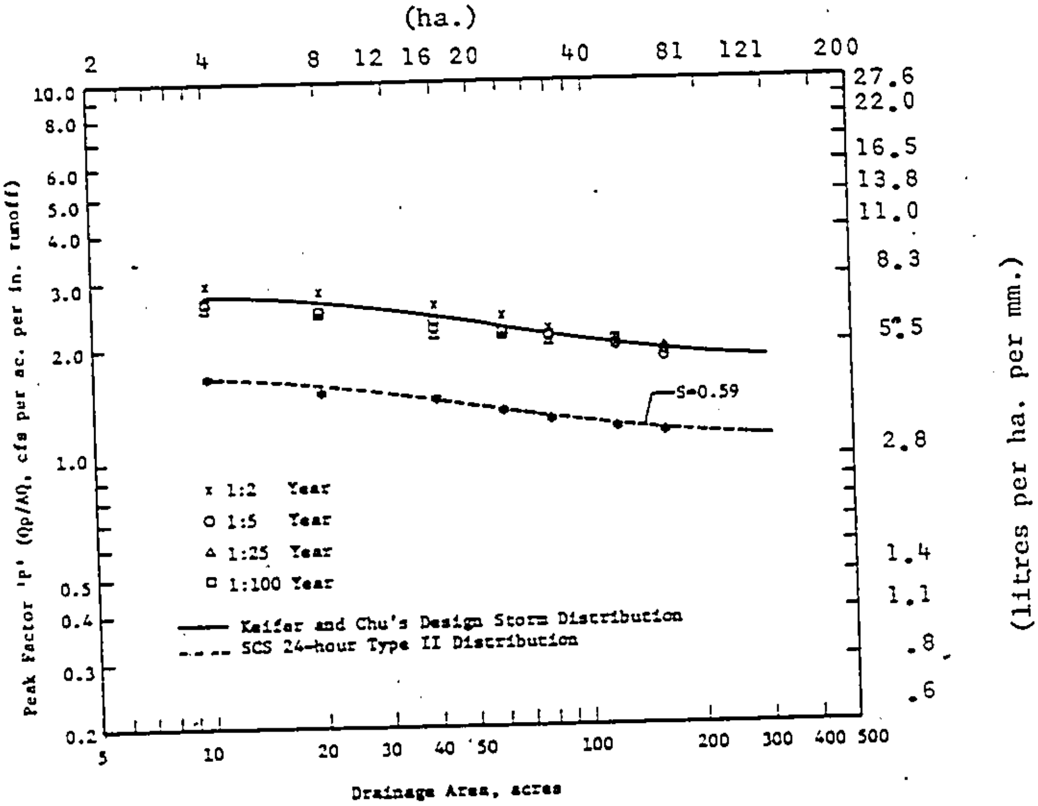


Figure 5.9: Peak Factor P Against Drainage Area for the Percentage Imperviousness of 35%

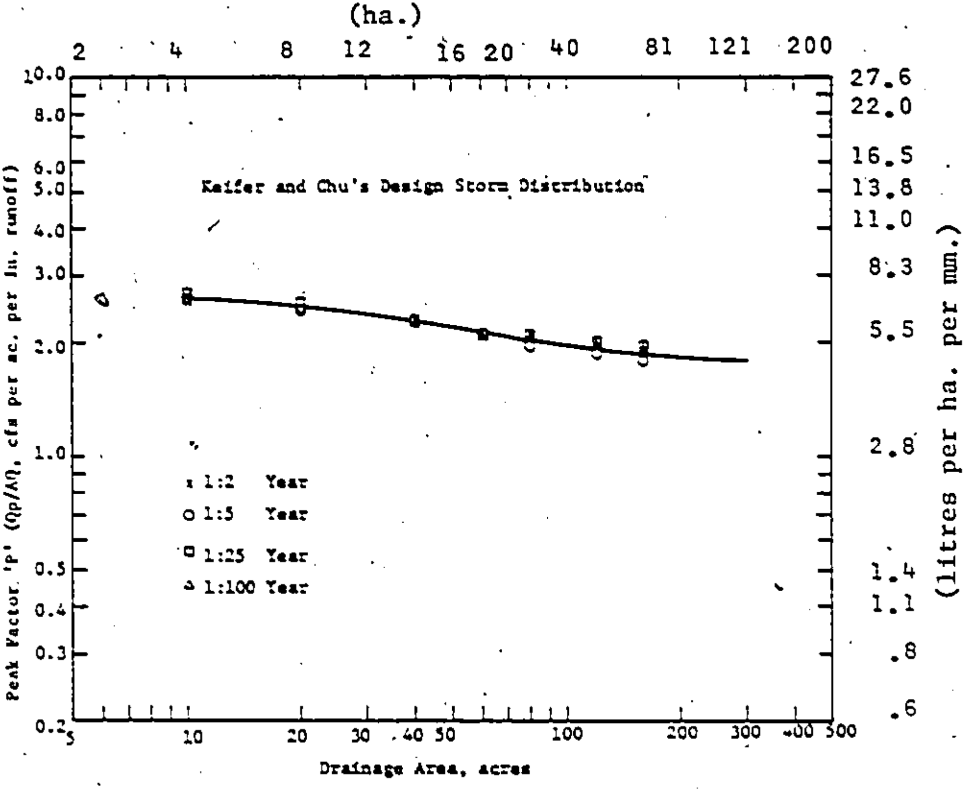


Figure 5.10: Peak Factor P Against Drainage Area for the Percentage Imperviousness of 50%

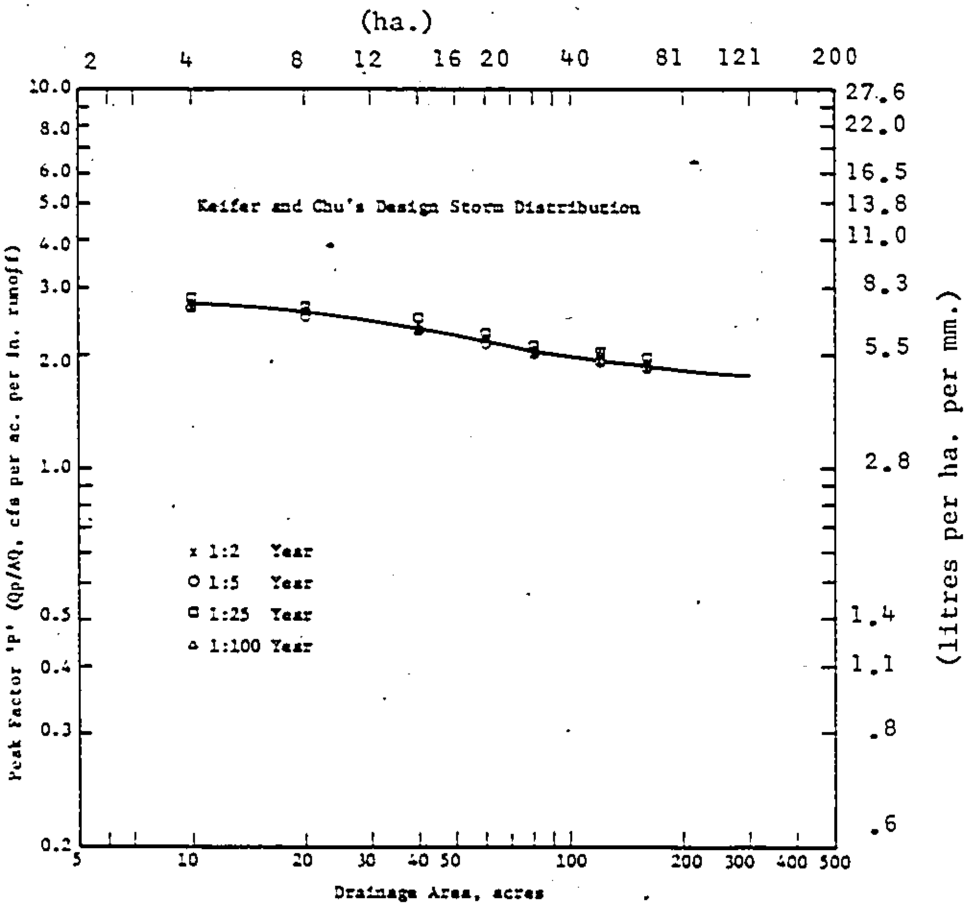


Figure 5.11: Peak Factor P Against Drainage Area for the Percentage Imperviousness of 70%

They show that a general relationship exists between Q_p/AQ (where Q = R.PRECIP) and the area of the watershed. A single curve is then drawn to represent the mean peak factors versus different areas of watersheds. It is also observed that curves derived for different imperviousness ratios are quite close and within 20%.

The above relationship has similar characteristics as the distribution graph employed by Bernard (1935) and Wisler and Brater (1959) for predicting runoff. Wisler and Brater (1959) have demonstrated that for a given runoff hydrograph the ordinate of the distribution graph during each time interval can be obtained by dividing the average runoff rate with the area of the watershed and the depth of runoff. They have found that by means of real watersheds similar distribution graphs can be obtained when the watershed characteristics are similar. A similar approach has also been undertaken by the SCS TR-55 method for predicting peak runoff rates (Section 2.3.4).

The peak factors derived for the SCS 24-hour Type II design storms are also plotted in Figures 5.8 and 5.9.

5.5.1.3 Derivation of S

It can be observed in Figures 5.8 and 5.9 that the peak factors derived for the SCS 24-hour Type-II distribution design storms are consistently lower than that derived for the

Keifer and Chu's design storm distributions. A constant ratio is seen between the two types of design storms. The S factors (Equation 5.22, Section 5.2.2) of the SCS design storms for the imperviousness ratios of 25% and 35% are 0.62 and 0.59, respectively. The mean rainfall peakiness factor for the SCS Type II design storms is therefore approximately 0.6 (relative to the Keifer and Chu's design storms).

5.5.2 SHM Dimensionless Hydrograph

Using the hydrographs simulated for the lumped watersheds with sizes of 20 acres, 80 acres and 120 acres, analyses are carried out in order to derive the SHM dimensionless hydrographs. The runoff hydrographs generated for each lumped watershed for design storms with different recurrence intervals of 1 in 2 years, 5 years, 25 years and 100 years are analyzed. The SCS 24-hour Type II design storm distribution is also employed for analyses. Since only one value of ground slope (1%) is adopted, the sensitivity of the dimensionless hydrograph to slope is not studied but it has been shown by Ragan et al (1975) to affect only the shape of the recession curve. However, two imperviousness ratios, namely 35% and 70%, are investigated.

The ordinates, $Q(t)/Q_p$, of the SHM dimensionless hydrograph are determined by dividing the average runoff rate occurred in subsequent time intervals with the peak runoff rate of the runoff hydrograph; the corresponding time at which that runoff rate occurs is also divided by the time to peak of the runoff hydrograph in order to determine the ordinates for t/t_p.

The dimensionless hydrographs estimated for the imperviousness ratios of 25% and 35% are plotted as shown in Figures 5.12 and 5.13, respectively. In each case, twelve dimensionless hydrographs are plotted. Considerable similarities are observed between them even when the size of the watershed and the design storms are very different. A single, common dimensionless hydrograph can therefore be drawn.

Similar results are obtained for the SCS 24-hour Type II design storms (Figure 5.14).

It is also observed from the runoff hydrographs that the time to peak is insensitive to watershed sizes and storm return periods. Even when the size of the watershed is varied from 20 acres to 120 acres, the time to peak of the runoff hydrographs has only varied between 70 minutes and 75 minutes (Table 5.2). This may be explained by the high intensities of the design storms, which are the characteristics of Keifer and Chu's design storms, and the fast response of runoff to rainfall in urban watersheds (Sections 3.3 and 5.2.4). The presence of impermeable surfaces in urban watersheds has caused the 'peaky' rainfall to be re-

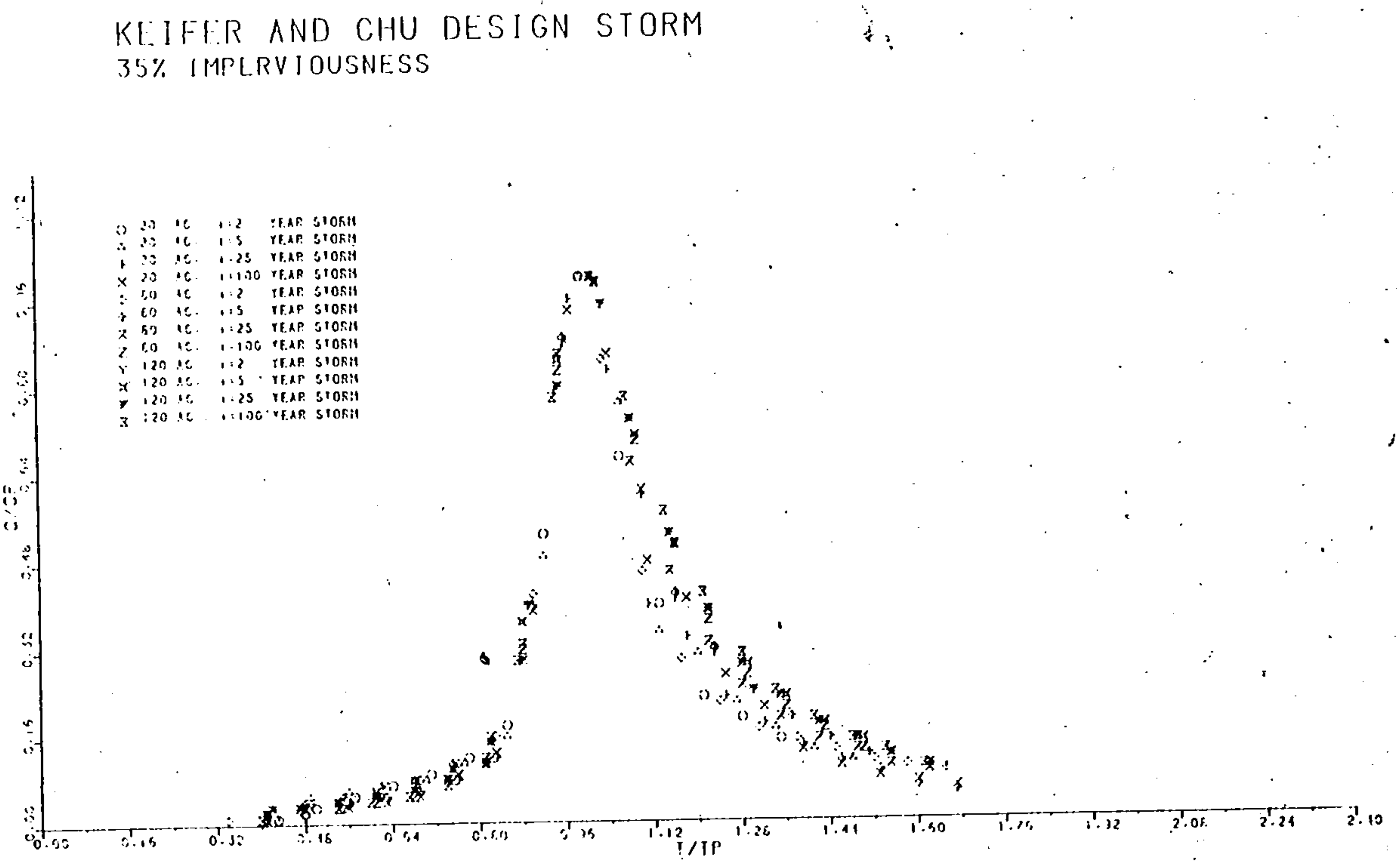


Figure 5.12: The SHM Dimensionless Hydrograph Derived for the Keifer and Chu's Design Storm Distribution and the Imperviousness Ratio of 35%

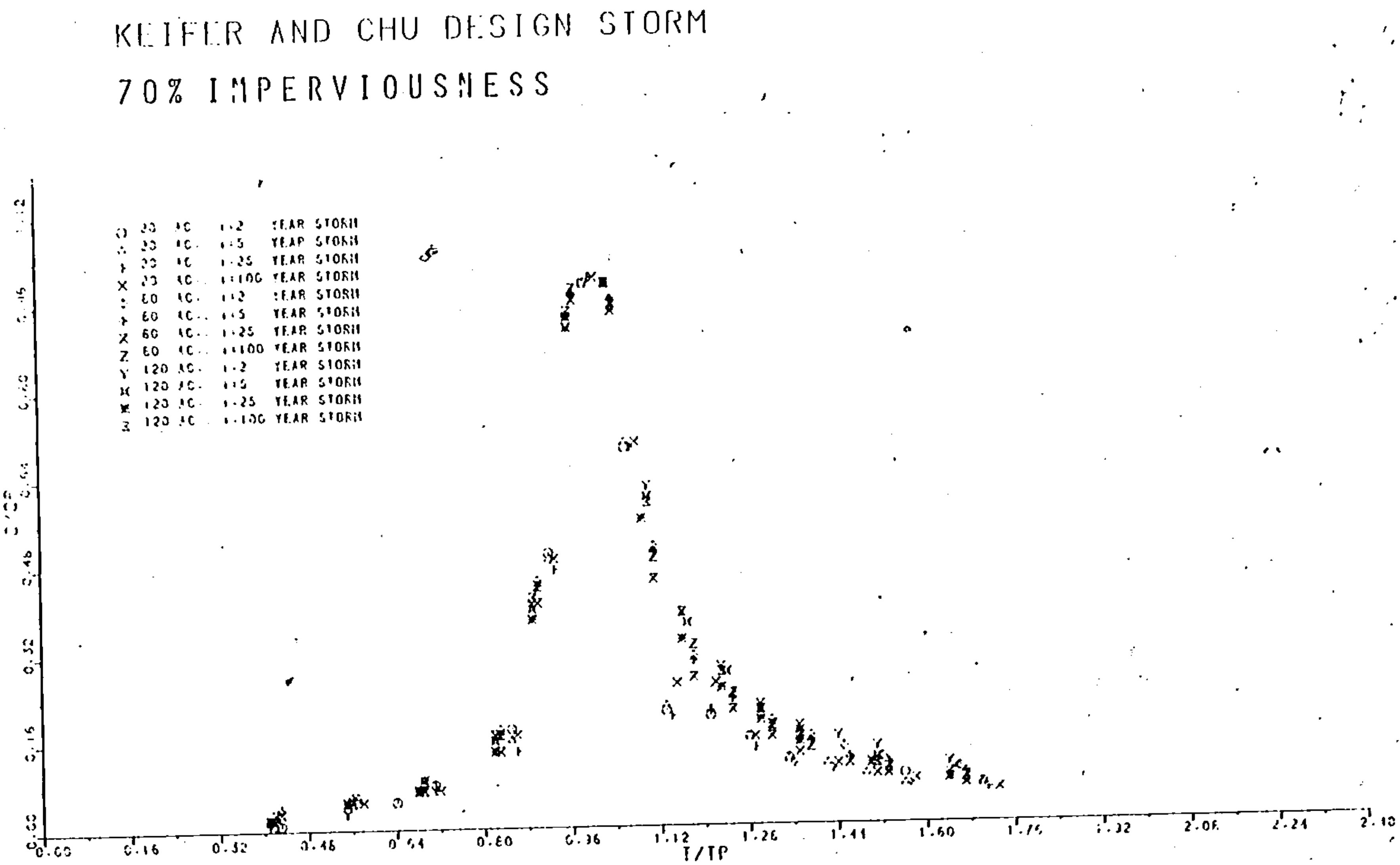


Figure 5.13: The SHM Dimensionless Hydrograph Derived for the Keifer and Chu's Design Storm Distribution and the Imperviousness Ratio of 70%

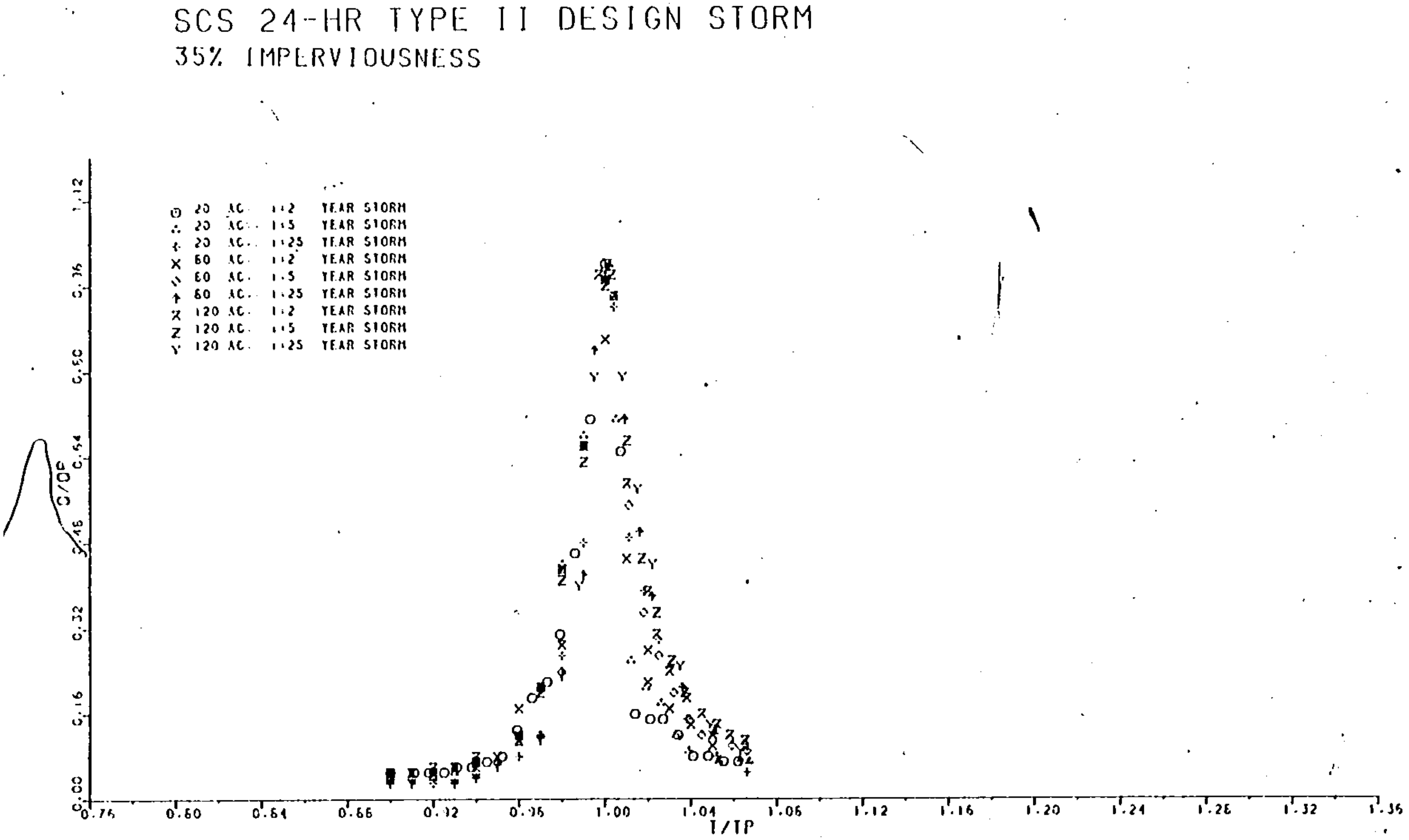


Figure 5.14: The SHM Dimensionless Hydrograph Derived for the SCS 24-hour Type II Design Storm Distribution and the Imperviousness Ratio of 35%

Table 5.2: Time to Peak Simulated by SWMM

Return Period of Design Storm	Time of Peak (hrs.)								
	25% Imperviousness			35% Imperviousness			70% Imperviousness		
	20 ac. 8.1 ha	80 ac. 32.4 ha	120 ac. 48.6 ha	20 ac. 8.1 ha	80 ac. 32.4 ha	120 ac. 48.6 ha	20 ac. 8.1 ha	80 ac. 32.4 ha	120 ac. 48.6 ha
1:2	1.167	1.203	1.217	1.175	1.21	1.217	1.175	1.200	1.217
1:5	1.182	1.217	1.225	1.183	1.217	1.225	1.175	1.2	1.213
1:25	1.2	1.25	1.267	1.2	1.233	1.233	1.167	1.2	1.218
1:100	1.217	1.242	1.267	1.2	1.233	1.242	1.158	1.2	1.217

Note: Peak Intensity is at 65 min. (1.083 hr.)
Total Rainfall Duration is 180 minutes

flected and conveyed very quickly in a short period of time. The peak intensity of the design storms used in this study occurs at approximately 65 minutes (total storm duration is 180 minutes). Thus the time of lag relative to the time of peak intensity is only between 5 to 10 minutes although they still have to be checked for larger watersheds.

5.6 ADVANTAGES AND LIMITATIONS OF SHM

5.6.1 Advantages

The previous section illustrates the steps by which the parameters of the SHM peak discharge equation and the dimensionless hydrographs can be derived using a given shape of design storm, a given model, and simple watershed configurations. Once these relations are obtained, they can be applied conveniently without lengthy computations or the use of complex computer modelling for preliminary studies, drainage designs in small urban watersheds, and verification of projects within the region of a municipality.

The SHM has several advantages as outlined below:

1. For determining the peak discharge, SHM uses the simple formula

$$Q_p = A.Q.P.S.$$

which is as easy to apply as the Rational Method but it is less subjective in the determination of parameters; for example, for a given watershed area and depth of runoff, there is only one value of peak factor to define the peak runoff rate. Consequently, the results obtained with the SHM will be consistent even when it is applied by different users. SHM is derived based on simulations with a computer model, in this case, SWMM. It therefore produces compatible results between SWMM and a simple, desk-top method. Thus compatible results between the preliminary and final design stages can be obtained.

2. The SHM dimensionless hydrograph developed is easy to use since it does not require a convolution procedure. The lengthy convolution procedure required for calculating the runoff hydrograph for a complete storm pattern is eliminated. The peak discharge of the dimensionless hydrograph can be first obtained by the SHM peak discharge equation.

   The SHM dimensionless hydrograph can be used further to derive hydrographs for the sizing of detention storage and the analyses of stormwater management facilities by means of channel and reservoir routing.

A preliminary version of the SHM method has already been used in the Storm Water Management Policies Guidelines by the Central Region of the Ontario Ministry of Natural Resources in Toronto (1981).

5.6.2 Limitations

Like other methods, the SHM procedure also has some limitations. Although the parameters of the SHM peak discharge equation and dimensionless hydrographs can be derived for other sets of I.D.F. curves, design storm distributions, or computer model, once they have been derived they are only applicable for those specific data. For other types of design storms or computer models, similar procedures as shown in the previous section have to be followed in order to derive another set of relations for the SHM peak discharge equation and dimensionless hydrographs. For the application of the SHM in non-homogeneous watersheds where separate land use characteristics exist, routing is needed to combine the runoff hydrographs generated from each subarea.

Chapter VI

RUNOFF CONTROL POLICIES AND DESK-TOP METHODS OF SIZING DETENTION STORAGE

6.1 RUNOFF CONTROL POLICIES

One of the most stringent runoff control policies imposed by some regulatory agencies is to require that there shall be a zero increase in stormwater runoff due to urban development (Chapter 1). It implies that the peak rates of runoff from an urbanized watershed have to be restricted to their pre-development levels for rainfalls of all return frequencies.

The concept of a zero increase in runoff is not as well defined as it would appear. Its technical implications, for instance, have been questioned by Marsalek (1978) who has observed from measurements that peak flow rates and magnitude of flow volumes are not highly correlated. Several factors can be cited for the deficiencies of the "zero runoff increase" concept:

1. When different methods (or models) are used for determining the pre- and post-development peak runoff rates, different estimates resulted. A single standard method or model for runoff computations must be

employed for both the pre- and post- development conditions.

2. The return period of pre-development flows does not necessarily correspond with that of the rainfall event due to different soil moisture conditions. Runoff from watersheds before development are very sensitive to the soil moisture condition because of the presence of permeable surfaces. When a high soil moisture level is employed, higher runoff will result. Proper antecedent soil moisture conditions must be specified to ensure that reasonable estimates of the pre-development peak flows are obtained.
3. The value of the pre-development peak flow, at which the post-development flow has to be restricted is unknown. There is no standard value to verify that the pre-development peak flow determined by a specific method and an antecedent moisture condition is adequate unless a flow-frequency curve is available to confirm the estimated values.
4. The selection of the type of storm input is important in order to produce correct pre-development flow estimations (see section 3.3). A storm event that is critical for the post-development condition may produce little runoff for the pre-development condition. Hence, the proper type of storm input must be select-

ed to ensure that the flows are adequately estimated.

5. It is felt that the policies of runoff control should be made on the basis of cost-effectiveness and not on a stringent and inflexible criterion.

6.1.1 Determination of Pre-Development Flows

For the determination of pre-development flows, a regulatory agency may prefer to conduct in-depth studies on an areawide basis and recommend runoff rates for particular development projects within that area. Long term records can be employed to derive a peak flow-frequency curve. However, the procedure is lengthy and costly, and the required data may not be always available at the development site. Therefore traditionally, pre-development flows are simulated with design storms and the return periods of flows generated are simply assumed to be equal to that of the design storms. A proper methodology is therefore required to estimate the pre-development flow such that it corresponds to the 'true' flow return period if design storms are used.

The return period of flow can be characterized by the return period of the total rainfall, average intensities and durations, areal and time distribution, and antecedent moisture condition (Laurenson, 1979).

When design storms are used in predicting flows, they are usually distributed with a standard pattern and a fixed duration (see Section 3.3), and assumed to have a uniform areal distribution. Hence, if the return period of the design rainfall is known, and if the return period of the soil moisture condition is defined, the design peak flow (Q_p) can be expressed mainly as a function of the total precipitation (P) and the antecedent moisture condition (AMC):

$$Q_p = g(P, AMC) \qquad 6.1$$

where g is a bounded function, P and AMC are bounded real random variables for the extreme rainfall precipitation and extreme soil moisture condition, respectively. This relationship has been studied by various researchers including Beran and Sutcliffe (1972), Laurenson (1979), Packman and Kidd (1980), and Hughes (1981).

The return frequency of design flows on the basis of design storms and soil moisture for the summer season has also been investigated in a recent study conducted jointly by the University of Ottawa and the Institute of Rural Engineering of the Federal Polytechnique School of Lausanne, EPFL (Sautier, 1981). The study has first involved the calibration of the computer model HYMO (Williams and Hann, 1973) using rainfall and runoff measurements collected for a 38.4 sq.km. watershed which is located near Geneva, Switzerland. The calibrated model is then used to predict design flows for the analysis of runoff control alternatives. A preliminary investigation has been carried out in order to obtain flows on the basis of the design rainfall and an appropriate soil moisture condition (Appendix C). It has been concluded that values of CN must be linked to a specific Antecedent Precipitation Index (API) in order to generate more realistic design flows for a rural watershed. A relationship which relates the CN with the API has thus been de-veloped.

6.1.2 Selective Runoff Control

It is concluded that the simulation of design flows from watersheds before development (i.e. rural watersheds) is very difficult if not impossible. The antecedent moisture condition, which is a random variable that is subjected to a wide range of variation, has to be defined. For the proper determination of the pre-development runoff one must consider the combined return frequency of both the design storm and the antecedent moisture level. If a municipality wants to implement the "zero runoff increase" criterion, guidelines must be provided for the proper selection of the antecedent moisture level.

The "zero runoff increase" concept is therefore concluded to be vaguely defined and impractical to apply. It is felt that the concept can be replaced by a runoff policy with which the degree of runoff control criterion is "selective". With a "selective runoff control" criterion, runoff control measures can be implemented by reducing the post-development flow to a more or less arbitrary degree, and to the pre-development level if it is strictly necessary (Wisner et al 1980). This latter criterion is also advantageous from a viewpoint of cost-effectiveness.

6.2 DESK-TOP METHOD FOR SIZING DETENTION STORAGE

Using the concept of "selective control", the peak of the controlled runoff hydrograph can be expressed as a percentage of reduction of the peak of the post-development runoff hydrograph. For a municipality who specifies the required outflow from a development based on downstream constraints, the percentage of peak reduction will simply be the peak of the controlled hydrograph divided by the peak of the uncontrolled hydrograph from the development. If necessary, the peak of the controlled hydrograph can be reduced to the pre-development flow level. Various percentages of peak reductions can therefore be resulted; in the present study, three percentages namely 10%, 25%, and 45% are selected to demonstrate the procedures for developing the simplified method for sizing detention storage volumes. In each case, the volume of detention storage, which is given by the area between the inflow and outflow hydrographs, required to meet such reduction criterion is calculated.

The inflow hydrograph is obtained by means of the SHM-dimensionless hydrograph (see Figures 5.12-5.13) based on Keifer and Chu's design storm distribution method. Two imperviousness ratios of 25% and 35% and various design storms return frequencies up to the 1 in 100 year period are considered.

While design storms of different distributions can be employed (see Section 3.3), the sensitivity of the detention volume to various types of design storms will not be investigated. The main goal of this part of the study is to demonstrate a simple methodology for the preliminary estimation of the detention volume only.

As reviewed earlier in Chapter 2, various simple shapes of outflow hydrographs for determining the detention storage can be used. For the present study, a curvilinear-shaped outflow hydrograph (Figure 6.6a) as used by Wycoff and Singh (1976, Figure 2.11, Sections 2.4.4 and 2.5.3) is assumed.

The detention volumes computed for the various peak reduction percentages are plotted against area as shown from Figures 6.1 to 6.4. It is possible to express the required detention in terms of the depth of water, that is, in terms of the storage per unit area of the watershed. By plotting

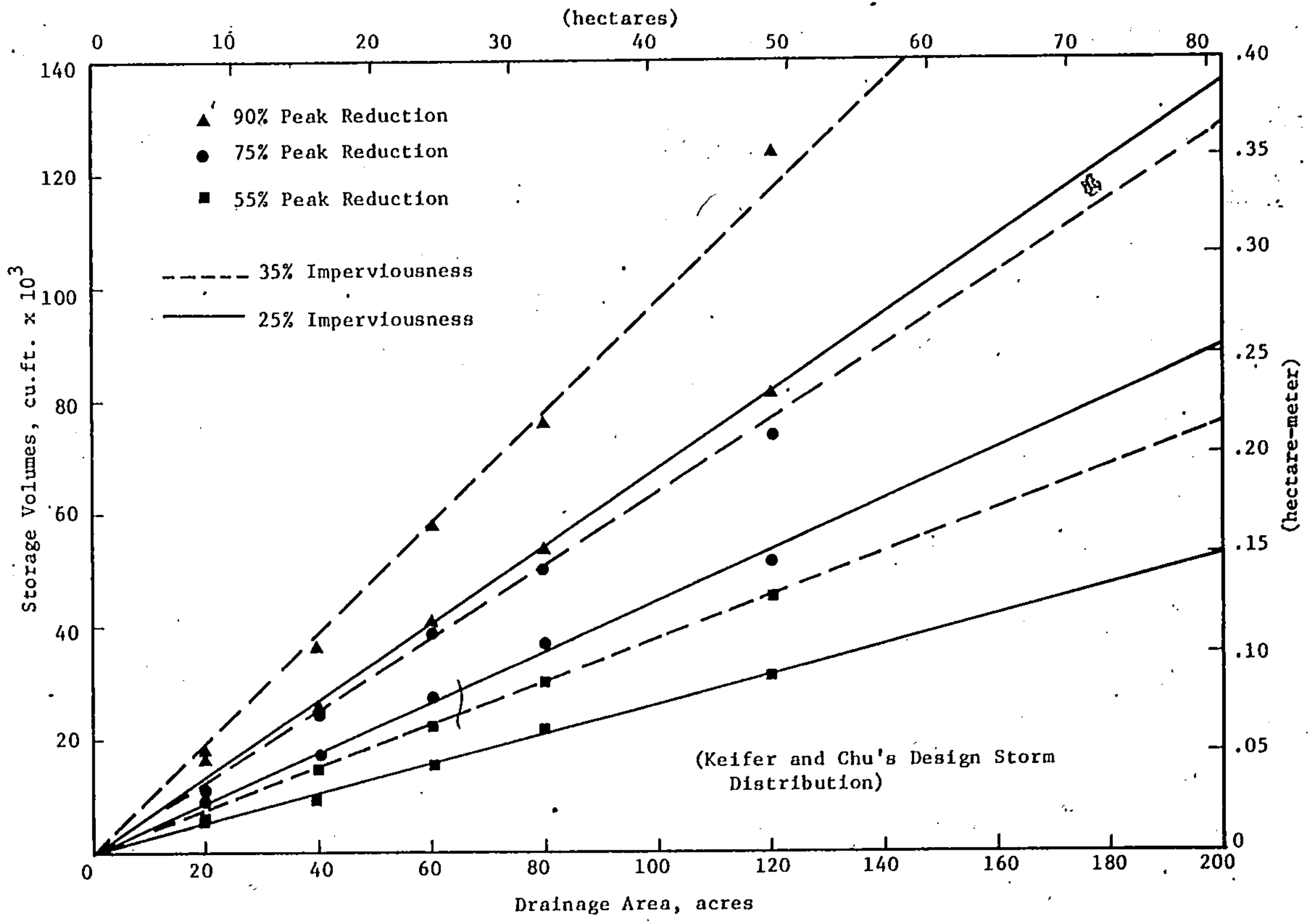


Figure 6.1: Storage Volumes Determined for Different Peak Reduction Rates for the 1:2 Year Design Storm

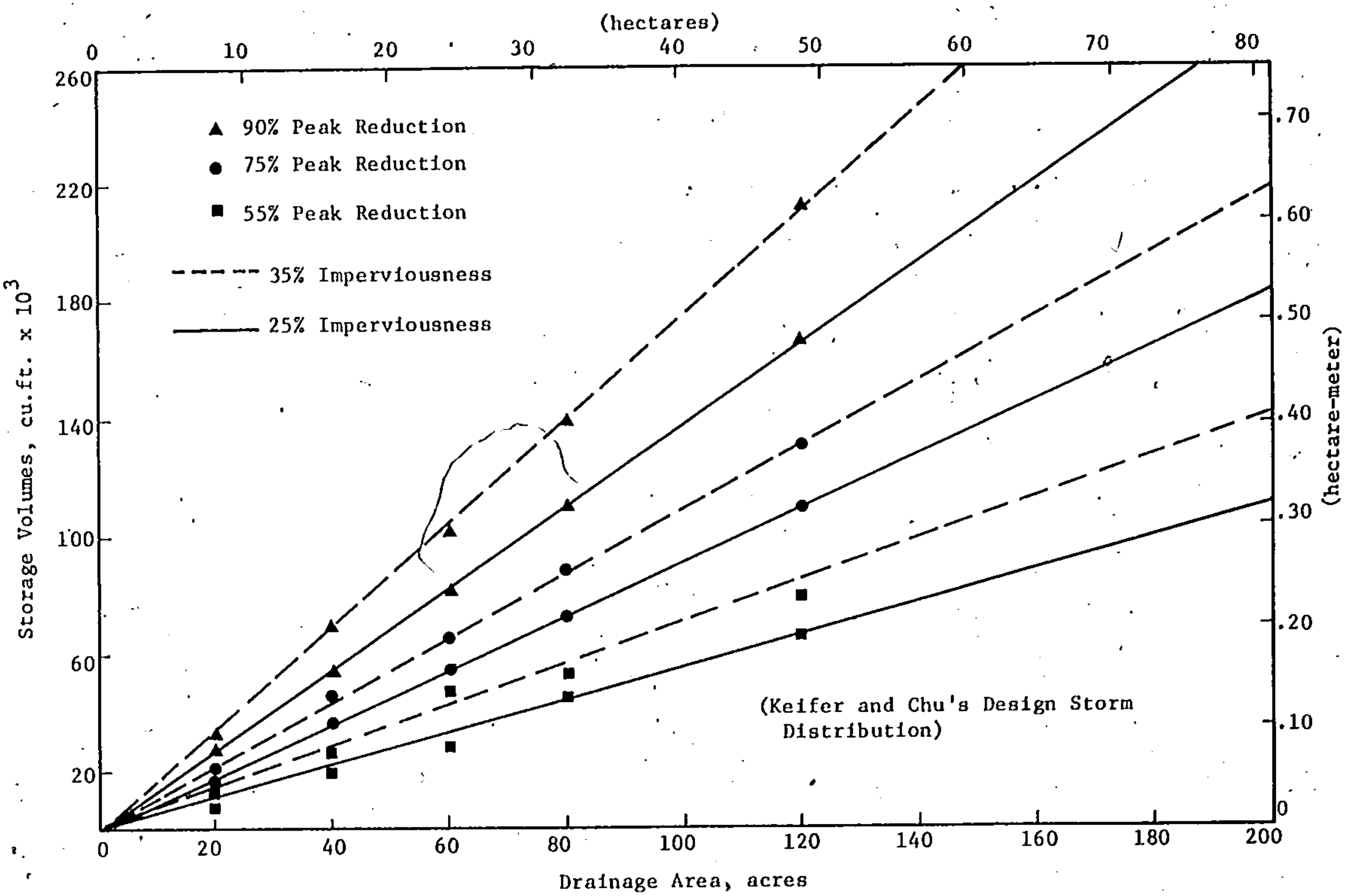


Figure 6.2: Storage Volumes Determined for Different Peak Reduction Rates for the 1:5 Year Design Storm

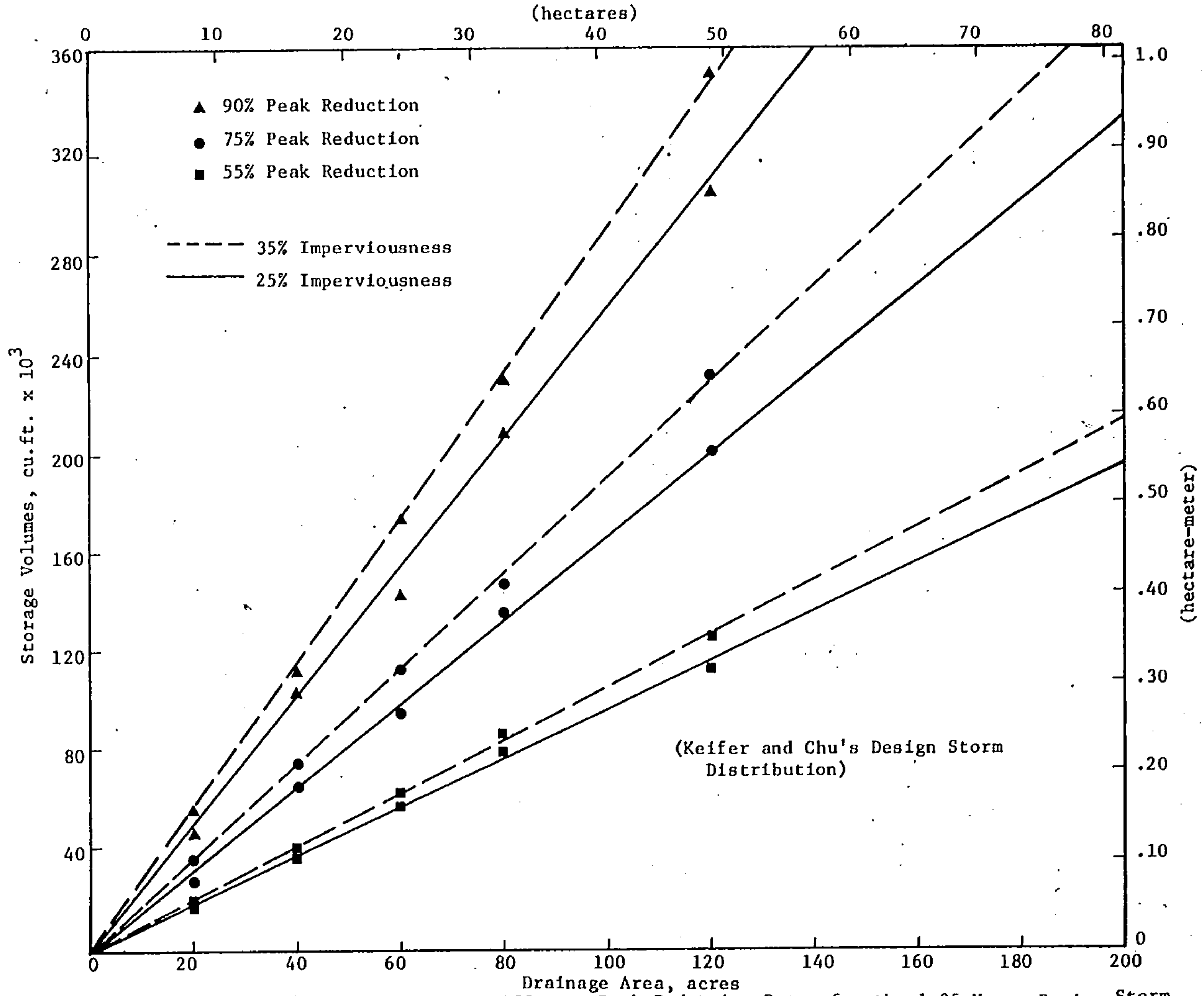


Figure 6.3: Storage Volumes Determined for Different Peak Reduction Rates for the 1:25 Years Design Storm

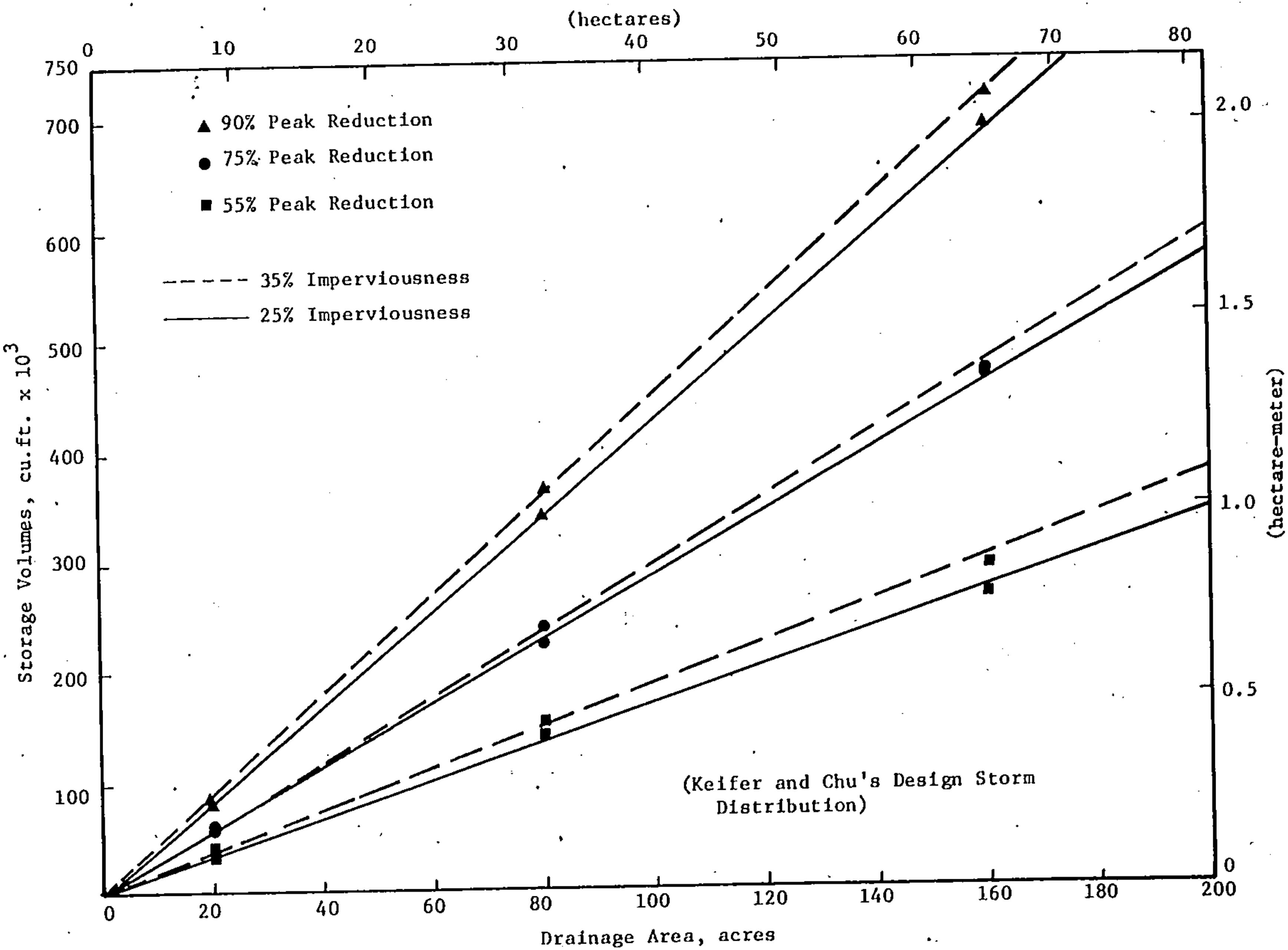


Figure 6.4: Storage Volumes Determined for Different Peak Reduction Rates for the 1:100 Year Design Storm

the depth of detention against the total depth of runoff, generalized relationships as shown in Figure 6.5 can be obtained. With this Figure, the depth of detention required for any given depth of runoff and corresponding percentage of peak reduction can be easily determined.

6.3. DESK-TOP METHOD FOR SIZING DUAL STORAGE

The "dual storage" concept, as described earlier in Chapter 2 (Section 2.4), is one of the methods recently applied for controlling runoff from new developments. It is a combination of storages in underground oversized storm sewers and park depressions provided in the development. Detention for frequent storms with the 1 in 2 or 1 in 5 years return period is accomplished by using an underground oversized storm sewer, (it operates during all storm events) and detention for less frequent storms with the 1 in 25 or 1 in 100 is accomplished by using both the oversized storm sewer and the park depression storage. Separation of flows between the two systems during major storm events is achieved by restricting the number of storm inlets (catchbasins), or the inlet capacities of storm inlets which allow inflow into the pipe system. The latter, called "inlet control", can be accomplished by installing restricting devices to limit inflow (Townsend et al, 1980).

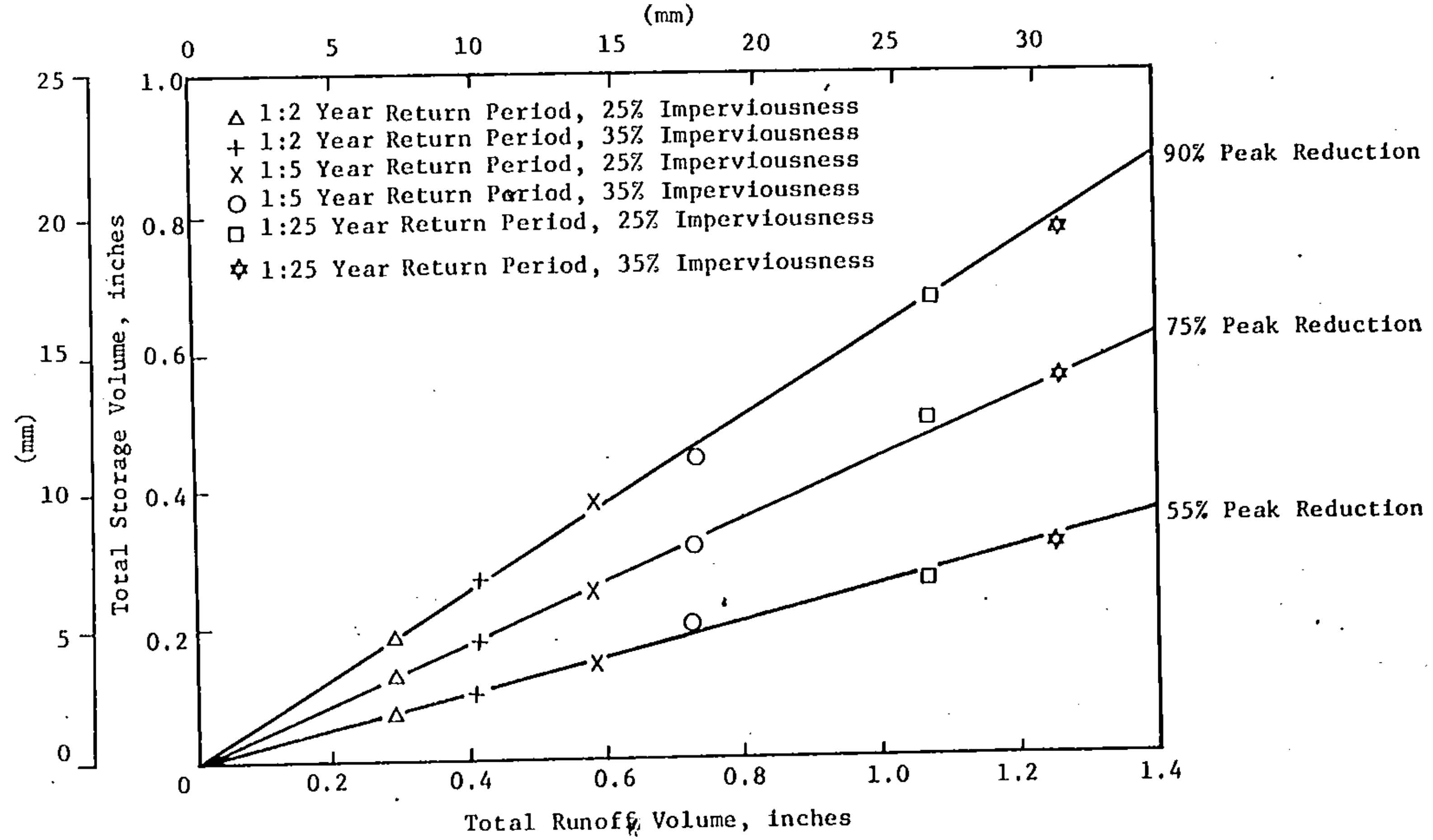
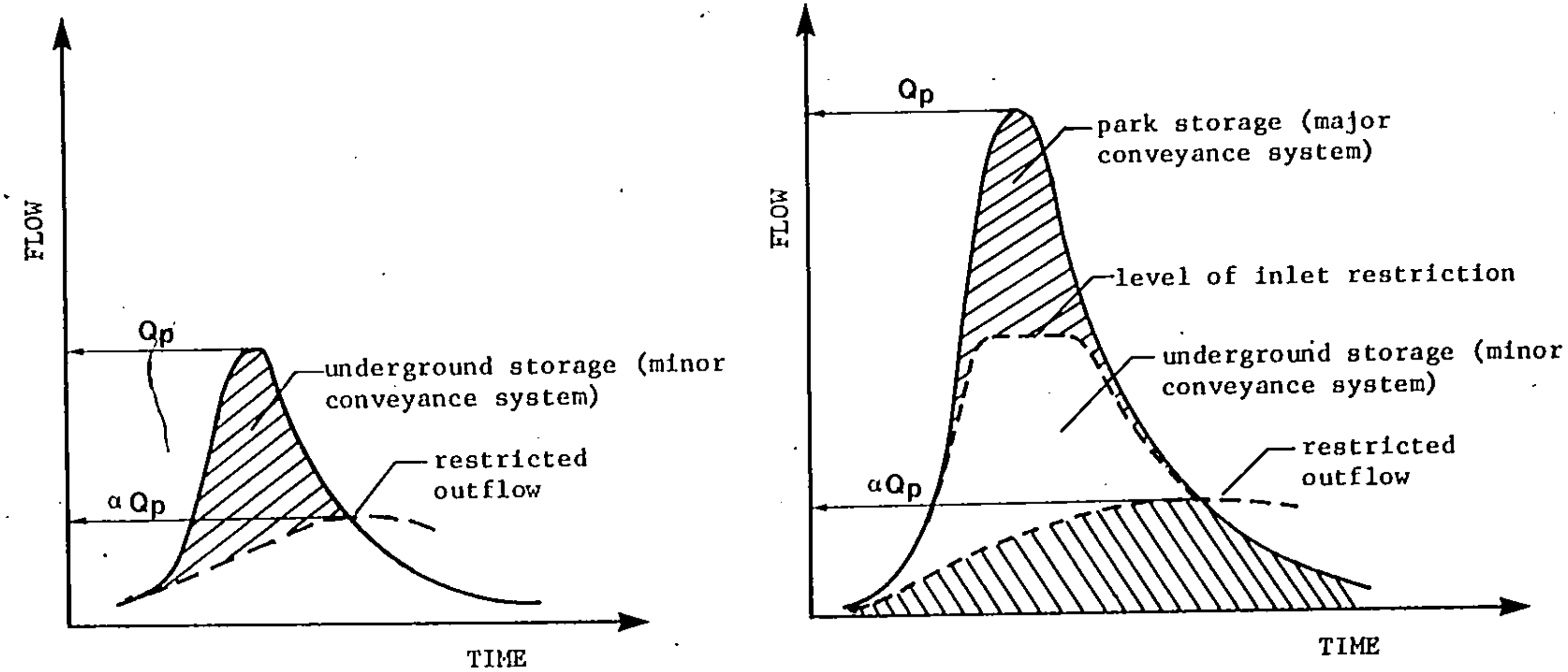


Figure 6.5: Storage Volumes Determined for Different Rates of Peak Reduction for a Given Total Runoff Volume

The operation of the "dual storage" system is illustrated with the hydrographs shown in Figure 6.6. The detailed modelling of the "dual storage" system has been conducted by Kassem (1982). As an approximation for determining the volumes of storages in the two separate systems it is, however, possible to employ a simplified method according to the following steps:

1. The design capacity (namely, for the 1:2 or the 1:5 year design storm return frequency) of the minor system is defined (see Figure 6.6(a));
2. If the maximum inflow rate into the minor system is restricted by "inlet control" devices, the balance of the flow during a major storm will be conveyed by the major system to the park depression storage; the level of flow separation is approximated by the line as indicated in Figure 6.6(b);
3. The required park storage volume for the 1:25 or 1:100 year storm is equal to the area between the complete runoff hydrograph and the line of separation which represents the level of inlet control;
4. The required underground storage volume which is in the "minor system" is equal to the total runoff volume minus the park storage volume and the runoff released through the minor system (Figure 6.6b).



(a) operation during minor storm events

(b) operation during major storm events

Figure 6.6: Determination of Storage Volumes for the "Dual Storage" System

Using the hydrographs determined with the SHM, Figures 6.7 and 6.8 show the volumes of park storage calculated for the 1 in 25 and 1 in 100 years design storms when the peaks of inflow into the minor system are limited by "restricting devices" to not greater than the 1 in 2 or the 1 in 5 years design storm levels, respectively. An imperviousness ratio of 25% is considered.

It is noted that the amount of underground storage volume increases substantially during major storms because the storm inlets can only restrict the peak but not the volume of inflow into the minor system. For the "dual storage" system, the underground storage should therefore be designed for the major storm frequencies with a volume as illustrated in Figure 6.6(b) and not Figure 6.6(a).

6.4 SUMMARY

A desk-top method for sizing detention storage is developed and presented. It is based on a "selective control" concept by which the peak of the post-development runoff hydrograph is reduced by any appropriately selected level instead of the pre-development runoff level which is a vaguely defined criterion. This "selective runoff control" criterion is more flexible and practical. The detention storages required for a range of percentages of peak reduction rates are determined.

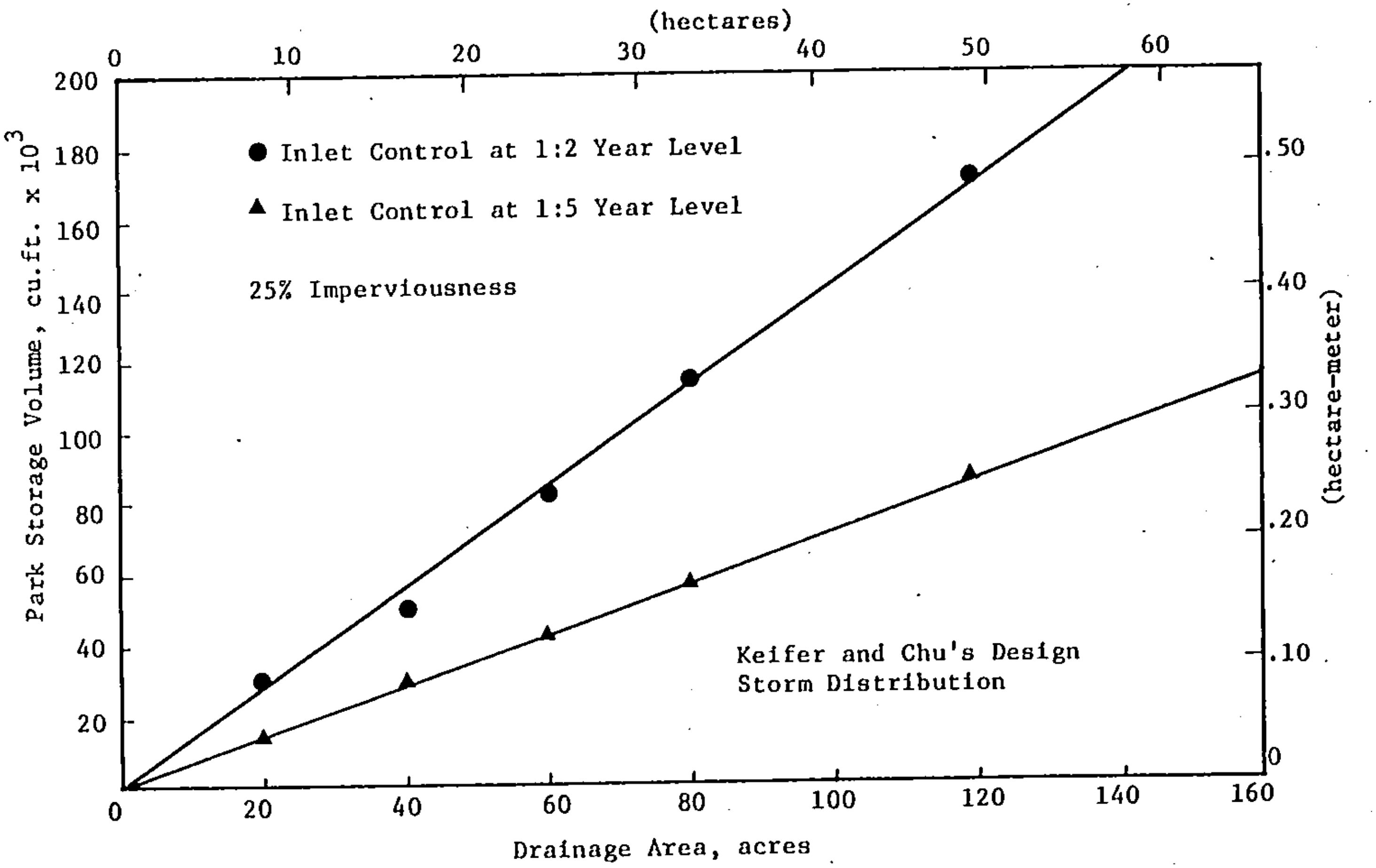


Figure 6.7: Park Storage Volumes Determined for the 1:25 Year Design Storm Return Period and Inlet Restrictions at the 1:2 and 1:5 Year Design Storm Levels

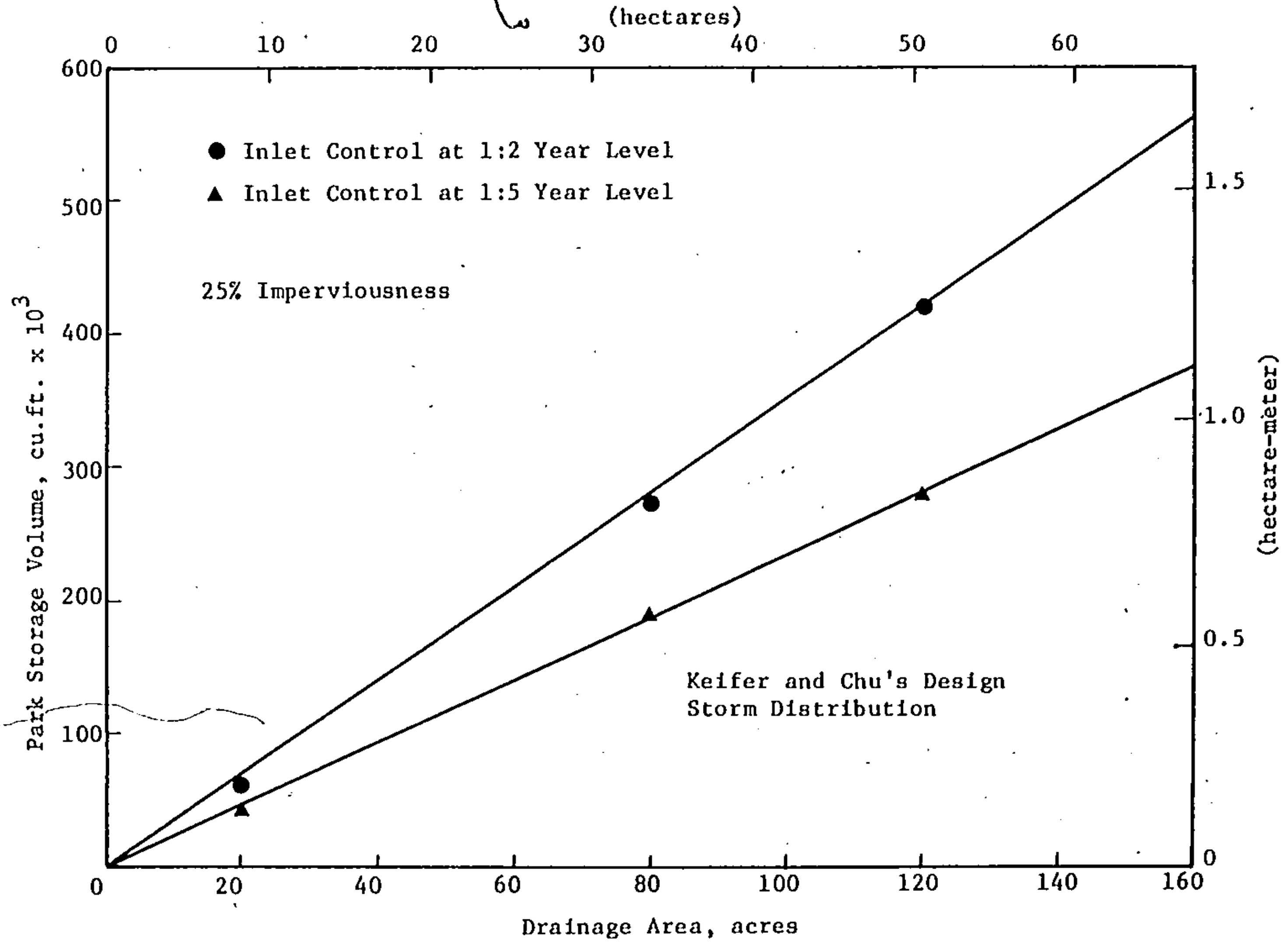


Figure 6.8: Park Storage Volumes Determined for the 1:100 Year Design Storm Return Period and Inlet Restrictions at the 1:2 and 1:5 Year Design Storm Levels

A desk-top method for sizing the "dual storage" system is also developed and presented. The method is also based on the "selective runoff control" criterion.

Both desk-top methods are applicable for preliminary design situations or may be used as independent check on final designs. The inflow hydrograph is based on SWMM simulated hydrographs and not on assumed simple hydrograph shapes; it does not inherit the weaknesses of the modified Rational Method hydrograph and the Linearized Hydrograph Method (Sections 2.3 and 2.4).

As a demonstration, the present study has used design storms distributed according to Keifer and Chu's method (1957), which are short duration and high intensity design storms. Other design storm distributions, such as the long duration and low intensities SCS Type II 24-hour design storm distribution, can also be employed. Since a long duration storm may produce higher runoff volume (see Section 3.3), further simulations should be carried out for comparison purposes. Other considerations are the use of a series or combination of design storms, and perhaps real storm events for the evaluation of the maximum detention volume.

The present study has not investigated the sensitivity of peak reduction to storage locations. Hawley et al (1981) have demonstrated that the timing of the controlled outflow from detention storage reservoirs can result in increases of the total peak discharges when the reservoirs are located at different parts of the watershed; the location of the detention storage reservoirs must therefore be carefully considered.

The effects of storage distribution when a series of detention reservoirs is used, are also additional factors to be considered in detention storage designs. Wycoff and Singh (1976) have shown that the efficiency of peak reduction reduces from 100% to 72% when six equal sizes of detention reservoirs are installed in series (Figure 6.9); an approximate method for determining the size of each detention reservoir arranged in series has also been proposed by Abt and Grigg (1978).

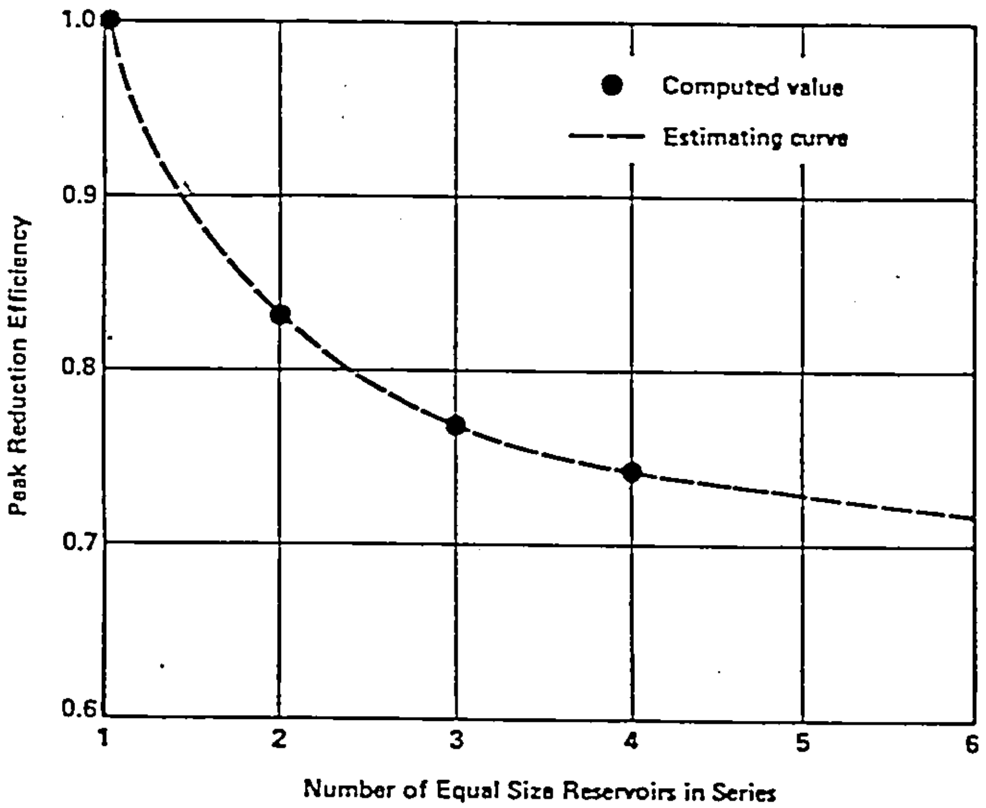


Figure 6.9: Effect of Storage Distribution on Flood Peak Reduction Efficiency (Wycoff and Singh 1976)

Chapter VII

TESTING OF SHM

7.1 RUNOFF HYDROGRAPHS AND PEAK FLOWS

Several watersheds located in Southern Ontario are selected for the application and testing of SHM. Since SHM is based on "lumped" watershed simulations, the peak discharge rates and runoff hydrographs obtained with SHM are compared with detailed SWMM simulations; the conventional Rational Method is also compared. For further comparison, the improved Rational Method according to Mitci (1974) (Section 2.2.2.5), who has suggested adjustments of the runoff coefficient with the storm duration, is also applied (Figure B.4)

Since the SHM procedure is a design method which cannot be used to reconstruct real storm events, it will be applied only with design storms. (However, the performance of a final design should always be checked with real events.) The same design storms are used for both the SHM and SWMM. Intense storms distributed with Keifer and Chu's design storm method are used because they are important for urban drainage design. These design storms are developed based on the I.D.F. curves of the Toronto-Bloor raingauge station (Figure

3.1). The same I.D.F. curves are used by the Rational Method.

7.1.1 Markham, Ontario

This watershed (Figure 7.1) is a residential watershed with a size of 150 acres and an average imperviousness ratio of 30%. It has three locations for schools and two parks. An average ground slope of 1 % is determined. The 1 in 5 year design storm is applied.

SHM is applied using a lumped approach and assuming a watershed with a homogeneous imperviousness ratio of 30 %. For the 1 in 5 year design storm, the total rainfall precipitation determined with Figure 5.4 is 1.85 inches. The runoff factor R determined for an imperviousness ratio of 30 % is 0.36 (Figure 5.5); therefore, the total runoff volume is 0.66 inches. By means of the peak factors P defined in Figures 5.8 and 5.9, the peak discharge rates at various design points of the watershed can be determined.

The above procedure is demonstrated by a series of diagrams shown in Appendix D.

Table 7.1 compares the values of peak discharges determined with SHM and detailed SWMM simulations. Values estimated by using the conventional Rational Method are also shown. It can be observed that the SHM simulations are within 5 to 6 % of the SWMM simulations, while the Rational

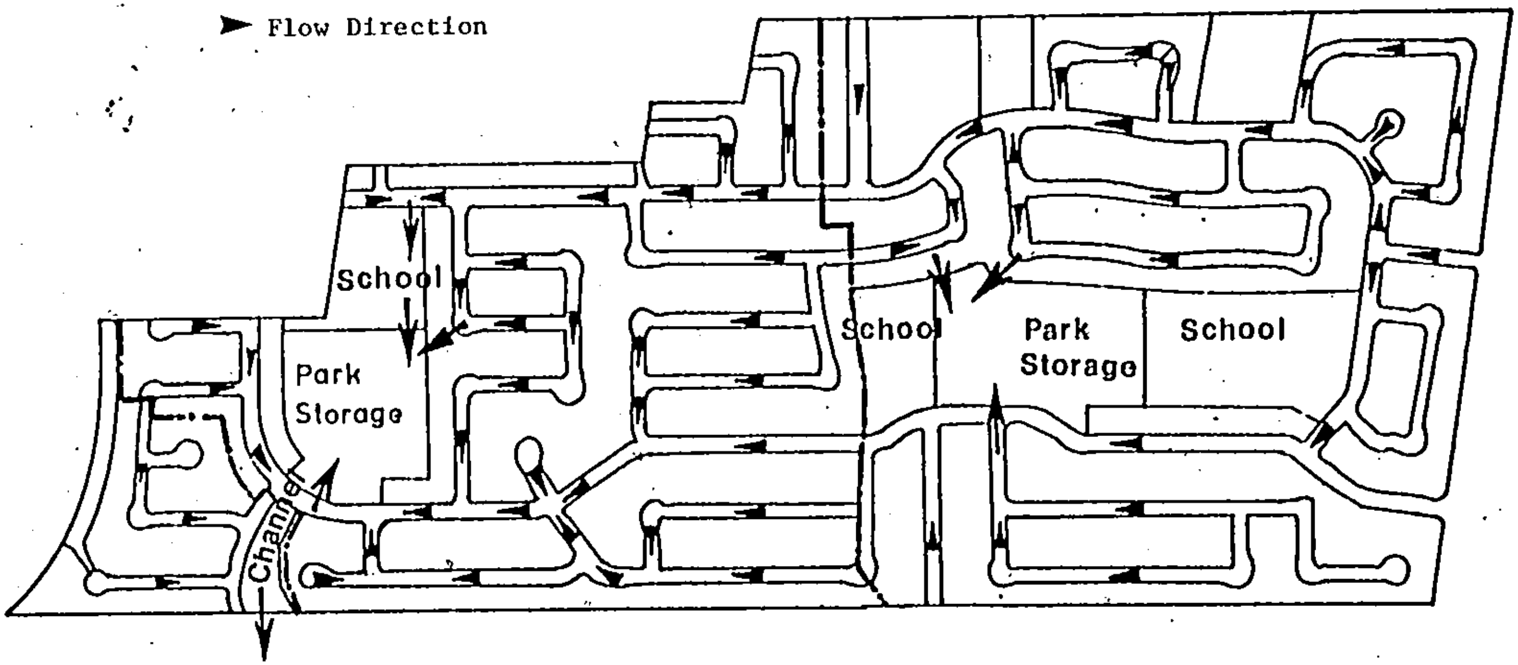


Figure 7.1: Markham Drainage System, Toronto

Table 7.1

Comparison of Design Flows Calculated by SWMM, SHM, and the Rational Method (RM) Markham, Ontario

Total area contributing (ac.) (ha.)	Detailed SWMM analysis (cfs) (cms)	SHM (cfs) (cms)	RM (cfs) (cms)	RM adjusted according to Mitci (1974) (cfs) (cms)
8 (3.2)	14 (.4)	14 (.4)	12 (.34)	11 (.31)
12 (4.9)	19 (.54)	20 (.57)	17 (.48)	17 (.48)
18 (7.3)	29 (.8)	30 (.85)	25 (.71)	25 (.71)
28 (11.3)	43 (1.2)	44 (1.25)	34 (.96)	36 (1.02)
34 (13.8)	50 (1.4)	53 (1.5)	40 (1.13)	44 (1.25)
48 (19.4)	66 (1.9)	70 (1.98)	50 (1.42)	57 (1.62)
96 (38.9)	130 (3.7)	130 (3.7)	108 (3.06)	121 (3.43)
144 (58.3)	189 (5.4)	190 (5.4)	147 (4.17)	169 (4.79)

Table 7.2

Comparison of Design Flows Calculated by SWMM, SHM, and the Rational Method (RM) Riseborough, Ontario

Total area contributing (ac.) (ha.)	Detailed SWMM analysis (cfs) (cms)	SHM (cfs) (cms)	RM (cfs) (cms)	RM adjusted according to Mitci (1974) (cfs) (cms)
22.0 (8.9)	36 (1.02)	36 (1.02)	30 (.85)	27 (.77)
85.5 (34.6)	118 (3.34)	120 (3.40)	99 (2.81)	103 (2.92)
121.8 (49.3)	160 (4.53)	161 (4.56)	132 (3.74)	147 (4.17)

Method results are all lower than SWMM by 14 to 24 %. As indicated earlier, this discrepancy with the Rational Method will increase with larger drainage areas (Chapter IV). The method suggested by Mitci (1974) is also applied to calculate the runoff coefficient for determining the peak discharge. For large drainage areas, the peak discharges estimated with Mitci's method are closer to SWMM although this is not the case for small drainage areas. However, the peak discharges calculated by Mitci's method are all still lower than SWMM by 7 to 21 %.

Since the peak runoff rate at the outlet is now known and the time to peak can be determined from Table 5.2, the total runoff hydrograph at the outlet is calculated by multiplying the ordinates Q(t)/Qp and t/tp of the SHM dimensionless hydrograph (Figure 5.12) to obtain the complete runoff hydrograph at the outlet. Figure 7.2 compares the hydrographs calculated with SHM and with detailed SWMM simulations. The hydrograph calculated with the "lumped" SHM method compares very well with the ones simulated by the detailed SWMM model.

7.1.2 Riseborough, Ontario

This watershed (Figure 7.3) is also for residential uses. The sewer system consists of two main branches. One branch drains the western part of the development and the

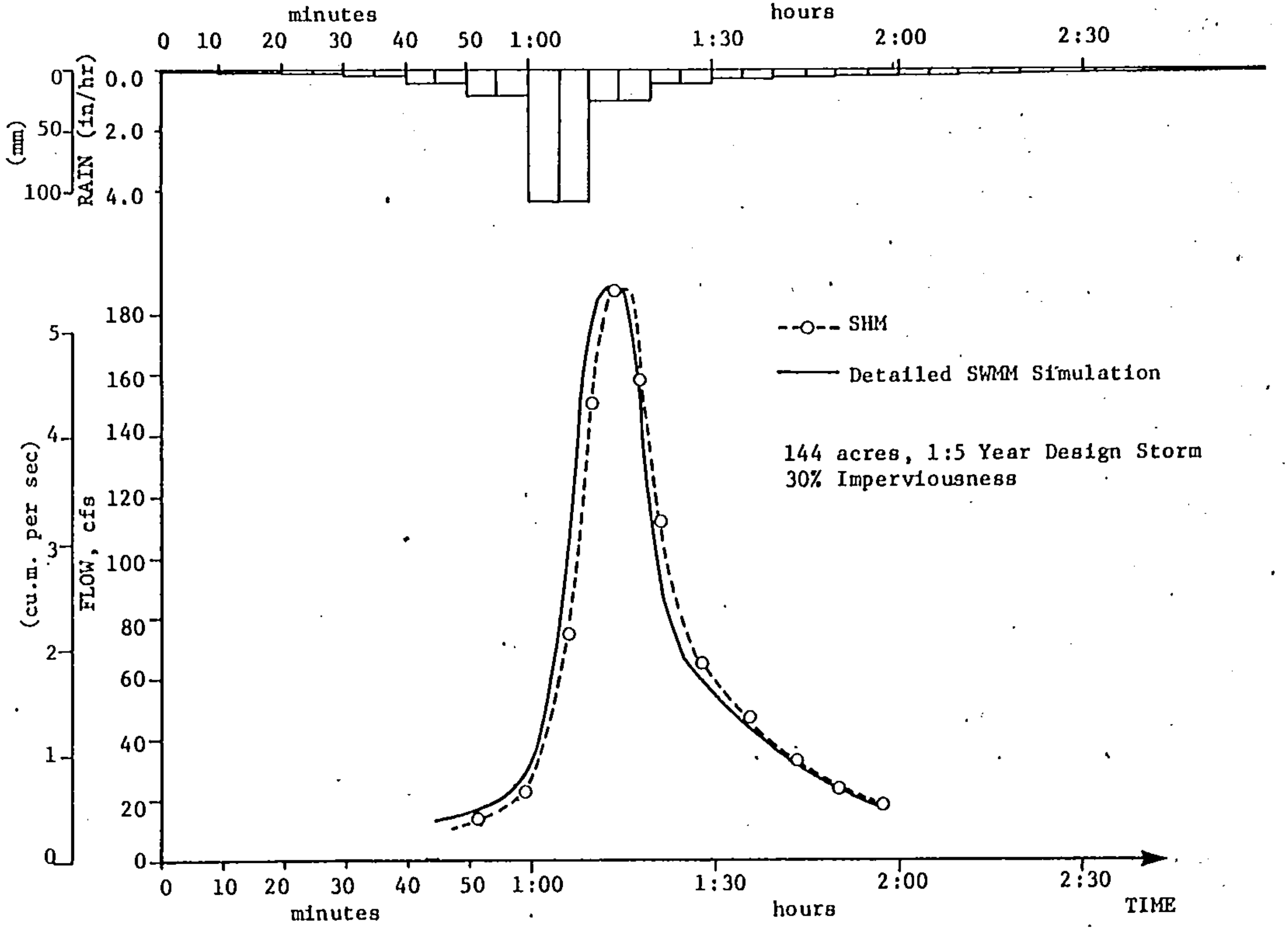


Figure 7.2 : Comparison of Hydrographs Obtained from Detailed Simulation and SHM for the Markham Drainage System, Toronto

Figure 7.3: Riseborough Drainage System, Toronto

other branch drains the eastern part of the watershed. The eastern branch is selected for analysis. It drains an area of 122 acres; an average imperviousness ratio of 30% is determined. The average ground slope is 1 %. The 1 in 5 year design storm is applied for analysis.

Similar computations are carried out using both the "lumped" SHM method and the detailed SWMM model; computations are also carried out using the Rational Method and Mitci's method (1974), respectively. Table 7.2 compares the values of peak discharges calculated with each method. It is observed that the Rational Method has underestimated all of the flows by 17 % while the flows estimated with the SHM are equal to SWMM. Peak discharges estimated with Mitci's method has reulted in lower flows for small drainage areas and increased flows for large drainage areas as compared with the Rational Method. These flows are still lower than SWMM by 25 and 8 %, respectively. Figure 7.4 compares the hydrographs calculated with SHM and SWMM at the point of outlet. The hydrograph calculated by SHM is in close agreement with SWMM. The time to peak is within 5 minutes of the detailed SWMM simulation.

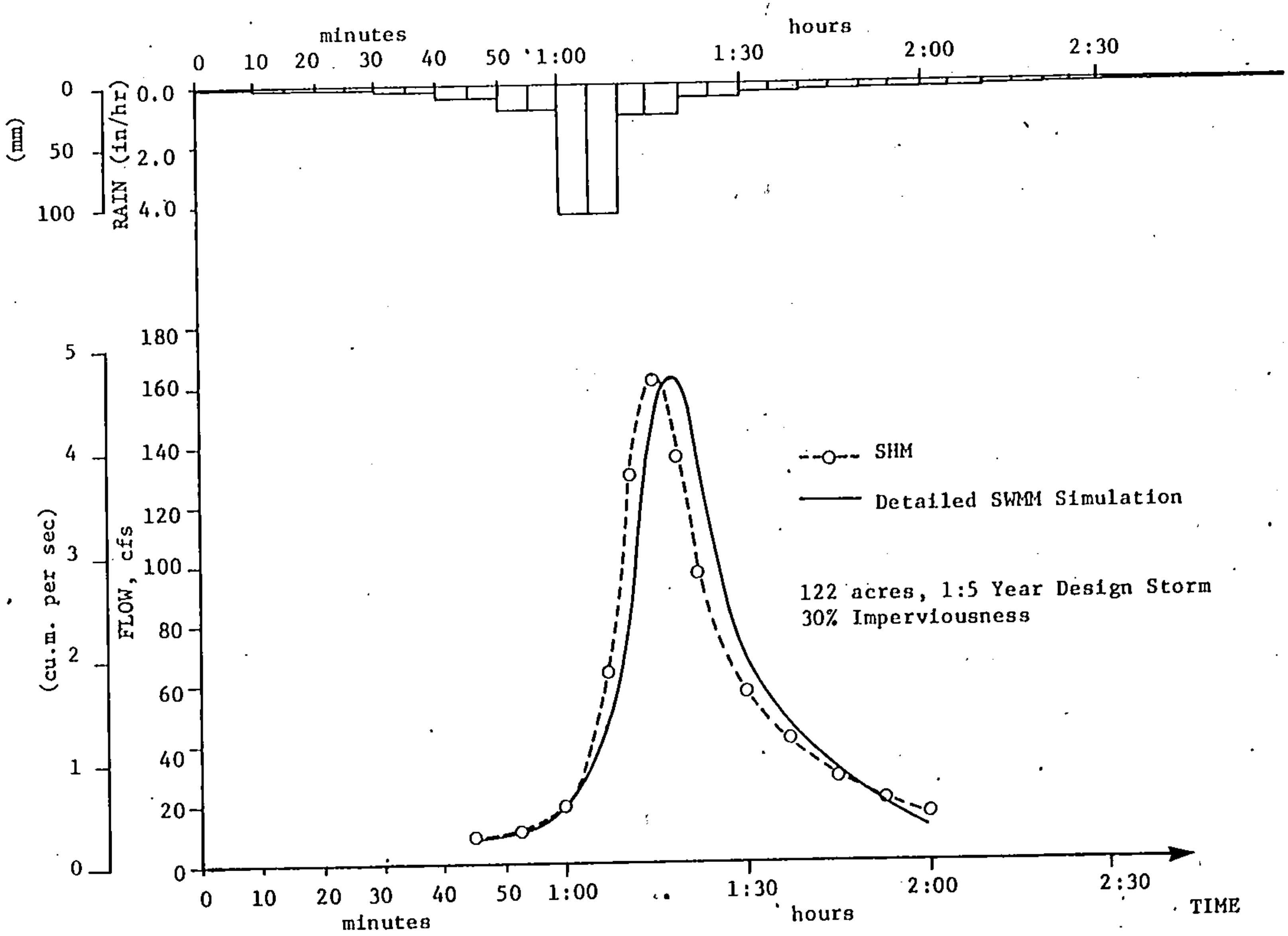


Figure 7.4: Comparison of Hydrographs Obtained from Detailed Simulation and SHM for the Riseborough Drainage System, Toronto

7.1.3 Pinetree, Ontario

This residential watershed (Figure 7.5) is drained by a sewer system with two main branches, and one of them (the eastern branch) is selected for analysis. It drains an area of 175 acres. For this sub-system, the land use characteristics are non-homogeneous. It has two schools, three parks, and a commercial area. Since the land use characteristics of this watershed are quite non-homogeneous, average imperviousness ratios and average SHM parameters have to be used. The steps used in averaging the SHM peak discharge parameters are illustrated in Appendix D. (For a more detailed approach, the system can be discretized into small sub-areas with homogeneous land use characteristics and analyzed individually by SHM.) An average ground slope of 1 % is determined. The 1 in 5 year design storm is applied.

Table 7.3 compares the peak discharges estimated with the SHM, SWMM, Rational Method, and Mitci's Method (1974), respectively. It shows again that the Rational Method tends to underestimate the peak discharge values as the drainage area increases (by 17 to 26 %). The peak discharges estimated by Mitci's method are lower than SWMM by 17 to 19 %. It is observed that the peak discharges estimated with the SHM are slightly inconsistent and vary between -5 % to +10%. This may be caused by the overall averaging of the imperviousness ratio. However, they can still be considered to be

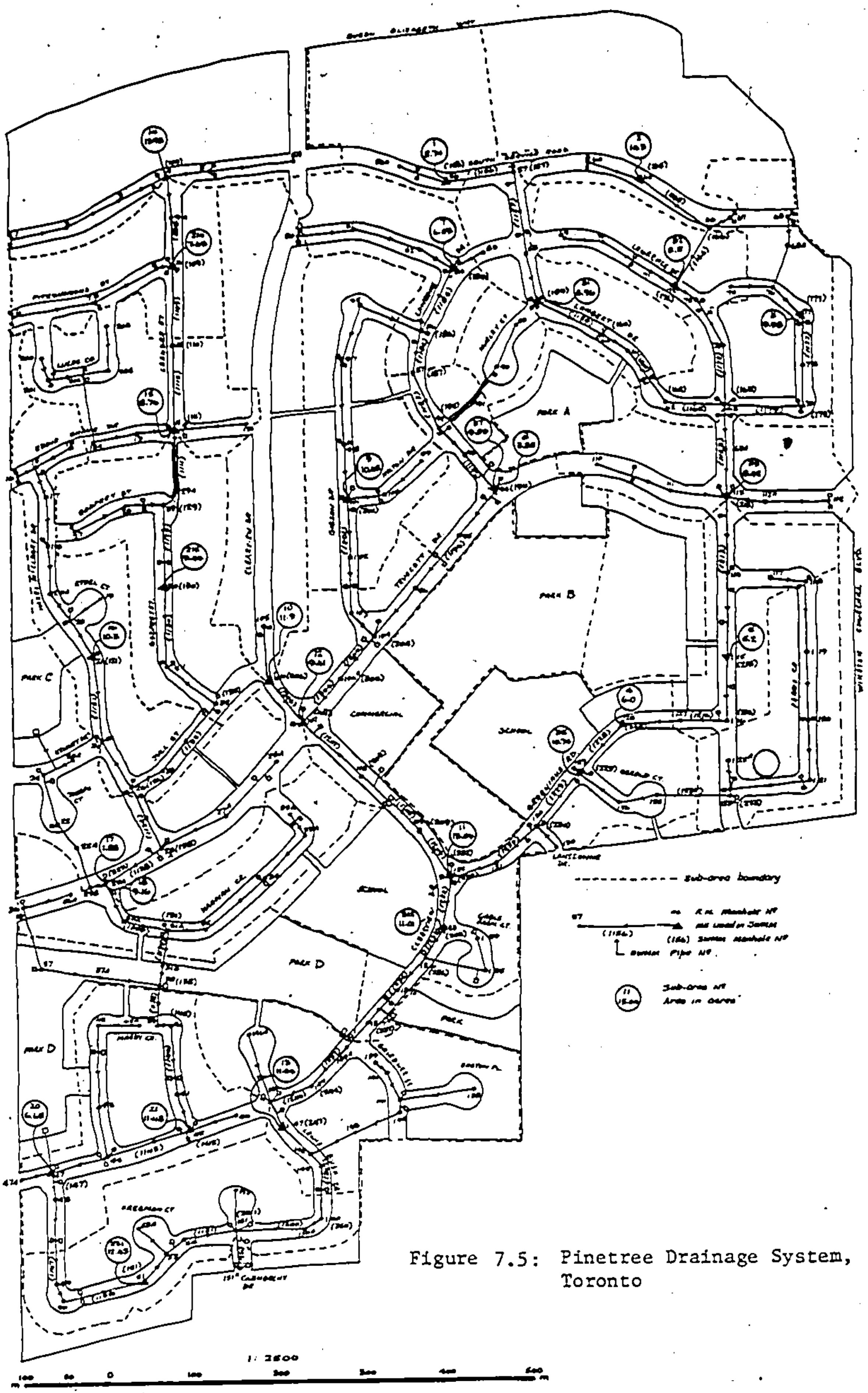


Figure 7.5: Pinetree Drainage System, Toronto

in reasonable agreement with the peak discharges simulated by SWMM. The total runoff hydrograph determined at the outlet is also in close agreement with that simulated by SWMM (Figure 7.6).

7.1.4 Discussion

The above comparisons show that as a desk-top method, flows calculated by the SHM are close to detailed simulations. Peak flows estimated by SHM for drainage areas larger than 50 acres are within 5 % of the detailed SWMM simulations, while peak flows estimated by the Rational Method tend to be underestimated (by as much as 26 %). For small drainage areas, the Rational Method estimates are quite close to SWMM (as indicated earlier in Chapter IV). Therefore, it can still be applied for peak discharge estimations for small drainage areas (Chapter IV).

Figure 7.7 compares the peak factor P obtained with detailed SWMM simulations for the various real watersheds with the lumped SHM. It is observed that the curve given by the SHM is in close agreement with the various peak factors. Peak factors obtained with design storms distributed with the I.D.F. curves for the city of Edmonton (Figure 3.2) using the Pinetree drainage system are also shown.

It is observed that the peak factors determined are less than those determined in this study by using the I.D.F.

Table 7.3

Comparison of Design Flows Calculated by SWMM, SHM, and the Rational Method (RM)

Pinetree, Ontario

Total area contributing (ac.) (ha.)	Detailed SWMM analysis (cfs) (cms)	SHM (cfs) (cms)	RM (cfs) (cms)	RM adjusted according to Mitci (1974) (cfs) (cms)
46.0 (18.6)	102 (2.89)	106 (3.00)	85 (2.41)	83 (2.35)
83.0 (33.6)	150 (4.25)	148 (4.19)	125 (3.54)	124 (3.51)
175.0 (70.9)	273 (7.74)	260 (7.37)	201 (5.70)	224 (6.35)

Table 7.4

Comparison of Detention Storages Determined with SHM and Detailed SWMM and EXTRAN Simulations

	SHM	SWMM
1:5 years peak flow	85 cfs (2.41 cms)	85 cfs (2.41 cms)
1:5 years minor system storage	46,000 cu. ft. (1,303 cu.m.)	45,800 cu. ft. (1,298 cu.m.)
1:25 years peak flow	150 cfs (4.25 cms)	139 cfs (3.94 cms)
1:25 years total storage	2.65 ac-ft. (0.33 ha-m.)	2.74 ac-ft. (0.34 ha-m.)
1:25 years park storage	0.96 ac-ft. (0.12 ha-m)	0.86 ac-ft. (0.11 ha-m.)
1:25 years minor system storage in excess of 1:5	27,400 cu. ft. (776 cu.m.)	36,150 cu. ft. (1,024 cu.m.)

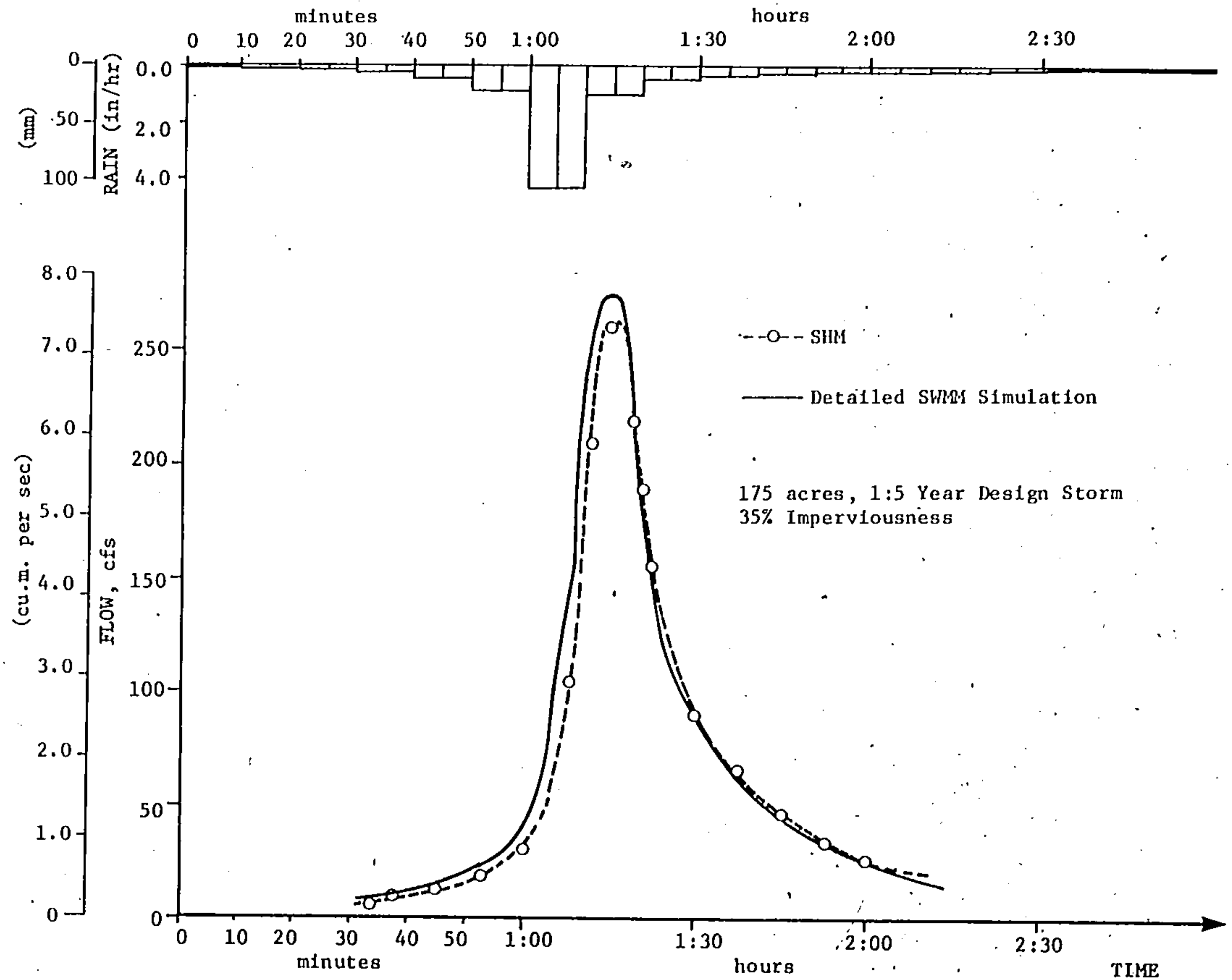


Figure 7.6: Comparison of Hydrographs Obtained from Detailed Simulation and SHM for the Pinetree Drainage System, Toronto

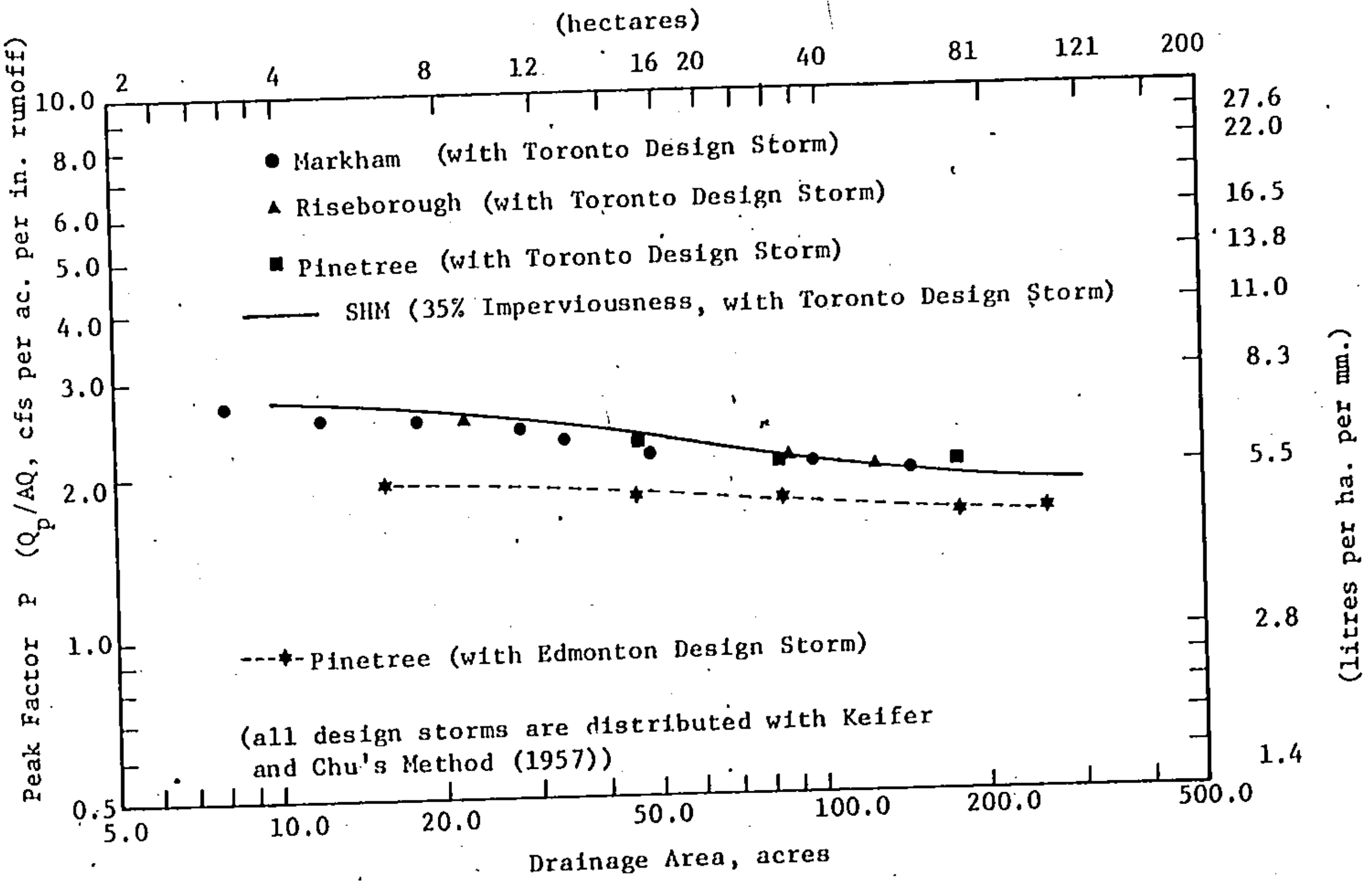


Figure 7.7: Comparison of Peak Factor P Obtained with Detailed Simulation and SHM

curves from the Toronto area. From Figures 3.1 and 3.2, it can be observed that the average intensities for the Edmonton I.D.F. curves are much lower than the Toronto I.D.F. curves. For example, for the 1 in 5 years design storm and a storm duration of 10 minutes, the average rainfall intensity determined with the former is 3.15 inches per hour while that determined with the latter is 4.3 inches per hour. Smaller peak factors are therefore obtained with the Edmonton I.D.F. curves.

The runoff hydrographs determined with the lumped SHM dimensionless hydrographs are very close to detailed simulations. The shapes of the hydrograph are in close agreement and the time to peak for the three comparisons are within 5 minutes.

However, further studies are required regarding the application of SHM for lumped non-homogeneous watersheds.

It is concluded that for the preliminary analysis of drainage systems the SHM can give results compatible with detailed computer simulations. As compared with the computer model SWMM, SHM does not require a mainframe computer once the parameters for the SHM peak discharge equation and the dimensionless hydrographs are derived.

7.2 DUAL STORAGE SYSTEM

For the residential development shown in Figure 7.8, detailed flow computations for the minor and major conveyance systems are carried out by applying the desk-top methods based on SHM and the detailed SWMM-RUNOFF and EXTRAN models. Schematics of the minor and major conveyance systems are shown in the Figure.

The total development has an area of 73 acres with an average imperviousness ratio of 30%. A park of 13 acres is located within the development, which can be used for temporary park storage. The pipe system is designed to carry the 1 in 5 year design storm runoff; all inlets to the storm sewer system are restricted to allow inflow only up to the 1 in 5 year design storm peak flow level (1:5 year inlet control).

The amount of on-line underground storage required for the minor system, when the outflow rate is controlled to 30 cfs for the 1:5 year design storm, is to be determined; the total park storage and minor system storage required during a major storm with a return period of 1 in 25 year are also to be determined.

The following steps are used to determine the peak runoff rates and storage volumes using the desk-top methods developed in this study:

1. Determine underground storage for 1:5 year storm

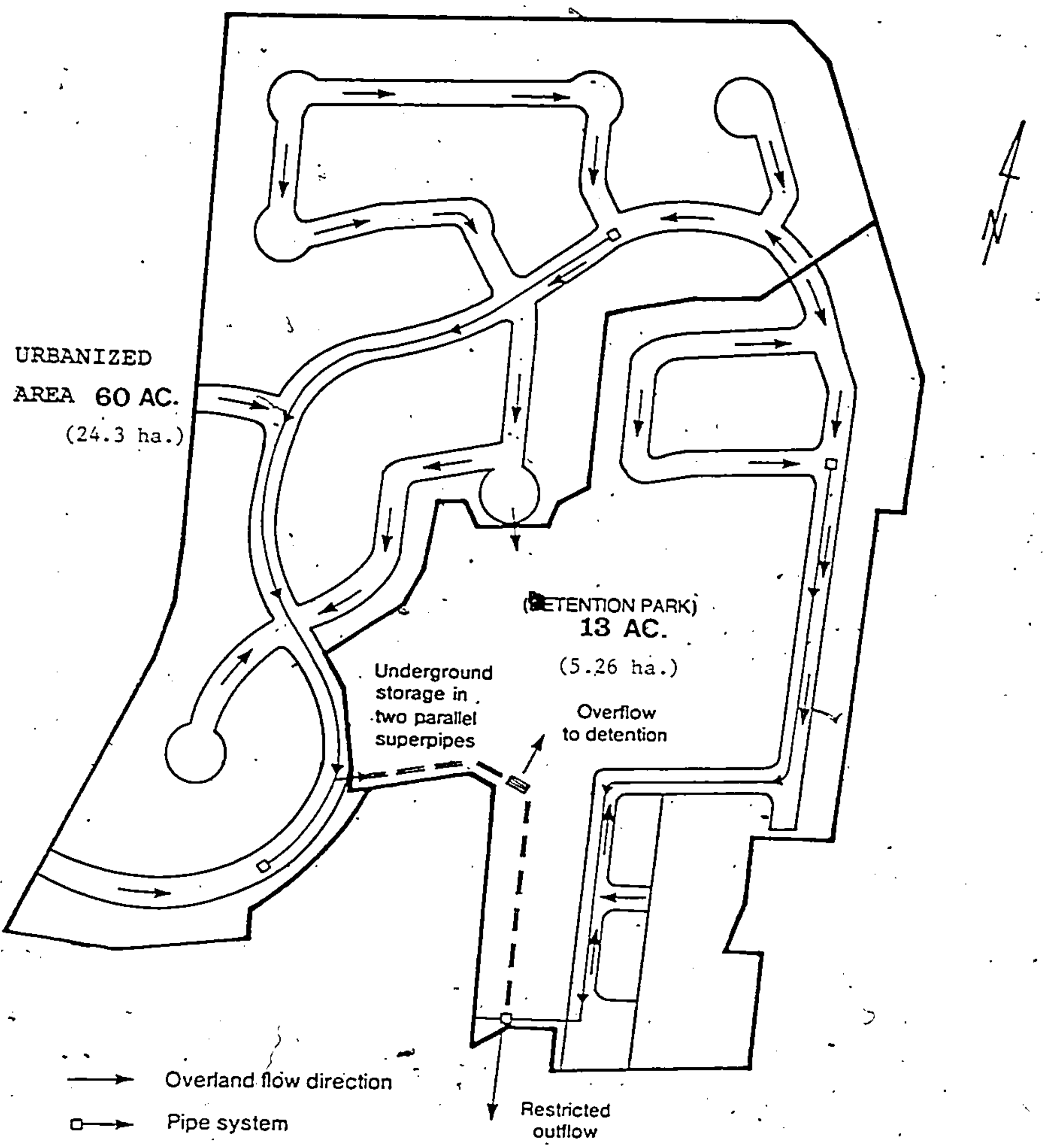


Figure 7.8: "Dual Storage" System

a) from Figures 5.4 and 5.5, the total runoff volume determined for the 30% imperviousness ratio and the 1 in 5 year storm is 0.66 in.

b) from Figure 5.8 and 5.9, the peak discharge determined for an urbanized area of 60 acres with 30% imperviousness ratio is 85 cfs.

c) peak flow reduction is therefore (30/85) x 100%, or 35%. Total underground storage required for a storm with 0.66 in. runoff volume and a peak reduction of 35% can be determined from Figure 6.5 which shows that 0.21 in. of storage is required. This is equivalent to 46,000 cu.ft. for an area of 60 acres.

2. Determine park storage and underground storage for 1:25 year storm

a) from Figures 5.4 and 5.5, the total runoff volume for the 30% imperviousness ratio and the 1 in 25 year design storm is 1.16 in.

b) from Figures 5.8 and 5.9, the peak discharge determined for an area of 60 acres and a 30% imperviousness ratio is 150 cfs.

c) the total controlled outflow is still 30 cfs, plus 5 cfs which is released from the park detention storage. Therefore, the total outflow for the 1 in 25 year storm is controlled to 35 cfs, or

(35/150) x 100% = 23%. The total storage required for this peak reduction is obtained from Figure 6.5 as 0.53 inch which is equivalent to 115,400 cu.ft. (2.65 ac-ft.) for an area of 60 acres.

d) park storage required for a 25-year storm with inlet controls at the 5-year level can be approximated by means of Figure 6.7. For an area of 60 ac. with an imperviousness ratio of 30 %, the park detention storage determined is 42,000 cu.ft., or 0.96 ac-ft.

e) the balance of the flow 115,400-42,000 = 73,400 cu.ft. would be carried by the pipe system. Since the underground storage designed previously in 1(c) can only accommodate 46,000 cu.ft., the extra storage volume of 27,400 cu.ft. (73,400 cu.ft. - 46,000 cu.ft.) will be needed. It can be observed that the volume of underground storage required for the control of major storms (1 in 25 years return frequency in this case) is approximately 1.6 times that of minor storms (1 in 5 years return frequency in this case).

Results obtained with the desk-top diagrams are compared with those obtained with detailed computer simulations using the SWMM model for simulating runoff and the EXTRAN model for the routing of flows in the minor and the major systems (Table 7.4).

While it is not considered that such a good agreement is possible in all applications, this example shows that the diagrams determined with the SHM hydrographs can give close results as compared with detailed simulations. The use of SHM, however, reduces the complexities invloved with the determination of the storage volumes in a "dual storage" system.

Chapter VIII

CONCLUSIONS

The conclusions of the study are summarized as follows:

1. It is concluded from the literature review that there exists a need for a desk-top method of runoff computation that is both consistent and compatible with computer models employed during the detailed design stage.
2. It is concluded that the traditional Rational Method generally produces peak flows which are less than the SWMM model. The inlet time is found to be a significant parameter in Rational Method flow computations for small watersheds with sizes less than 25 acres or time of concentrations of less than 20 minutes. It is concluded that flows calculated by the Rational Method can be made to match SWMM by systematically adjusting the inlet time. However, for larger watersheds (up to 460 acres) close results with SWMM cannot be obtained even when the inlet times are adjusted. Additional 'correction factors' are required to adjust the Rational Method flows. However, these

'correction factors' are limited to homogeneous watersheds and will vary for different design storm distributions.

3. An alternative desk-top method, herein entitled the Standard Hydrograph Method (SHM), has been developed. The SHM peak discharge equation is obtained on the basis of computer modelled simulations, for example, SWMM, by deriving a general relationship between the peak runoff rate, the watershed area, and the runoff depth. SHM utilizes the dimensionless hydrograph which incorporates a design storm profile for determining the runoff hydrograph.

4. SHM has been extended to develop two desk-top methods for determining detention storages in an urban drainage system for the Selective Runoff Control criterion. It is concluded that post-development runoff should be reduced by means of the Selective Runoff Control Criterion because the Zero Runoff Increase Criterion uses the vaguely defined pre-development flow value. It is concluded that these two desk-top methods can give results compatible with detailed computer simulations.

5. For the cases tested, results obtained with SHM are practically the same as detailed computer simulations. It is concluded that SHM can reduce the in-

consistencies involved in design by replacing the Rational Method in flow estimations. Another asset of SHM is that it is simple enough to be used on a programmable calculator. A particular application of the SHM is as a standardized tool for regulatory agencies. SHM provides a desk-top method which can be developed to be consistent with any desired combination of computer model, design storm distribution, and I.D.F. curve in a specific hydrologic region.

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Appendix A

THE RATIONAL METHOD

The Rational Method was first introduced in the United States by Emil Kuichling (1889) though its basic concept is said to have been implicitly applied by Mulvaney (1851). In Kuichling's paper entitled "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts" published in the ASCE transactions, he observed that the method of computing sewer sizes must be improved since "no fault can be found with the execution of the sewerage works.....failure must be attributed to the assumptions with regard to rainfall upon which the calculations are based." A series of gaugings of a number of large sewers in the city of Rochester were then carried out in order to find a relationship between rainfall and the corresponding maximum discharge. By examining the data collected for five districts, the percentages of rainfall discharged by the sewers during the period of maximum flow for fifteen major storm events were tabulated. The following conclusions were made:

1. "the percentage of rainfall discharged from any given drainage area is nearly constant for rains of all considerable intensities and lasting equal periods of

time...to the fact that the amount of impervious surface on a definite drainage area was also practically constant...";

2. "the said percentage varies directly with the degree of urban development of the district; or, in other words, with the amount of impervious surface";
3. "the said percentage increases....with the duration of the maximum intensity of the rainfall, until a period is reached which is equal to the time required for the concentration of the drainage waters from the entire tributary area at the point of observation; but if the rainfall continues at the same intensity for a longer period, the said percentage will continue to increase for the additional interval of time at a much slower rate than previously....to the fact that the permeable surface is gradually becoming saturated....in other words, the proportion of impervious surface slowly increases with the duration of the rainfall";
4. "the said percentage becomes larger when a moderate rain is immediately preceded by a heavy shower".

These conclusions constitute the basis of a "rational" method of sewer computation:

$$Q = mAr \tag{A.1}$$

$$m = at \tag{A.2}$$

$$r = b - ct \tag{A.3}$$

where m is the proportion of impervious surface, which is also the same as the proportion of the rainfall discharged during the period of greatest flow; A is the magnitude of the drainage area in acres; r is the maximum intensity of the rainfall in inches per hour; and t is the duration in minutes of such intensity.

A.1 PRESENT STATE OF THE RATIONAL FORMULA

Since 1889, the rational formula has become quite popular due to its simplicity and has been traditionally applied in most North American cities for determining storm sewer sizes. Its form has remained unchanged although improvements have been sought in defining the coefficient m, later referred to as the runoff coefficient C, the time of concentration tc of storm-waters, and the maximum rainfall intensity. Some success has been achieved by improving Equation (3) through the development of the Intensity-Duration-Frequency (I.D.F.) curves. This has permitted the rainfall intensity and hence the peak rate of runoff to be related with a recurrence interval. Few improvements, however, have been made with the runoff coefficient and the time of concentration. The interpretation of these parameters have been usually left to the judgement of the person using the formula, and this has led to significant variations in the amount of peak runoff calculated.

The Rational Method is expressed with the following equation:

$$Q = CiA \tag{A.4}$$

where Q = the maximum rate of runoff in cubic feet per second

C = the runoff coefficient which expresses the ratio of peak runoff rate to average critical rainfall rate with values ranging between 0.0 and 1.0

i = the average critical rainfall intensity in inches per hour for a specified return frequency and with a duration equal to the time of concentration of the watershed in consideration

A = the watershed area in acres tributary to the point of design

The conversion factor which converts acres-inch/hour into cubic feet per second is equal to 1.008 and is usually neglected. However, for metric units of ha-mm/hr., a conversion factor of 0.000278 is needed to convert the runoff rates into units of cubic meters per second.

A.2 APPLICATION OF RATIONAL METHOD FOR SEWER DESIGNS IN SMALL URBAN WATERSHEDS

Traditionally, the computation of flows in urban watersheds begins upstream at the first section of the pipe system and proceeds downstream. The time of concentration of the flow that travels from the most upstream sub-watershed to the first pipe section is mainly determined by the time for flow that travels overland from the edge of the development over grass areas (backyards and frontyards) and street surfaces to a storm sewer inlet which is connected to the first pipe section. This period of overland flow time has been traditionally referred to, as the 'inlet time'. Flows downstream of this 'inlet area' travel entirely through the storm sewer system. The time that flows are travelling through the sewer system has been traditionally referred to as the 'travel time'. The total time of concentration downstream of the first pipe section is therefore equal to the sum of the inlet time (t_i) and the travel time (t_t):

$$t_c = t_i + t_t \qquad \text{A.5}$$

Since the time of concentration is defined as the longest travel time when all flow paths are considered, it must be compared at each design point with the total time of concentration from all upstream watersheds and the longest time is then selected as the local time of concentration.

The velocity of overland flow is usually much smaller than the velocity of flow in the storm sewer. The inlet time can therefore be comparatively long and constitute a large percentage of the total time of concentration of the watershed especially when its size is small. The ASCE Design Manual No. 37 (1970) reported that the inlet time used for design normally varies between 5 to 30 minutes, with 5 to 15 minutes most commonly used. In densely developed areas, an inlet time of 5 minutes is often reported. In well developed areas with relatively flat slopes, an inlet time of 10 to 15 minutes is common, and in residential areas with flat slopes and widely spaced street inlets, inlet time of 20 to 30 minutes is customary.

In order to determine the peak runoff rate of a particular return frequency for the design of storm sewers in an urban drainage area, the following steps, as illustrated in Figure A.1 are in general followed:

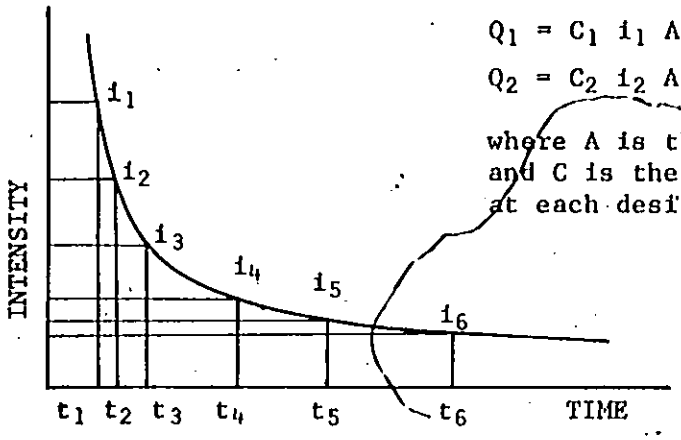
1. The area that contributes flow to the storm sewer section is defined;
2. An appropriate runoff coefficient is determined based on the soil characteristics and percentage of impervious surface of the drainage area;
3. A uniformly distributed rainfall is assumed to occur long enough for runoff from all parts of the area to contribute at the design point; this period of time

Figure A.1: Illustration of the Rational Method

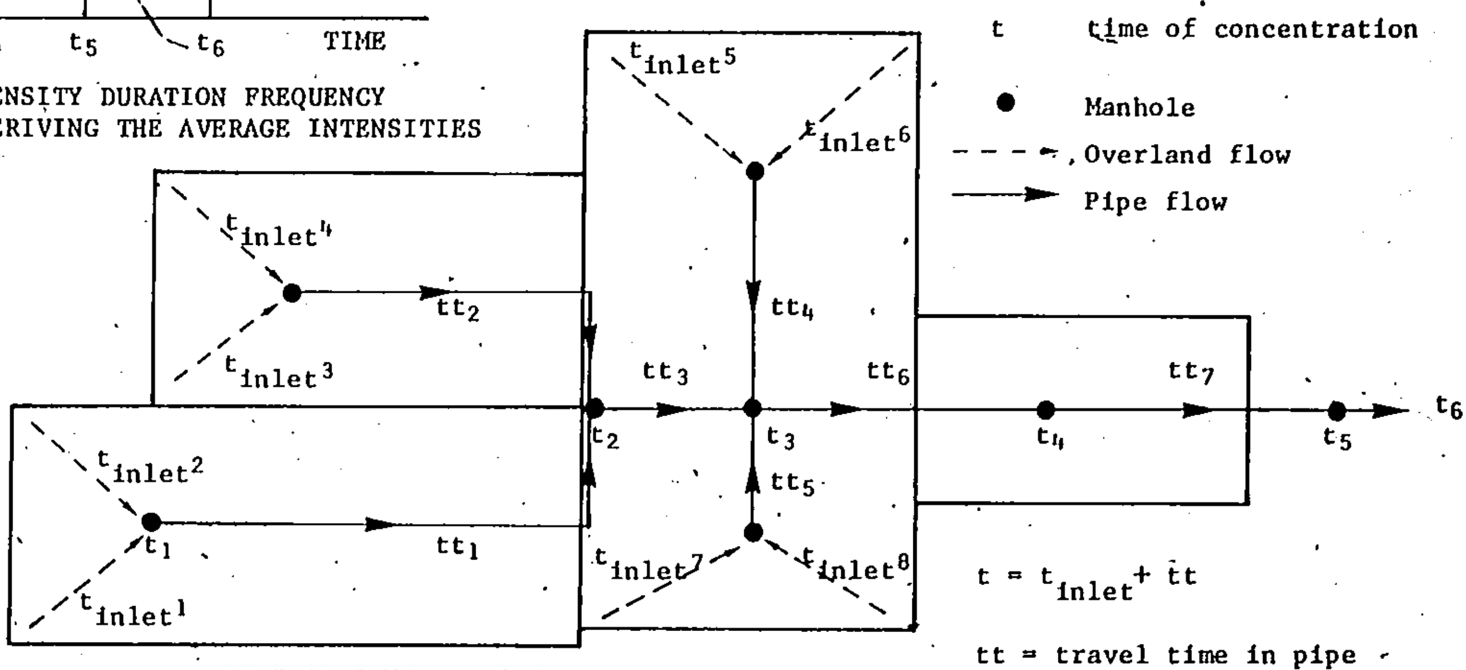
$Q_1 = C_1\ i_1\ A_1$

$Q_2 = C_2\ i_2\ A_2$ etc.

where A is the accumulated area to the point considered and C is the runoff coefficient for the accumulated area at each design point.



TYPICAL INTENSITY DURATION FREQUENCY CURVE FOR DERIVING THE AVERAGE INTENSITIES



TYPICAL SEWER DESIGN

is the time of concentration t_c, which is the longest travel time for water to travel from the hydrautically most distant point of the watershed to the point of design. In urban drainage designs, it is composed of the time for flow to travel overland from the most distant point of the watershed to the first storm sewer inlet (t_i), and on through the storm sewer system to the design point (t_t).

4. By means of tc, the maximum average rainfall intensity, with duration assumed to be equal to tc, is obtained for the chosen return frequency from the Intensity-Duration-Frequency curve;
5. the Rational Formula is applied to calculate the peak runoff rate.

A.2.1 Assumptions

Several major assumptions used in the Rational Method are listed as follows:

1. Rainfall

Rainfall is assumed to be spatially and temporally distributed over the entire watershed and storm duration. In order to obtain the maximum discharge, the duration of rainfall is assumed to be equal to the time of concentration, but in reality, real storm durations can be quite variable. It can be shorter or longer than the time of concentration. The rainfall intensity also normally varies considerably during a real storm, but an average rainfall intensity is used in the Rational Method.

2. Runoff Frequency

The return frequency of computed peak runoff rates is assumed to be the same as the return frequency of the rainfall. In reality, the return frequency of runoff is also dependent on other factors such as the antecedent moisture condition of the soil due to rainfall precipitation prior to the storm event.

3. Time of Concentration

As stated by Kuichling, for drainage areas of moderate sizes, the peak rate of runoff occurs when the time of concentration tc is reached. In some situations, this assumption depends on the shape of the watershed. A higher peak flow can be obtaind by choosing a different flow path to determine the time of concentration. For the example given Figure A.2, a shorter flow path has resulted in a higher runoff value (Mechler 1976). Also, this assumption is true only when equilibrium conditions exist.

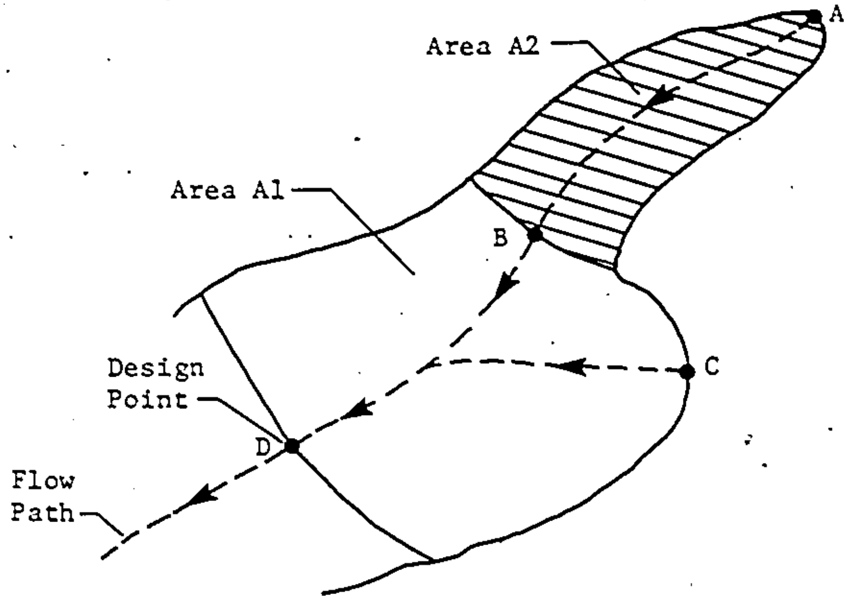


FIGURE A.2

4. Runoff Coefficient

It is assumed in the Rational Method that the runoff coefficient is independent of the combined effects of surface imperviousness, depression storage, soil type, soil moisture condition and infiltration, rainfall intensity, and duration of storm on the amount of runoff and can be represented by a simple fraction. Also, although it is clear that some of these parameters are quite variable with time, the runoff coefficient is usually assumed to have a constant value.

The Rational Method uses only a weighted average runoff coefficient to estimate the rate of runoff from non-homogeneous watersheds. This assumes that the watershed is homogeneous.

A.2.2. Limitations

One of the major limitations of the Rational Method is that it only provides an estimate of the peak discharge - a single point on the runoff hydrograph. The shape and volume of the runoff hydrograph cannot be defined as in other more complex methods, which simulate runoff hydrographs.

The Rational Method is also limited to small areas due to the assumption that rainfall is uniformly distributed over the entire watershed. A uniform rainfall is true only for small areas and storm durations. Unfortunately, the maximum area of watershed for which the Rational Method applies has never been clearly defined. A wide range of sizes, ranging from 200 acres (American Iron and Steel Institute 1980) to 5 square miles (Williams 1950, Corrugated Steel Pipe Institute 1972) have been recommended. Clarification regarding this limitation is essential.

The Rational Method is limited to the analysis of homogeneous watersheds. For a non-homogeneous watershed, The Rational Method can only use a weighted average runoff coefficient for runoff estimations.

Appendix B

SUPPLEMENTARY FIGURES AND TABLES FOR SECTION 2.2

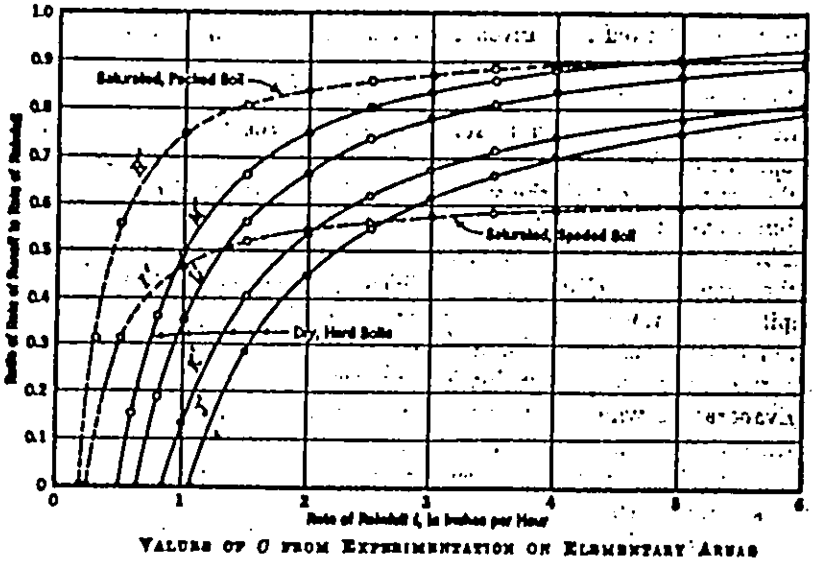


Figure B.1: C in terms of Rainfall Intensity
(Gregory and Arnold 1932)

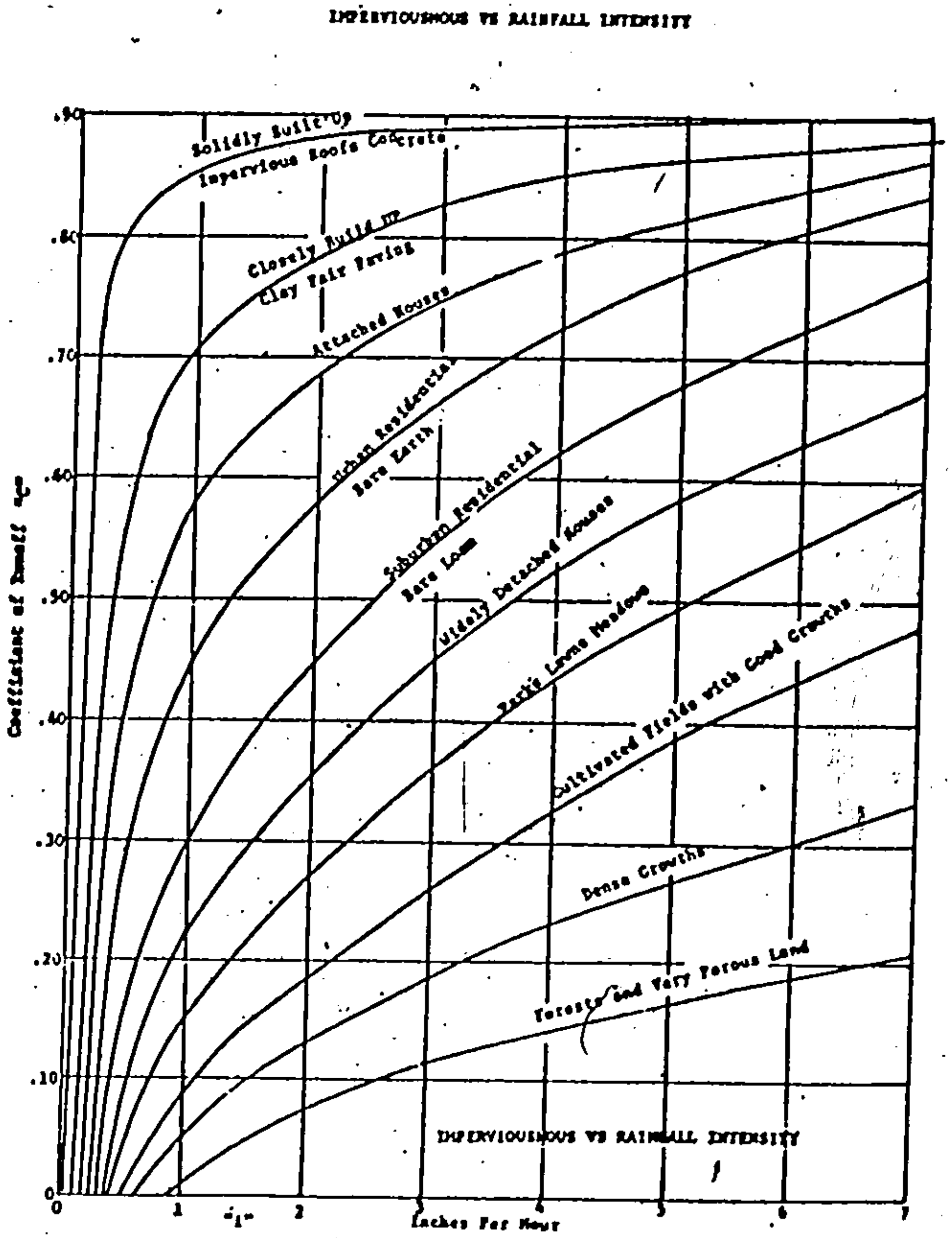
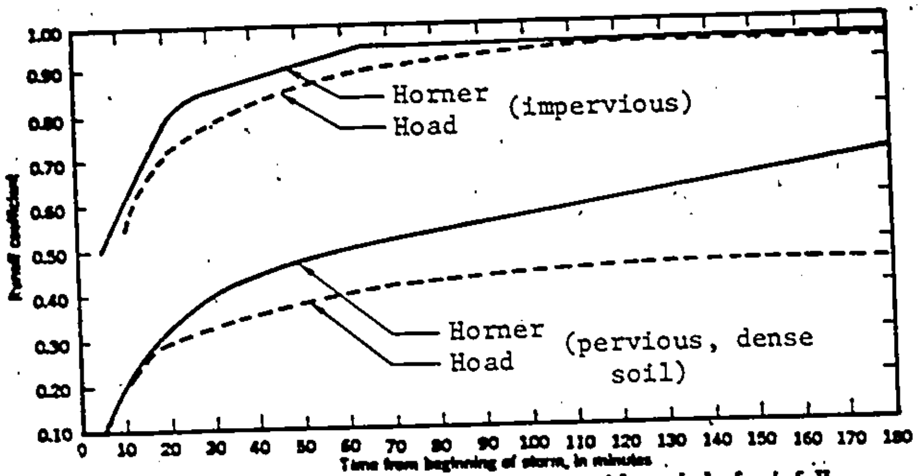


Figure B.2: C in terms of Rainfall Intensity
(Ordon 1973)



Variation in runoff coefficient with period of rainfall.

Figure B.3: C in Terms of Rainfall Duration
(Horner 1910, Hoad 1959)

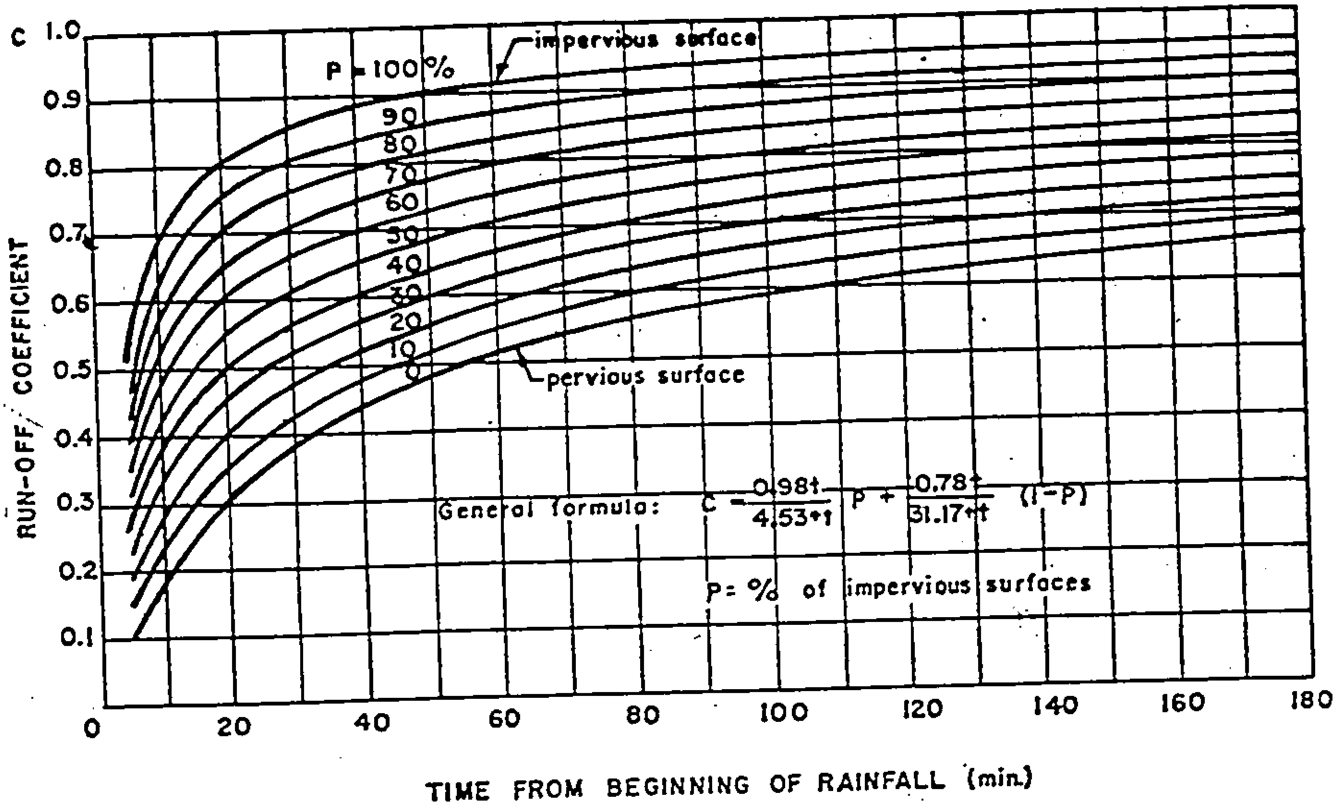
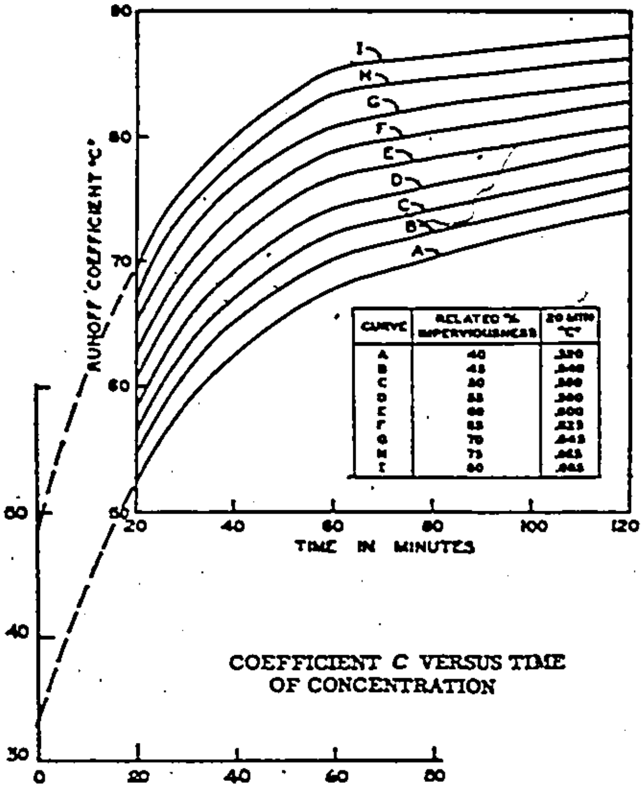


Figure B.4: C in Terms of Rainfall Duration
(Mitci 1974)



CURVE	RELATED % IMPERVIOUSNESS	20 MIN "C"
A	40	.520
B	45	.540
C	50	.560
D	55	.580
E	60	.600
F	65	.625
G	70	.645
H	75	.665
I	80	.685

Figure B.5: C in Terms of Time of Concentration
(McKinney 1967)

Table B.1

Average Runoff Coefficient for
Use in the Rational Formula
(ASCE 1970)

Description of Use	Runoff Coefficients
Business:	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential:	
Single family areas	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.70
Residential (suburban)	0.25 to 0.70
Apartment dwelling units	0.50 to 0.70
Industrial:	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemetaries	0.10 to 0.25
Playgrounds	0.20 to 0.40
Railroad yard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30

Character of Surface	Runoff Coefficients
Streets:	
Asphaltic	0.70 to 0.95
Concrete	0.80 to 0.95
Brick	0.70 to 0.85
Drives and walks	0.85 to 0.85
Roofs	0.75 to 0.95
Lawns; Sandy soil:	
Flat, 2%	0.05 to 0.10
Average, 2% to 7%	0.10 to 0.15
Steep, 7%	0.15 to 0.20
Lawns; Heavy soil:	
Flat, 2%	0.18 to 0.17
Average, 2% to 7%	0.18 60 0.22
Steep, 7%	0.25 to 0.35

Table B.2

Runoff Coefficients for Use in the Rational Formula

Land Use	Hydrologic Soil Group and Slope Range											
	A			B			C			D		
	0-2%	2-6%	6% +	0-2%	2-6%	6% +	0-2%	2-6%	6% +	0-2%	2-6%	6% +
Industrial	0.67[1] 0.85[2]	0.68 0.85	0.68 0.86	0.68 0.85	0.68 0.86	0.69 0.86	0.68 0.86	0.69 0.86	0.69 0.87	0.69 0.86	0.69 0.86	0.70 0.88
Commercial	0.71 0.88	0.71 0.89	0.72 0.89	0.71 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.89	0.72 0.90	0.72 0.89	0.72 0.89	0.72 0.90
High Density[3] Residential	0.47 0.58	0.49 0.60	0.50 0.61	0.48 0.59	0.50 0.61	0.52 0.64	0.49 0.60	0.51 0.62	0.54 0.66	0.51 0.62	0.53 0.64	0.56 0.69
Medium Density[4] Residential	0.25 0.33	0.28 0.37	0.31 0.40	0.17 0.35	0.30 0.39	0.35 0.44	0.30 0.38	0.33 0.42	0.38 0.49	0.33 0.41	0.36 0.45	0.42 0.54
Low Density[5] Residential	0.14 0.22	0.19 0.26	0.22 0.29	0.17 0.24	0.21 0.28	0.26 0.34	0.20 0.28	0.25 0.32	0.31 0.40	0.24 0.31	0.28 0.35	0.35 0.46
Agricultural	0.08 0.14	0.13 0.18	0.16 0.27	0.11 0.16	0.15 0.21	0.21 0.28	0.14 0.20	0.19 0.25	0.26 0.34	0.18 0.24	0.23 0.29	0.31 0.41
Open Space	0.05 0.11	0.10 0.16	0.14 0.20	0.08 0.14	0.13 0.19	0.19 0.26	0.12 0.18	0.17 0.23	0.24 0.32	0.16 0.22	0.21 0.27	0.28 0.39
Freeways and Expressways	0.57 0.70	0.59 0.71	0.60 0.72	0.58 0.71	0.60 0.72	0.61 0.74	0.59 0.72	0.61 0.73	0.63 0.76	0.60 0.73	0.62 0.75	0.64 0.78

[1] Lower runoff coefficients for use with storm recurrence intervals less than 25 years

[2] Higher runoff coefficients for use with storm recurrence intervals of 25 years or more

[3] High Density Residential - greater than 15 dwelling units per acre

[4] Medium Density Residential - 4 to 15 dwelling units per acre

[5] Low Density Residential - 1 to 4 dwelling units per acre

(Erin and Niagara Counties, New York 1977)

Table B.3

Runoff Coefficients for Use in Rural Watersheds (Schwab et al 1971)

Values of C in Q = CiA

Topography and vegetation	Soil Texture		
	Open sandy loam	Clay and silt loam	Tight clay
Woodland			
Flat 0-5% slope	0.10	0.30	0.40
Rolling 5-10% slope	0.25	0.35	0.50
Hilly 10-30% slope	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.70
Hilly	0.52	0.72	0.82

Table B.4

C in Terms of Rainfall Intensity (Rossmiller 1980)

Variation of C with I_a

Average Intensity in./hr.	P in.	F in.	I_a in.	SRO in.	C in.
0.0	0.0	0.0	0.0	0.0	0.00
0.5	0.5	0.5	0.0	0.0	0.00
1.0	1.0	0.5	0.5	0.0	0.00
1.5	1.5	0.5	0.5	0.5	0.33
2.0	2.0	0.5	0.5	1.0	0.50
2.5	2.5	0.5	0.5	1.5	0.60
3.0	3.0	0.5	0.5	2.0	0.67
3.5	3.5	0.5	0.5	2.5	0.71
4.0	4.0	0.5	0.5	3.0	0.75
4.5	4.5	0.5	0.5	3.5	0.78
5.0	5.0	0.5	0.5	4.0	0.80
5.5	5.5	0.5	0.5	4.5	0.82
6.0	6.0	0.5	0.5	5.0	0.83

Table B.5

C in Terms of Rainfall Duration
(Hoad 1959, Mitci 1974)

Equation by	Land Use Pervious	 Impervious
Hoad	C = t/(t+8)	C = 0.5t/(t+15)
Mitci	C = 0.98/(t+4.53)	C = 0.78t/(t+31.17)

Table B.6

(Rossmiller 1980)

Hydrologic Soil Group	X
A	39
B	61
C	74
D	80

Table B.7

Roughness Coefficients used in Kerby's Formula (Kerby 1959)

n in Kerby Formula	Type of Surface
0.02	smooth impervious
0.10	smooth packed soil
0.20	poor grass cover
0.40	pasture
0.60	timberland
0.80	deep forest with dense grass

Table B.8

Resistance Coefficients used in Izzard's Formula (Izzard 1946)

Surface	Value of C_r
Very smooth pavement	0.007
Concrete pavement	0.012
Dense bluegrass turf	0.060

Appendix C

THE EFFECT OF ANTECEDENT MOISTURE CONDITIONS ON RURAL WATERSHEDS

A joint study by the Department of Civil Engineering of the University of Ottawa and the Institute of Rural Engineering of the Federal Polytechnique School of Lausanne (EPFL) has been conducted to develop a methodology for the hydrologic modelling of flood control alternatives with short records of data. The method is first applied to two watersheds located near Geneva: the Seymaz and Foron. Three years of detailed rainfall and streamflow records (1977-1980) and twelve years of rainfall precipitation data (1965-1976) are available for the Seymaz, but very little data are available for the Foron; data calibrations are therefore mainly carried out for the Seymaz watershed. With completion of calibration, design flows are generated for the analysis of flood control alternatives.

Previous studies of the Seymaz watershed have been carried out by the EPFL (Sautier, 1979a, 1979b). In these studies, the rainfall and flow data collected at several locations within the basin have been statistically analyzed and Intensity-Duration-Frequency curves, Precipitation-Frequency curve, and Peak Discharge-Frequency curve are obtained. The unit hydrograph method has also been used to model two real storm events measured within the watershed in 1977 and 1978, respectively. The unit hydrograph obtained can be used to calculate hydrographs for other storms; however it cannot be applied to analyze future land use changes and flood control facilities. The computer model HYMO is therefore selected and applied in the second stage of the study.

Some of the findings of the study are summarized in the following sections.

C.1 DESCRIPTION OF STUDY AREA

The Seymaz watershed is situated on the north-east border of the City of Geneva (Figure C.1). Its total area is 38.4 sq.km. comprising of 80% of rural area in the upstream part (upstream of Pont Bouchet) and 20% of urban area in the downstream part of the watershed. The rural part is at present used mainly for vineyards, farming, and recreation. The urban part is mainly residential with 50% of industrial use. The basin is relatively flat with a general slope of less than 2%; the major soil type of the rural area is of highly permeable limono-sand and silty-limoneur.

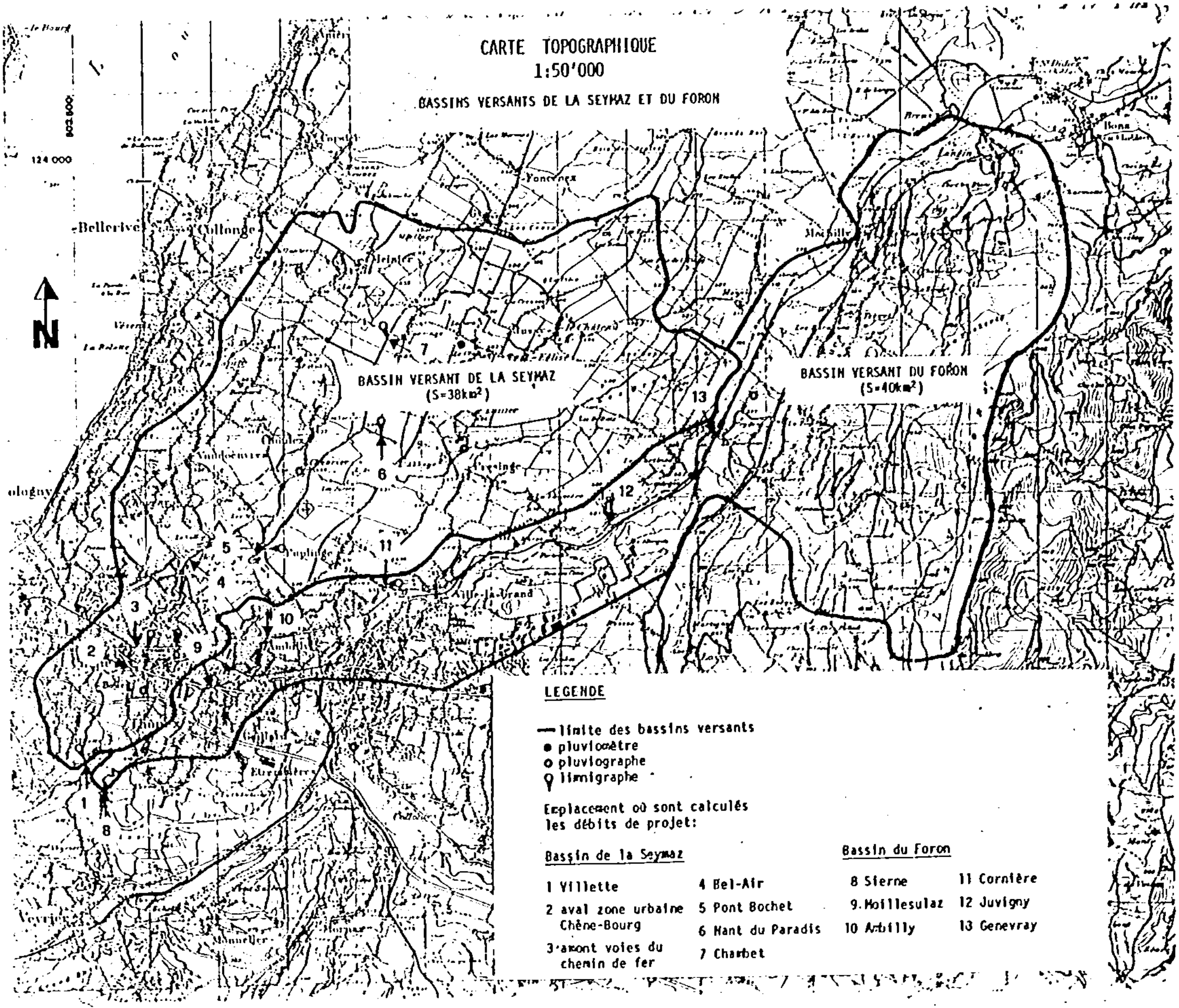


Figure C.1: The Seymaz Watershed

C.2 CALIBRATION

The Seymaz watershed is discretized according to land use and soil type and wherever possible at locations where the magnitudes of flows are of interest. Cross-sections along the Seymaz are measured and are used in HYMO for routing. Runoff producing rainfall and runoff data are selected for calibration.

The main parameters used by HYMO for simulating the runoff hydrograph, namely K, tp, and CN, are obtained by calibrating and verifying with the measured data for the rural areas. The runoff hydrographs simulated are compared with the measured data and close calibrations for two of the calibrated events are shown in Figures C.2 and C.3.

C.3 FLOW PREDICTIONS

After the calibrations are completed, the calibrated K, tp, and CN values are used to predict flows for designs using the Precipitation-Frequency curve that is obtained by the EPFL (Figure C.4). With this curve, the total rainfall precipitation that is expected to exceed for any given return period can be obtained. For all return frequencies, the total depth of precipitation is distributed according to a standard real storm pattern that has a duration of 12 hours.

PONT-BOCHET

7-9 août 1978

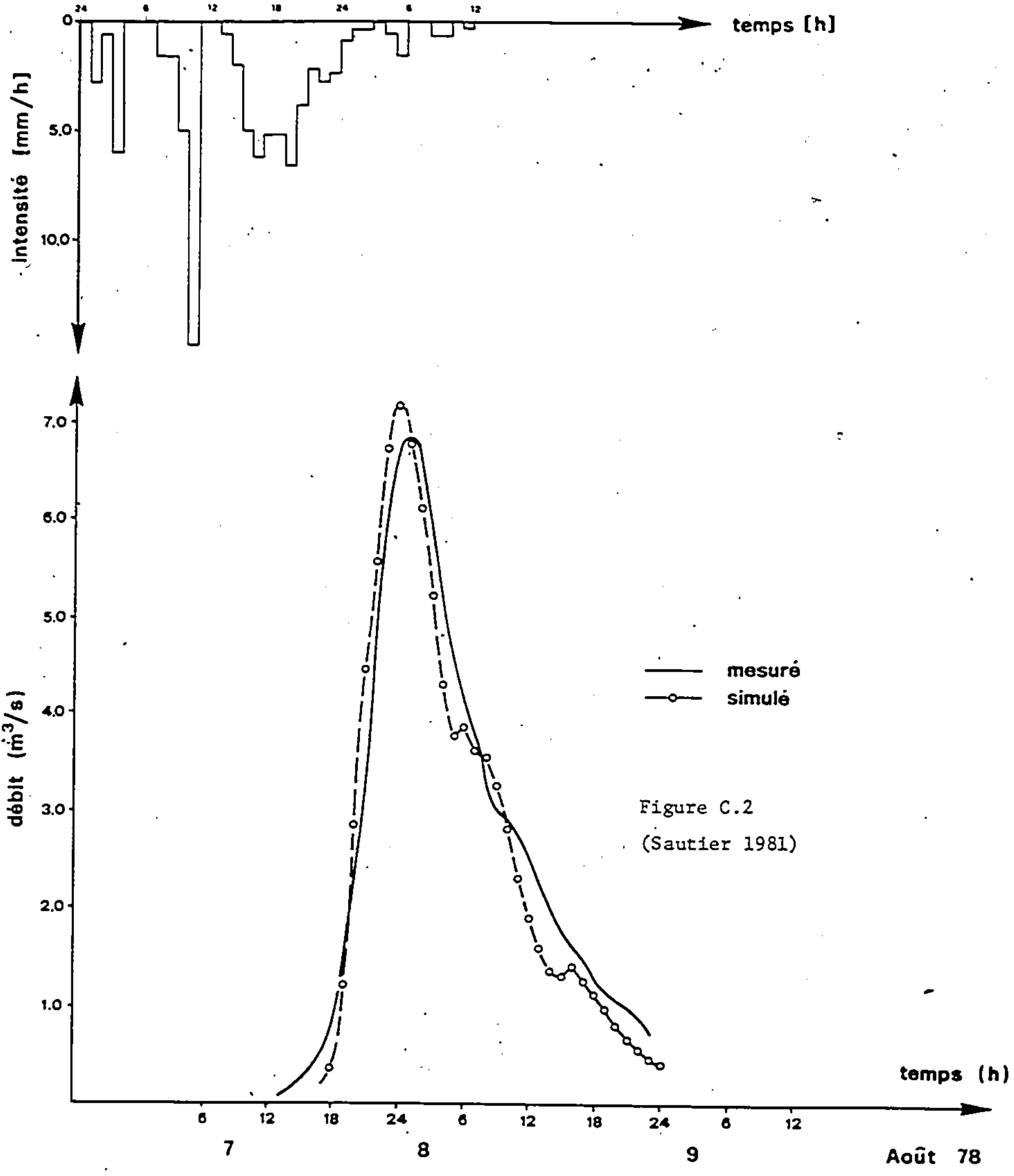


Figure C.2
(Sautier 1981)

VILLETTE

7-9 juillet 1980

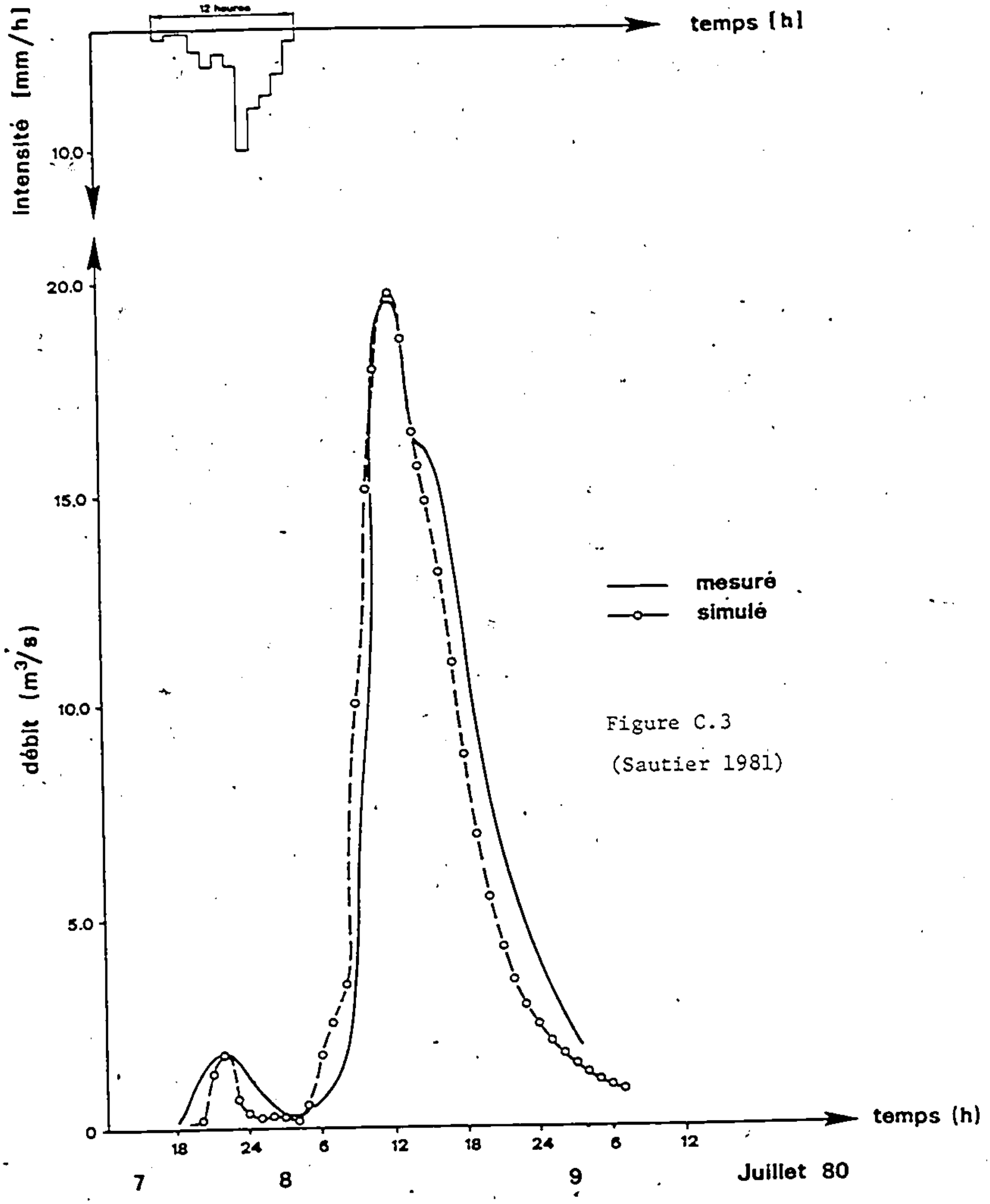


Figure C.3

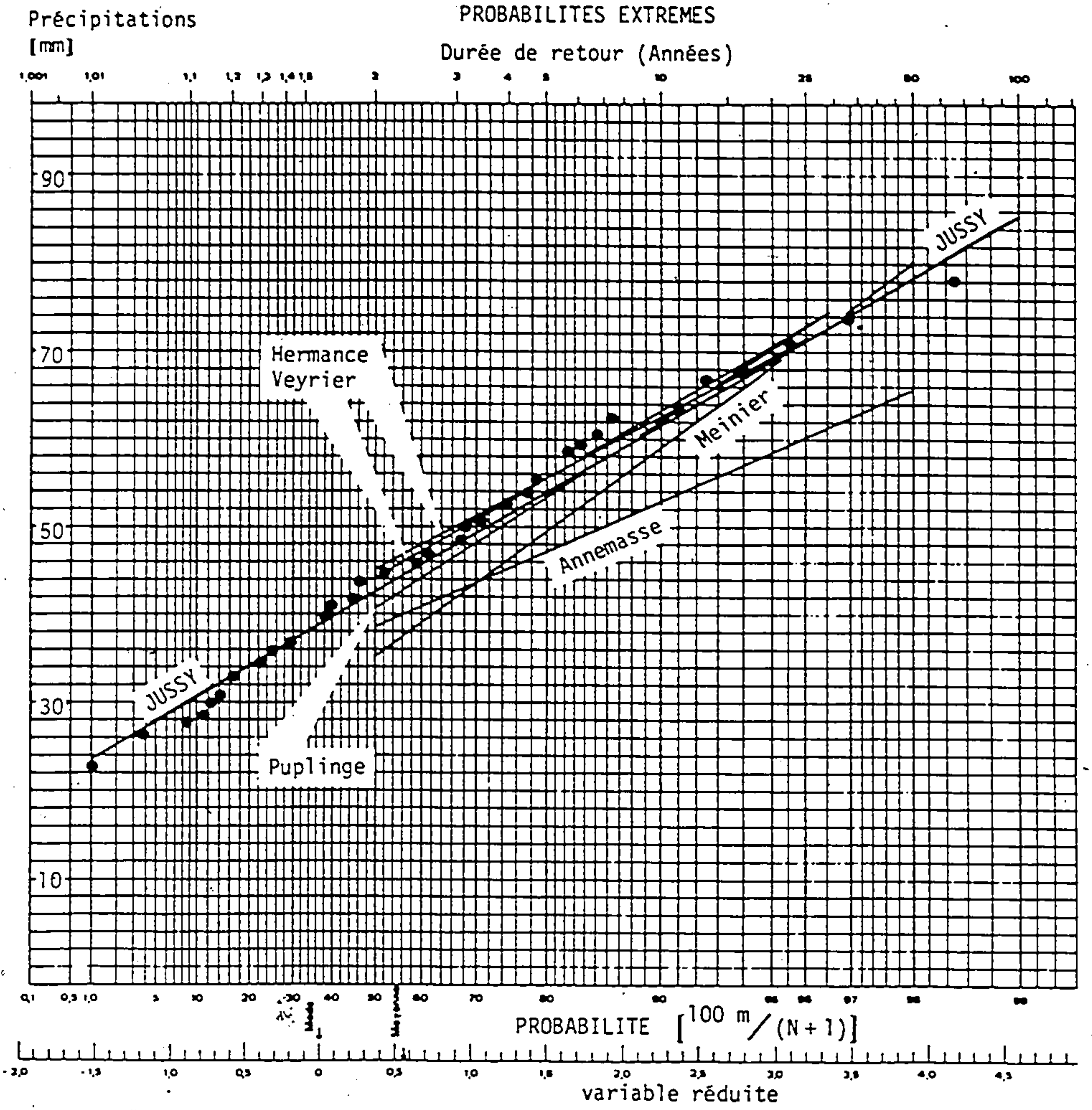
(Sautier 1981)

Figure C.4

PLUIES JOURNALIERES

MAXIMUM ANNUEL

(Sautier 1979)



In current practice, it is common to assume, either by fact or implication, that the return frequency of the design flow will have the same return frequency as the total rainfall. This assumption is not necessarily true since the saturation level of the soil also dictates significantly the amount of runoff from a storm (Patry and McPherson, 1979). Since HYMO uses the Curve Number, to calculate the volume of runoff, consideration must be given to the selection of CN on the basis of the antecedent moisture condition (AMC), which in turn determines the 'correct' return frequency of the design flow.

An attempt is therefore made to investigate the exceedence probabilities of design flows and soil moisture conditions for summer storm events. The procedure involves several steps. First, instead of classifying the moisture level of the soil in three discrete ranges of values as suggested by the SCS procedure (SCS 1971), the Antecedent Precipitation Index (API) is used to express the saturation level of the soil; hence, a relation of CN and one specific moisture condition (i.e. API) is established for each measured event; second, an exceedence probability function for the range of API values expected during the summer season of each year is established; hence, the exceedence probability of the CN values for the given watershed can also be established; third, the exceedence probabilities of the maximum soil moisture condition (API) and of the design rainfall can be combined to approximate the design runoff rates.

C.4 RELATIONSHIP OF CN AND API

Using the historical rainfall data, the value of CN can be related with one specific magnitude of moisture condition for the Seymaz watershed. The API is used as an indicator of the saturation level of the soil. For individual storms, Kohler and Linsley (1951) proposed that

$$P = b_1 P_1 + b_2 P_2 + \ldots\ldots + b_t P_t \qquad \text{C.1}$$

where b_t is a constant that is less than unity, P is the total precipitation occurring t days prior to the storm under consideration, and bt is assumed to be a function of time. The soil moisture level is considered to decrease exponentially with time during periods of no rainfall precipitation. The API on a day-by-day basis is therefore:

$$API(i) = K \; . \; API(i-1) \qquad \text{C.2}$$

where API(i) = antecedent predipitation index for current day;

API(i-1) = antecedent precipitation index for previous day;

K = recession factor ranging normally between 0.85 to 0.98.

If rain occurs on any day, the amount of precipitation (P) is added to the index

$$API(i) = K \; . \; API(i-1) + P(i-1) \qquad \text{C.3}$$

Since only a fraction of the precipitation is infiltrated, an index of precipitation minus runoff should be more accurate than using the total precipitation alone. However, the minor improvements gained do not justify the added computation (Linsley et al, 1975).

The API on any day theoretically depends on precipitation over an infinite antecedent period, but if a reasonable initial value is assumed, the computed index will approach the true value within 30 days. To ensure that this is correct, several initial values of API, namely 15 mm, 30 mm, and 60 mm, respectively, are used. The same value of API is approached within five weeks of calculation.

By calculating the API of each storm event used for calibration and by using the calibrated CN value of the storm event, a relationship between the API and CN can be established. A plot of these points is shown in Figure C.5. With this curve, the corresponding CN number for a known API can be determined.

C.5 RETURN FREQUENCY OF API

In order to obtain the return frequency of API, 15 years (1964-1978) of rainfall precipitation records measured during the summer season at Meinier station are analyzed by calculating the daily API with Equation C.3. The maximum API prior to storms with rainfall larger than 1.5 mm (since

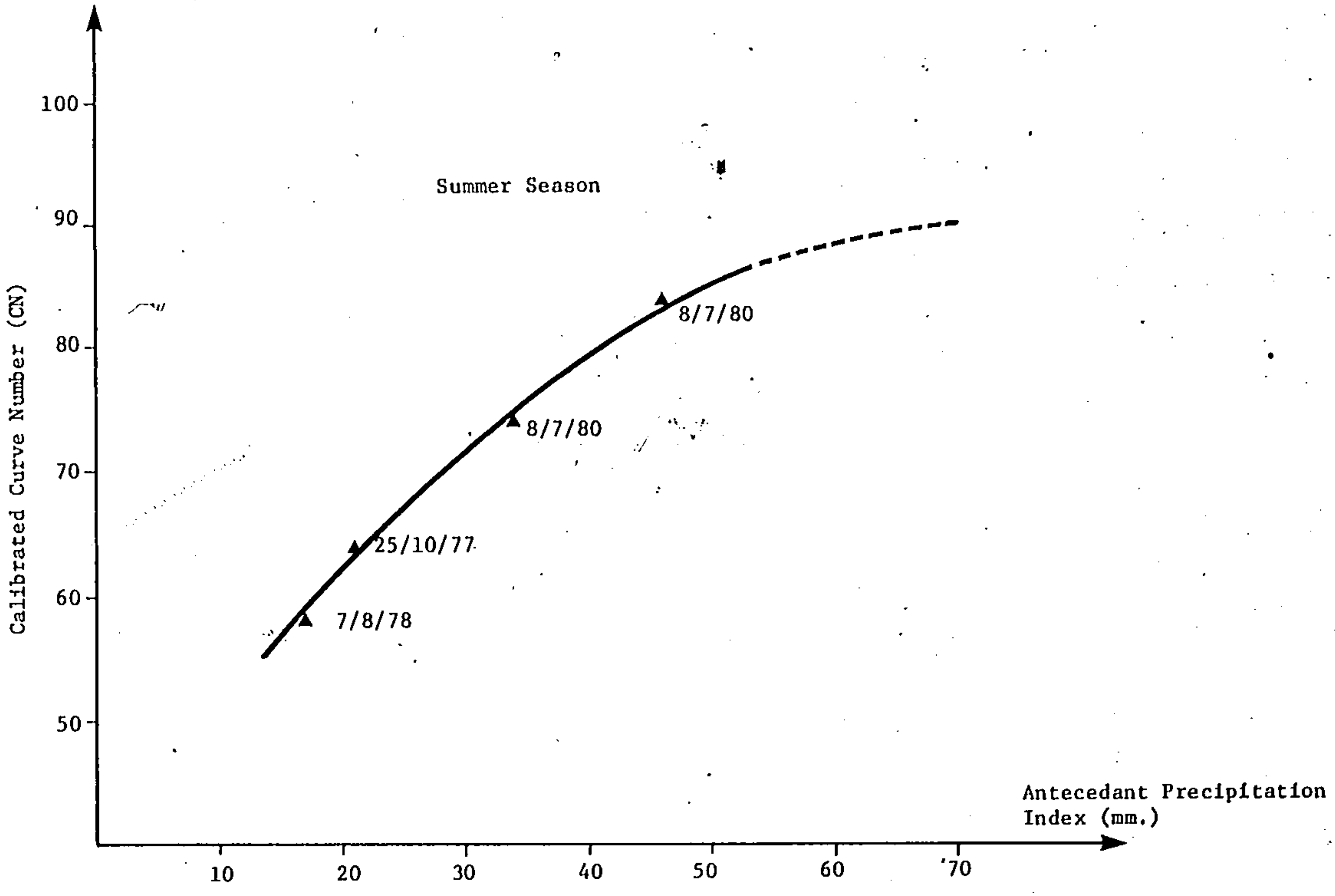


Figure C.5: Relationship of Curve Number and Antecedant Precipitation Index this is the value when runoff occurs as determined by EPFL in the 1979 Report) are examined. The maximum API, prior to a storm during the summer season (seasonal maximum) of each year is selected and statistically analyzed. The probability, or return frequency, of API calculated is plotted in Figure C.6. It can be observed that the worst API expected to return every year is approximately in the range of 20 mm. For a probability of 50%, the worst API is approximately 43 mm, which is expected to return every two years.

The return frequency of API defined in two other manners are also explored. It is possible to define the exceedence curve in terms of the mean API during the summer season as a whole (seasonal mean) as well as in terms of the maximum monthly mean API during each summer season (monthly mean). The two probability curves obtained are also shown in Figure C.6. It can be seen that the exceedence probability curve defined for the maximum seasonal API is most conservative while that defined for the seasonal mean during the summer season is least conservative.

Since CN is related directly with API, the return frequency of CN corresponding to the various levels of API can be defined.

The above study demonstrates a methodology of selecting the API and CN to overcome some of the difficulties of defining the pre-development flow. However, a methodology for

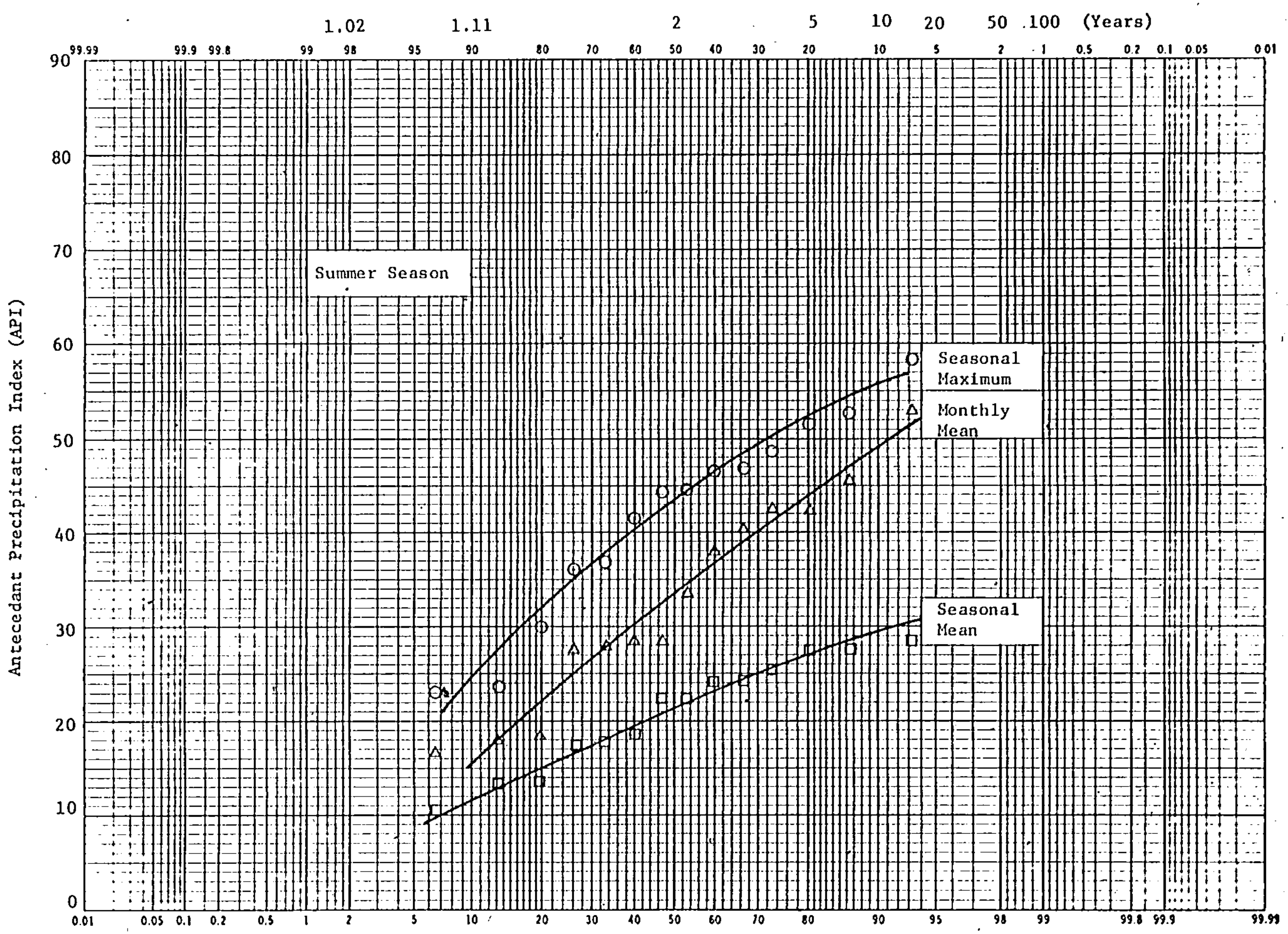


Figure C.6: Return Frequency of Antecedant Precipitation Index (API) deriving the true return frequency of the flow simulated by a design storm is still to be investigated. This preliminary study constitutes the basis on which further research is now being carried out by Mr. D. Jobin, a graduate student at the Department of Civil Engineering, University of Ottawa..

Appendix D

STEPS OF APPLYING SHM

D.1 MARKHAM, ONTARIO

D.1.1 Determination of Peak Runoff Rates

The steps for determining the peak runoff rates by means of the SHM peak discharge equation are shown as follows:

1. Determine the total rainfall precipitation PRECIP - as shown in Figure D.1 for the 1 in 5 year design storm distributed with Keifer and Chu's method, the total depth of rainfall PRECIP is 1.85 inches (47 mm.).
2. Determine the total runoff volume Q - the runoff factor R is first determined as shown in Figure D.2; for a percentage imperviousness of 30% and a 1 in 5 year design storm, the runoff factor R is 0.36; the total runoff volume in depth is

   $$Q = 1.85 \text{ in.} \times 0.36 = 0.66 \text{ in.} \ (16.8 \text{ mm.})$$

3. Determine the peak factor P (Figure D.3) - using Figures 5.8 and 5.9 given in Chapter 5, the peak factors P for the imperviousness of 30% are interpolated; P for sub-watershed sizes of 18 ac. (7.3 ha.), 48 ac. (19.4 ha.), and 144 ac. (58.3 ha.) are 2.5, 2.2, and 2.0, respectively;

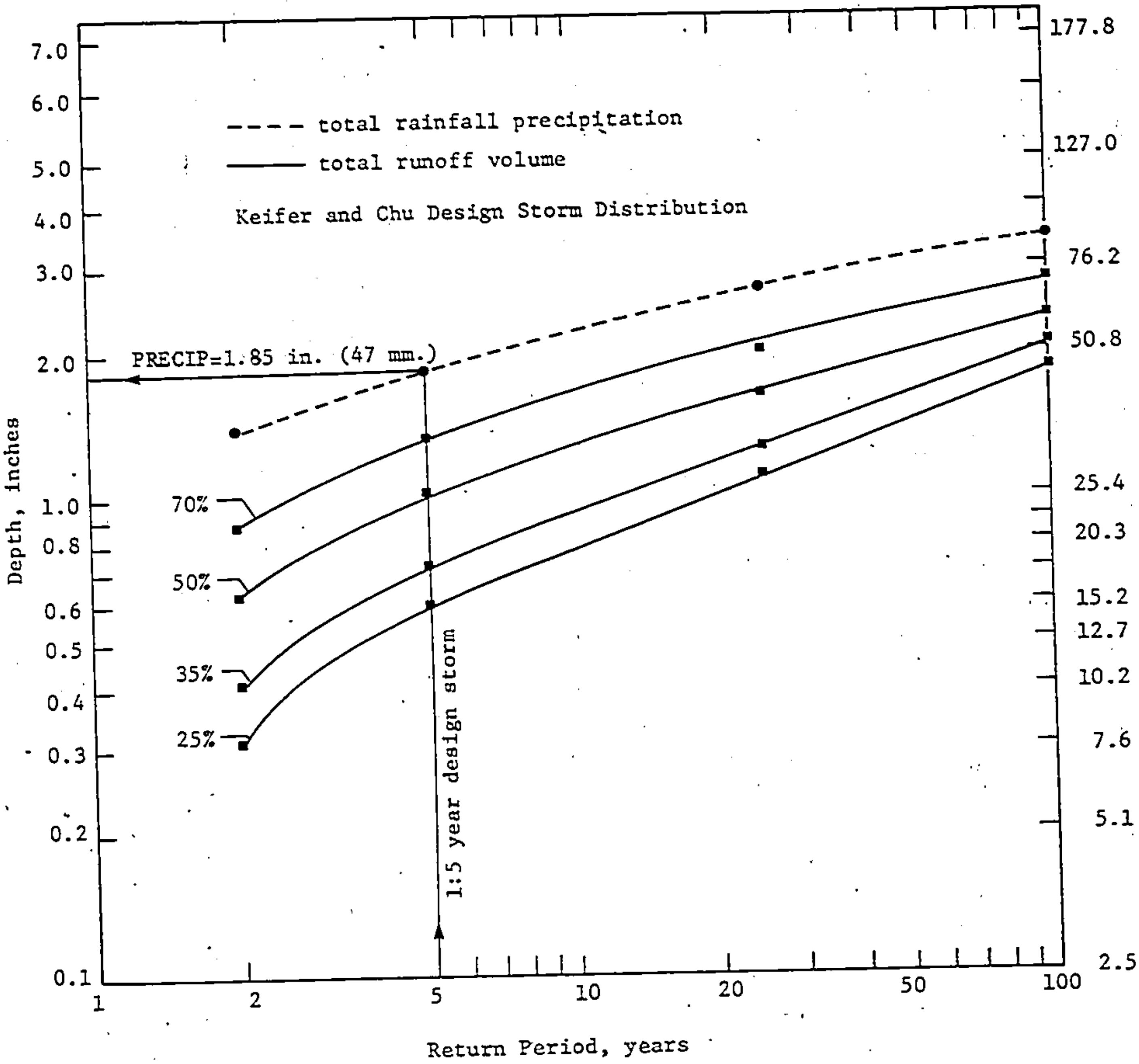


Figure D.1: Determination of PRECIP (Markham, Ontario)

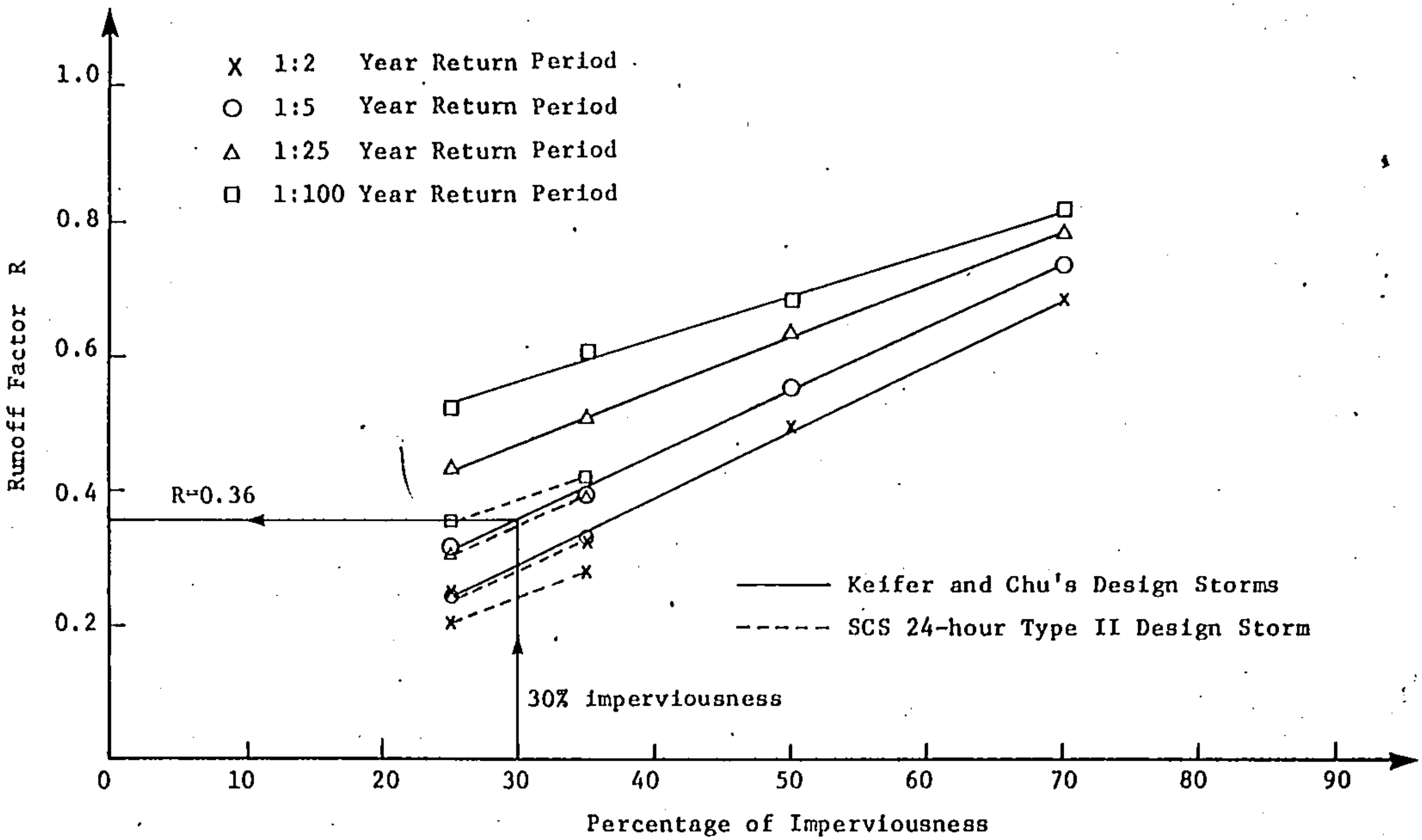


Figure D.2: Determination of R (Markham, Ontario)

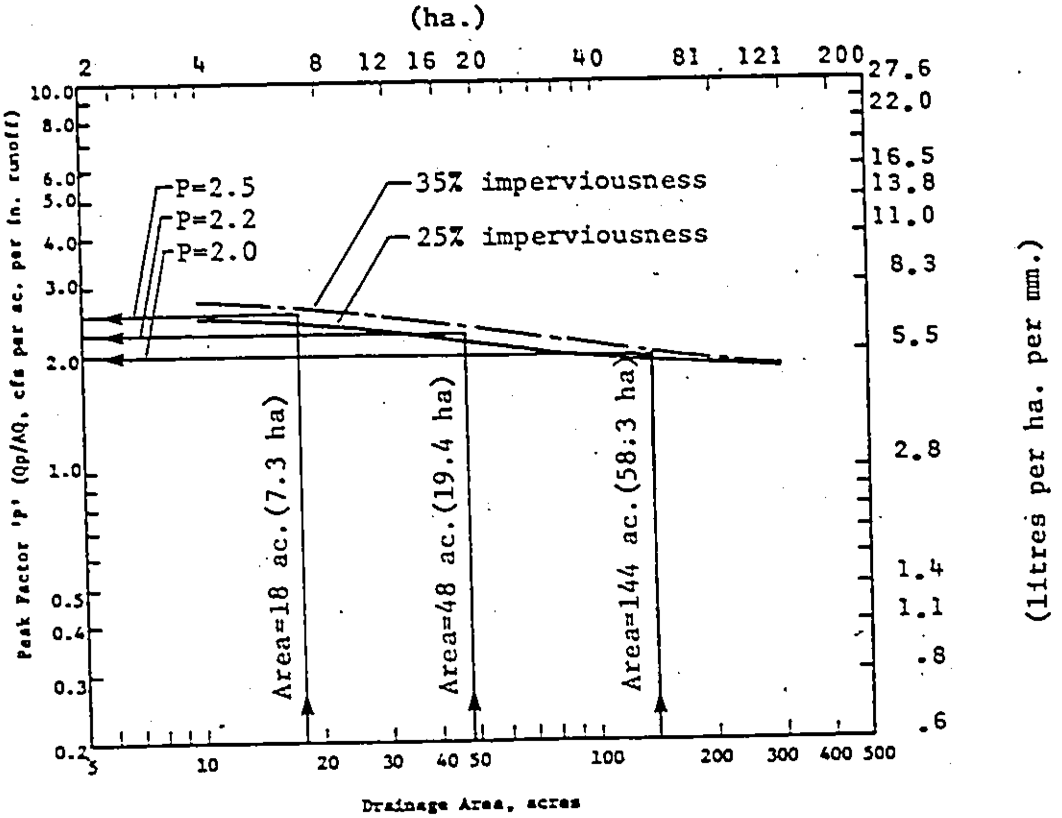


Figure D.3: Determination of P (Markham, Ontario)

P for other sub-watershed sizes are determined in a similar manner.

4. Calculate the peak runoff rates - the peak runoff rate for each sub-watershed is calculated by multiplying P with A and Q:

Sub-Watershed Area A		Peak Factor P	Peak Runoff Rate	
(ac.)	(ha.)	(cfs per ac. per in. runoff)	(cfs)	(cms)
18	7.3	2.5	30	0.9
48	19.4	2.2	70	2.0
144	58.3	2.0	190	5.4

D.1.2 Determination of Runoff Hydrograph

The hydrograph at the outlet of the development is calculated by multiplying the ordinates (Q/Q_p and t/t_p) of the SHM dimensionless hydrograph with the peak runoff rate which has been determined in the last section (190 cfs, or 5.4 cms) and the time to peak at the outlet. The time to peak for this watershed area of 144 ac. (58.3 ha.) and 30% imperviousness is determined based on Table 5.2 of Chapter 5.

A time to peak of 74 minutes is determined. The ordinates of the runoff hydrograph at the outlet are calculated as follows:

SHM Dimensionless Hydrograph		Runoff Hydrograph at Outlet	
t/t_p	Q/Q_p	Time (min.) t/t_p x 74	Flow Rates (cms) Q/Q_p x 5.4
.30	.01	22.1	.05
.45	.02	33.1	.11
.50	.035	36.8	.19
.60	.05	44.1	.27
.70	.07	51.5	.38
.80	.12	58.8	.65
.90	.40	66.2	2.20
.95	.80	69.8	4.32
1.00	1.00	74.0	5.40
1.05	.84	77.2	4.54
1.10	.60	80.9	3.24
1.20	.35	88.2	1.89
1.30	.25	95.6	1.35
1.40	.18	102.9	.97
1.50	.13	110.3	.70
1.60	.10	117.6	.54

D.2 PINETREE, ONTARIO

D.2.1 Determination of Peak Runoff Rates

1. Determine the total rainfall precipitation PRECIP - as shown in the last section, the total rainfall precipitation of the 1 in 5 year Keifer and Chu design storm distribution is 1.85 inches (47 mm.).
2. Determine the total runoff volume Q - average imperviousness ratios are first determined for each sub-watershed of the Pinetree development; the runoff factors R are then obtained as shown in Figure D.4; the runoff volumes Q in depths are calculated by multiplying PRECIP with R.

Area of Sub-Watershed		Average Percentage Imperviousness	Runoff Factor R	Runoff Volume Q	
(ac.)	(ha.)			(in.)	(mm.)
46	18.6	54%	.58	1.07	27.3
83	33.6	43%	.48	.89	22.6
175	70.9	38%	.42	.78	19.7

3. Determine the peak factor P - using Figures 5.8, 5.10, and 5.11 in Chapter 5, which give the peak factors for imperviousness of 25%, 50%, and 70% respectively, the peak

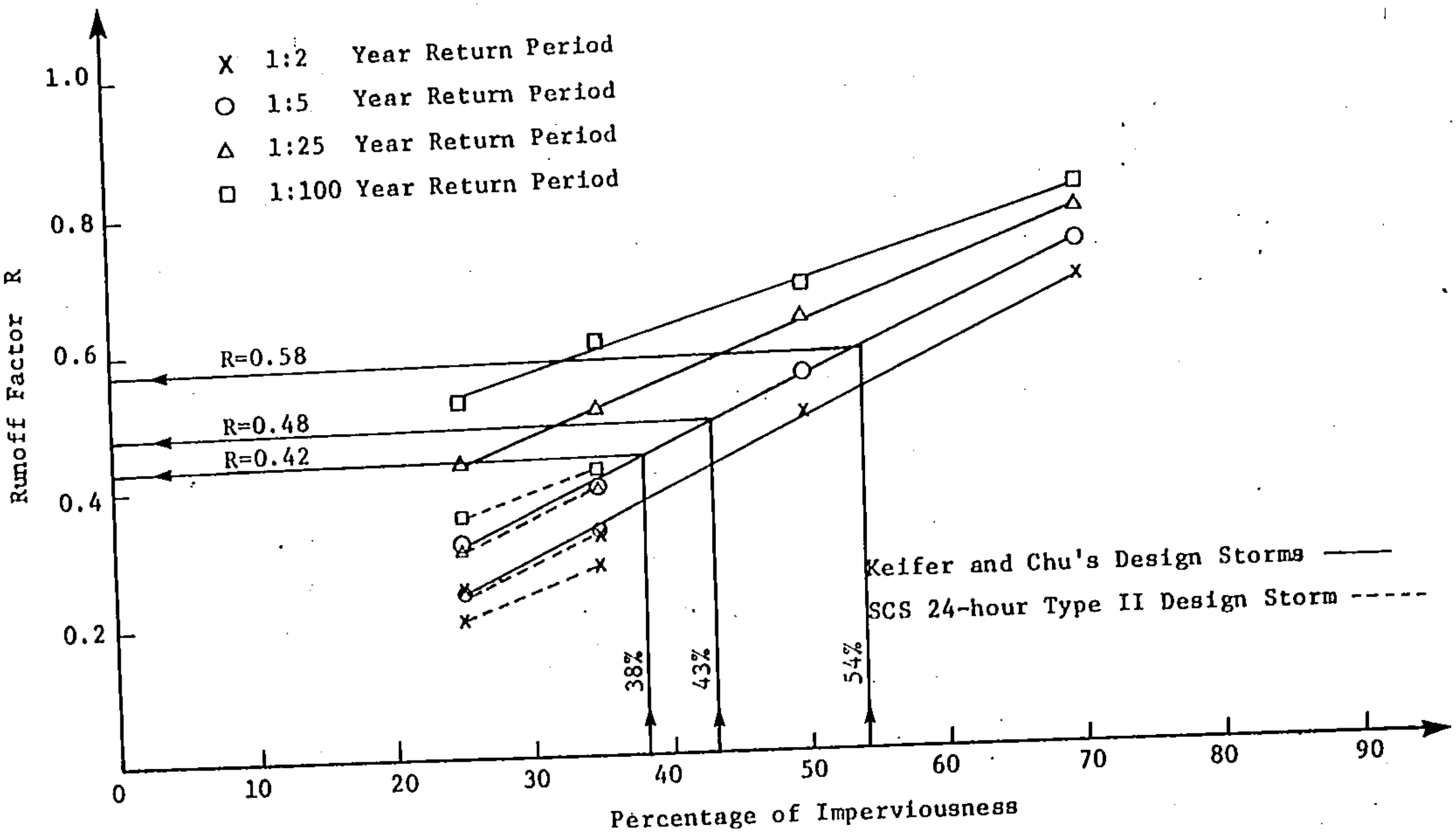


Figure D.4: Determination of R (Pinetree, Ontario) factors for the different sub-watersheds are determined by interpolation (Figure D.5).

4. Determine the peak runoff rates - the peak runoff rate for each sub-watershed is calculated by multiplying P with A and Q:

Sub-Watershed Area A		Peak Factor P	Peak Runoff Rate	
(ac.)	(ha.)	(cfs per ac. per in. runoff)	(cfs)	(cms)
46	18.6	2.15	106	3.0
83	33.6	2.0	148	4.2
175	70.9	1.9	260	7.4

D.2.2 Determination of Runoff Hydrograph

Since the peak runoff rate from each sub-watershed is now known, by determining the time to peak of each sub-watershed with Table 5.2 of Chapter 5 the runoff hydrograph from each sub-watershed can be easily determined by following the same procedures demonstrated in Section D.1.2. In this example, only the runoff hydrograph at the outlet (175 ac., 70.9 ha.) is determined. The time to peak determined with Table 5.2 is 75 minutes.

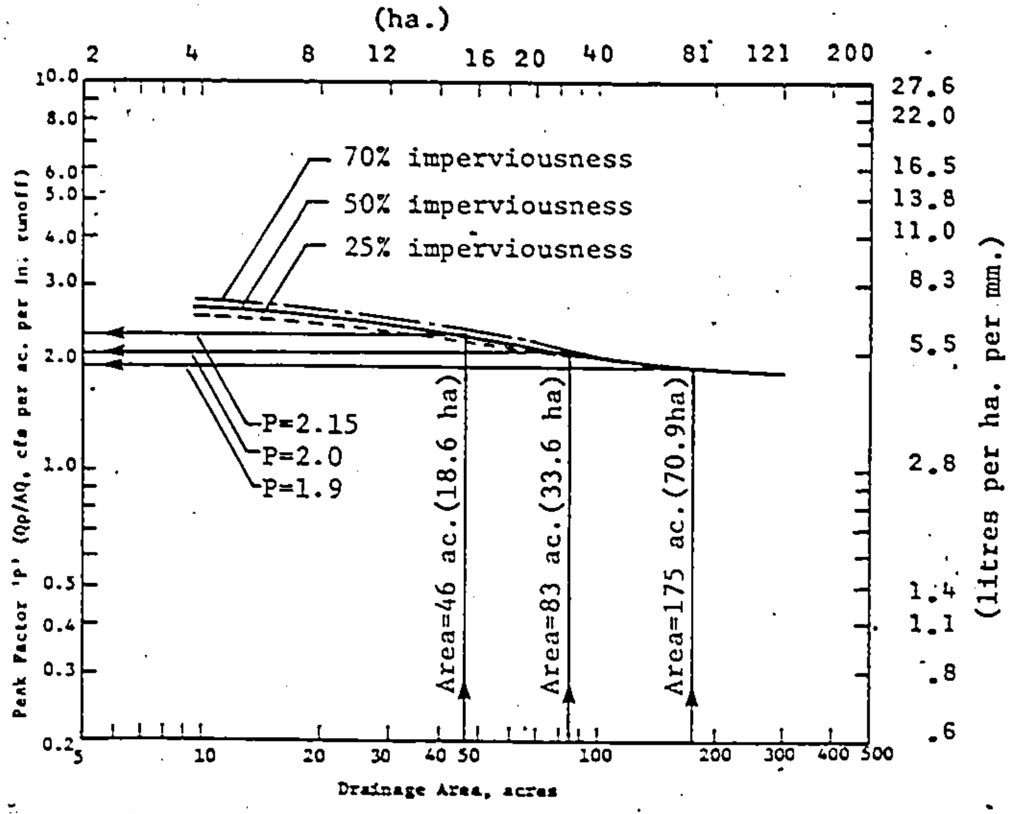


Figure D.5: Determination of P (Pinetree, Ontario)