Marsalek, J

Environment Canada | Environnement Canada

National Water Research Institute | Institut National de Recherche sur les Eaux

CONTRIBUTIONS TO THE DESIGN FLOOD GUIDE FOR CANADA

by

J. Marsalek[1] and G.G. Patry[2]

Inland Waters Directorate | Direction Générale des Eaux Intérieures

Marsalek (65)

Study 83-330

This manuscript has been submitted to
The Editorial Board for Design Flood Guide for Canada
for inclusion and publication in the
Design Flood Guide for Canada
Chapter 7 - Urban Design Floods
and the contents are subject to change

This copy is to provide information prior to publication

CONTRIBUTIONS TO THE DESIGN FLOOD GUIDE FOR CANADA

by

J. Marsalek[1] and G.G. Patry[2]

[1]Environmental Hydraulics Section
Hydraulics Division
National Water Research Institute
Burlington, Ontario

[2] McMaster University
Hamilton, Ontario

January 1984

CHAPTER 7 - URBAN DESIGN FLOODS

Table of Contents

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. The mechanism of generation of urban floods and their analysis for design purposes are discussed in this chapter. Before doing so, however, the reasons for treating urban floods separately from floods in natural catchments are examined. In general, the separate approach seems to be justified by the special characteristics of urban catchments and the resulting need for special computational procedures and flood abatement measures.

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 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, conventional flood frequency techniques applied to 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 inter-relationships 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 10 to 20% of the gross rainfall on a natural watershed. 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 receiving waters generally deteriorates through surface pollutant washoff. However, the primary objective of urban flood control 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 displace and worsen flooding conditions dowstream, with massive and costly contruction of conveyance systems. In recent years, urban drainage has been addressed on a more scientific basis by carefully examining the mechanisms of surface runoff generation in urban areas, and by investigating means of reducing the rate and volume of runoff. 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 watersheds is typically many times more important than that 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 thereby leading to the dual drainage concept. The minor (pipe) drainage system is usually used to convey flows associated with the more common rainfall-runoff events. On the other hand, the major drainage system, consisting of roadways and surface channels, is generally used to route flows generated by the more unusual events. While both systems have always existed, it is only recently that engineers have incorporated 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..

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 the 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 watercourse has the right to drain his land to this watercourse. Such a right is lost, however, if the rainwater which would find its way by percolation or surface runoff to a natural watercourse 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 the federal law.

Provincial legislation related to urban drainage varies from province to province. Urban drainage is governed by provincial legislation which deals with environmental 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 generally include storm drainage policies; specification of design rainfall data; specifications 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 in Canada. Experience shows that although the average initial capital costs for drainage schemes with on-site controls 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 an 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 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 drainage alternatives which are derived from considerations of 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. Considering the dynamics of urban area development and changes in drainage technology, the master drainage plans should be regularly updated and modified, about once every 5 years.

The preparation of the master drainage plan 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 types of information may be required.

Site plans of the watershed, development, topography, watercourses, the present and proposed land use patterns, the proposed major drainage system, regional storm floodlines (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 watercourse crossings, details of watercourse and valley reaches, simulated flows at key points for 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 hydrometeorological 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-developement 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, water-courses, 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 above information needs to be determined for each of these subcatchments.

7.3.2 Process parameters In deterministic hydrological calculations, it is assumed that the relationships between many interactive factors affecting the water balance can be defined analytically and the numeric 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. Numeric 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 the 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 the 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.

<u>Depression storage</u> 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. 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 conveyance 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 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 shortcomings of such procedures is their empiricism and the fact that these methods yield only the peak flow and not the entire flow hydrograph.

Synthetic design storms 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 meteorologic 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 (1983). Design storms are best applicable to urban catchments of limited areas, significant imperviousness, and without large storage facilties. Under those circumstances, the use of point rainfall 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 than 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. Three types of temporal distributions are particularly popular in Canadian practice - the Atmospheric Environment Service (Pugsley, 1981), Chicago-type (Keifer and Chu, 1957), and SCS 24-hour distributions (U.S. Department of Agriculture, 1975). Whenever possible, the first distribution is preferred because it has 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. 1). In urban applications, it is recommended to use the 30-percentile probability curve (Hogg, 1982).

The Chicago-type distribution is based on the assumption that, for a particular return period, the design storm should contain all rainfall 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 some of its weaknesses can be remedied, it is inferior to the AES distribution. 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. Further developments of the AES distribution which are currently underway (Watt, 1983) will further limit the usefulness of the Chicago-type distribution in Canadian practice.

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. 2.

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 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 are the STORM and SWMM models which are described later in this chapter. Both these models require hourly precipitation data which are available on magnetic tape for many locations in Canada (see the Appendix). 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 records 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 evaluate 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 produced those flows. Such historical storms would be then 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 hydrograph 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 physiographic and hydrometeorological data. All four approaches 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 users' 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.

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:

$$Q_T = a\, A^b\, I^c\, S^d\, R_T^e\, \Phi^f \tag{1}$$

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 3 as a function of the percent impervious cover and the weighted main channel Manning 'n'. Coefficients a through f are given in Table 2 for all 60 cathments taken together, and for the 26 East coast catchments analysed seperately. Mean absolute errors in fitted flows ranged from 19 to 36% for the 26 East coast catchments, and from 30 to 34% for all 60 watersheds in the study.

7.4.2 Empirical Methods 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, $q_p=Q_p/A$, 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:

$$Q_p = C_u\, C\, I\, A \tag{2}$$

in which Q_p is the peak runoff in m^3/s (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.

The 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 frequency;
- 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 subcatchment.

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 of 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 (t_e) and the time of travel (t_f) in the pipe network:

$$t_c = t_e + t_f \qquad (3)$$

The inlet time varies with surface slope, rainfall intensity, 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 4 describes a convenient graphical estimate of overland and gutter flow times, while Table 3 summarizes some 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 coefficient 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.

<u>Runoff coefficient</u> 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 surface, 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 (1983) 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 waterseds.

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:

$$Q = [P-IA]^2/[P-IA+S] \tag{5}$$

and

$$S= [1000/CN] - 10 \tag{6}$$

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:

$$CN = \frac{100}{1+P}\left\{0.5 - C\left[\left(\frac{1.25}{C}+1\right)^{1/2} - 1\right]\right\} \tag{7}$$

7.4.3 Synthetic Hydrograph 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, depression storage, and infiltration losses. In urban areas, interception losses are usually not very significant and are traditionally neglected or lumped with other rainfall losses without any serious effect 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:

$$V_d(t) = S_d \, [1 - e^{-k \, P_e(t)}] \tag{8}$$

in which $V_d(t)$ 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 are listed in Table 5.

Many of the more popular rainfall-runoff models make use of the initial storage depression model, in which the total depression storage 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 storage is zero, i.e, immediate surface runoff. The initial storage 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 a 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:

$$f(t) = f_c + (f_0 - f_c)\, e^{-kt} \tag{9}$$

in which f_0 and f_c are the initial and final infiltration rates in 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_0 (Terstriep and Stall, 1974). Great care should be exercised in specifying the time variable in Horton's model. Only if the rainfall intensity is consistently greater than the potential infiltration rate will t represent the time from the beginning of the storm event. Whenever 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.

Synthetic Hydrograph Computation 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 hydrograph methods differ from one another in their definition of the D-hour unit hydrograph ordinates. However, all D-hour unit hydrograh 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 5, has a peak runoff rate given by:

$$Q_p = 7.0\ C_p\ A\ /\ T_l \tag{11}$$

in which Q_p is the peak flow in m^3/s; A is the catchment area in km^2; C_p is a coefficient ranging from 0.59 to 0.69; and T_l 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 5, 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 3. 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 <u>Simulation Models</u> 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 by providing a more elaborate description of the physics involved from input (rainfall) to output (runoff), the complexity of the natural processes 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 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 6 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 effluents on the receiving water body. A flowchart of the model is provided in Figure 7. STORM also handles non urban catchments, snowpack accumulation, snowmelt, 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 Holtan's conceptual infiltration model. Design or evaluation modes can be specified for any reach in a network. The latest version (1978) of ILLUDAS provides for both hydrologic and hydraulic channel routing options. A simplified flowchart of the model is provided in Figure 8. A major drawback of the ILLUDAS model 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.

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 9. The linkage among the five computational blocks is described in Figure 10. 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.

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 particularly 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 Brandstetter (1976), Brandstetter 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 different from those used in calibration. Finally, validation is concerned with the actual performance of the model in practice (ASCE, 1982). Until the model can be independently verified, the model remains strictly hypothetical. A calibration/verification analysis is typically conducted by splitting the data base in half, using one set of rainfall-runoff events for calibration, the other for verification. Care should be taken to insure 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 based on a group of high frequency events, if the immediate objective is to evaluate runoff conditions for low frequency events.

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 it is to be used, 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 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 raised by McPherson. 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 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.

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 has 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 systems - 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 detailled 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), 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).

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 and abatement is facilitated by progressive drainage criteria, guidelines and policies which have been proposed or adopted by some municipalities and provinces. Further improvement can be expected as more jurisdictions will follow this practice.

To illustrate the current practices in urban flood analysis, several case studies have been selected and are presented in the next section.

7.6 References

AMERICAN SOCIETY OF CIVIL ENGINEERS, 1969. Design and Construction of Sanitary and Storm Sewers. Manual of Practice No. 37, New York, N.Y., 382 p.

AMERICAN SOCIETY OF CIVIL ENGINEERS, 1982. Modeling hydraulic phenomena. ASCE, Journal of the Hydraulics Division, 108(HY7), pp. 845-852.

ARON, G., and WHITE, E.L., 1982. Fitting a gamma distribution over a synthetic unit hydrograph. Water Resources Bulletin, 18(1), pp. 95-98.

BRANDSTETTER, A., 1976. Assessment of mathematical models for storm and combined sewer management. EPA-600/2-76-175a, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohio, 530 p.

BRANDSTETTER, A., FIELD, R., and TORNO, H.C., 1976. Evaluation of mathematical models for the simulation of time-varying runoff and water quality in storm and combined sewerage systems. In: Proceedings of the Conference on Environmental Simulation, April 19-22, EPA-600/9-76-016, Office of Research and Development, Washington, D.C., pp. 548-552.

DELLEUR, J.W. and DENDROU, S.A., 1980. Modeling the runoff process in urban areas. Critical Review in Environmental Control, 10, pp. 1-68.

DENDROU, S.A., 1982. Overview of urban runoff models. Chapter 7. IN: Urban Stormwater Hydrology. Kibler, D.F. (Editor), American Geophysical Union Monograph No. 7, Washington, D.C., 271 p.

ESPEY, W.H. and WINSLOW, D.E., 1974. Urban flood frequency characteristics. ASCE Journal of the Hydraulics Division, 100(HY2), pp. 279-291.

HAWKINS, R.H., 1978. Runoff curve numbers with varying soil moisture. ASCE Journal of Irrigation and Drainage Division, 104(IR4), pp. 389-398.

HOGG, W.D., 1982. Distribution of rainfall with time: design consideration. Paper presented at the AGU Chapman Conference on Rainfall Rates, Urbana, Illinois, April 27-29.

HORTON, R.E., 1933. The role of infiltration in the hydrologic cycle. Trans. of the Amer. Geoph. Union, 14, pp. 446-460.

HUBER, W.C., HEANEY, J.P., NIX, S.J., DICKSON, R.E., and POLMAN, D.J., 1982. Storm water management model - users' manual, version III. Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohio.

KEIFER, C.J. and CHU, H.H., 1957. Synthetic storm pattern for drainage design. ASCE Journal of the Hydraulics Division, 83(HY4), pp. 1332-1 - 1332-25.

KIBLER, D.F. (Editor), 1982. Urban Stormwater Hydrology. American Geophysical Union Monograph No. 7, Washington, D.C., 271 p.

LAGER, J.A., DIDRIKSON, T., and OTTE, G.B., 1976. Development and application of a simplified stormwater management model. EPA-600/2-76-218, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohio, 153 pages.

LEE, K.B., 1981. Versatile stormwater quantity and management model. Master of Engineering Thesis, McMaster University, Hamilton, Ontario.

LINSLEY, R.K., KOHLER, M.A., and PAULHUS, J.L.H., 1982. Hydrology for engineers. Third Edition. McGraw-Hill Book Company, New York, N.Y., 508 p.

MARSALEK, J. and WATT, W.E., 1983. Design storms for urban drainage design. Proceedings of the 6th Canadian Hydrotechnical Conference, Ottawa, June 2-3, pp. 953-978.

McCUEN, R.H., 1982. A guide to hydrologic analysis using SCS methods. Prentice Hall, Inc., Englewood Cliffs, N.J., 145 p.

McPHERSON, M.B., 1969. Some notes on the rational method of storm drain design. Technical Memorandum No. 6, ASCE Urban Water Resources Research Program, New York, N.Y., 74 p.

McPHERSON, M.B., 1979. Overview of urban runoff tools of analysis. Report No. GREMU-79/01. Ecole Polytechnique de Montréal, Montréal, Québec, 31 p.

MEDINA Jr., M.A., 1979. Level III receiving water quality modeling for urban stormwater management. EPA-600/2-79-100. Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, Ohio, 217 pages.

MINISTRY OF THE ENVIRONMENT, 1983. Urban drainage design guidelines (discussion draft). Toronto, Ontario.

PUGSLEY, W.I. (Editor), 1981. Flood hydrology guide for Canada: hydrometeorological design techniques. Publication No. CL13-81, Atmospheric Environment Service, Environment Canada, Downsview Ontario, 102 p.

ROESNER, L.A., GIGUERE, P.R., and DAVIS, L.C., 1977. User's manual for the storm runoff quality model RUNQUAL. Prepared for the Southeast Michigan Council of Governments, Detroit, Michigan.

SHEAFFER, J.R., WRIGHT, K.R., TAGGART, W.C., and WRIGHT, R.M., 1982. Urban Storm Drainage Management, Marcel Dekker, Inc., New York, N.Y., 271 p.

SHUBINSKI, R.P., KNEPP, A.J., and BRISTOL, C.R., 1977. Computer program documentation for the continuous storm runoff model SEM-STORM. Report to the Souheast Michigan Council of Governments, Detroit, Michigan.

SCHROETER, H.O. and WATT, W.E., 1983. Practical simulation of sediment transport in urban watersheds. Proceedings, 10th International Symposium on Urban Hydrology, Hydraulics, and Sediment Control, Lexington, Kentucky.

SMITH, A.A. and FALCONE, J., 1983. MIDUSS - A design program for stormwater sewerage. Department of Civil Engineering, McMaster University, Hamilton, Ontario.

SMITH, A.A. and LEE, K.B., 1983. The rational method revisited. Department of Civil Engineering, McMaster University, Hamilton, Ontario.

TERSTRIEP, M.L. and STALL, J.B., 1974. The Illinois drainage area simulator, ILLUDAS. Bulletin 58, Illinois State Water Survey, Urbana, Illinois.

U.S. ARMY, CORPS OF ENGINEERS, 1976. Storage, treatment, overflow, runoff, model, "STORM". Hydrologic Engineering Center, Davis, California.

U.S. ARMY, CORPS OF ENGINEERS, 1977. Guidelines for calibration and application of STORM. Training Document No. 8, Hydrologic Engineering Center, Davis, California

U.S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE, 1972. National Engineering Handboog, Section 4, Hydrology. Washington, D.C.

U.S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE, 1975. Urban hydrology for small watersheds. Technical Release No. 55, Engineering Division, Soil Conservation Service, U.S. Department of Agriculture, Washington, D.C.

VIESSMAN, Jr., W., KNAPP, J.W., LEWIS, G.L., and HARBAUGH, T.E., 1977. Introduction to hydrology. Second Edition. Harper and Row, Publishers, Inc., New York, N.Y., 704 p.

WATT, W.E., 1983. Personal communication.

WHIPPLE, W., GRIGG, N.S., GRIZZARD, T., RANDALL, C.W., SHUBINSKI, R.P., and TUCKER, L.S., 1983. Stormwater management in urbanizing areas. Prentice Hall, Inc., Englewood Cliffs, N.J., p. 234.

WISNER, P.E. and P'NG, C.E., 1982. OTTHYMO - a planning model for master drainage plans in urban areas. In: Urban Drainage Systems. Pitman Advanced Publishing Program, London, pp. 4.3-4.17.

WISNER, P.E., 1983. Hierarchical approach to urban hydrological modeling: the IMPSWMM methodology. Proceedings, 10th International Symposium on Urban Hydrology, Hydraulics, Sediment Control, Lexington, Kentucky.

WISNER, P.E., 1983. IMPSWMM urban drainage modelling procedures. Second Edition. Faculty of Science and Engineering, University of Ottawa, Ottawa, Ontario.

LIST OF 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 obtained 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	c	d	e	f
			Model Parameters			
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	c	d	e	f
			Model Parameters			
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 $\leq$ 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

Slope		Runoff coefficient: 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).

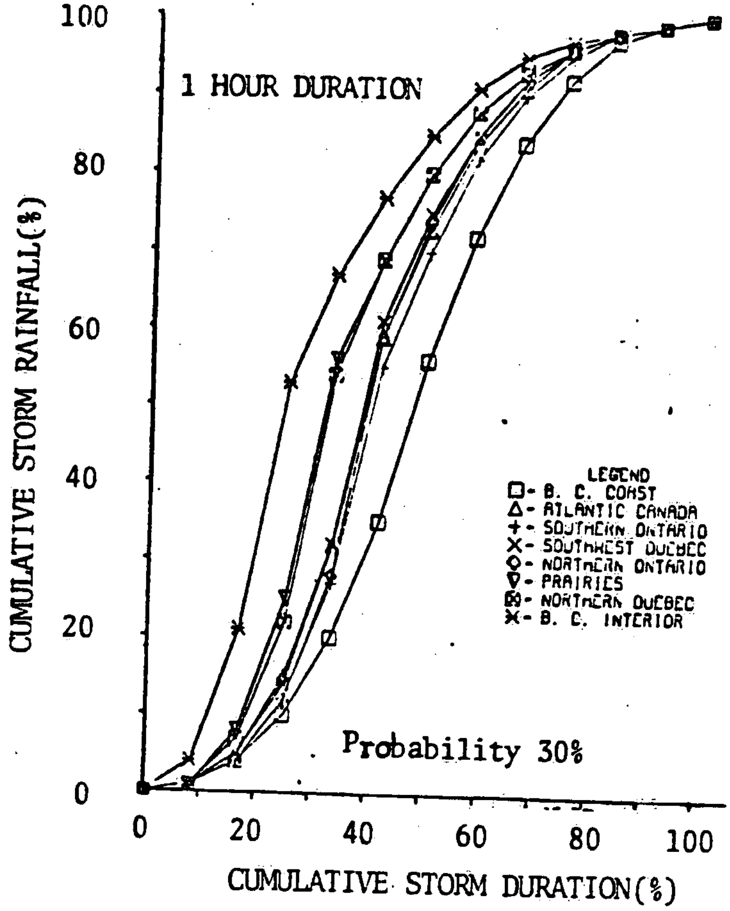
Equations	Total Explained Variation
$T_R = 3.1\ L^{0.23} S^{-0.25} I^{-0.18} \phi^{1.57}$	0.802
$Q = 31.62 \times 10^3\ A^{0.96} T_R^{-1.07}$	0.936
$T_B = 125.89 \times 10^3\ A\ Q^{-0.95}$	0.844
$W_{50} = 16.22 \times 10^3\ A^{0.93}\ Q^{-0.92}$	0.943
$W_{75} = 3.24 \times 10^3\ A^{0.79}\ Q^{-0.78}$	0.834

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	

LIST OF FIGURES



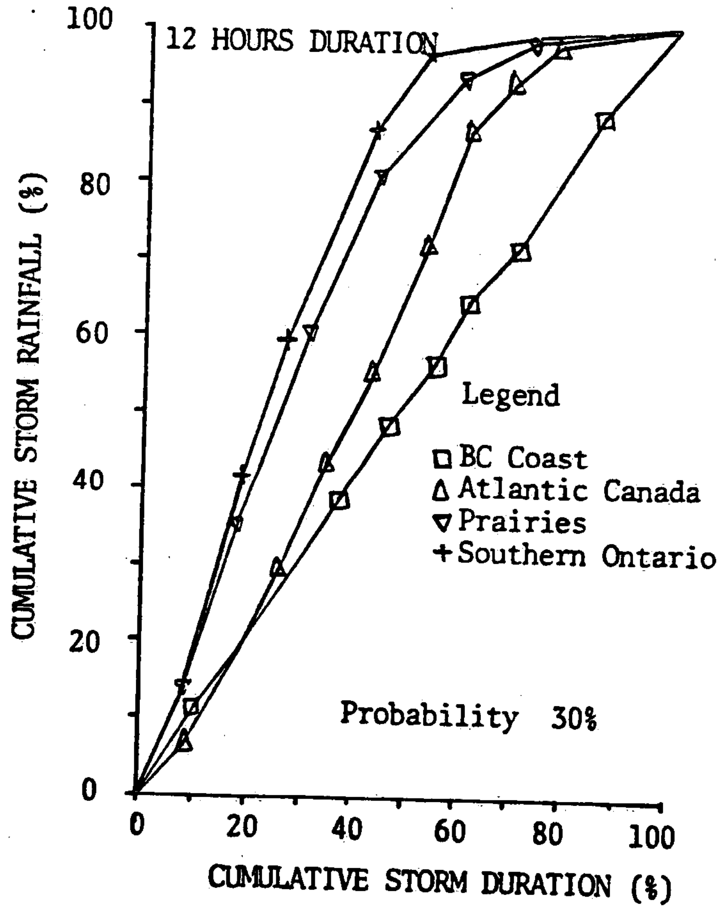


Fig.1. AES Temporal Storm Distributions for Durations 1 and 12 Hours

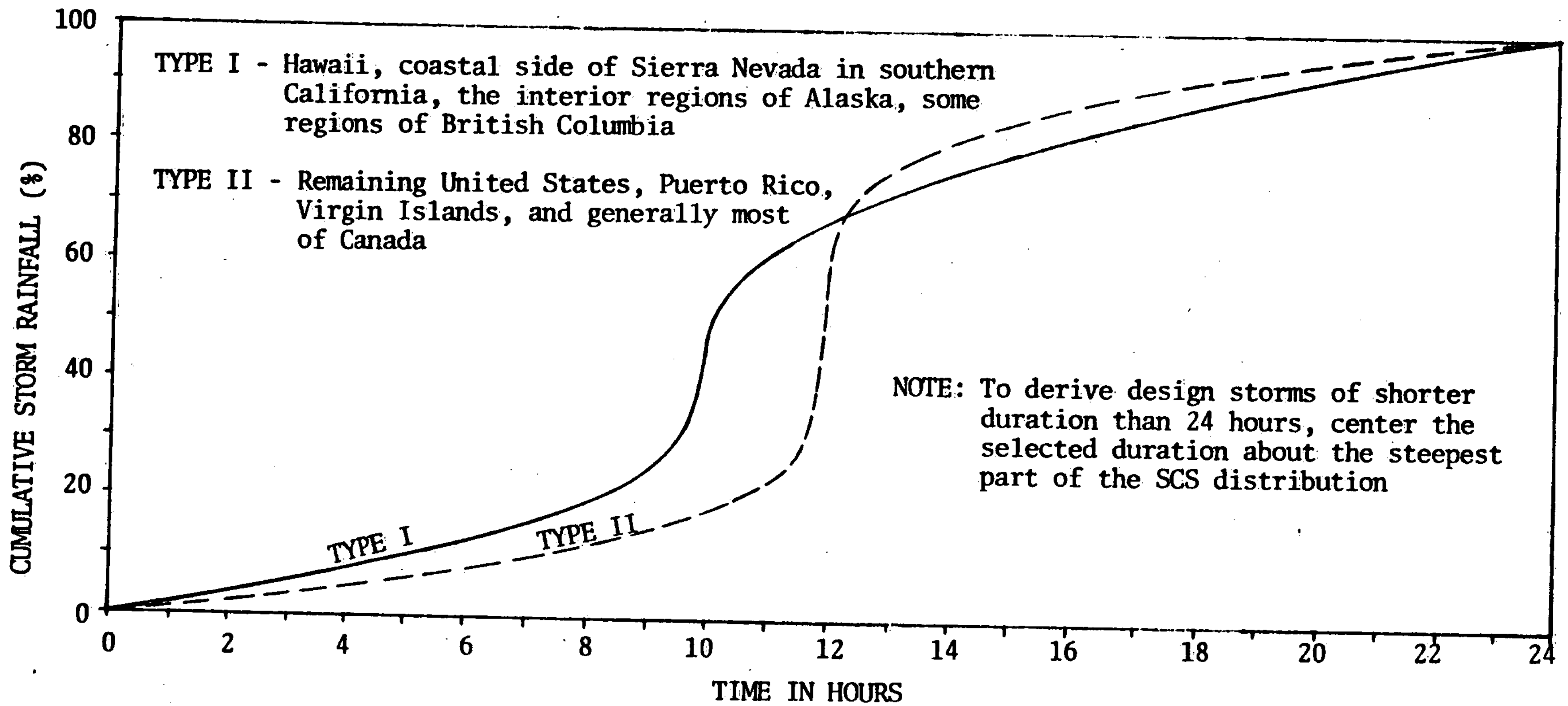


Fig.2. SCS Twenty-Four Hour Rainfall Distributions

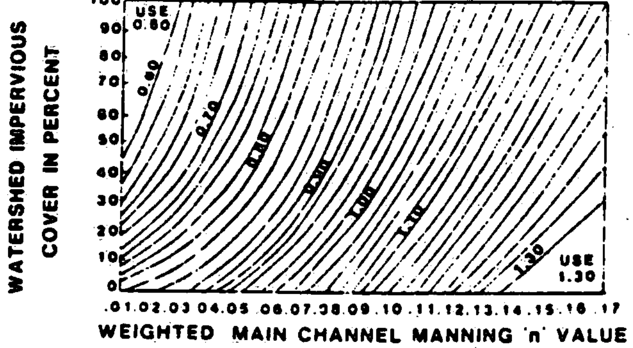


FIGURE 3. Watershed Conveyance Factor, Φ, as a Function of Percent of Impervious Cover and Weighted Mean Channel Manning 'n' Value (Whipple, et al., 1983).

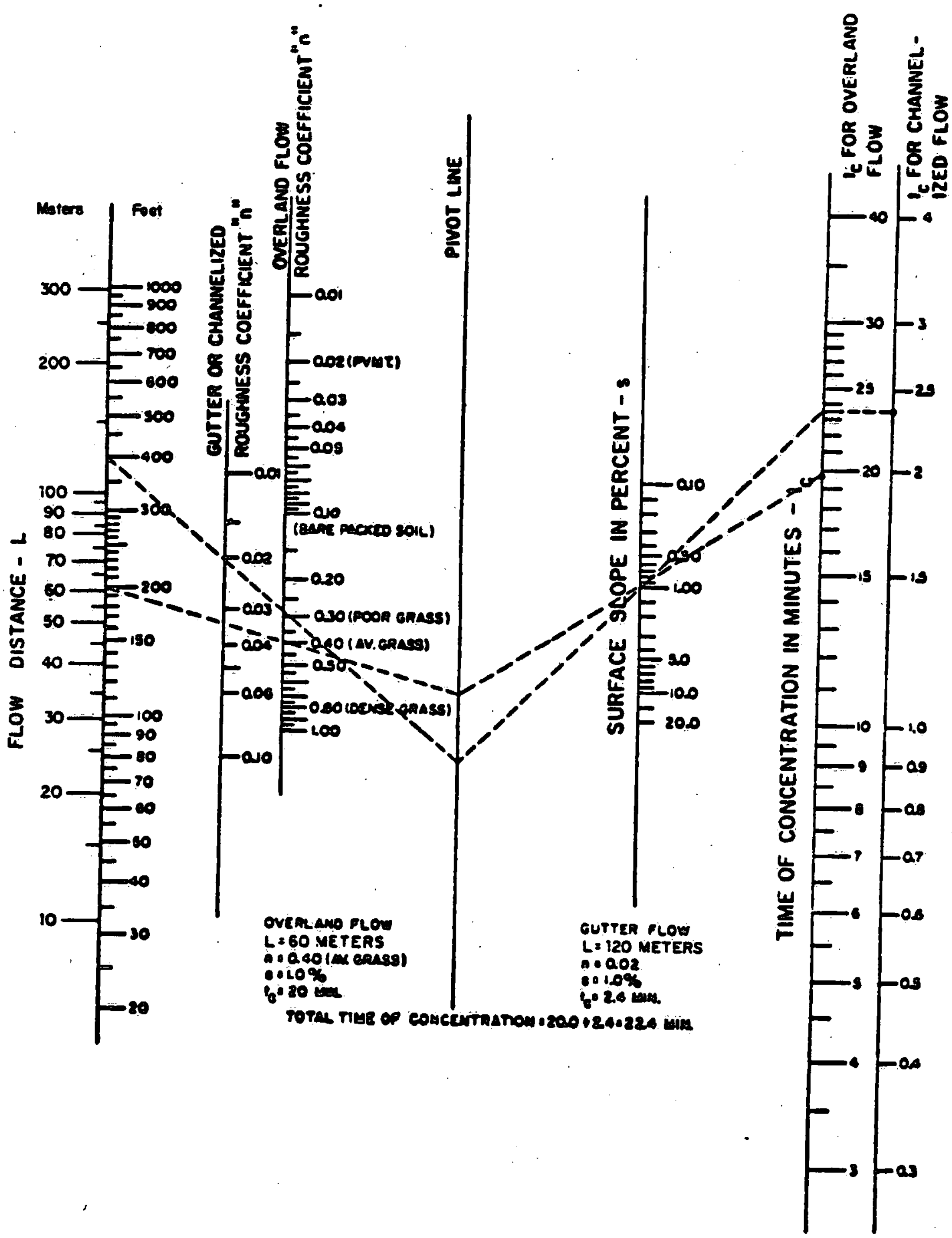


FIGURE 4. Nomograph for the Time of Concentration (Sheaffer et al., 1982).

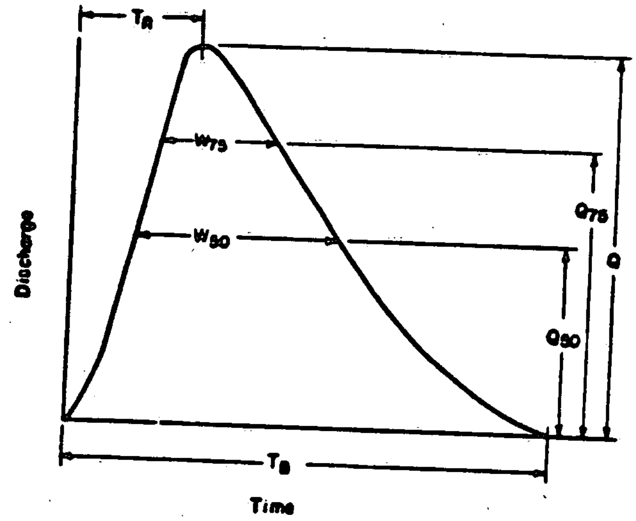


FIGURE 5. Definition of Espey 10-min. Unit Hydrograph Parameters.

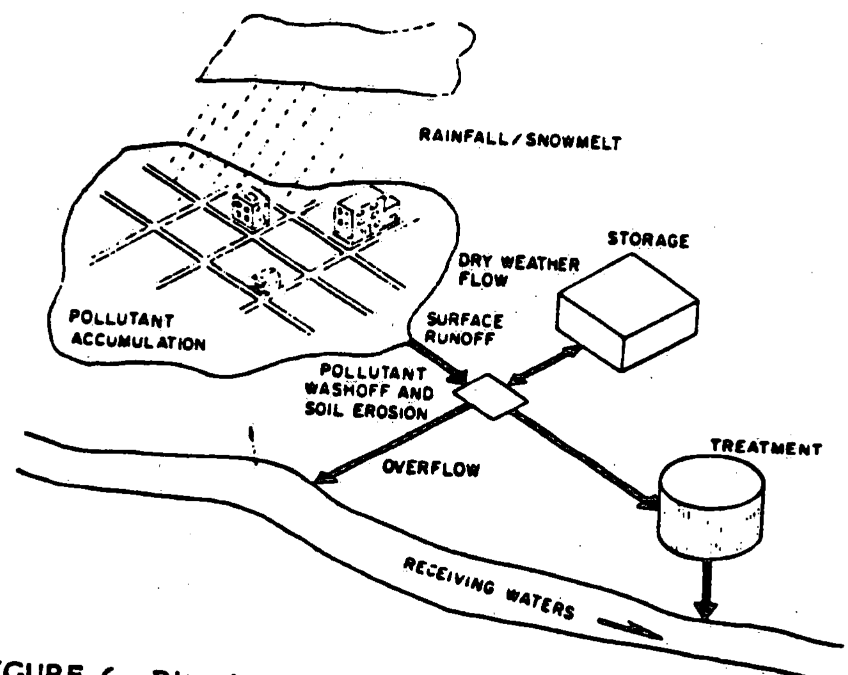


FIGURE 6. Planning Level Basin Modelling Conceptualization (U.S. Army Corps of Engineers, 1976).

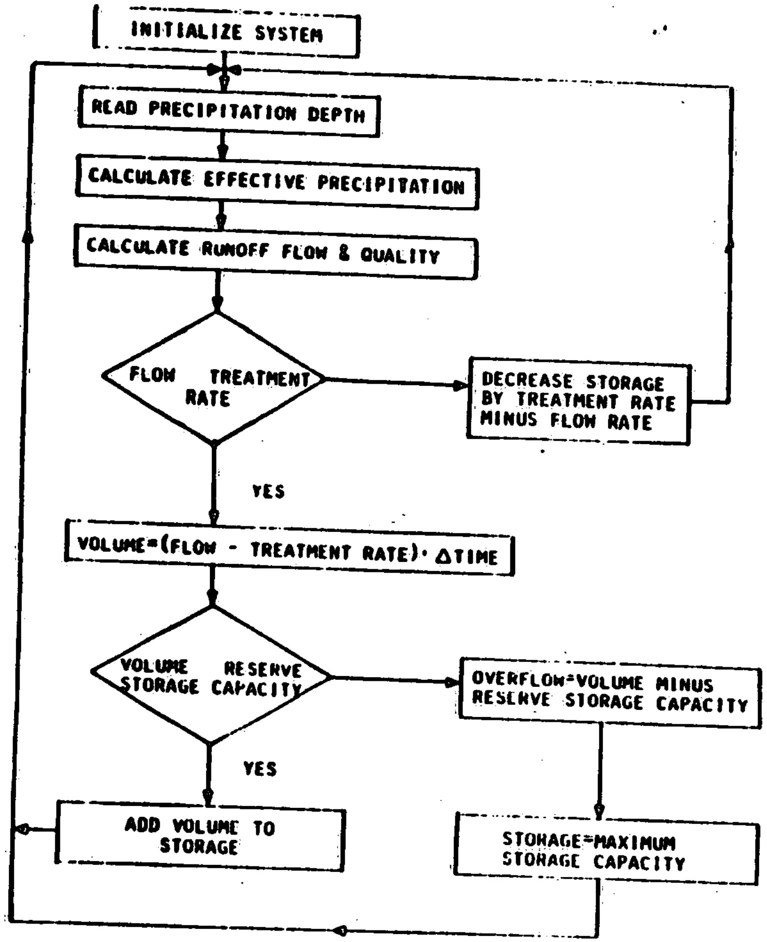


FIGURE 7. Simplified Flowchart of STORM (U.S. Army Corps of Engineers, 1976).

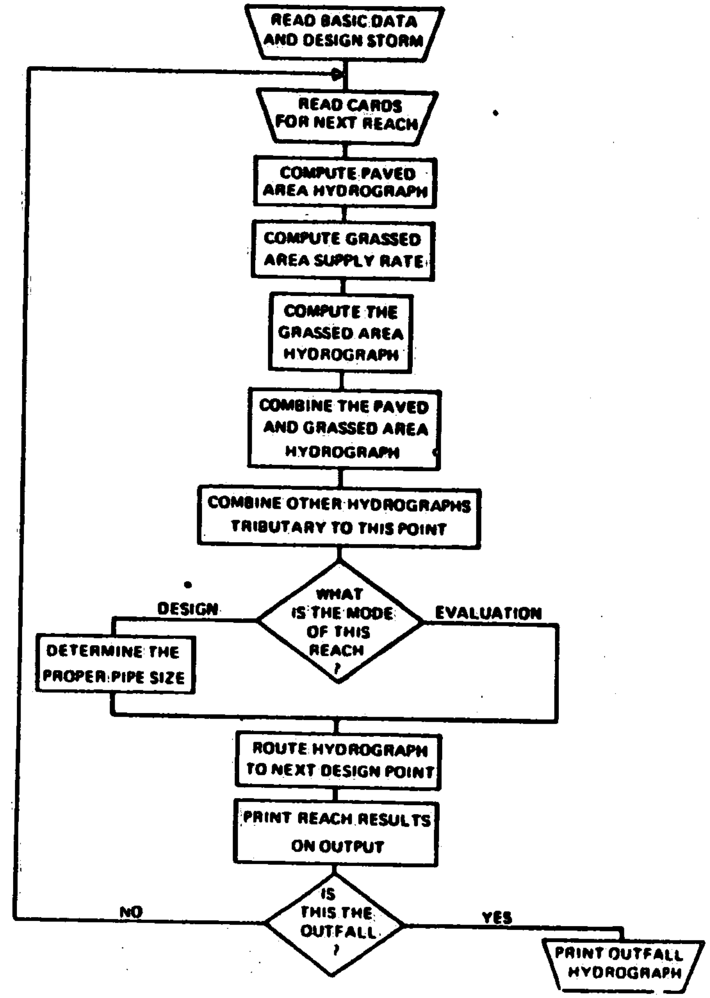


FIGURE 8. Flowchart for ILLUDAS (Terstriep and Stall, 1974).

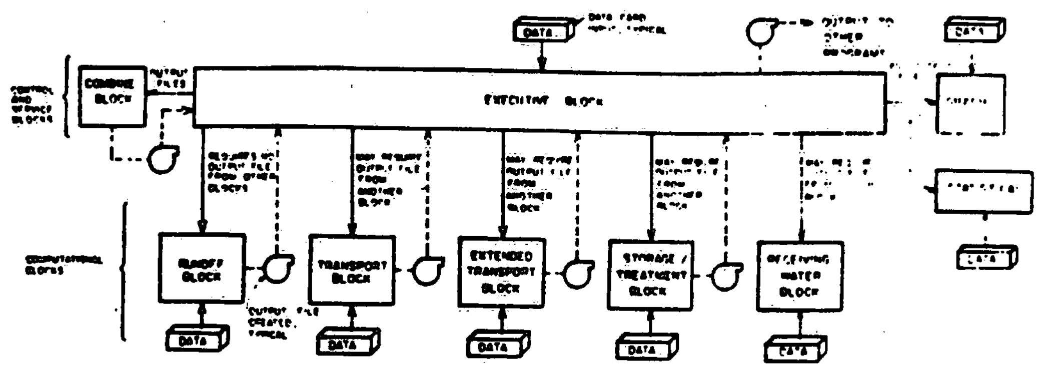


FIGURE 9. Relationship of Executive Block with Other SWMM Blocks (Huber et al., 1982).

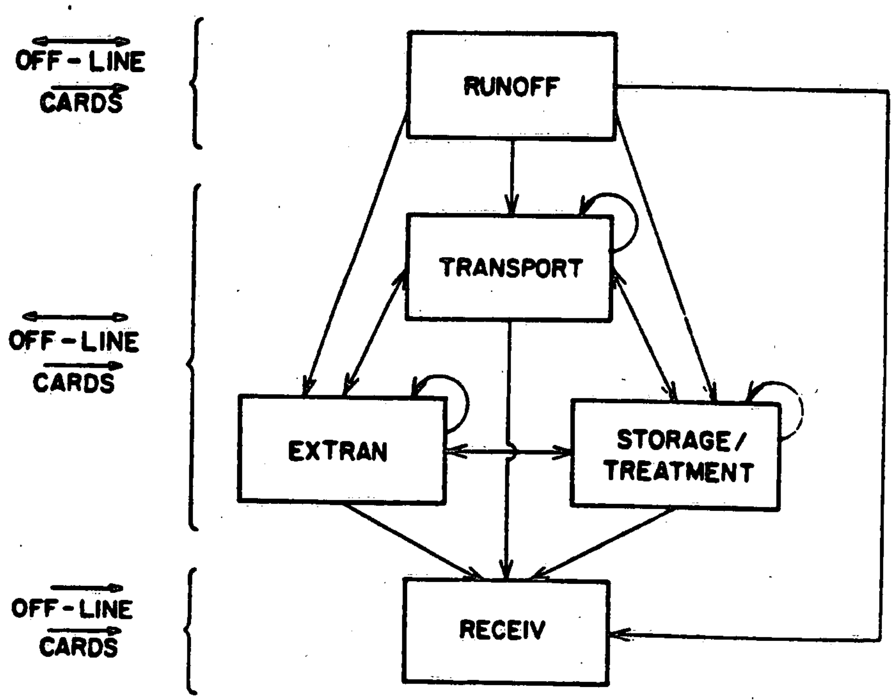


FIGURE 10. Overview of SWMM Structure, Indicating Linkages Among the Five Computational Blocks (Huber et al, 1982).

15912