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A Contribution to the Design Flood Guide for Canada

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Urban Design Floods

by

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ABSTRACT

Approaches to design of flood protection in urban areas are examined. Such examination starts with a description of characteristics of urban floods and basic concepts of urban flood control and stormwater management. Further discussion deals with input data required for the planning and design of flood control measures. Such data include process parameters and physiographic and hydrometeorological data. The hydrological design of flood protection is based on special computational methods which were developed or adapted for urban catchments. The methods discussed include urban flood frequency analysis, empirical methods, synthetic hydrograph methods and simulation models. Finally, a brief overview of design procedures currently used in the Canadian practice is given.

SOMMAIRE

On examine différentes approches pour la construction d'ouvrages de lutte contre l'inondation dans les zones urbaines. On décrit d'abord les caractéristiques des inondations en zone urbaine et les notions fondamentales de lutte contre l'inondation et de gestion des eaux pluviales. La discussion porte ensuite sur les données d'entrée qu'il faut recueillir pour planifier et construire les ouvrages de lutte contre l'inondation. Ces données comprennent entre autres des paramètres hydrologiques ainsi que des données physiographiques et hydrométéorologiques. La conception hydrologique des ouvrages de lutte contre l'inondation se fonde sur des méthodes de calcul qui ont été élaborées ou adaptées spécialement pour les bassins versants englobant une zone urbaine. Les méthodes présentées dans la discussion comprennent l'analyse de fréquence de crues, les méthodes empiriques, l'hydrogramme unitaire synthétique et les modèles de simulation. L'étude se termine par un survol des procédures de lutte contre l'inondation qui sont couramment utilisées au Canada.

MANAGEMENT PERSPECTIVE

In Canada, annual expenditures on structures requiring design flood calculations are about 2 billion dollars while the annual flood damages, due to inappropriate developments and hydrologic design, even exceed this total. Recognizing such large expenditures, the Associate Committee on Hydrology has initiated the preparation of the Design Flood Guide for Canada. The report that follows represents one of the chapters of the Guide dealing with urban design floods.

Urban areas and floods possess special characteristics which led to the development of special procedures for flood design in such areas. In particular, urban floods are flashy with fast-rising peaky hydrographs. They generally cause large damages because of high density and property values of urban developments subject to flood damages. Consequently, special hydrologic methods have been developed for urban flood design. Such methods use urban rainfall inputs, abstractions characteristic for urban areas, short computational time steps, and detailed descriptions of catchments and transport systems. At the same time, urban areas offer an opportunity for flood control and management by controls at the source and various forms of storage.

This report offers guidelines for urban flood design starting with descriptions of basic approaches and concepts, followed by input data requirements, computational methods and an overview of the current Canadian design practice.

The report should be of interest to water managers, planners and designers dealing with flood protection design in urban areas.

PERSPECTIVE GESTION

Au Canada, on consacre chaque année environ 2 milliards de dollars à la construction d'ouvrages nécessitant des calculs de crue nominale. Mais, les dégâts causés par les inondations en raison d'une mauvaise utilisation des terres et des erreurs de conception sur le plan hydrologique dépassent ce montant. À la lumière de ces coûts élevés, le Comité associé d'hydrologie a mis en chantier le Design Flood Guide for Canada. Le rapport qui suit constitue l'un des chapitres du guide traitant du problème des crues nominales en zones urbaines.

Les zones urbaines et les inondations auxquelles elles sont sujettes possèdent des caractéristiques particulières qui ont motivé l'élaboration de procédures spéciales pour la construction des ouvrages de lutte contre l'inondation. En particulier, les inondations en zones urbaines se caractérisent par un hydrogramme dont la courbe monte brusquement et retombe rapidement. Le montant des dégâts causés par les inondations est en général dû à la forte densité des habitations et à la valeur élevée des propriétés dans les zones urbaines menacées. Par conséquent, des méthodes hydrologiques spéciales ont été mises au point pour la construction des ouvrages de lutte contre l'inondation en zones urbaines. Ces méthodes se fondent sur les données de précipitations en zones urbaines, la portion des eaux pluviales qui ne fait pas partie des eaux de ruissellement, des données chronologiques à intervalles rapprochés ainsi que des descriptions détaillées des bassins versants et des réseaux de drainage. Dans les zones urbaines, il est également possible de lutter contre l'inondation en prévoyant des ouvrages de régularisation ou de dérivation à la source et différentes formes de réservoirs de crue.

Dans le présent rapport, on établit des lignes directrices régissant la conception des ouvrages de lutte contre l'inondation à partir d'une description des approches et des notions fondamentales, suivie des exigences en matière de données d'entrée et de méthodes de calcul et se terminant par un survol des pratiques actuelles au Canada dans le domaine de la conception de ces ouvrages.

Cette étude suscitera vraisemblablement l'intérêt des gestionnaires, des planificateurs et des concepteurs qui ont à se pencher sur les problèmes de conception des ouvrages de lutte contre l'inondation dans les zones urbaines.

TABLE OF CONTENTS

	Page
ABSTRACT	i
MANAGEMENT PERSPECTIVE	ii
LIST OF TABLES	v
LIST OF FIGURES	vi
7.1 Characteristics of Urban Floods	1
7.2 Basic Concepts in Urban Flood Control and Stormwater Management	3
7.2.1 Fundamental Principles	3
7.2.2 Institutional Aspects	5
7.2.3 Urban Drainage and Flood Control Planning Process	8
7.3 Input Data for Urban Flood Design and Analysis	10
7.3.1 Catchment Physical Parameters	10
7.3.2 Process Parameters	11
7.3.3 Hydrometeorological Data	14
7.4 Computational Methods	22
7.4.1 Urban Flood Frequency Analysis	22
7.4.2 Empirical Methods	23
7.4.3 Synthetic Hydrograph Methods	30
7.4.4 Simulation Models	32
7.4.5 A Note on Model Selection	38
7.5 Applications and Current Practice	38
7.5.1 Overview	39
7.6 References	42

LIST OF TABLES

		Page
1.	Methodology for Master Drainage Plan Preparation	47
2.	Derived Flood Frequency Parameters for Equation 1 (Espey and Winslow, 1974)	48
3.	Summary of Time of Concentration(t_c) Estimation Methods (Kibler, 1982)	49
4.	Rational Method Runoff Coefficients(ASCE, 1969)	50
5.	Rational Method Runoff Coefficients for Composite Analysis(ASCE, 1969)	50
6.	Typical Depresssion Storage Depths for Various Land Covers(Kibler, 1982)	51
7.	Factors Used in Claculations of Horton's Standard Infiltration Curve(Terstriep and Stall, 1974)	51
8.	Espey 10-Minute Unit Hydrograph Equations(Kibler, 1982)	52
9.	Model Applicability to Various Urban Drainage Problems (Torno, 1982)	53

LIST OF FIGURES

		Page
1.	Urbanization Impacts on Basin Response Without Increased Detention Storage(After Kibler, 1982)	55
2.	AES Temporal Storm Distributions for Durations of 1 and 12 Hours	56
3.	SCS Twenty-Four Hour Rainfall Distributions	57
4.	Watershed Conveyance Factor, O, as a Function of Percent Impervious Cover and Weighted Mean Channel "n" Value (Whipple et al., 1983)	58
5.	Nomograph for the Time of Concentration(Scheaffer et al., 1982)	58
6.	Definition of Espey 10-Minute Unit Hydrograph Parameters	59
7.	Planning Level Basin Modelling Conceptualization (U.S. Army Corps of Engineers, 1976)	59
8.	Simplified Flowchart of STORM(U.S. Army Corps of Engineers, 1976)	60
9.	Flowchart of ILLUDAS(Terstriep and Stall, 1974)	60
10.	Relationship of Executive Block With Other SWMM Blocks (Huber et al., 1982)	61
11.	Overview of SWMM Structure Indicating Linkages Among the Five Computational Blocks(Huber et al., 1982)	61

7 URBAN DESIGN FLOODS

7.1 Characteristics of Urban Floods

Urbanization is known to alter the land phase of the hydrologic cycle by increasing the volume and speed of surface runoff in the affected areas. Such increases then result in a greater incidence of flooding in the urban area, receiving streams, and downstream segments of the watershed. Because of special characteristics of urban floods and catchments and the resulting need for special computational procedures and flood abatement measures, it is desirable to discuss urban floods separately from those occurring in natural catchments. Differences between both types of floods are demonstrated in Fig. 1 showing flood hydrographs before and after catchment urbanization.

Field observations indicate that urban floods differ from those in natural catchments by their hydrograph shape, the peak magnitude relative to the contributing area, and the occurrence during the year. The high imperviousness of urban areas and hydraulic efficiency of urban runoff transport elements contribute to the flashy nature of urban floods and the concomitant narrow, peaky hydrographs. Such floods usually result from high intensity thunderstorms occurring during the summer months. Urban floods often cause large damages because of high density of urban population and high values of urban properties subject to flood damage.

Urban flood design is done for a future state of the catchment which differs dramatically from the pre-development

state. During the development process, a significant portion of the catchment becomes impervious and natural drainage channels are replaced by hydraulically efficient channels and conduits. Thus, frequency analysis of actual flow records for the natural state of the catchment would yield results which are irrelevant to the post-development conditions and other approaches need to be adopted. Among these, hydrologic synthesis is the most common.

Hydrologic synthesis calculations are generally accomplished by means of urban runoff models which account for special features of urban catchments. The features typical for such models and their applications include the use of urban rainfall inputs, the rainfall abstraction characteristics of urban areas, short computational time steps, detailed catchment descriptions, and flow routing in two interconnected transport systems - the minor and major drainage systems.

Another special feature of urban flood analysis is the consideration of urban flood control measures. These measures attempt to modify the response of developing urban areas by reversing the impact of urbanization on runoff - reducing runoff volumes through enhanced infiltration and reducing the rate of runoff by detention in impoundments and minor or major transport systems.

Thus, it appears appropriate to treat urban floods separately because of their special nature, specific features of urban catchments, and the use of special design techniques and flood control measures. This chapter deals with fundamental concepts

of urban flood control and stormwater management, input data for urban flood design analysis, computational methods, and current Canadian practices.

7.2 Basic Concepts in Urban Flood Control and Stormwater Management

7.2.1 Fundamental Principles The urban drainage system is part of the larger urban environmental system. Ideally, stormwater management and urban flood control should take into consideration all of the important interrelationships between the drainage system and its related subsystems, including both flow quantity and quality.

Prior to urbanization a large fraction of the total rainfall contributes to groundwater recharge through infiltration and percolation. In fact, surface runoff typically represents less than 25 to 30% of the gross rainfall on a natural watershed (Chow, 1964). Following urbanization both the volume and rate of surface runoff increase sharply with a corresponding decrease in groundwater contribution. The groundwater table typically recedes after urbanization, while the quality of the receiving waters generally deteriorates through surface pollutant washoff. However, groundwater and water quality problems are frequently neglected in urban flood control where the primary objective is to minimize flooding problems.

For many years, common practice was to provide for rapid evacuation of all surface runoff to drainage (pipe) collection systems. The result of such practice was to worsen flooding conditions in downstream areas, with massive and costly construc-

tion of conveyance systems. In recent years urban drainage design, has focussed on methods of reducing the rate and volume of runoff by careful examination of the mechanisms of surface runoff generation in urban areas. Non-structural source control measures, ranging from enhanced infiltration to rooftop and parking lot storage have been examined along with more direct structural measures such as the construction of stormwater reservoirs and urban lakes. However, even with source control measures, the rate and volume of runoff from urbanized watershed are typically much greater than those which existed prior to urbanization. In fact, designers soon realized that the collection (pipe) system could only handle a fraction of the total surface runoff during high intensity rainfall events and this realization led to the development of the dual drainage concept. This concept assumes that the minor (pipe) drainage system will be used to convey flows associated with fairly common rainfall-runoff events. On the other hand, the major drainage system, consisting of roadways and surface channels, will be used to route flows generated by the less frequent events. While both drainage systems or routes followed by flood waves of different magnitude always exist, it is only recently that engineers have started to incorporate some consideration of the major drainage system in their design. Basically, the tools of analysis described later in this chapter are applicable to both systems. In fact, some of the more recent computational models now have provisions to handle both the minor and major drainage systems simultaneously.

Although drainage systems have been traditionally designed as networks of conduits with open-channel flow, practical experience shows that most drainage systems surcharge when higher than design flows, resulting from rare storms, are encountered. Sewer surcharging is not necessarily harmful as long as the pressure gradient does not exceed critical elevations above which flood damages occur. This fact can be used to achieve significant cost savings by designing some parts of the sewer system so that they surcharge during some storms or periods of runoff. Such cost savings result from the fact that surcharged sewers generally convey higher discharges than the same sewers with open-channel flow. Sewer surcharging, however, has to be properly considered in the design. These considerations include the use of appropriate computational tools for routing of pressurized flow in sewers and implementation of inlet controls or other measures for limiting the degree of surcharge. Furthermore, the pressure gradient has to be calculated throughout the network and checked or adjusted so that no excessive damage (e.g., through basement flooding) occurs in the area under design.

7.2.2 <u>Institutional Aspects</u> Urban floods are directly related to urban drainage and, consequently, their analysis has to be considered within the institutional arrangements for planning, design, approval, and operation of drainage systems. From the legal point of view, federal and provincial laws, and municipal bylaws have to be considered. Besides the government agencies, other action groups involved may include developers, designers, and property owners.

Drainage and the resulting floods are covered by the common law or civil code which form the basis for drainage laws, unless federal or provincial statute laws take precedence. The common law regarding drainage consists of two parts - the rules governing the riparian rights and obligations of the landowners whose lands are immediately adjacent to natural water courses, and the rules governing the landowner rights relating to surface or percolating waters. Any landowner whose land abuts a natural water course has the right to drain his land to this water course. Such a right is lost, however, if the rainwater which would find its way by percolation or surface runoff to a natural water course is collected in man-made channels, as it is always done in urban areas. In that case, the landowner is responsible to avoid discharging his collected water on the lands of another, and he must, at his expense, take the water to a sufficient capacity outlet.

The authority to establish land drainage laws rests with the provinces, except for drainage works extending beyond the boundaries of one province or work declared for the "general advantage of Canada" which come under federal law.

Provincial legislation related to urban drainage varies from province to province. Urban drainage is governed by provincial legislation which deals with environmental assessment protection, water resources protection, environmental assessment of proposed projects and activities, flood control, municipal planning, authorization of municipalities to enact bylaws relating to urban drainage and water control, and local improvements including

major drainage works.

Technical detailed aspects of drainage and flood control are often established in municipal drainage criteria which are mandatory within the municipality jurisdiction. Many municipalities have established drainage criteria of various scope and detail. The aspects covered by such criteria include storm drainage policies; specification of design rainfall data; specification of runoff calculations; flood plain analysis; controls of volume, rate, and quality of urban runoff; and, detailed storm sewer design criteria. Implementation of such criteria and policies provides for orderly urban growth and protection of urban areas and receiving waters against flooding. Where municipal boundaries cross a watershed, cooperation and common acceptance of the watershed-scale planning are necessary.

Implementation of stormwater control and management to abate urban floods is becoming fairly common practice in Canada. Experience shows that although the average initial capital costs for drainage schemes with local (on-site) runoff controls in the area under development are slightly higher than for conventional drainage, the need for any downstream control structures is greatly reduced. The resulting savings may reach millions of dollars. The success of on-site controls depends on the cooperation of all parties involved and on early consideration of such controls during the planning process. Finally, public awareness and participation in urban drainage and flood control projects is desirable to ensure a complete consideration of all alternatives and to obtain support for implementation.

7.2.3 Urban Drainage and Flood Control Planning Process The past lack of coordination between urban drainage and land use planning resulted in costly flood abatement programs which many municipalities had to undertake. To eliminate such problems in the future, there is a trend in Canadian practice towards comprehensive planning of urban drainage and flood control. A general outline of the applicable planning strategy is shown in Table 1. The planning considerations described below are limited to urban floods, although other objectives are often also included.

For any flood control planning to be effective, it must be done on the watershed basis. Consequently, as a part of watershed plans, master drainage plans are prepared for the entire watershed at an early stage of watershed development and all future work in various parts of the watershed must comply with the master plan which is regularly updated. The master drainage plan incorporates the whole drainage system including the interrelationship of major and minor drainage systems.

The preparation of the master drainage plan starts with the identification of problems and definition of objectives which usually include the abatement of local flooding inconvenience (through the implementation of minor drainage), and reduction in local and downstream flood damages or threat to human life. Such objectives need to be accomplished under a given set of constraints which include natural constraints, policy and regulations constraints, and cost constraints.

The next step consists in defining the drainage system

components including the inputs, elements, and outputs. Examples of inputs are design rainfall data and unit costs. System elements are established for the proposed development and for various non-structural and structural measures employed in urban drainage design. Non-structural measures include land use policies, prohibition of flood plain occupancy, or the floodway-flood fringe concepts for flood plains. Structural measures include various drainage conduit and channel configurations, storage structures, diversion structures, channelization, dikes, and flood proofing. Finally, the system outputs are produced in the form of flood hydrographs and costs for various alternatives. These outputs are produced by means of various computational procedures which are described later in this chapter.

All drainage alternatives are screened, compared, and the best alternative is selected on the basis of decision and evaluation criteria. The selected alternative is then implemented. Experience with the dynamics of urban development and changes in drainage technology indicates that the master drainage plans should be regularly updated and modified, about every 5 years.

The preparation of the master drainage plans requires an appreciable volume of information depending on the requirements of the municipality and the approval agency. Below is listed an example of such requirements as given in the Ontario Drainage Guidelines (Ministry of the Environment, 1983).

For the preparation of the master drainage plan, the following type of information may be required.

Site plans of the watershed, development, topography, water courses, present and proposed land use patterns, the proposed major drainage system, regional storm flood lines (if applicable), and points of various water resources problems. The information prepared in a tabular form includes subcatchment characteristics for the pre-development and post-development stages, details on water course crossings, details of water course and valley reaches, simulated flows at key points for the pre-development and post-development conditions, calculated flood elevations at all sections, benefit-cost matrices of alternatives, sizes of flow control facilities, and volumetric runoff coefficients. Finally, the information presented pictorially includes pre-development and post-development peak flows, uncontrolled and proposed controlled flow peaks, flood control works proposed, plots of peak flows versus area for a range of storms, and profiles of flood levels in the major drainage system.

7.3 Input Data for Urban Flood Design and Analysis

Computations of urban floods require various types of input data depending on the procedure used. For deterministic procedures which are predominant in urban hydrology, input data can be classified in three categories - catchment physical characteristics, process parameters, and hydro-meteorological data. General discussions of individual categories follow. For details of input data scope and formats, the users manuals which are referred to in the next section should be consulted.

7.3.1 Catchment physical parameters Physical characteristics of

the catchment are required to establish drainage patterns in the catchment, linkages of various conveyance elements, and numerical values of process parameters. Because the computations often involve comparisons of pre-development and post-development flows, physical characteristics are needed for both states of the catchment development. The types of information required include catchment topography, drainage plans, and soil maps, as further discussed below.

Site maps are required showing the entire watershed and the area under consideration; details of topography including contours, watercourses, wooded areas, rock outcrops, and marshes; details of existing and proposed land use; and the existing and proposed major drainage channels. Using such maps, it is possible to establish the study area drainage boundaries, general drainage patterns in the area, surface slopes, and the total catchment area. The total catchment area is further subdivided into impervious and pervious parts. Of particular interest are the effective impervious areas which drain directly into transport elements and the contributing pervious area. All these areas need to be delineated and characterized in terms of the area, plan geometry, and surface slopes. For various reasons, the catchment studied is usually subdivided into a number of subcatchments and then the information needs to be determined for each of these subcatchments.

7.3.2 <u>Process parameters</u> In deterministic hydrological calculations, it is assumed that the relationships between many interactive factors affecting the water balance can be defined analyti-

cally. The numerical values used to quantify the movement and storage of water are called parameters. Although such parameters can be quite numerous, only a few of these are used in typical urban flood computations. Such parameters include infiltration rates, depression storage, flow roughness coefficients, and runoff coefficients. Numerical values of these parameters are obtained by field measurements, calibration, or most frequently, by transposition from other similar catchments. General descriptions of hydrologic process parameters have been given in Chapter 6. The discussion that follows concentrates on urban applications.

Infiltration Rates A proper presentation of infiltration in a catchment is a complex task which often extends beyond the scope of hydrologic concepts and methods employed in urban drainage design. Most often, infiltration rates are evaluated from soil physical properties which are obtained from soil maps. For this purpose, soils are classified according to their drainage properties and the corresponding infiltration rates are selected from the literature. In urban applications, the U.S. Soil Conservation Service (SCS) classification of soils into four hydrologic groups A-D is the most common (U.S. Department of Agriculture, 1972). The knowledge of the hydrologic soil group and initial soil moisture conditions is sufficient to estimate the infiltration rates based on the SCS method and to select th appropriate parameter values for Horton's equation, or to select a runoff coefficient for pervious areas (see next section).

In physically-based approaches, such as those described by

Holtan and Green-Ampt equations, more information on soils is required. In particular, the division of the soil profile into various horizons needs to be known, together with the soil porosity and various types of water storage (see Chapter 6).

Simplified approaches to infiltration may be acceptable for urban catchments in which the generation of runoff is mainly controlled by impervious elements. In other cases, the most comprehensive approach which can be supported by the available data should be used. Such approaches are generally physically-based and involve continuous simulation of water storage in soils.

Depression storage Depression storage accounts for rainwater trapped on the catchment surface in minute depressions, that does not run off or infiltrate into the soil. Generally, it represents a combination of several hydrologic abstractions. For urban floods, the depression storage is of secondary importance. Typical values are 1.6 mm for impervious areas and 5 mm for pervious areas (Kibler, 1982). Other values for specific surfaces are presented later in this Chapter (see Table 6).

Roughness of transport elements In flow routing calculations, the roughness of individual elements needs to be determined. In urban applications, this is estimated most often by means of the Manning roughness coefficient, n. For concrete conduits, concrete-lined channels, and impervious overland flow planes, the value of n=0.013 is widely used.

Manning's n values for overland flow on grassed areas vary from 0.2 to 0.35. Extensive listings of n-values for various convey-

ance elements were presented by Linsley et al (1982) and Huber et al (1982).

7.3.3 Hydrometeorological Data Various hydrometeorological data needed in hydrologic synthesis were discussed in Chapter 5. The purpose of this section is to describe their application in urban flood analysis. The discussion starts with rainfall data, followed by soil moisture, streamflows, and the data required in snowmelt computations.

Rainfall data Rainfall data are used in urban flood calculations in a variety of forms. The type of rainfall data used is governed by the computational procedure which in turn is given by the type of problem to be solved and the level of analysis. The following forms of rainfall data are used in urban flood calculations: Intensity-Duration-Frequency (IDF) curves, synthetic design storms, historical design storms, and actual or synthetic long-term rainfall records.

The IDF curves which are described in Chapter 5 are used in urban flood analysis as rainfall inputs for empirical peak flow formulae and they also serve to develop synthetic design storms.

Empirical formulae for runoff peak calculation are described in the next section. Such formulae assume that, for the runoff equilibrium conditions, the peak flow can be expressed as a function of the catchment area, runoff coefficient, and a constant rainfall intensity. The duration of rainfall must be sufficient to reach the equilibrium state and equals the time of concentration of the catchment, as defined in the next section.

Thus for a known time of concentration and the selected return period, the designer can determine the corresponding rainfall intensity from the IDF curve. Applications of runoff peak formulae and constant intensity rainfall to urban flood analysis are rather rare., they are better suited for minor drainage design. The main inherent short-comings of such procedures is their empiricism and the fact that these methods yield only the peak flow and not the entire hydrograph.

Synthetic design forms are derived by generalization and synthesis of properties of a large number of actual storm and then used for calculation of urban flood hydrographs. In these applications, the specific meteorological input is used as a design flood criterion (see Chapter 3).

In spite of difficulties with the definition of design storms and the corresponding antecedent moisture conditions, the concept of design storms is very popular in Canadian urban drainage practice. Under such circumstances, efforts should be made that the 'best' design storms are used properly within the range of their applicability as discussed by Marsalek and Watt (1984). Design storms are best applicable to urban catchments of limited areas, significant imperviousness, and without large storage facilities. Under those circumstances, the use of point rainfall for the whole catchment is justified, runoff is controlled by impervious areas (thus antecedent conditions are less important), and the flow hydrographs are not modified by large storage facilities. As the catchment characteristics depart from those above, the design storm concept will yield less reliable

results.

Applications of design storms require the knowledge of storm characteristics and antecedent moisture conditions in the form required by the computational method used. The storm characteristics include the return period, duration, total rainfall depth, temporal distribution of rainfall, and spatial distribution of rainfall.

The design storm return period is assumed to be approximately equal to the calculated peak flow return period. The storm return period is generally specified in the drainage criteria and it varies from two years in minor drainage design to 100 years in major drainage design.

The storm duration is selected in relation to the catchment physical characteristics. It is taken as equal to or greater that the time of concentration which is defined the same way as in the rational method (see section 7.4). The total storm rainfall is determined for a given return period and duration from the IDF curves.

The temporal rainfall distribution is generally characterized by its peak intensity and peak timing as expressed in terms of the discretization time interval. Four types of temporal distributions are available for use in Canadian practice - the Atmospheric Environment Service distributions (Pugsley, 1981), the Hydrotek design storms (Hydrotek, 1985), Chicago-type storm (Keifer and Chu, 1957), and SCS 24-hour distributions (U.S. Department of Agriculture, 1975). Whenever possible, the first two distributions are preferred because they have been derived from

Canadian data for a large number of locations in Canada as explained in Chapter 5.

The atmospheric Environment Service (AES) distributions are available for 1 and 12 hour durations and various levels of probability. These distributions are plotted as the normalized cumulative rainfall depth versus the normalized lapsed time and applied to a selected rainfall depth to obtain a design hyetograph (see Fig. 2). In order to determine which percentile distribution should be recommended for practice, runoff peak frequency curves were derived from runoff simulations for actual and selected design storms. Simulations with a rather simple rainfall-runoff model indicated that the 30 percentile distribution performed best among all percentile distributions for all regions of Canada (Hogg, 1982)

Uncertainties in choosing a percentile distribution led to the development of a new urban design storm model by Hydrotek Inc. (Hydrotek, 1985). This two-component distribution model comprises linear rise and exponential decay components for early peaking storms and vice-versa for late peaking storms. For each site studied, the model was capable of simulating individual actual rainfall events as well as average or design events. One-hour urban design storms have been developed for five AES long-term data records across Canada using the proposed model. Such work was then extended to 45 stations across Canada and regional values of model parameters were derived so that the model can be applied to areas without rainfall data. Reliability and sensitivity tests on this design storm on a test catchment produced very

encouraging results (Hydrotek, 1985).

The Chicago-type distribution is based on the assumption that, for a maxima corresponding to various durations. This distribution can be expediently derived from local IDF curves and a set of historical storms which are needed to determine the average timing of the peak intensity (Keifer and Chu, 1957). Although the Chicago-type distribution is still used fairly extensively in engineering practice, and with some modification, its weaknesses can be remedied, it is inferior to the AES and Hydrotek distributions. Recent investigations indicate (Pugsley, 1981) that the Chicago-type distribution is totally inappropriate for some parts of Canada and, in the remaining parts, it is not among the most probable distributions.

For storm durations longer than 12 hours, the SCS 24-hour design storm distribution is sometimes used in Canada, particularly Type II (Type I is generally used in west coast regions, including parts of British Columbia). This distribution is expressed as a percentage of the accumulated rainfall to the total rainfall depth. The SCS 24-hour design storm distribution is shown in Fig. 3.

Spatial distributions of rainfall are of interest on large catchments. Limited sizes of storm cells and storm kinematics and dynamics affect the catchment average rainfall which differs from point rainfall. Spatial effects are sometimes accounted for by using areal reduction factor (see Chapter 5), or by using different (local) rainfall inputs for various parts of the catchment. The problem of moving storms is pertinent to the

operation of large drainage systems. At present, there is not enough knowledge on this subject for applications under design conditions.

Difficulties with synthetic design storms led to the proposal of historical design storms. Such storms are selected either on the basis of historical flood records in the area, or from runoff simulations for historical storms. In the former case, the severity of historical storms and the resulting floods are usually well documented in terms of the discharge and flood damages. Some shortcomings of synthetic design storms, such as the uncertainty regarding the storm return period and antecedent moisture, apply to the historical design storms as well.

Inherent difficulties with design storms and concomitant single event runoff simulation can be avoided by establishing design flows from frequency analysis of simulated flow records, which were produced for the study area and a local precipitation record. Precipitation is converted into flows by means of continuous simulation models of various levels of sophistication. The popular modelling tools for this purpose, at a planning level, are the STORM and SWMM models which are described later in this chapter. To reduce computer costs, both these models use hourly precipitation data which are available on magnetic tape for many locations in Canada (see the Appendix). The use of a relatively long computational time step results in a loss of accuracy which may be acceptable in preliminary calculations for simulation purposes. The most comprehensive tool for continuous runoff simulation is the HSPF model described briefly

in the next section.

To reduce the effort required to convert long precipitation records with long periods of low or zero flows, surrogate continuous simulation has been used for sequences of historical events. Such sequences are sometimes identified in the rainfall record by means of a simple screening model (e.g., the STORM) model). Conceivably, where actual precipitation records are not available, synthetic precipitation could be used instead.

Soil moisture data Soil moisture data are required in some runoff calculations which involve physically-based approaches to infiltration. Such data can be obtained from AES for a limited number of stations across Canada (see the Appendix). In most urban runoff computational procedures, however, the need for soil moisture data is eliminated by using antecedent precipitation data to evaluated the initial catchment wetness. The antecedent precipitation can be determined from actual records, or from AES listings for their design storm distributions (Pugsley, 1981).

Streamflow data Exceptionally, streamflow data may be required in urban flood analysis to identify design-level discharges in urban streams and to trace back the storms which produce those flows. Such historical storms would then be used in further design in the area under consideration. For this purpose, streamflow data are available from the Water Survey of Canada as described in the Appendix.

Snowmelt computation data The contribution of snowmelt and frozen ground to the generation of floods in urban areas is not well understood. The discussion that follows provides some

guidance for such cases. Floods result from adverse combinations of precipitation and catchment conditions which can be characterized by such factors as antecedent soil moisture, precipitation stored on the catchment (e.g, in the snow cover), and frozen ground. In urban areas of large imperviousness, the state of the impervious areas contributing most runoff remains virtually unchanged throughout the year and, consequently, it is the severity of precipitation input that controls flood generation. Because the highest precipitation intensity are typically observed during the summer months, urban floods occur in highly impervious catchments during the same period.

In partly developed urban areas of low imperviousness, the significance of catchment conditions increases and various combinations of catchment conditions and precipitation inputs should be considered in flood analysis. The annual flood may result from a combined snowmelt-runoff event or runoff from a frozen ground, although the concomitant precipitation event does not belong to the most intense events of the year. Furthermore, the flooding situation can be further aggravated by high water levels in the receiving waters and the resulting reduction in the sewer outfall capacity. In these cases, the designer has to consider seasonal design events that may produce flooding as a result of joint occurrence of rainfall, snowmelt, and high water levels in the receiving waters. The details of snowmelt computations are given in Chapter 5.

Very little guidance is available for consideration of frozen ground in runoff computations. Such conditions are

generally accounted for by reducing infiltration rates, or increasing the runoff coefficient.

7.4 Computational Methods

Computational methods in urban flood analysis usually fall in one of the following four categories a) frequency analysis; b) empirical models; c) synthetic hydrographic models; and d) conceptual or simulation models. Selection of the most appropriate approach depends on the objectives of the investigation and on the availability of the physiographic and hydrometeorological data. All four approaches will be discussed briefly in this section. More emphasis will be discussed briefly in this section. More emphasis will be placed on the concepts underlying each group of models than on actual computational techniques which are described in the various user's manuals.

7.4.1 Urban Flood Frequency Analysis The basic concepts of flood frequency analysis described in Chapter 4 can, to some extent, be applied to urban flood flow analysis. However, evaluation of the flood frequency characteristics of urban watersheds is complicated among other things by: a) the limited availability of suitable data; and b) the dynamic (time-varying) physiographic characteristics of urban watersheds. Increasing interest in urban stormwater data collection programs over the last 10 to 15 years, has made possible the preliminary application of flood frequency approaches to urban watersheds (Espey and Winslow, 1974).

The most timely contribution in urban flood frequency analysis has been provided by Espey and Winslow (1974) who developed empirical urban flood frequency equations based on the log-Pearson type III distribution. Sixty urban watershed located throughout the United States were used to predict the T-year peak flows. The derived flood frequency models were given by the following general equation:

$$Q_T = aA^b I^c S^d R_T^e \phi^f \qquad [1]$$

in which a through f are empirical coefficients; T is the recurrence interval in years; A is the drainage area in mi^2; S is the average catchment slope in ft/ft; I is the impervious cover in percent; and ϕ is a channel improvement factor given in Figure 4 as a function of the percent impervious cover and the weighted mean channel Manning 'n". Coefficients a through f are given in Table 2 for all 60 catchments taken together, and for the 26 East coast catchments analyzed separately. Mean absolute errors in fitted flows ranged from 30 to 34% for all 60 watersheds in the study.

7.4.2 <u>Empirical Methods</u> The rational method is probably the most popular, but also the most controversial model for urban runoff estimation. The method is based on the assumption that a rainfall of uniform intensity and sufficient duration will generate a maximum runoff rate per unit area, $q_p = Qp/A$, after a time equal to the time of concentration (t_c) of the catchment. The time of concentration is usually taken to be the time for a particle of water to travel from the hydraulically most remote

part of the catchment to the basin outlet. The ratio of q_p to the constant rainfall intensity i, (q_p/i) is termed the runoff coefficient (C). The runoff coefficient can also be thought of as a volumetric coefficient, defined as the ratio of total runoff to rainfall volumes. The computational form of the rational method is given by:

$$Q_p = C_u \; C \; I \; A \qquad [2]$$

in which Q_p is the peak runoff in ø̸s (cfs); I is the average rate of rainfall intensity in mm/h (in./hr.) for a T-year event having a duration equal to the time of concentration; A is the area of the watershed in km^2 (acres); and C_u is a unit conversion factor equal to 1.008 in British units, and 2.78 in metric units.

he underlying assumptions of the rational method are usually sufficient to emphasize its limitations:

- time-invariant response of the catchment irrespective of antecedent moisture conditions;
- linear catchment response;
- rainfall frequency equal to flow;
- uniform and constant rainfall intensity over the whole catchment during the entire duration (t_c) of the event;
- uniform runoff coefficient across the watershed or sub-catchment.

This latter restriction can be circumvented partly by providing a weighted estimate of the runoff coefficient.

McPherson (1969) noted that an outstanding limitation of the rational method was its complete independence of storm patterns. Notwithstanding some of the very restrictive assumptions, the

rational method can still provide reasonable peak flow estimates (Kibler, 1982; Whipple et al., 1983). Steps necessary in the application of the rational method can be summarized as follows:

1. Identify the drainage area, A, tributary to the point under investigation;
2. Determine the runoff coefficient (C) for the tributary area;
3. Estimate the time concentration (t_c) to the design point;
4. Calculate the average rainfall intensity from the T-year intensity-duration curve, taking t_c as the duration of the event;
5. Compute the peak flow, Q_p, from Equation 2.

Time of concentration In urban catchments, the time of concentration at the design point is equal to the sum of the inlet time (te) and the time of travel (tf) in the pipe network:

$$t_c = t_e + t_f \qquad [3]$$

The inlet time varies with surface slope, rainfall intensity and storm pattern, depression storage, surface cover, antecedent moisture conditions, distance and infiltration capacity of the soil (Sheaffer et al., 1982). A variety of formulae have been proposed to estimate t_e in urban areas providing a wide range of entry time estimates (Kibler, 1982). Figure 5 describes a convenient graphical estimate of overland and gutter flow times, while Table 3 summarized some of the most popular entry time formulae in use today.

Under certain circumstances, particularly where land use characteristics vary significantly over a catchment, peak flows might not be associated with the time of concentration of the whole catchment. In fact, a highly non-uniform runoff coeffic-

ient might cause peak flows to result from a rainfall duration less than t_c, for which only a fraction of the catchment will be contributing to runoff under a higher rainfall intensity.

Intensity The average (constant) T-year rainfall intensity, I, for an event of duration t_c, is obtained from local intensity-duration-frequency curves discussed in chapter 5.

Runoff coefficient A great deal of judgement must be exercised in estimating the average (weighted) runoff coefficient of a catchment or subcatchment, as the values of C vary as a function of surface cover, slope, depression storage, moisture conditions, and rainfall intensity. Table 4 summarizes typical values of C as a function of land use, while Table 5 gives values of C for different homogeneous areas. As indicated earlier, these values should be adjusted to reflect actual surface runoff conditions and initial soil moisture state. For composite areas a weighted runoff coefficient should be calculated. Alternately, the following equation has found widespread use in practice:

$$C_{avg} = C_{perv} (1-Imp) + C_{imp}\ Imp \qquad [4]$$

in which C_{perv} is the pervious area runoff coefficient; C_{imp} is the impervious area runoff coefficient; and Imp is the fraction of impervious surfaces.

Modifications to the basic rational method have been suggested by many investigators to overcome some of the limitations listed earlier. Sheaffer et al. (1982) reported on the use of a correction factor for infrequent storm, while Yen (1978) incorporated a full hydrograph construction to the basic model. Others have suggested that the inlet time be adjusted to agree

more closely with the more accepted conceptual models (Wisner, 1983). Finally, Mitci (1974) and Smith and Lee (1984) support the use of time-varying runoff coefficients. The reader should also realize the limitations of these modified methods, some of which have more merit than others. In any event, most everybody agrees that the rational method should be restricted to small urban watersheds. However, authors tend to disagree on the maximum watershed area to which it is believed to be applicable. The following upper limits have been recommended: 40 ha (Sheaffer et al., 1982), 80 ha (Whipple et al, 1983), 250 ha (Kibler, 1982), and 500 ha (Viessman et al, 1977).

The Soil Conservation Service of the U.S. Department of Agriculture (SCS,1975) has developed a method of estimating urban runoff based on the design storm concept. In contrast to the rational method, the SCS method provides an estimate of the runoff volume, as well as the time-history of the design storm flows. The governing equations in the SCS model are given by:

$$Q = [P-IA]^2 / [P-IA=S] \qquad [5]$$

and

$$S = [1000/CN] - 10 \qquad [6]$$

in which Q is the accumulated runoff since the beginning of the storm; IA represents the initial abstraction losses or sum of interception, depression storage, and infiltration losses to be satisfied prior to runoff; S is the maximum potential soil retention; CN is the runoff curve number, determined from soil type, land cover, and antecedent moisture conditions. Parameter

IA is often taken to be 0.2S, however this value has been said to be on the high side (McCuen, 1982). A few authors have developed empirical relationships between the runoff coefficient and the SCS curve number (Hawkins , 1978); Smith and Falcone, 1983). The equation proposed by Smith and Falcone is given by:

$$CN = \frac{100}{1 + P_{tot}(0.5 - C((1.25/C + 1)^{0.5} - 1))} \qquad (7)$$

7.4.3 Synthetic Hydrographic Methods The empirical and flood frequency methods described earlier are peak flow estimation techniques. Usually, most stormwater management studies require that the time-history of flow be assessed in order to better evaluate alternative stormwater control measures such as source control, detention storage, etc. All synthetic unit hydrograph methods are based on the definition of a unit hydrograph that is used in estimating runoff by convoluting the appropriate unit hydrograph ordinates with the excess rainfall hyetograph. Accordingly, the first step in the analysis is to estimate the appropriate excess rainfall pattern.

Rainfall excess estimate The term **rainfall abstraction** refers to that component of rainfall that does not contribute to surface runoff. It is usually, made up of interception losses are usually not very significant and are traditionally neglected or lumped with other rainfall losses without any serious effects on the results.

Depression storage refers to that component of rainfall that remains trapped on the ground surface into small puddles without

infiltrating or running off. In general, there are two basic depression storage models: a) Linsley et al.'s (1982) gradual accumulation depression storage model; and b) the initial depression storage model. Linsley's model can be written as:

$$V_d(t) = S_d \left[1 - e^{-k P_e(t)} \right] \qquad [8]$$

in which Vd(t) is the volume of water in depression storage at time t; S_d is the maximum depression storage volume; $p_e(t)$ is the cumulative precipitation in excess of infiltration at time t; and k is a coefficient equal to $1/S_d$. Typical depression storage depths for various surfaces are listed in Table 5.

Many of the more popular rainfall-runoff models make use of the initial storage depression must be satisfied prior to any surface runoff. SWMM for example also provides the opportunity to identify a fraction of the tributary area for which the depression model appears to be quite valid in cases where depression storage volumes represent an insignificant fraction of the total rainfall depth.

Finally, the third component necessary in evaluating rainfall excess is the estimation of infiltration losses. Modelling of infiltration losses in urban areas is usually quite primitive. The variability in infiltration characteristics throughout a watershed coupled with the usual lack of site-specific infiltrometer data have led many investigators to treat the infiltration component of the rainfall-runoff process more as a black-box than an physically-based process. In fact, infiltration parameters are often prime targets in the calibration

process.

Among the various infiltration models available, Horton's (1933) empirical relationship has received significant attention in urban runoff modelling. The rate of infiltration according to this exponential model is given by:

$$f(t) = f_c + (f_o - f_c)\, e^{-kt} \qquad [9]$$

in which f_o and f_c are the initial and final infiltration rates mm/h, respectively, and k is the exponential decay rate in h^{-1}. Table 7 gives an indication of Horton's infiltration parameters as a function of soil type. Initial soil moisture conditions can also be considered by appropriately specifying f_o (Terstriep and Stall, 1974). Great care should be exercised in specifying the time variable in Horton's model. Only if the rainfall intensity falls below the potential infiltration rate, the time variable t becomes the effective infiltration time, and must be calculated from the accumulated infiltration curve. This computation is readily incorporated in most computer codes.

<u>Synthetic Hydrograph Computation</u> Synthetic hydrograph methods for urban runoff analysis are based on the discrete convolution of a D-hour unit hydrograph with the estimated excess rainfall hyetograph, i.e.,

$$Q(t) = \sum_{i=0}^{t} U(i)\, R(t-i) \qquad [10]$$

in which U(i) represents the ordinates of the discrete D-hour unit hydrograph, and R(t-i) is the excess rainfall hyetograph. Synthetic unit hyetograph methods differ from one another in

their definition of the D-hour unit hyetograph ordinates. However, all D-hour unit hyetograph methods are based on the following assumptions:

- rainfall excess is distributed uniformly over the entire catchment;
- rainfall excess is distributed uniformly over the incremental duration D;
- the catchment behaves as a linear system, i.e., the principles of proportionality and superposition are assumed to hold;
- runoff duration is constant for a given rainfall duration irrespective of moisture conditions.

For ungauged watersheds, several unit hydrograph procedures have been proposed over the years (Viessman et al., 1977). Snyder's unit hydrograph defined in Figure 6, has a peak runoff rate given by;

$$Q_p = 7.0\ C_p\ A\ /\ T_l \qquad [11]$$

in which Q_p is the peak flow in m^3/s; A is the catchment area in km^2; CP is a coefficient ranging from 0.59 to 0.69; and tl is the lag time. The Corps of Engineers (Viessman, et al., 1977) later introduced characteristic time widths at 50 and 75% of peak flow in order to assist in defining the shape of the unit hydrograph. Espey and Altman (1978) developed functional relationships between the unit hydrograph parameters given in Figure 6, and the physiographic features of the watershed for a 10-min. unit hydrograph. The resulting relationships derived from actual urban rainfall-runoff sequences are given in Table 8. The watershed conveyance factor, Φ, has been discussed earlier in

this section, and is given in Figure 4. The unit hydrograph should be constructed such that the area below the curve corresponds to 1 inch of direct runoff. Aron and White (1982) described an explicit method of finding the ordinates of the unit hydrograph by fitting a gamma distribution. The procedure is particularly well suited for desktop and microcomputer applications.

7.4.4 Simulation Models Simulation models, also referred to as conceptual or internally-descriptive models, are generally characterized by a more or less detailed mathematical description of the major rainfall-runoff processes, including rainfall, hydrologic abstractions, surface runoff, channel transport, and receiving water components. While such models are designed to reduce the level of empiricism in modelling of the rainfall-runoff process, including rainfall, hydrologic abstractions, surface runoff, channel transport, and receiving water components. While such models are designed to reduce the level of empiricism in modelling of the rainfall-runoff process by providing a more elaborate description of the physics involved from input (rainfall) to output (runoff), the complexity of the natural process involved precludes the full realization of that objective. The reader must recognize that any model, no matter how sophisticated, is only a crude representation of the prototype. Moreover, the reader should realize that the most sophisticated model is not necessarily the most appropriate one. In fact, in recent years, with the advent of powerful microcomputers, there has been a general tendency to move away from the more sophisticated

conceptual models to simpler desktop algorithms designed to achieve a similar level of accuracy.

The purpose of this section is to provide a brief overview of the more popular simulation models available. The description has been limited to public domain models that have received widespread acceptance by the practicing community. On the other hand, section 7.5.1 provides an overview of Canadian modelling practice.

Many authors (McPherson, 1979; Dendrou, 1982) suggested that simulation models can be classified in one of the following applications categories: planning, analysis/design, and operation. Only the first two groups of models will be described hereafter as they are of particular concern in urban flood analysis.

Planning models Planning models are designed to provide an overview of the water quantity and quality impacts of alternative stormwater management schemes. Accordingly, most planning models are lumped parameter, continuous simulation models. The most popular planning model is the Storage, Treatment, Overflow, Runoff Model (STORM) developed by the U.S. Army Corps of Engineers (1976) Hydrologic Engineering Center. The conceptual framework of the model is described in Figure 7 in which dry--weather flow, pollutant accumulation, pollutant washoff, and surface runoff are simulated at one hour time intervals. Storage and treatment alternatives, based on a simplified accounting scheme are integrated in the model to assess the significance of pollutant loadings from combined sewer overflows (CSO) and

treatment plant effluent on the receiving water body. A flow-chart of the model is provided in Figure 8. STORM also handles non-urban catchments, smowpack accumulation, smowmelt, and land erosion from urban and non-urban areas (U.S. Army Corps of Engineers, 1976 and 1977).

Medina (1979) developed a receiving water model that uses the output from STORM to assess the impacts of CSO on the receiving water body. Other planning models include a distributed parameter version of STORM called SEMSTORM (Shubinski, et al. 1977), and a simplified version of the Storm Water Management Model (SWMM) developed for the Environmental Protection Agency (EPA) by Lager et al. (1976). A major drawback of all planning models is the lack of reported verification results. The continuous modelling approach on which all such models are based, requires an extensive data base for both calibration and verification. In fact, most planning exercises have proceeded without the benefit of local field data calibration (McPherson ,1979).

Analysis/design models Analysis and design models are used either to assess the performance of an existing stormwater drainage system or to design one based on a proposed land use master plan. While data requirements usually vary from one model to another, they are typically more elaborate than those required at the planning stage.

One of the simpler and widely accepted analysis/design model is the Illinois Urban Drainage Area Simulator (ILLUDAS; Terstriep and Stall, 1974). Derived from the Road Research LAboratory

method, the ILLUDAS model differs from its predecessor in the way it handles runoff from the various surface components. In addition to runoff from impervious surfaces, the ILLUDAS model generates a surface runoff hydrograph for pervious surfaces, taking into consideration contributions from the indirectly connected impervious surfaces. All surface runoff hydrographs are based on the isochronal method of flow computation. Infiltration characteristics are modelled in accordance with a form of Horton's model, modified to reflect possible antecedent moisture conditions using Horton's conceptual infiltration model. Design or evaluation modes can be specified for any reach in a network. The latest version (1978) of ILLUDAS provides both hydrologic and hydraulic channel routing options. A simplified flowchart of the model is provided in Figure 9. A major drawback of using the ILLUDAS model in the analysis of existing systems lies in its inability to handle looped network and its simplistic handling of surcharge flow conditions, viz., immediate storage of any excess at a reach. Such problems are usually not encountered in the design of new systems.

The Storm Water Management Model (SWMM) developed for EPA (Huber, et al., 1982) is probably the most widely used model in North America today. The model has been undergoing continuous development ever since its introduction in the early seventies. In addition to the usual surface runoff and pipe flow components, SWMM provides a comprehensive simulation of water quality parameters in storm and combined sewer networks, as well as in receiving water bodies. This large-scale model consists of 5

major computational blocks in addition to a variety of control and service blocks, including the EXECUTIVE, COMBINE, GRAPH, and STATISTICS blocks as shown in Figure 10. The linkage among the five computational blocks is described in Figure 11. The RUNOFF block generates the quantity and quality characteristics of the surface runoff component. Infiltration is computed either from Horton's equation described earlier in this chapter or from Green-Ampt's model discussed in Chapter 6. The TRANSPORT block routes hydrographs and pollutographs through the sewer network system based on a finite-difference formulation of the kinematic wave equation. AS an alternative to the transport block, the EXTRAN block provides a complete solution of the continuity and momentum equations, allowing modelling of looped networks and surcharge flow conditions. The STORAGE/TREATMENT block simulates the operation of storage and treatment plant facilities, while the RECEIVE block examines the water quality impacts of pollutant discharges on the receiving water body. Currently, the SWMM model is available in a microcomputer version which can be run on the IBM personal computer or other compatible microcomputers.

The Hydrocomp Simulation Program - Fortran (HSPF) is a derivative of the Stanford Watershed Model. This continuous simulation modular program includes a complete water balance and accounts for both surface and groundwater components in addition to exchanges and interactions between them. HSPF uses the kinematic wave equation for both surface and channel routing. Empirical equations are used to estimate surface runoff water quality parameters. The nonpoint source (NPS) module is particu-

larly well suited to study the long term effects of NPS of pollution on the receiving water bodies.

A host of other urban runoff models have been developed over the last 15 years. The interested reader is referred to Brandsetter (1976), Brandsetter et al. (1976), Huber et al. (1979), and Delleur and Dendrou (1980), for a more complete account of the characteristics and features of all of these models.

Calibration, verification and validation The reader must recognize that any model, no matter how sophisticated, is only a crude representation of the actual system. Consequently, an important step in the application of any rainfall-runoff model is concerned with calibration and verification of the selected model.

Calibration is the process of adjusting model parameters to minimize the differences between observed and simulated flows. Care should be taken to preserve the physical significance of any physically-based parameters. Verification on the other hand, is concerned with assessing the performance of the calibrated model on a set of rainfall-runoff events for calibration, the other for verification. Care should be taken to ensure that each set of events spans the range of rainfall-runoff conditions of interest. For example, it would serve little purpose to calibrate a model for a group of high frequency events, if the immediate objective is to evaluate runoff conditions for low frequency events. Although the transposition of 'calibrated' parameter values is not generally recommended, some guidance in value selections can be obtained from model manuals (Huber et al.,

1982; Terstriep and Stall, 1974).

7.4.5 A Note on Model Selection On the subject of model selection, McPherson (1979) stated that "reality dictates that a model should be selected on the basis of the type of application involved, how much can be invested in its use, how often it would be used, what levels of precision are required or desired, what kinds of outputs are wanted, how much time can be spent to get the model to work, and how much time can be spent to get the model to work, and how much time can be committed to verify and calibrate the model." Model selection will thus involve answering all of these questions and examining the corresponding model features. Delleur (1980) provided a very comprehensive analysis of a large number of rainfall-runoff models by indirectly answering to many of the questions listed above. Table 9 provides a summary of the applicability of the various models described in this chapter. It is obvious that there is no 'universal' model. In fact, it has been observed that the most experienced users will make use of a hierarchy of models, ranging from the very simple to the more complex, depending on the nature of the problem. In any event, it should be emphasized that the simplest model that provides the desired level of accuracy should be used. The designer should keep in mind that models are just tools helping to achieve the design goal - a well-functioning economical drainage system.

7.5 Applications and Current Practice

This section deals with an overview of current practices on urban flood analysis and case studies illustrating such practices.

7.5.1 Overview Canadian practice in urban flood analysis nas been significantly refined during the last 10 years as the earlier simplistic empirical approaches have been replaced by a more comprehensive analysis of urban floods and flood abatement measures through stormwater management. A brief listing of advances in this field follows.

In the overall approach, the problem of urban floods is now approached on the watershed basis with full consideration of two interconnected drainage system - the minor and major drainage. Towards this end, master drainage plans, which include flood abatement and stormwater management measures, are developed and updated as discussed earlier in Section 7.2.3.

Numerous advances have been made in urban flood computations. The rainfall inputs used in such computations include constant rainfall intensity data, synthetic design storms, historical design storms, and rainfall records for use in continuous simulation. In general, these inputs are available from AES as described in the Appendix.

Flood flows are generally computed by means of computer models of various nature. Besides the earlier listed widely-used models, there is a fair number of other models which are well suited for certain tasks of urban flood analysis and should be fully considered in the model selection process. A brief listing

of such models follows.

At the planning level, the commonly used models include the SCS procedure and various versions of the STORM and HYMO models. In particular, a number of applications has been reported for the OTTHYMO model (Wisner and P'ng, 1982) which is an expanded version of the HYMO model designed to reflect special properties of urban catchments.

In detailed modelling, the majority of applications is undertaken with the SWMM and ILLUDAS models. Other models used in practice include OTTSWMM, Queen's University Urban Runoff MOdel (QUURM), SIRDU model, and the Versatile Stormwater Quantity and Management Model (VSQMM).

OTTSWMM is a modified version of the SWMM model which divides inlet supply hydrographs into minor and major flows and routes these flows through the respective transport systems (Wisner, 1983).

QUURM (Watt and Schroeter, 1983) simulates runoff generation and routing in urban catchments. Further developmental work is underway to account for dual minor/major drainage (Watt, 1983).

SIRDU is a modified version of the ILLUDAS model based on the time-area curve concept with a number of enhancements (Béron, 1983).

VSQMM (Lee, 1981) simulates runoff generation and routing in urban catchments. For various processes, the user can select the preferred approach from a number of options.

The most widespread flood abatement measure is on-site runoff storage which is considered early in the planning process.

Popular forms of runoff storage include distributed storage on the catchment surface (e.g., created by inlet restrictions) and stormwater ponds. Experience with these measures shows that although the drainage schemes with on-site controls may be slightly more expensive than conventional schemes without controls, significant savings are achieved in the former case by reducing or eliminating the need for downstream controls (Ministry of the Environment, 1983).

The implementation of modern approaches to urban flood analysis abatement is facilitated by drainage criteria, guidelines and policies which have been proposed or adopted by some municipalities and provinces. Further improvements can be expected as more jurisdictions will follow this practice.

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TABLES

Table 1. Methodology for Master Drainage Plan Preparation

1. Identification of Problems and definition of objectives

   1.1 Reduction of local flooding inconvenience
   1.2 Reduction of local flood damage and threat to life
   1.3 Reduction of downstream flooding

2. Identification of constraints

   2.1 Natural constraints
   2.2 Policy and regulation constraints
   2.3 Cost constraints

3. Definition of drainage system components

   3.1 Inputs - Design rainfall, unit costs
   3.2 Elements - System elements for various alternatives derived from considerations of both non-structural and structural measures.

   Non-structural measures:

   Land use planning
   Prohibition of flood plain occupancy
   Floodway - flood fringe concept

   Structural Measures:

   Drainage conduit and channel configurations
   Storage structures
   Diversion structures
   Channelization
   Dikes
   Floodproofing

   3.3 Outputs - Flows, volumes, and costs for various alternatives as obtains from various computational procedures

4. Comparison of alternatives and selection of the best alternative

   4.1 Decision-making matrix
   4.2 Cost comparison

5. Plan implementation and regular updating

Table 2. Derived Flood Frequency Parameters for Equation 1 (Espey and Winslow, 1974)

A) All 60 urban watersheds

Recurrence Interval years	a	b	Model Parameters c	d	e	f
2.33	169	0.77	0.29	0.42	1.80	-1.17
5	172	0.80	0.27	0.43	1.73	-1.21
10	178	0.82	0.26	0.44	1.71	-1.32
20	243	0.84	0.24	0.48	1.62	-1.38
50	297	0.85	0.22	0.50	1.57	-1.61

B) Twenty-six east coast urban watersheds

Recurrence Interval years	a	b	Model Parameters c	d	e	f
2.33	11700	0.73		0.75		
5	16800	0.75		0.76		
10	19800	0.76		0.75		
20	21000	0.77		0.72		
50	21200	0.78		0.68		

TABLE 3. Summary of Time of Concentration (t_c) Methods (Kibler, 1982)

Method and Date	Formula for T_c (min.)	Remarks
Kirpich [1940]	$T_c = 0.0078\ L^{0.77}\ S^{-0.385}$ L = length of channel/ditch from headwater to outlet, ft S = average watershed slope, ft/ft	Developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%); for overland flow on concrete or asphalt surfaces multiply T_c by 0.4; for concrete channels multiply by 0.2; no adjustment for overland flow on bare soil or flow in roadside ditches
California Culverts Practice [1942]	$T_c = 60\ [11.9\ L^3/H]^{0.385}$ L = length of longest watercourse, mi H = elevation difference between divide and outlet, ft	Formula is essentially the Kirpich equation; developed from small mountainous basins in California; [U.S. Bureau of Reclamation, 1973, pp. 67-71]
Izzard [1946]	$T_c = [41.025\ (0.0007\ i + c)\ L^{0.33}]/[S^{0.333}\ i^{0.667}]$ i = rainfall intensity, in./h c = retardance coefficient L = length of flow path, ft S = slope of flow path, ft/ft	Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement, c = 0.012 for concrete pavement, and c = 0.06 for dense turf; solution is extremely tedious and requires iteration; product i times L should be ≤ 500
Federal Aviation Agency [1970]	$T_c = 1.8(1.1 - C)L^{0.50}/S^{0.333}$ C = rational method runoff coefficient L = length of overland flow, ft S = surface slope, %	Developed from air field drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems but has been used frequently for overland flow in urban basins
Kinematic wave formulas Morgali and Linsley [1965] Aron and Egborge [1973]	$T_c = 0.94\ L^{0.6}\ n^{0.6}/[i^{0.4}\ S^{0.3}]$ L = length of overland flow, ft n = Manning roughness coefficient i = rainfall intensity in./h S = average overland slope, ft/ft	Overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces; method requires iteration since both i (rainfall intensity) and T_c are unknown; superposition of intensity-duration-frequency curve gives direct graphical solution for T_c
SCS [1975] lag equation	$T_c = \{100\ L^{0.8}[(1000/CN)-9]^{0.7}\}/[1900\ S^{0.5}]$ L = hydraulic length of watershed (longest flow path), ft CN = SCS runoff curve number S = average watershed slope, %	Equation developed by SCS from agricultural watershed data; it has been adapted to small urban basins under 2000 acres; found generally good where area is completely paved; for mixed areas it tends to overestimate; adjustment factors are applied to correct for channel improvement and impervious area; the equation assumes that T_c = 1.67 x basin lag
SCS [1975] average velocity charts	$T_c = 1/60(\Sigma\ L/V)$ L = length of flow path, ft V = average velocity in feet per second from Fig. 3-1 of TR 55 for various surfaces	Overland flow charts in Fig. 3-1 of TR 55 show average velocity as function of watercourse slope and surface cover

TABLE 4. Rational Method Runoff Coefficient (ASCE, 1969).

Description of area	Runoff coefficients
Business:	
Central business areas	0.70-0.95
District and local areas	0.50-0.70
Residential:	
Single-family areas	0.35-0.45
Multiunits, detached	0.40-0.60
Multiunits, attached	0.60-0.75
Residential 1/4-hectare (1/2-acre) lots or larger	0.25-0.40
Industrial:	
Light areas	0.50-0.80
Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.35
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30

Source: Reprinted with permission of the American Society of Civil Engineers [3].

TABLE 5. Rational Method Runoff Coefficients for Composite Analysis (ASCE, 1969).

For Impervious Surfaces	
Character of surface	Runoff coefficient
Streets:	
Asphaltic	0.70-0.95
Concrete	0.80-0.95
Drives and walks	0.75-0.85
Roofs	0.75-0.95

For Pervious Surfaces					
		Runoff coefficient			
Slope		A soils	B soils	C soils	D soils
Flat:	0-2%	0.04	0.07	0.11	0.15
Average:	2-6%	0.09	0.12	0.16	0.20
Steep:	Over 6%	0.13	0.18	0.23	0.28

Sources: (Top) Reprinted with permission of the American Society of Civil Engineers [3]; (bottom) reprinted with permission of Kurt W. Bauer and The Southeastern Wisconsin Regional Planning Commission [4].

Table 6. Typical Storage Depression Depths for Various Land Covers (Kibler, 1982)

Land Cover	Depression and Detention, mm	Recommended, mm
Impervious		
Large paved areas	1.25-3.75	2.50
Roofs, flat	2.50-7.60	2.50
Roofs, sloped	1.25-2.50	1.25
Pervious		
Lawn grass	5.00-12.50	7.60
Wooded areas and open fields	5.00-15.20	10.20

Table 7. Factors Used in Calculating Horton's Standard Infiltration Curve (Terstriep and Stall, 1974).

Item	VALUE			
Hydrologic soil group				
USDA designation	A	B	C	D
ILLUDAS designation	1	2	3	4
Final constant infiltration rate, f_c, mm/h	25.4	12.5	6.4	2.5
Initial infiltration rate, f_0, mm/h	255	200	125	75
Shape factor, k, of infiltration curve	2	2	2	2

Note:

Soil Groups	Description
A	Low runoff potential
B	Moderate infiltration rates
C	Slow infiltration rate
D	High runoff potential

TABLE 8. Espey 10-min Unit Hydrograph Equations (Kibler, 1982).

Equations	Total Explained Variation
$T_R = 3.1\ L^{0.23} S^{-0.25} I^{-0.18} \phi^{1.57}$	0.802
$Q = 31.62 \times 10^3\ A^{0.96} T_R^{-1.07}$	0.936
$T_B = 125.89 \times 10^3\ A\ Q^{-0.95}$	0.844
$W_{50} = 16.22 \times 10^3\ A^{0.93}\ Q^{-0.92}$	0.943
$W_{75} = 3.24 \times 10^3\ A^{0.79}\ Q^{-0.78}$	0.834

Notes: Where L is the total distance (in feet) along the main channel from the point being considered to the upstream watershed boundary; S is the main channel slope (in feet per foot) as defined by H/(0.8L), where L is the main channel length as described above and H is the difference in elevation between two points, A and B (A is a point on the channel bottom at a distance of 0.2L downstream from the upstream watershed boundary; B is a point on the channel bottom at the downstream point being considered); I is the impervious area within the watershed (in percent); ϕ is the dimensionless watershed conveyance factor as described elsewhere in the text; A is the watershed drainage area (in square miles); T_R is the time of rise of the unit hydrograph (in minutes); Q is the peak flow of the unit hydrograph (in cubic feet per second); T_B is the time base of the unit hydrograph (in minutes); W_{50} is the width of the hydrograph at 50% of the Q (in minutes); and W_{75} is the width of the unit hydrograph at 75% of Q (in minutes).

TABLE 9. Model Applicability to Different Urban Drainage Problems (Torno, 1982).

Application	Rational Method	Unit Hydrograph	STORM	SWMM	ILLUDAS	HSPF
Selection of critical rainfall events	unsuitable	unsuitable	very good	good	poor	good
Preliminary analysis of urban areas	fair	good	good	good	very good	good
Detailed analysis of urban areas	poor	poor	fair	very good	good	very good
Analysis of detention retention storage	unsuitable	unsuitable	very good	good	good	good
Design of detention/ retention storage	unsuitable	unsuitable	good	very good	good	very good
Analysis of surcharged sewer systems	unsuitable	unsuitable	unsuitable	good	unsuitable	good
Prediction of peak flows in small systems	good	fair	poor	good	very good	good
Design of sewer systems (open-pipe flow)	fair (for small areas)	fair	poor	good	good	

FIGURES

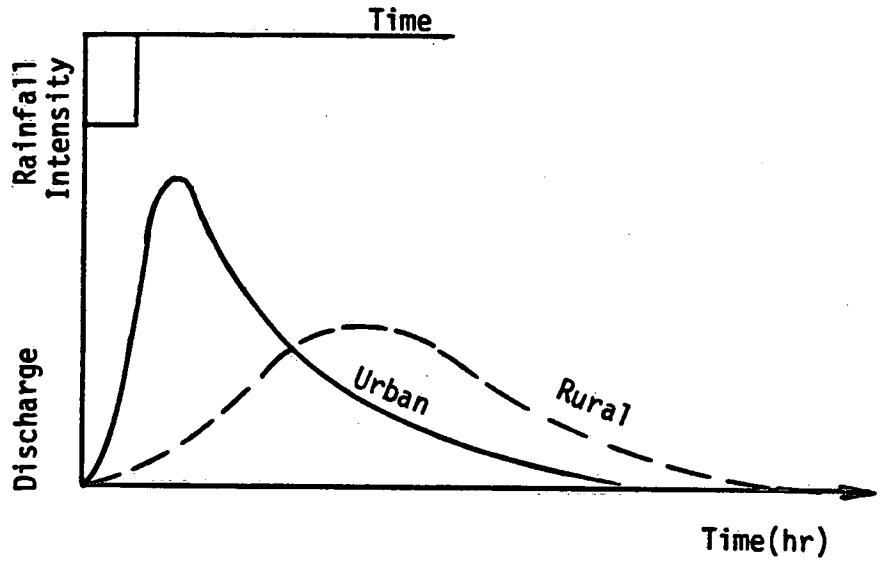
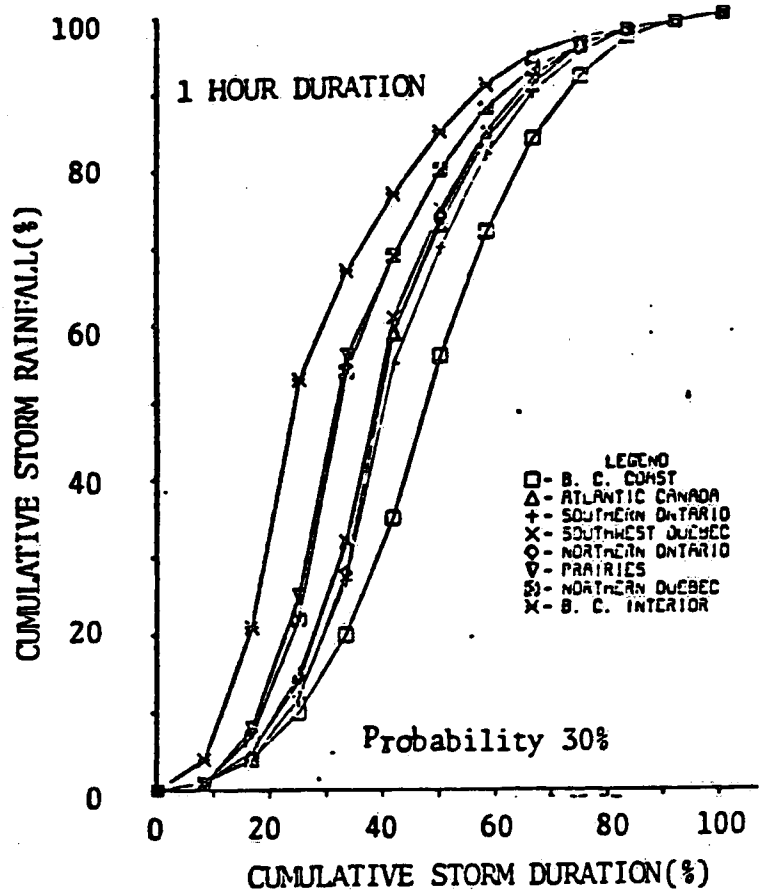


Fig.1. Urbanization Impacts on Basin Response Without Increased Detention Storage(After Kibler, 1982)



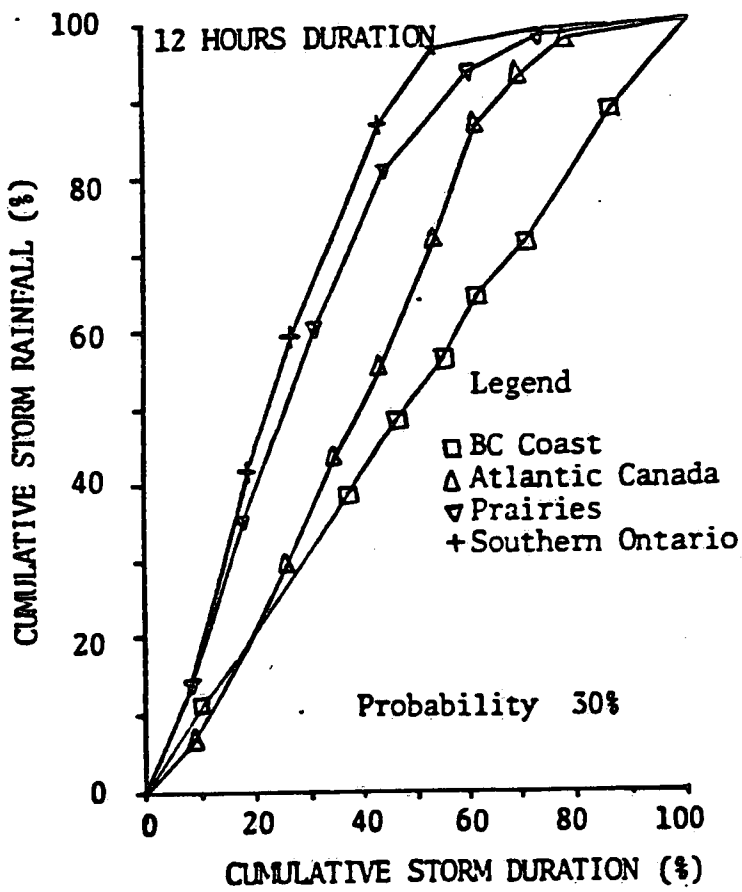


Fig.2. AES Temporal Storm Distributions for Durations of 1 and 12 Hours

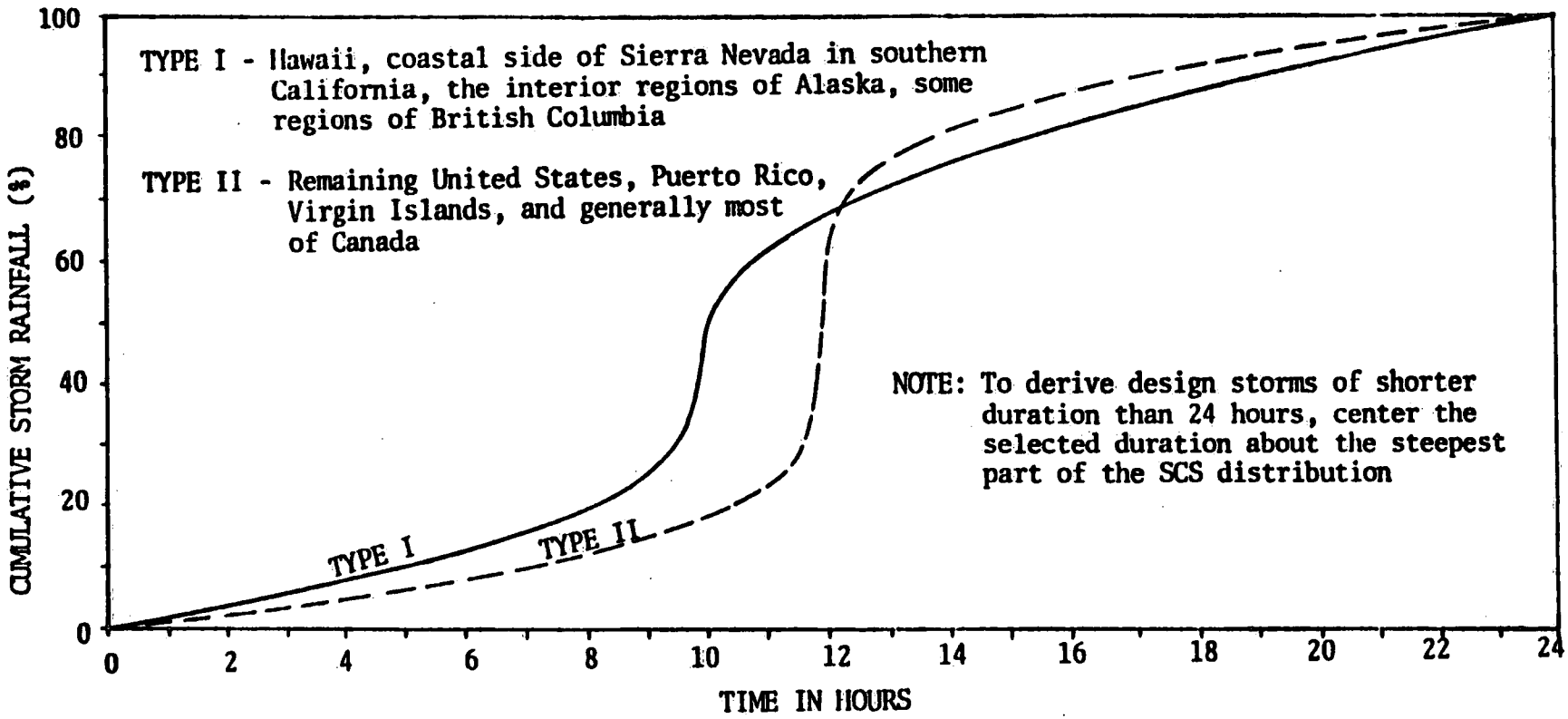


Fig.3. SCS Twenty-Four Hour Rainfall Distributions

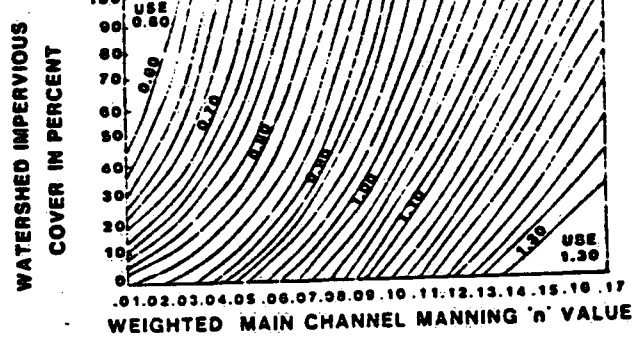


Fig.4. Watershed Conveyance Factor, Φ, as a Function of Percent of Impervious Cover and Weighted Mean Channel Manning 'n' Value (Whipple, et al., 1983).

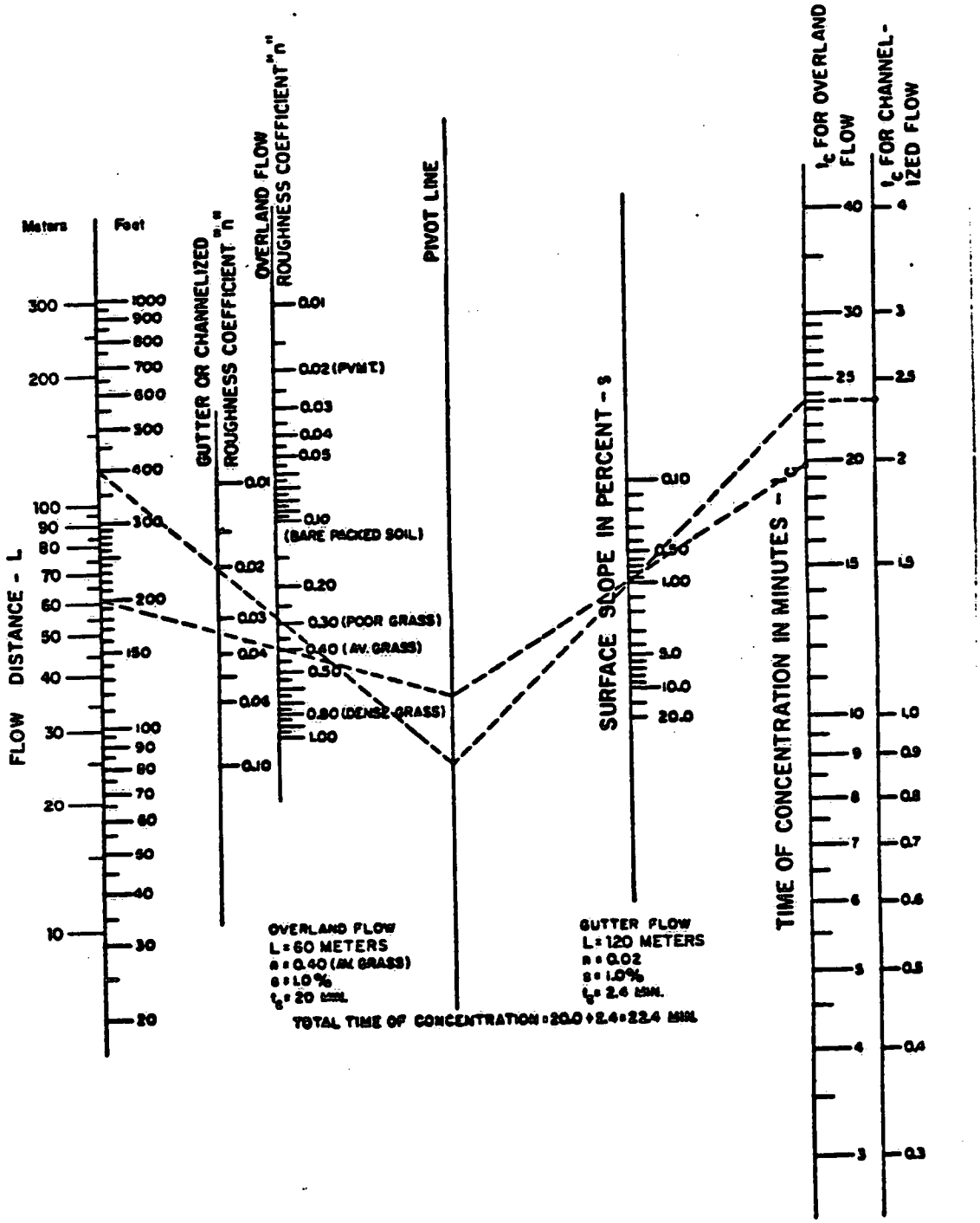


Fig.5. Nomograph for the Time of Concentration (Sheaffer et al., 1982).

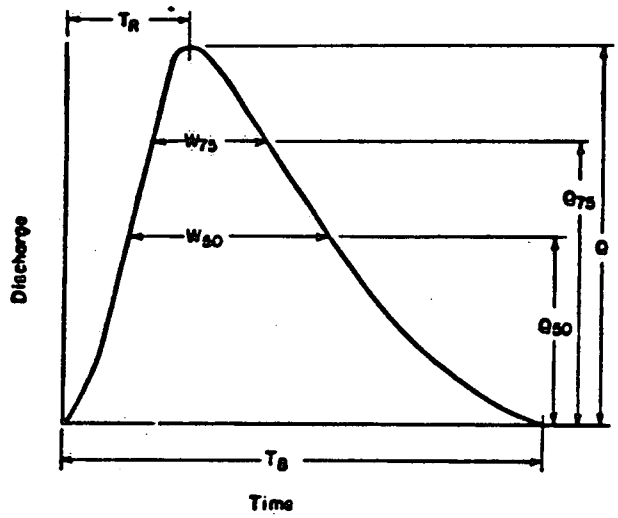


Fig.6. Definition of Espey 10-min. Unit Hydrograph Parameters.

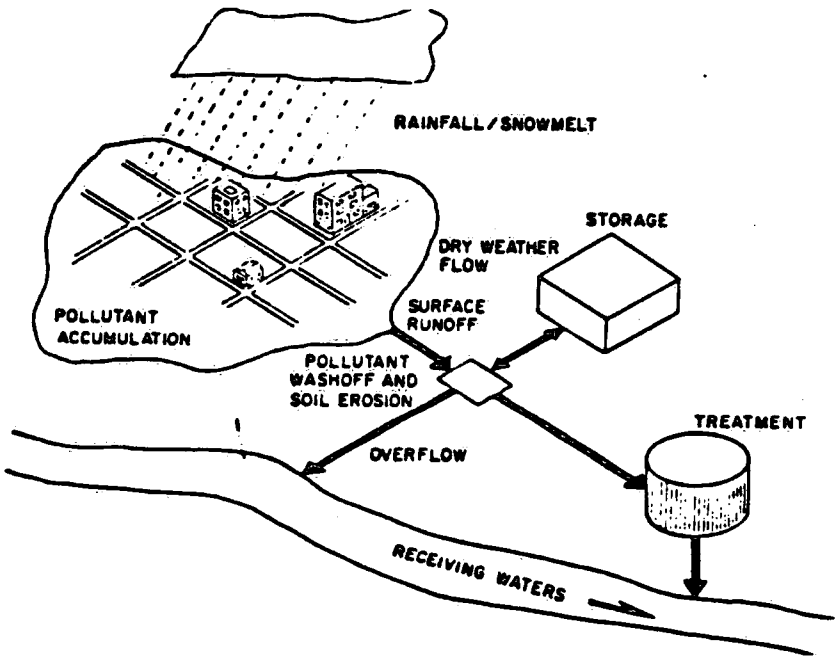


Fig.7. Planning Level Basin Modelling Conceptualization (U.S. Army Corps of Engineers, 1976).

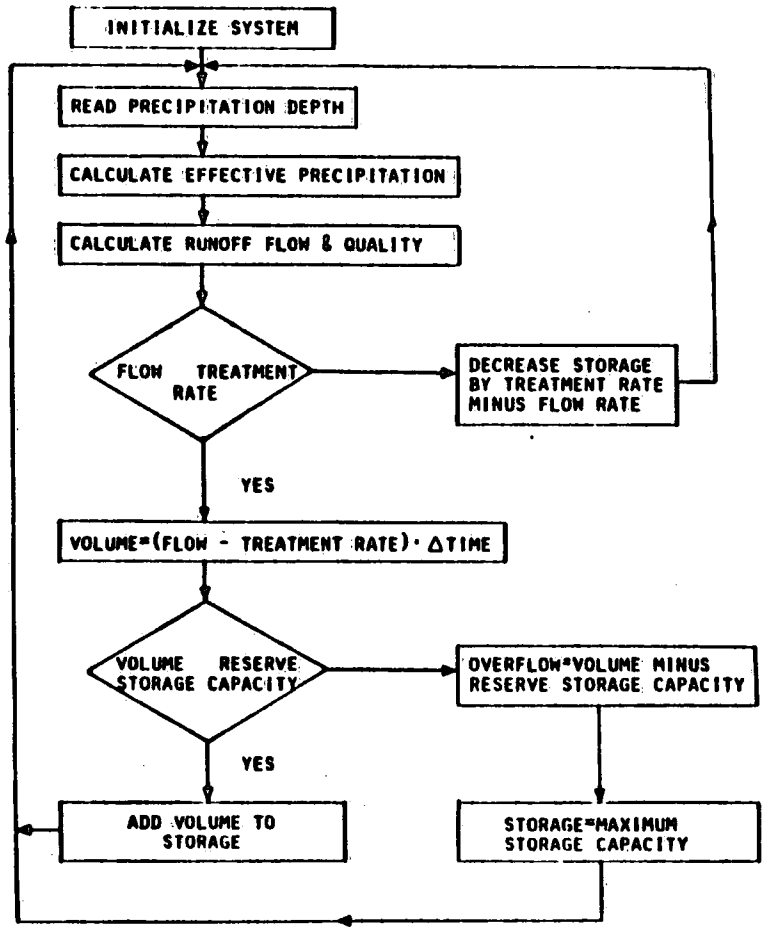


Fig.8. Simplified Flowchart of STORM (U.S. Army Corps of Engineers, 1976).

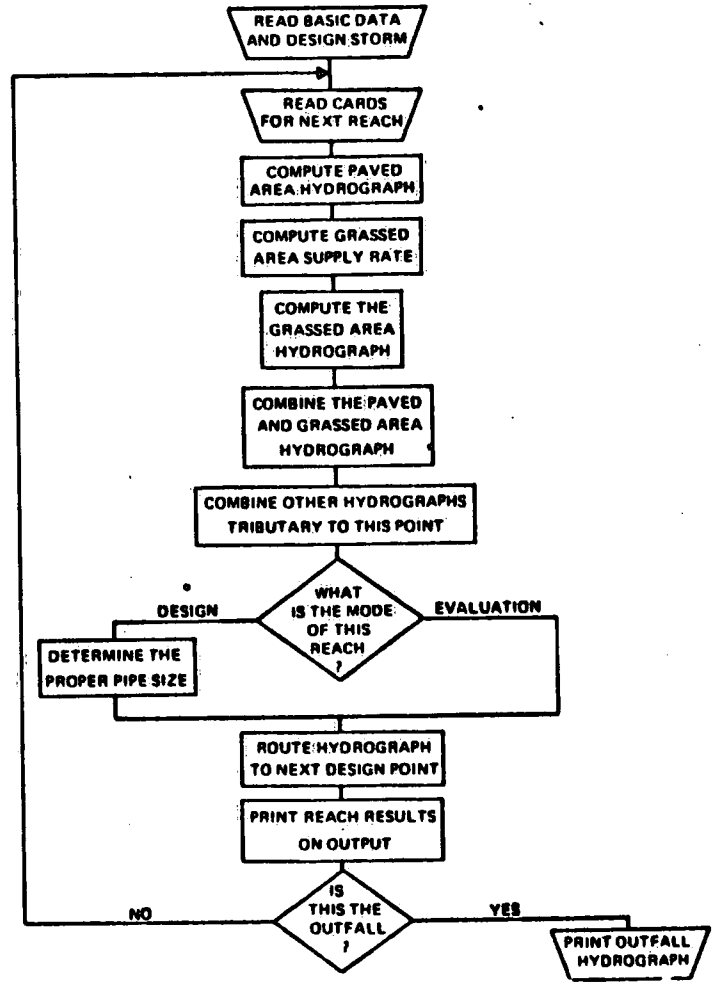


Fig.9. Flowchart for ILLUDAS (Terstriep and Stall, 1974).

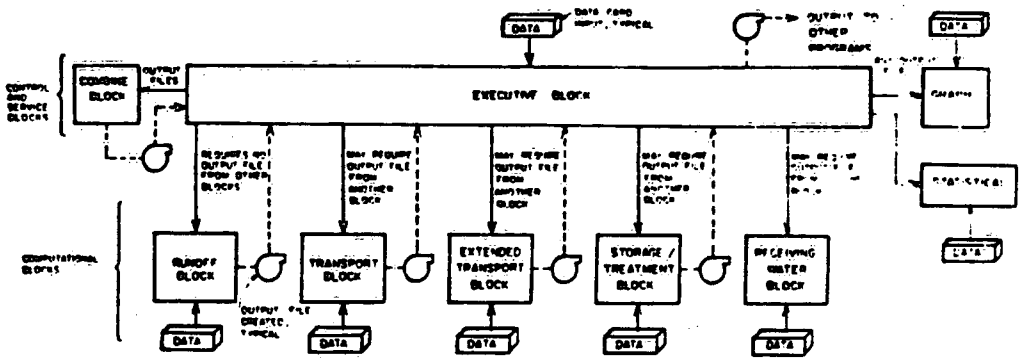


Fig.10. Relationship of Executive Block with Other SWMM Blocks (Huber et al., 1982).

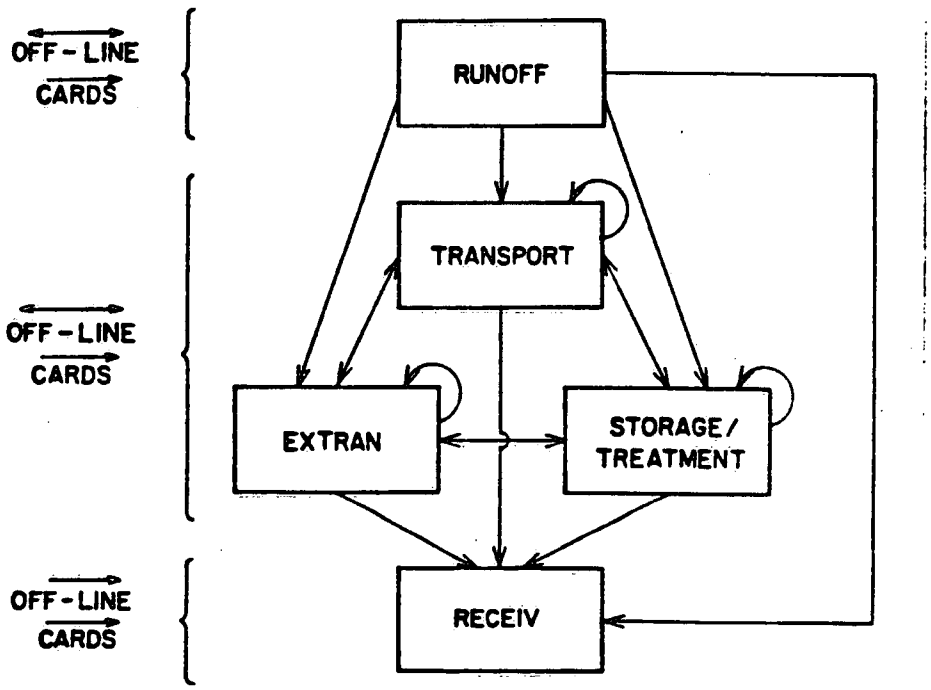


Fig.11. Overview of SWMM Structure, Indicating Linkages Among the Five Computational Blocks (Huber et al, 1982).