Marsalek

Environment Canada / Environnement Canada

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

DESIGN STORMS FOR URBAN DRAINAGE DESIGN

By

J. Marsalek[1] and W. E. Watt[2]

This manuscript has been submitted to the
Proceedings of the 6th Canadian Hydrotechnical
Conference, June 1983 for publication
and the contents are subject to change.

This copy is to provide information
prior to publication.

DESIGN STORMS FOR URBAN DRAINAGE DESIGN

By

J. Marsalek[1] and W. E. Watt[2]

[2] Dept. of Civil Engineering
Queen's University
Kingston, Ontario

[1] Environmental Hydraulics Section
Hydraulics Division
National Water Research Institute
Canada Centre for Inland Waters

January 1983

DESIGN STORMS FOR URBAN DRAINAGE DESIGN

J. Marsalek[1] and W.E. Watt[2]

SYNOPSIS

The design storm concept is well established in Canadian urban drainage practice, although some confusion results from an incomplete definition of design storms and their applications. To remedy this situation, it is recommended that design storms be developed for a wide range of return periods; these storms should be based on local AES rainfall data, given for both the rational method and hydrograph model applications, and supplemented by specifications of the computational procedure and normal antecedent conditions. The design storm hyetograph would be defined for a particular region, return period and catchment time constant and, when applied in conjunction with the specified computational procedure and antecedent conditions, would produce a peak flow of approximately the same return period as that of the design storm. None of the existing design storms has all these features but further work on temporal distributions and antecedent condtions would result in an acceptable set of design storms.

KEYWORDS

Design storms, urban drainage, stormwater, hydrological design; precipitation; runoff computations;

---

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

[2] Department of Civil Engineering, Queen's University, Kingston, Ontario

SOMMAIRE

Le concept de l'averse nominale est bien établi dans le domaine du drainage urbain au Canada, même si la définition incomplète du terme et de ses applications entraîne quelque confusion. En guise de remède à cette situation, il est recommandé que les averses nominales soient établies pour toute une gamme de périodes de récurrence, qu'elles s'appuient sur les données locales du SEA, qu'elles comportent les spécifications nécessaires à l'application de la méthode rationnelle et des modèles de ruissellement urbain, qu'elles s'accompagnent des spécifications de la méthode de calcul et des conditions antérieures normales. L'hyétogramme de l'averse nominale serait établi pour une région particulière, une période de récurrence précise et selon la constante de temps du bassin versant. S'il était appliqué concurremment avec la méthode de calcul spécifiée et les conditions antérieures, il donnerait un débit maximal ayant à peu près la même période de récurrence que celle de l'averse nominale. Même s'il n'existe à l'heure actuelle aucune averse nominale comportant toutes les caractéristiques précitées, il est possible d'y apporter d'autres perfectionnements. Plus particulièrement, il y a lieu d'approfondir l'étude des distributions temporelles et des effets des conditions antérieures.

INTRODUCTION

The arbitrary specification of a precipitation input for design purposes is well-established as a criterion for the design of hydrotechnical structures. Early examples were the 'standard project storm' and the 'probable maximum precipitation' in the United States and more recently Hurricane Hazel and the Timmins rain (McMullen 1962) in Ontario. Often the specification of precipitation input was linked implicitly to a specification of antecedent soil moisture conditions and a computational technique to determine the design flood. In the area of urban storm drainage, a common example is the specification of a rainfall input of given return period and inlet time together with the specification of a computational technique, the rational method, and an implicit specification of antecedent soil moisture conditions which is included in the runoff coefficient. The advent of stormwater modelling techniques has spawned a plethora of so-called urban design storms but to date none has received the acceptance of the early examples listed above.

Although this paper is restricted to urban design storms, there is no logical reason to adopt a definition different from that accepted for design storms on larger, natural drainage basins. Therefore, a design storm, or more precisely a design storm event, will be defined as a somewhat arbitrary specification of the temporal distribution of a precipitation input, together with the specification of antecedent soil moisture conditions and a computational technique to determine design flows.

Although simple in concept, the subject of urban design storms is somewhat controversial. Much of this controversy arises from two sources: a lack of accurate definition of design storms and confused thinking regarding the areas of application. As a result of these factors, the design storm concept has been misused. In this paper, attempts will be made to clarify the areas of application of design storms, to critically review design storms in common practice and to recommend urban design storms for Canada.

Before doing this, however, consider whether the design storm concept should be employed at all. The justifications for using design storms are as follows.

(i) There is no network of urban hydrometric stations and so clearly there can be no frequency analysis of observed flows and resulting selection of a design flow on the basis of either economic efficiency, specified risk and service period, or specified return period.

(ii) The cost of many installations does not justify an extensive simulation from historical precipitation data to yield simulated flows and further analysis as in (i).

(iii) The fact of local jurisdiction and the perception of a uniform level of protection as an objective preferable to economic efficiency lead to the specification of an input and a method of computation in urban drainage design.

(iv) The arbitrary specification of a design storm event (in terms of the rational method) is well accepted in practice and will continue to be used by practitioners.

(v) The specification of design storms requires minimal resources in terms of time and money.

(vi) The specification of a design storm minimizes misunderstandings on the parts of both the client and the approval agency.

In summary, for a large portion of urban storm drainage structures, the design storm concept has been and will continue to be used. Clearly, the 'best' design storm should be selected for these purposes. At the same time, however, there is a significant number of cases where the design storm concept is not appropriate -- just as there are such cases for natural drainage basins. It is counterproductive to attack the design storm concept on the basis that it does not apply in these cases. Rather, alternative approaches should be developed but a discussion of these alternative approaches is beyond the scope of this paper.

DESIGN STORM CONSIDERATIONS

Range of Practical Applications

The most common application of design storms is the calculation of peak flows for the design of hydraulic transmission elements such as gutters and sewers. This is perhaps the best application of conventional design storms if the catchment is largely impervious so that the impervious areas dominate the peak flow and the contributions from pervious areas are of secondary importance. For significant contributions from pervious areas (i.e. largely pervious catchments for all return periods or largely impervious catchments for high return periods) the reliability of conventional design storms is reduced because antecedent conditions and a computational method are not specified. Obviously, the largest uncertainties will occur in the case of undeveloped catchments for rainfalls of low to moderate return period.

Design storms are sometimes applied to the calculation of storage for the design of hydraulic storage elements. In general, such applications are outside the accepted range and an alternate approach is required.

Design storms are also occasionally applied to the design of hydraulic elements for quality control and/or treatment. Such applications are also outside the accepted range. The list of inputs to be specified now includes such items as pollutant accumulation and street sweeping frequency, etc. Moreover, a precipitation input which produces a peak flow suitable for sewer design is not guaranteed to produce water quality conditions which can be used for design. Clearly, an alternative approach is required.

Design Storm Characteristics

In a general case, a precipitation event is defined by its return period, its total depth, its temporal distribution at a point, its spatial characteristics including average spatial distribution, storm movement and spatial development and decay. The temporal distribution may be summarized by the following statistics: storm duration, peak intensity, and time of peak intensity all in terms of the discretization interval for the event. The relative importance of each of these factors varies with the application of the design storm and with the catchment characteristics.

Design Return Period (T)

Ideally, a design return period should be selected on the basis of economic efficiency, i.e. to minimize total cost-investment plus damage, so as to yield an optimal design. But, as noted by Watt and Marsalek (1977),

> "In practice, the economic and hydraulic analyses which are necessary to determine the optimal design have not been undertaken for urban hydraulic structures. What has occurred, in place of this concept of optimal design, is the development of a concept of a 'level of protection'. The selection of this level is made locally, often in the absence of any information on the relative construction or damage costs, and consequently, appears to be fairly arbitrary. In addition, the level of protection often applies to the exceedance probability of some precipitation event and not to the probability of failure of the hydraulic structure."

Total Precipitation Depth (d)

The total precipitation depth at a point is a function of T and the storm duration t_s. The Atmospheric Environment Service of Environment Canada (AES) publishes intensity-duration-frequency (IDF) summaries for over 500 recording gauge stations in Canada. For other sites, the value of d for any specified T and t_s can be determined by reference to depth-duration-frequency maps. These maps, prepared by AES, will be distributed to the public in 1983. Each map will show for a particular region in Canada isolines of the mean $\bar{d}$ and standard deviation s_d of the observed precipitation series corresponding to selected values of t_s. For a particular return period, the rainfall depth can be calculated from

$$(1) \qquad d_T = \bar{d} + K_T s_d$$

where K_T is the frequency factor which, for a given two-parameter probability distribution, depends only on the return period. The AES has determined that the extreme value type 1 or Gumbel distribution is appropriate for short duration rainfalls so that the expression for K_T is

$$(2) \qquad K_T = \frac{\sqrt{6}}{\pi} \left(-\ln - \ln (1 - 1/T) - 0.5772 \right)$$

Storm Duration (t_s)

The storm duration is an important factor which, as noted above, defines d for a given T. It also affects the storm intensity i and hence the resulting peak flows. The design storm duration depends on the catchment time constant which has been traditionally defined as the time of concentration, t_c. In its usual sense, t_c is not adequate as a design storm duration for a number of reasons. It is not a constant but varies with precipitation intensity and antecedent conditons. Its definition as the travel time from the most remote point ignores the relative runoff-producing capabilities of pervious and impervious areas. Nevertheless, there is a tendency in present practice to select the design storm duration as the time of concentration or longer. Some guidance on the selection of t_s is offered in a later section of this paper.

Temporal Precipitation Distribution (i(t))

The variation of precipitation intensity over the duration of the storm is an important factor in determining the timing and the magnitude of the peak flow. A realistic estimate of temporal distribution can be obtained only from an analysis of precipitation data from the recording gauge network. In Canada, this network is maintained by the AES for all of Canada except Québec and by Environnement Québec for that province. In practical terms, the maximum daily intensity is reported for discrete intervals from five minutes to 24 hours. Therefore, five minutes appears to be a suitable minimum interval that can be used for the discrete representation of design storms unless the original records are reanalyzed. The peak intensity and the time of occurrence of this peak are then defined in terms of this interval.

Storm Spatial Characteristics

These characteristics are important only for larger drainage basins. The average basin precipitation may be significantly lower than the point value because of the limited extent of storm cells (James and Drake 1980, Shtifter 1981). In this event, data must be obtained from the precipitation gauge network to estimate the ratio of basin average to point values usually as an average for the storm. In addition, for these larger basins, storm movement and storm development and decay may also affect runoff both in terms of magnitude and timing. These considerations are particularly relevant to the case of operation and/or control of large systems.

Catchment Characteristics

Important considerations are the total area of the catchment and whether there is an opportunity for runoff control; these considerations lead to four cases. If the area is sufficiently small that point precipitation values may be used and if no control is contemplated, then there is a good chance that the design storm concept can be used for design. For this case, important catchment characteristics are total area, degree of imperviousness, antecedent soil moisture and catchment time constant. As noted above, the catchment time constant governs the duration of the design storm which, with the return period, defines the total precipitation depth. This, together with the total area, degree of imperviousness and antecedent soil moisture determines the total volume of storm runoff. In order to determine the temporal distribution of this storm runoff at a point (i.e. a hydrograph), one further basin characteristic is required. This is a description of the drainage system including the distribution and linkages of watershed, channel and reservoir elements. The design storm can then be applied to this particular catchment for specified antecedent conditions and, using a particular computational procedure, the runoff hydrograph can be determined at any point in the system.

If the area is large but no control is contemplated, the design storm concept can still be used but point values of precipitation must be reduced to represent average values over the catchment.

For the two remaining cases where control is contemplated (regardless of catchment size), the conventional design storm concept may not be applicable because the objectives of the control (e.g. runoff quality control) may not be accomplished by structures or systems designed on the basis of this input. Clearly, these two cases are outside the range of practical applications of conventional design storms. The designer may have to develop special precipitation inputs or use an alternative approach, for example continuous simulation.

Effect of Computational Procedure

The selection of a design storm is obviously tied to the computational procedure and vice-versa. For example, in the case of a small area, largely impervious with no opportunity for storage, the rational method may be specified as the computational procedure. In this case, the appropriate design storm is a block of precipitation of uniform intensity of duration just long enough for equilibrium conditions to exist. Antecedent conditions must be specified explicitly in the runoff coefficient or by an infiltration capacity which together with the percentage of impervious area will define the runoff coefficient.

Other cases will require the use of a runoff simulation model which in turn requires a design storm of variable intensity which should be based on local data. The catchment response will be given by the simulation model provided that it has been properly calibrated for local conditions. Antecedent soil moisture conditions can be specified in terms of the loss submodels (e.g. depression storage and infiltration) assuming proper calibration. It must be emphasized that the simulation model parameters should be estimated for design conditions.

Design Storm Characteristics-Urban Runoff Quantity

For the most common and generally accepted application of urban design storms, the characteristics in Table 1 are identified as being necessary.

Table 1. Necessary Characteristics for a Design Storm

Characteristic	Symbol	Comment
Return period	T	
Storm duration	t_s	In intervals of Δt $\Delta t \geq 5$ minutes
Total depth of precipitation	d	For small t_s precipitation = rainfall
Temporal distribution of precipitation intensity	$i(t)$	"
Antecedent soil moisture conditions	θ_o	Often given in terms of C or f
Computational method	M	Either M = RM rational method or M = URM urban runoff model

OVERVIEW OF COMMON DESIGN STORMS

The design storms discussed in this section are applicable to the design of sewer systems without large detention facilities in small to intermediate catchments. Under such circumstances, the number of important storm characteristics is somewhat reduced and the development of the design storm may be simplified. In particular, because of the limited catchment area, storm spatial characteristics may be neglected.

A number of design storms have been proposed and described in the literature. Nine of these storms and their characteristics are listed in Table 2. Five of these storms are pertinent to Canadian drainage practice and only these storms are discussed here in more detail. It should be emphasized that these descriptions and discussions are based on the original references, and that other storm modifications, sometimes contradictory to the original author's intentions, may be available.

AES Design Storms

The AES design storm distributions were recently developed to encourage the use of Canadian data and to offer an alternative to the widely used storm distributions which are based on U.S. data.

Temporal storm distributions for 1 and 12 hour durations are available for 35 stations across Canada. The durations of 1 and 12 hours were selected to provide samples of both convective shower events as well as synoptic scale cyclonic circulation events. For the 1 hour duration, the analysis was based on 5-minute intervals. For the longer duration, hourly rainfall depths were considered. In addition to the storm rainfall depth, 1 and 5 day antecedent precipitation values are given. The rainfall temporal distributions are given as percentage rainfall vs. percentage time for various probability percentiles. For practical use, the 50 or 30 percentile curves are recommended. An example of the AES design storm distribution is shown in Fig. 1.

In an overall evaluation, the main advantages of the AES design storms follow from the fact that they are based on actual data and are available on a nationwide basis. Shortcomings include the lack of guidance for the selection of other storm characteristics (duration, rainfall depth), the limited availability for only two durations, and a possible neglect of the variability of the temporal distributions with return period. It appears, however, that such shortcomings could be overcome with a relatively small additional developmental effort. For example, the AES distributions could be applied in conjunction with locally derived storm durations and rainfall depths.

Chicago Design Storm

The Chicago design storm is perhaps the most widely used design storm in Canadian practice (McKelvie 1982). This storm was developed more than 25 years ago (Keifer and Chu 1957) to provide a rainfall input for applications of the Chicago hydrograph method to the design of sewers. Originally developed for a 5-year return period and a duration of 180 minutes, the Chicago storm distribution has since been applied to a wide range of durations and return periods (Bandyopadhyay 1972) and sometimes arbitrarily modified to reduce the peakedness of the storm. The Chicago storm has been widely incorporated in North American practice because it can be readily derived from available rainfall IDF relationships and partly because of limited alternative approaches. A contributing factor was undoubtedly its inclusion in a widely used handbook (Water Pollution Control Federation 1970). When the method was presented, it was criticized on the grounds that it retained too many of the fallacies and empiricisms inherent in the

Table 2. Basic Characteristics of Selected Design Storms

	Storm Return Period	Recommended Storm Duration	Total Storm Rainfall Depth	Temporal Distribution	Primary Intended Application	Computational Method	Antecedent Conditions Specified	Reference
A E S	User specified	1 and 12 hours	$i_{av} \times t_s$	Tabulated for both storms	Urban hydrological design	Not Specified	No, but antecedent prec. listed	Hogg 1980
Chicago	5 Years (or User specified)	3 hours (or t_c)	$i_{av} \times t_s$	Described by eq.(5)	Sewer sizing	Chicago Hydrograph	No	Keifer and Chu 1957
Desbordes	User specified	4 hours	$i_{av} \times t_s$	Derived by analysis of actual data	Sewer sizing	Linear Reservoir Model	No, but antecedent prec. analyzed	Desbordes 1978
Flood Studies Report	User specified	$2t_c - 3t_c$	$i_{av} \times t_s$	Tabulated	Flood studies, rural basins	Unit Hydrograph	Yes	Natural Environment Research Council 1975
Hamburg	User specified	6 hours	$i_{av} \times t_s$	Statistically defined	Planning & Design of combined sewers.	Version of SWMM	No	Abraham et al 1976
ISWS	User specified	1 hour	$i_{av} \times t_s$	Tabulated	Sewer sizing in Illinois	ILLUDAS	No	Terstriep and Stall 1974
Packman & Kidd	User specified	User specified	$i_{av} \times t_s$	Flood Stud. Report, 50 percentile	Hydrol. design, urban & rural	Wallingford Model	Yes	Packman and Kidd 1980
SCS	Max. Probable (or User spec.)	6 hours or more	Given in Maps	Tabulated	Small dam design	SCS Hydrograph Analysis	Yes	SCS 1975
Uniform Intensity	User specified	t_c	$i \times t_s$	Uniform	Sewer sizing	Rational Method	No	Water Pollution Control Federation 1970

rational method to recommend its adoption for general use (McPherson 1958).

In an attempt to preserve correspondence with actual rainfall events, the Chicago storm method takes into account the maximum rainfalls of individual durations, the average amount of rainfall antecedent to the peak intensity, and the relative timing of the peak intensity. The first step in applying the method is determination of the time antecedent to the peak intensity, expressed as a dimensionless ratio. This ratio, r , which divides the hyetograph into two parts, is defined as

$$(3) \qquad r = t_p \,/\, t_s$$

where t_p is the elapsed time from the onset of rainfall to the peak intensity and t_s is the total storm duration. Values of r are determined individually for a number of historical storms and the mean value is used for the design hyetograph. The intensities on either side of the peak are obtained from applicable local IDF curves which are expressed as

$$(4) \qquad i_{av} = a \,/\, (t_d^b + c)$$

where i_{av} is the average maximum rainfall intensity over a duration t_d , and the constants a, b, c satisfy the fit of data. Typically, one to six hours is selected as the total storm duration, t_s . However, the choice of t_s does not affect the magnitudes of the peak rainfall intensity or the dimensionless time to peak.

Finally, the storm hyetograph is expressed as

$$(5) \qquad i = \frac{a \left[(1-b) \left(\frac{t_p - t}{r} \right)^b + c \right]}{\left[\left(\frac{t_p - t}{r} \right)^b + c \right]^2} \qquad \text{for} \quad t \leq t_p$$

$$i = \frac{a \left[(1-b) \left(\frac{t-t_p}{1-r} \right)^b + c \right]}{i \left[\left(\frac{t-t_p}{1-r} \right)^b + c \right]^2} \qquad \text{for} \quad t > t_p$$

In the overall evaluation, the popularity of the Chicago method seems to follow from its simplicity -- it can be expediently derived from the existing IDF curves and a set of historical storms for any duration and return period. The major shortcomings of the method arise from its temporal distribution which is based on an assumption that the design storm contains all the maximum intensities for various durations and from the determination of the peak intensity timing r. A further discussion of these two important aspects follows.

The assumption that the design storm should contain all rainfall maxima of a particular return period contradicts findings for actual storms, certainly for lower return periods. In fact, this assumption is contradicted by the storm data in the original paper (Keifer and Chu 1957). Recently, Hogg (1980) pointed out that the Chicago-type distribution is totally inappropriate for some Canadian climates and, for the bulk of the country, it is not among the most probable distributions. It was of interest to note that Keifer and Chu had been fully aware of the weakness of their assumption as evidenced by the following quotation from their paper.

> "The Synthetic Storm Pattern having the same average intensities as that given by the rate-duration curve for all durations, would not likely to occur at the same frequency as the rate-duration curve from which it was derived. It would undoubtedly have a greater return period, that is, less frequent occurrence."

They argued, however, that the use of the Chicago storm was justified because this storm, when applied in conjunction with the Chicago Hydrograph Method, produced runoff peaks not greater than those produced by the separate uniform intensity rainfall with a correct antecedent precipitation. From a historical perspective, the Chicago storm represented an improvement over the uniform rainfall when applied with a particular procedure. Note, however, that this argument is no longer valid because of changes in computational procedures and the discontinued use of uniform rainfall in runoff modelling.

Additional problems arise from determination of the parameter r. Extensive analyses by Chen (1975) indicate that it is a random variable. The mean r values are very much affected by the selection of historical storms from which r is determined. Note that even for the data set used by Keifer and Chu (1957), the mean values of r for individual stations and durations vary from 0.323 to 0.583 for storms with return periods 2 - 10 years. Such variations are lost when the whole set of 83 storms for 4 stations is considered (the recommended value of r is then 0.375).

ISWS (Illinois State Water Survey) Design Storm

The ISWS design storm for the state of Illinois is described in the ILLUDAS model manual (Terstriep and Stall 1974). It is conceivable that the ISWS method could be used in conjunction with local data elsewhere and, therefore, it is included in this detailed discussion.

The ISWS design storm is derived from IDF curves and temporal distributions. First, one needs to determine the critical storm duration, which in the earlier studies with the ILLUDAS model was established as one hour (for catchments from 1.2 km^2 to 21.5 km^2). For this critical duration, the total storm rainfall depth is determined from the IDF curves for a particular return period.

Finally, this rainfall depth is distributed in time according to the standard distribution. For Illinois, Huff's distributions (Huff 1967) were utilized and the median distribution of the first quartile storms was adopted as the standard distribution. In other areas, one would first group heavy storms according to the quartile in which the heaviest rainfall occurred and the median distribution for the predominant quartile would be adopted.

In general, the ISWS design storm cannot be readily applied to other areas. One would need to determine the local temporal distributions and also to determine the critical storm duration. The recommended duration of 1 hour may not be universal. Antecedent conditions also would need to be specified. However, instead of attempting to adopt the ISWS design storm to Canadian conditions, it would be more productive to overcome the shortcomings in the AES design storms which are based on a similar type of analysis.

SCS (Soil Conservation Service) Design Storm

The SCS design storm was developed for various storm types, storm durations and regions of the United States (SCS 1975). The SCS storm version which is used most often in Canada is a general type 6-hours storm applicable east of the 105° meridian.

The SCS design storm is derived from the maximum probable precipitation, areal reduction factors, and standard temporal distributions. The total storm rainfall depth can be determined from maps of 6-hour maximum probable precipitation for areas of 25.6 km^2 (10 sq.miles). It should be stressed that such precipitation was derived by maximizing observed storms, thus representing a more severe combination of meteorological events that has yet been observed. Alternatively, the values read from the IDF curves for various return periods could also be used.

The 6-hour precipitation is then adjusted for various catchment areas and extrapolated to longer durations (up to 48 hours, if required) using a set of graphs for nine geographical zones of the U.S.A. Finally, a temporal distribution which is given in graphical and tabular forms is applied to the adjusted storm rainfall depth and the storm hyetograph is obtained. Such a storm hyetograph is then applied in conjunction with specific antecedent moisture conditions and the SCS hydrograph analysis procedure.

In an overall evaluation, the SCS design storm belongs to the most comprehensive category; it is defined for various zones (of the U.S.A.) and catchment sizes, and appropriate antecedent moisture conditions and a runoff computational technique are specified.

Notwithstanding these positive aspects, the application of the SCS design storm and computational procedure in their present forms should not be encouraged for a number of reasons. They were developed for natural drainage basins of larger size in the U.S.A. Hence, neither the criteria (i.e. maximum probable precipitation) on which the temporal distributions are based nor the loss component of the simulation model are particularly appropriate for urban conditions. Even if this were not the case, the temporal distributions, area-duration data and loss model parameters are based on U.S. data which may not be representative of Canadian conditions.

Uniform Distribution Storm

The uniform distribution storm is typically used in conjunction with the rational method which could be referred to as an equilibrium runoff model.

As noted above, for specified T and t_s, d and hence $i_{av} = d/t_s$ can be obtained from data published by AES.

Among the limitations of the uniform storm approach, one can name the limited choice of computational methods (more or less the rational method and its variations), the dependency of the storm intensity on the ill-defined time of concentration, and the neglect of antecedent conditions.

Because t_s is set equal to the time of concentration, t_c, i_{av} varies inversely with t_c and thus errors in t_c are then reflected in i_{av} and the calculated discharge. In general, the time of concentration is not well-defined. It is described as the time of travel from the most remote point of catchment to the point under consideration. The time of concentration is sometimes further broken into two components — the inlet time (typically specified as a fairly arbitrary constant) and the time of travel in the sewer system. In urban catchments, the largest time of travel is likely to apply to pervious areas and such maximum t_c does not necessarily produce the maximum discharge, which may be largely produced by impervious areas with shorter times of concentration and higher rainfall intensities. It would appear, therefore, that further work and standardization on the use of this uniform distribution design storm and the associated computational procedure would be desirable. Note also that, although the antecedent conditions are not specified for the uniform distribution storm, they could be considered in the choice of the runoff coefficient.

A RECOMMENDED DESIGN STORM

In this section, the institutional aspects of the definition and use of design storms are listed from the perspectives of various action groups, followed by a listing of desirable features of a design storm, assessment of existing design storms and suggestions for future research and development.

Institutional Aspects and Perspectives

1. Clients The client may be in the private sector, for example, a developer or in the public sector, for example, a municipality, regional government or conservation authority. In either case, the client wants a reasonable balance between investment

and level of protection, design costs which are commensurate with the cost of the structure and a standardized procedure which is based on accessible data. Public sector clients also want a uniform level of protection.

2. Practitioner In most cases, the designer prefers that the design criteria be specified rather than developed as part of the design. The designer also prefers a standardized procedure which is based on accessible data and is not particularly concerned about whether or not the procedure is scientifically rigorous.

3. Hydrologist The hydrologist recognizes that the design costs should be commensurate with the cost of the structure and within this constraint, would specify the following objectives.

(i) The model or methodology adopted must be appropriate to the case at hand.

(ii) The simulation must be approximately correct.

(iii) Any probability statements, either explicit or implicit, must be approximately true.

(iv) The precipitation input must be based on local data.

4. Approval Agency In most cases, the approval agency will be a government agency, but occasionally, the government agency will issue a contract for checking to a private firm. The approval agency wants an appropriate level of protection and a standardized procedure which results in straightforward checking. An additional objective is minimizing problems which occur at boundaries between municipalities.

Desirable Features of a Design Storm

Recognizing the role and relative weight of the above 'institutions', the specification of a design storm as set out in the following list should satisfy the major concerns of each.

1. The design return periods for various types of development should be established by local authorities to ensure a uniform level of protection and to keep the costs of such protection commensurate with potential damages. In general, the minor drainage systems are designed for return periods of 2 to 10 years, major drainage systems are designed for return periods up to 100 years.

2. Design storms must be specified for a wide range of return periods from 2 up to 100 years.

3. Design storms will be based on AES data, either tabulated for a site or interpolated from a map.

4. Design storms will be given for two cases: (i) small homogeneous areas with no potential for storage, and

(ii) all other urban areas to which design storms apply.

5. For each case, the design storm will include:

   (i) specification of a computational method which yields approximately correct flow, and

   (ii) specification of 'normal' antecedent conditions.

6. The design storm will be given in the form of a hyetograph which

   (i) is fully defined by the region, T and the catchment time constant, and

   (ii) results in peak flow of approximately the same return period as the precipitation when used in the specified computational procedure with the 'normal' antecedent conditions.

Design Storms for Small Homogeneous Areas with No Storage

In this case, the area is assumed to be small and homogeneous and because there is no opportunity for storage, the complete hydrograph is not required. A design storm is selected such that flow is allowed to just reach equilibrium conditions, i.e. the so-called equilibrium flow model (see Figure 2).

(6) $Q_e = (i - f)\,A$

where Q_e = flow at equilibrium
i = precipitation intensity
f = infiltration rate
A = area

If inconsistent units are used then (6) must be written as

(7) $Q_e = \alpha\,(i - f)\,A$

The values of α and units for the variables in (7) for two common systems of units are given in Table 3.

Table 3. Equilibrium Runoff Formulae: Variables and Units

Variable	Imperial	SI
Q_e	ft^3/s	m^3/s
i	in/h	mm/h
f	in/h	mm/h
A	acres	ha
α	≈ 1	0.00278

The rational method is the application of this equilibrium runoff model in design. The following assumptions and/or modifications are made.

(i) The design return period T is specified by the local authority.

(ii) The time to equilibrium, t_e is estimated (traditionally, t_e has been set equal to t_c but this practice is not encouraged (Overton and Meadows 1976)).

(iii) Design storm duration, t_s, is set equal to t_e.

(iv) The design rainfall intensity is taken from the local IDF curve (Figure 2) for the specified T and t_s.

(v) The term $(i - f)$ is replaced by $C.i$ where C is the runoff coefficient.

Hence from (7)

$$Q = \alpha\ C\ i\ A \tag{8}$$

Although the method is straightforward, there are a number of difficulties involved in determining appropriate values of t_s and C. For the case of a completely impervious area, $C = 1$ and no modifications are required. However, when $C \neq 1$, three problems arise with the specification of t_s and antecedent conditions if the method is not applied carefully. For example, consider a small area which is 50 percent impervious ($C = 1$) and 50 percent pervious (assume $C = 0.1$). For typical antecedent conditions, the peak flow from an input block of specified T will be maximum for a duration close to a duration equal to t_e for the impervious area rather than t_e for the entire area and this fact should be reflected in the selection of t_s.

The second problem occurs when the contributions from pervious areas are significant. The runoff coefficient ($C = 1 - f/i$) depends on antecedent conditions and the rainfall intensity. Even if average antecedent conditions are specified, different values of C must be specified for significantly different values of T.

The third problem arises because of the arbitrary specification of an inlet time (or minimum t_e) and a runoff coefficient by public sector clients. Reasons for this may be to make up for a lack of technical expertise on the part of the practitioner, to provide an extra degree of protection, or simply historical accident. In any event, such practice should be discouraged. Information on C and t_e applies beyond municipal boundaries and hence these variables should not be specified locally. The level of protection should be reflected in the specification of T.

In order to minimize the inappropriate application of the rational method including the design storm specification, a users' manual should be prepared. This manual would include the following items:

(i) examples of and reference to AES single site IDF data and regional maps,

(ii) tables of C for typical values of T, soil and cover combinations and antecedent conditions,

(iii) formula and nomographs for determining t_e for a given area, slope, length, imperviousness, surface roughness and rainfall intensity, and

(iv) guidance on the selection of t_s for non-homogeneous areas in terms of percent imperviousness and t_e for various area types.

Design Storms for Other Urban Areas

As the size and complexity of the drainage system increases, more sophisticated design techniques become appropriate. Characterization of the design runoff event by the peak flow only is inadequate and runoff simulation must be used in conjunction with a design event to produce the required runoff hydrograph. As discussed above, the design storm of variable intensity (or the design event) should be characterized by its return period, storm duration, total rainfall depth, temporal rainfall distribution, and furthermore, the normal antecedent catchment conditions and the computational method should be specified. In the following discussion, individual attributes of the design event and of its application are presented.

The first step is the specification of T by the local authority. The next step is to determine the design storm duration. In general, such a duration should reflect both the nature of local storms and the response of the catchment under consideration. The recommended storm duration can be expressed as

(9) $t_s = m\, t_k$

where t_k is the catchment time constant and m is a parameter equal to or larger than two. The catchment time constant characterizes the catchment response in terms of the area, slope, length, imperviousness, surface roughness, and rainfall intensity. Detailed descriptions of formulas for calculating t_k can be found in the literature (Desbordes 1978, Overton and Meadows 1976). For small urban catchments (≈20 ha), t_k typically varies from 15 to 30 minutes. The appropriate coefficient m should be at least two, thus yielding the storm duration from 30 to 60 minutes. It would appear that for small urban catchments, the storm duration of 1 hour is appropriate but as the catchment time constant increases, greater durations will apply.

The total storm rainfall depth can be determined, for the recommended duration, from the IDF curves as shown in Fig. 3. This rainfall depth is then distributed in time using temporal distributions derived from local data. In the absence of such distributions, one should use the AES distributions from the nearest AES station. As an interim measure, the 50 percentile distribution is recommended. Such distributions have been tabulated by Hogg (1980) and Pugsley (1981). Further revisions of this recommendation may be forthcoming as more work on the AES distributions is done.

The storm hyetograph is obtained from the cumulative distribution by differentiation and discretization. The discretization interval, Δt, should be between 5 minutes and $t_k/2$ to conform with the general practice of AES to provide maximum rainfall intensity data for durations greater than 5 minutes and to select Δt appropriate for the catchment response.

Apart from the design storm hyetograph, the applicable antecedent catchment conditions should be also specified. Depression storage is unimportant for the computation of peak flows and hence antecedent conditions can be specified in terms of an infiltration capacity. By specifying parameters of the Horton, Holtan, and Green-Ampt infiltration equations for various applicable soil types, the most frequently used urban runoff models would be covered. Such a procedure would not be without a precedent, because both the SCS Hydrograph Method (SCS 1975) and the Flood Studies Method (Natural Environment Research Council 1975) consider the antecedent catchment conditions in relation to the design storm.

Finally, the computational method to be used in conjunction with a particular design storm should be specified. Such a specification may be fairly general, e.g. one could specify an urban runoff model and list the applicable models with a qualification that other equivalent models also apply. Again, there are precedents to this recommendation as is apparent from Table 2.

The steps to be taken in the definition of the design storm event are shown pictorally in Fig. 3.

Assessment of the Design Storms Used in Canadian Practice

The specifications listed above for uniform and variable intensity design storms may be used to assess the design storms currently used in Canadian practice.

For the uniform intensity storm, it appears that the criteria given earlier are mostly met, except for the implications of the recommendations to refine the runoff coefficient to reflect the storm return period, antecedent catchment conditions, and catchment characteristics and to provide guidance for the selection of t_s.

When examining the design storms with time-variable intensities, it appears that none of the storms discussed meets all the criteria. In general, the definitions of existing design storms are either incomplete, or their temporal distributions do not reflect the local climate. Detailed evaluations follow.

The AES design storms are not fully defined. Further guidance is needed on the storm duration, the role of distribution probability percentiles, and the applicable antecedent conditions. Furthermore, the applicability of the existing temporal distributions to durations other than 1 and 12 hours, and the possible dependence of these distributions on the storm return period should be investigated. Pending such further development, the AES storms hold promise to become standard design storms for urban applications.

Although the Chicago design storm is rather popular in current engineering practice, it suffers from certain shortcomings and it is often applied outside of the range of its applicability. In particular, the temporal distribution of the Chicago design storm is unrealistic and refuted by both the original data of Keifer and Chu (1957) and by recent extensive analyses of Canadian rainfall data by Hogg (1980). As with the other design storms, the design hyetograph should be supplemented by information on the antecedent conditions and the computational procedure.

The ISWS design storm was developed for the state of Illinois and this limits its applicability. Although the basic approach could be applied elsewhere, the AES design storms are based on a similar type of analysis of Canadian data and therefore should replace the ISWS design storm.

Among the discussed storms, the SCS design storm is defined most completely. In addition to the design hyetograph, antecedent conditions and the computational procedure are both specified. The main problems with the SCS storm arise from the fact that it was developed for larger drainage basins in the U.S.A. The temporal distributions were intended for the maximum probable precipitation in the United States and may be irrelevant for Canadian climates, the return periods used in design applications and storm durations much less than 6 hours. More guidance is needed on the applicability of the loss component of the simulation model.

Therefore it can be concluded that none of the existing design storms has all the necessary attributes.

Future Research and Development

Recognizing that none of the existing design storms has all the features which were listed as prerequisites for standardization of design storm applications, it is recommended that the most promising design storms be further developed to increase the reliability of their use. In particular, future work should concentrate on the following aspects.

(1) Delineate the applicability of simple design storms.

(2) Advance the work on temporal distributions of design storms. In particular, the approach taken by AES holds promise and should be expanded to other durations than 1 and 12 hours (e.g. 0.5, 2,3 and 6 hours).

(3) Develop temporal distributions for various return periods (if applicable).

(4) Develop descriptions of normal antecedent conditions for various regions and types of catchments.

(5) Provide complete descriptions of design storms for various regions.

SUMMARY AND CONCLUSIONS

The design storm concept is well established in Canadian drainage practice and is likely to remain in use in spite of other, sometimes more costly, alternatives and in spite of shortcomings of the existing design storms. The main problems with the existing design storms arise from their incomplete definitions and the transposition of temporal distributions which appear to be unrealistic and unconfirmed by actual data.

Two types of design storms are recognized — an uniform intensity storm which is suitable for applications of the rational method and design storms of varying rainfall intensity (stationary). For the former type, enough information on extreme rainfall intensities is available from AES in the form of IDF curves or rainfall data maps. However, some further work needs to be done on the corresponding computational method. In particular, better definitions of the runoff coefficient C and the time to equilibrium (which affects the rainfall intensity) are needed.

For design storms of variable intensity, the traditional definition of the design storm needs to be expanded. Design standardization will become possible only if the design storm description includes the hyetograph (for a given region, return period, and a recommended catchment time constant) computational procedure and normal antecedent conditions.

Finally, it should be recognized that there are classes of design problems (e.g. water quality design, moving and developing storms over large areas) that are not suitable for applications of conventional design storms and such applications should be avoided, unless special rainfall inputs are developed for each case. Note also that there are other alternative approaches to the design storm concept, notably continuous simulation and surrogate continuous simulation. Such approaches may gain more prominence in the future.

REFERENCES

Abraham, C., Lyons, T.C. and Schulze, K.W. (1976): Selection of a Design Storm for Use with Simulation Models. National Symposium on Urban Hydrology, Hydraulics, and Sediment Control, University of Kentucky,exington, Kentucky, July 26-29, (preprint).

Bandyopadhyay, M. (1972): Synthetic Storm Pattern and Run-Off for Gauhati, India. Journal of the Hydraulics Division, ASCE, Vol. 98, No. HY5, pp. 845-857.

Chen, C. (1975): Synthetic Storms for Design of Urban Highway Drainage Facilities. Report PRWG106-4, Utah Water Research Laboratory, Utah State University, Logan, Utah.

Desbordes, M. (1978): Urban Runoff and Design Storm Modelling. In: Urban Storm Drainage, Editor P.R. Helliwell, Proceedings of the International Conference Held at the University of Southampton, April, 1978, Published by Pentech Press, London.

Hogg, W.D. (1980): Time Distribution of Short Duration Storm Rainfall in Canada. Proceedings of the Canadian Hydrology Symposium 80, Toronto, Published by National Research Council, Ottawa.

Huff, F.A. (1967): Time Distribution of Rainfall in Heavy Storms. Water Resources Research, American Geophysical Union, Vol. 3, No. 4, pp. 1007-1019.

James, W. and Drake, J.J. (1980): Kinematic Design Storms for Urban Hydrology. Proc. of Canadian Hydrology Symposium, Toronto, pp. 79-84, May.

Keifer, C.J. and Chu, H.H. (1957) Synthetic Storm Pattern for Drainage Design. Journal of the Hydraulics Division, Proc. American Society of Civil Engineers, New York, Vol. 83, No. HY4, pp. 1332-1 - 1332-25.

McKelvie, S.A. (1982): Preparing a Design Storm. Proceedings of Stormwater and Water Quality Management Modeling Users Group Meeting, 25-26 March 1982, Report EPA-600/9-82-015, U.S. Environmental Protection Agency, Athens, GA 30613.

McMullen, D.N. (1962): Timmins Flood August 31 - September 1, 1961, A Design Storm for Ontario. Report CIR-3746, TEC-428, Meteorological Branch, Department of Transport - Canada, October 30.

McPherson, M.B. (1958): Discussion of Synthetic Storm Pattern for Drainage Design. Journal of the Hydraulics Division, Proc. ASCE, Vol. 84, No. HY1, pp. 49-57.

Natural Environment Research Council. (1975) Flood Studies Report. Vol. 2, Meteorological Studies, Natural Environment Research Council, London.

Overton, D.E. and Meadows, M.E. (1976): Stormwater Modeling. Academic Press, New York.

Packman, J.C. and Kidd, C.H.R. (1980): A Logical Approach to the Design Storm Concept. Water Resources Research, Vol. 16, No. 6, pp. 994-1000.

Pugsley, W.I. (Editor) (1981): Flood Hydrology Guide for Canada: Hydrometeorological Design Techniques. Publication No. CL13-81, Atmospheric Environment Service, Environment Canada, Downsview, Ontario.

Shtifter, Z. (1981): A Kinematic Storm Model for Urban Drainage Study. M. Eng. Thesis, Dept. of Civil Engineering, McMaster University, Hamilton, Ont., May.

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.

Terstriep, M.L. and Stall, J.B. (1974): The Illinois Urban Drainage Area Simulator. Illinois State Water Survey, Urbana, Ill., Bulletin 58.

Water Pollution Control Federation. (1970): Design and Construction of Sanitary and Storm Sewers. WPCF Manual of Practice No. 9, Washington, D.C.

Watt, W.E. and Marsalek, J. (1977): What the Practising Urban Hydrologist Needs From the Hydrometeorologist. Preprint Volume: Second Conference on Hydrometeorology, October 25-27, 1977, Toronto, Ontario.

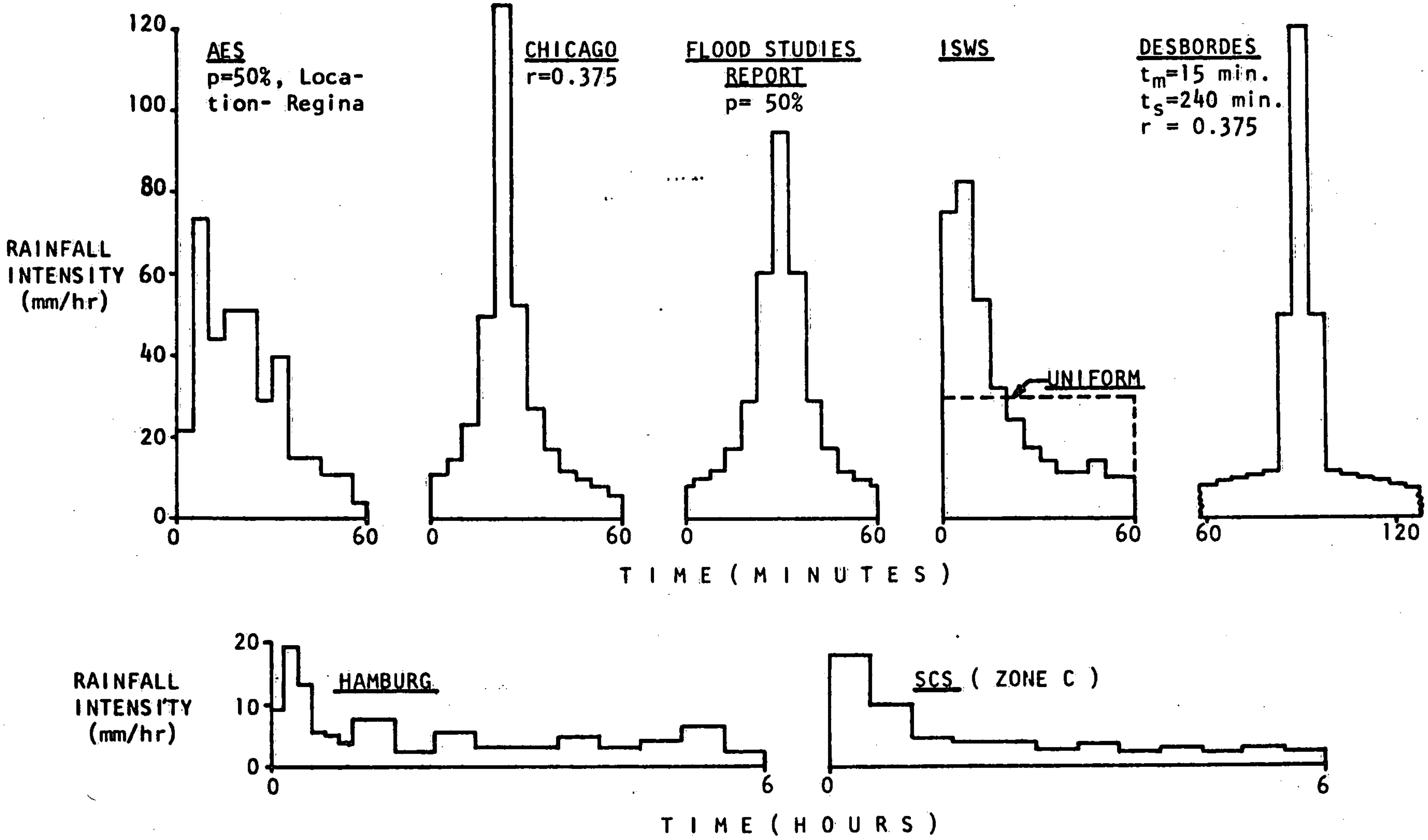


Fig. 1. Hyetographs for Selected Design Storms

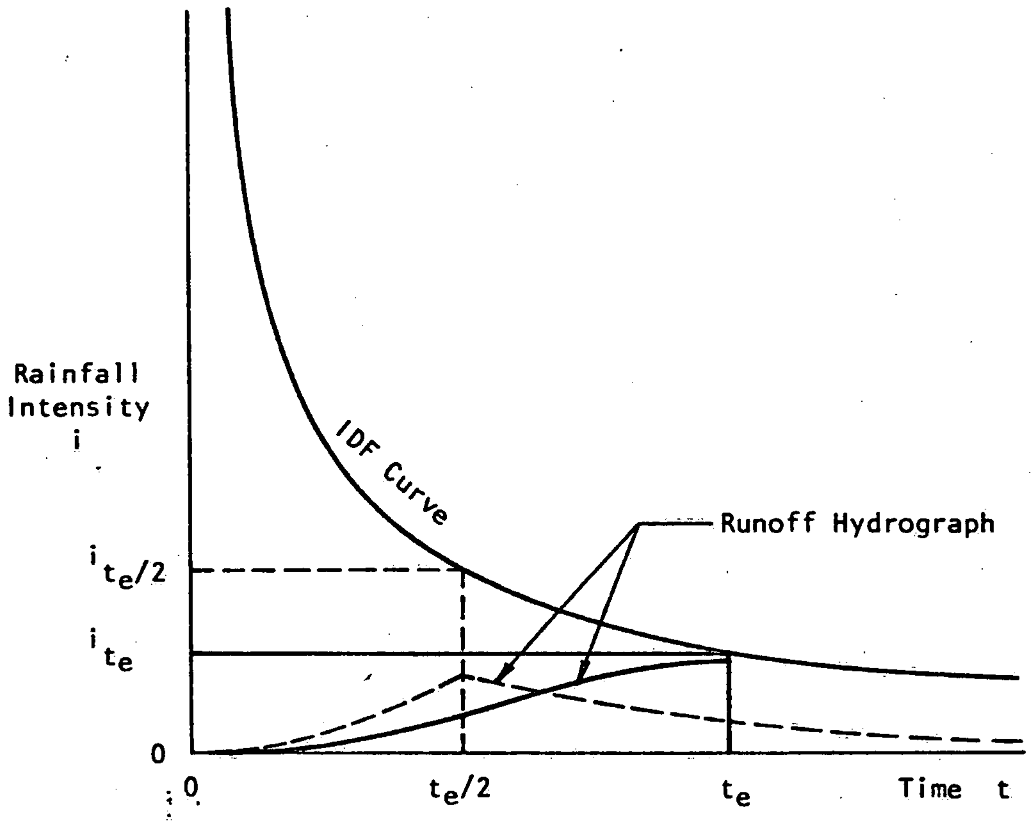
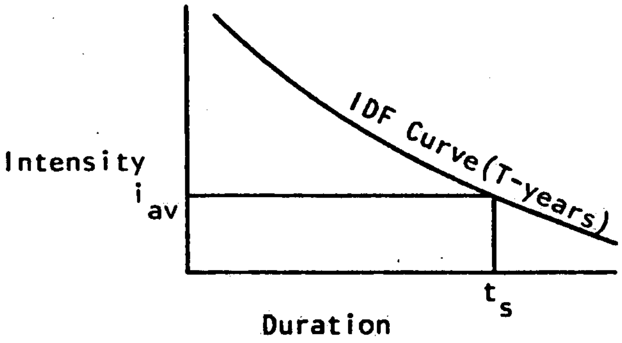
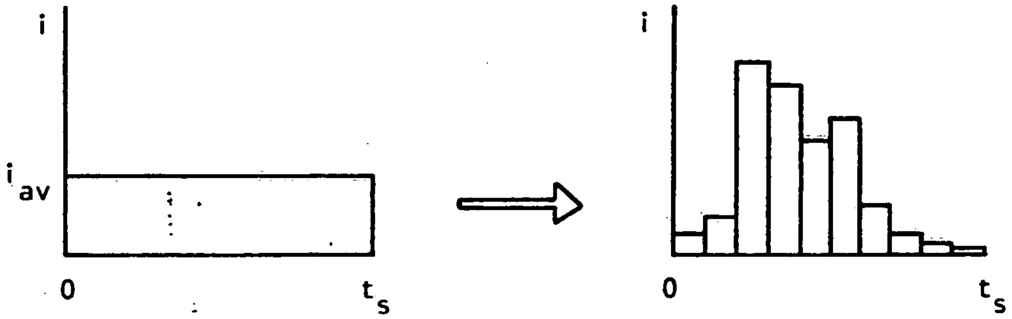


Fig.2. Runoff Equilibrium Model

1. SELECT THE DESIGN STORM RETURN PERIOD ACCORDING TO LOCAL CRITERIA
2. DETERMINE THE STORM DURATION t_s = (2 to 3) x t_k, WHERE t_k IS THE CATCHMENT TIME CONSTANT
3. DETERMINE THE TOTAL STORM RAINFALL DEPTH FROM IDF CURVES FOR THE RETURN PERIOD T AND DURATION t_s



4. DISTRIBUTE THE STORM RAINFALL DEPTH OVER THE DURATION t_s USING A SPECIFIED TEMPORAL DISTRIBUTION AND TIME INTERVAL Δt ($\geq$ 5 min.)



5. SPECIFY THE COMPUTATIONAL PROCEDURE, e.g. URBAN RUNOFF MODELS $URM_1 \ldots URM_j$, OR EQUIVALENT
6. SPECIFY ANTECEDENT CATCHMENT CONDITIONS (SOIL MOISTURE)

Soil Type	Infiltration Capacity Equation Parameters				
	X_1	X_2	X_3	....	X_m
S_1					
S_2					
⋮					
S_n					

Fig.3. Recommended Procedure for Variable Intensity Design Storms