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CONTRIBUTIONS TO THE ASCE MANUAL OF PRACTICE ON DESIGN AND CONSTRUCTION OF STORM DRAINAGE SYSTEMS

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ASCE MANUAL OF PRACTICE ON DESIGN AND
CONSTRUCTION OF STORM DRAINAGE SYSTEMS

Chapter 4, Section B

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CONTRIBUTIONS TO THE ASCE MANUAL OF PRACTICE ON DESIGN AND CONSTRUCTION OF STORM DRAINAGE SYSTEMS

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PART I DESIGN STORMS

The specification of a rainfall event, sometimes called a design storm, as a design criterion for drainage structures is used widely in engineering practice. In spite of the widespread use, the subject of urban design storms is somewhat controversial. Much of this controversy, however, arises from the lack of realistic and accurate definitions of design storms and confused thinking about their applications. Main criticism of design storms arises from the practice of assigning a particular frequency to a design storm, neglect of antecedent catchment conditions, and the design on the basis of specified rainfall rather than the runoff discharge. From the practical point of view, the design storm may be viewed as a convention established for a particular type of design work. Furthermore, the use of design storms requires minimal resources in terms of time and money and this fact as well as the lack of well defined and inexpensive alternatives contributed to the popularity of this approach. Although the design storm has to reflect the required level of protection, the local climate, and catchment conditions, it does not have to be scientifically rigorous. It is probably more important to define the design storm and the range of its applicability fairly precisely, in order to ensure safe, economical, and standardized usage.

Two types of design storms are recognized - synthetic and actual (historic) design storms. The former ones are derived by synthesis and generalization of a large number of actual storms. The latter ones are events which have occurred in the past and which are sometimes well documented in terms of their impact on the drainage system. This manual concentrates on the more common synthetic design storms. For further information on actual design storms, references (11, 17) should be consulted.

Conventional urban design storms are best applicable to sewer sizing in relatively small (say less than 100 acres) catchments in which the generation of runoff peaks is controlled by the impervious areas. Under such circumstances, the considerations of spatial effects and antecedent catchment conditions may be omitted. In the case of larger catchments, or catchments with low imperviousness, the reliability of conventional design storms decreases. As the catchment area increases, there may be more opportunities to incorporate runoff storage into the design and also the spatial effects may become more significant. In catchments with low imperviousness, the pervious segments will control the generation of runoff peaks and the neglect of antecedent catchment conditions may no longer be acceptable. Finally, the application of conventional urban design storms to the design of drainage systems with storage or treatment facilities should be avoided. The design of such complex and intricate systems should be based on continuous rainfall/runoff simulation or on comparable approaches.

In spite of the above discussed limitations of the design storm concept, it is a useful engineering tool which, if understood and correctly used, may be applied to the appropriate design problems in urban drainage.

1. Design Storm Characteristics

Generally, a design storm is defined by its return period, the total rainfall depth, the temporal distribution at a point, and spatial characteristics which include the average spatial distribution, storm movement, and spatial development and decay. The temporal distribution is further characterized by the storm duration, the peak rainfall intensity, and the chronological location of the peak. The relative importance of individual storm characteristics varies with the intended application, as discussed below.

a. Design storm return period

Ideally, the return period of a design storm should be selected on the basis of economic efficiency with the objective to minimize the total drainage costs which are defined as the investment costs plus the damages. In practice, however, the concept of economic efficiency is typically replaced by the concept of a level of protection. The selection of this level of protection, or the design storm return period, which actually refers to the exceedance probability of the design storm rather than the probability of the failure of the drainage structure, is largely based on local experience with design and operation of drainage systems. Recent engineering experience in the United States and Canada indicates the following design return periods:

Minor drainage systems	
Storm sewers in residential areas	2 to 5 years
Storm sewers in high-value districts and in commercial areas	5 to 10 years
Major drainage system elements	
Swales, streets, channels, culverts and bridges	up to 100 years

Some additional considerations are made in the selection of the return period. In particular, greater return periods are recommended for those parts of the system which are not susceptible to future relief, for design of combined sewers because of basement flooding, and for design of those structures whose failure would seriously disrupt an important facility (2).

b. Total storm rainfall depth

The total storm rainfall depth at a point, for a given rainfall duration and frequency, is given by the local climate. Rainfall depths for various durations and frequencies are published in maps of precipitation and in reports of governmental agencies collecting precipitation data. Rainfall depths can be further processed and converted into rainfall intensities (intensity = depth/duration) which are then presented in rainfall intensity-duration-frequency (IDF) curves. Such curves are particularly useful in storm drainage design because many computational procedures require the rainfall input in the form of intensities. The sources and analyses of rainfall data for drainage design are further discussed below.

Although some municipalities collect and process their own rainfall data, the drainage design is more often based on rainfall data compiled and processed by national agencies. In the United States, the best source of rainfall data are the data banks and reports of the National Weather Service (NWS). Besides the basic rainfall data, it is possible to obtain maps containing isolines of constant rainfall depth for specific durations and return periods (16). Other similar maps were prepared by Hershfield (4), Miller et al. (13), and Frederick et al. (3). The last reference is particularly useful because it provides rainfall depths for the central and eastern United States for durations as short as five minutes. Rainfall data from these reports can be abstracted, converted into intensities, and presented in the form of IDF curves which are shown in Fig. 1 (2). Similarly, the total storm rainfall depth can be determined from such maps for a chosen storm duration and return period.

Besides the graphical presentations, the rainfall intensity-duration curves for individual frequencies are sometimes approximated by mathematical expressions in one of the following forms:

$$i = \frac{a}{t_d + b}, \quad \text{or} \quad i = \frac{a}{(t_d + b)^c}, \quad \text{or} \quad i = \frac{a}{t_d^c + b} \qquad (1)$$

where i is the average intensity for duration t_d, and constants a, b, c satisfy the fit of data.

Recognizing that the precipitation data in maps were subject to some interpolation and smoothing, it may be justifiable to develop IDF curves directly from local basic rainfall data. Procedures for performing such analysis are well established and described elsewhere (17).

c. <u>Storm duration</u>

The design storm duration is an important storm parameter which, as noted earlier, defines the rainfall depth or intensity for a given frequency, and consequently affects the resulting runoff peak. The design storm duration which produces the maximum runoff peak depends on the catchment time constant traditionally defined as the time of concentration. The time of concentration is commonly expressed as the travel time from the most remote catchment point to the point under design. Such a definition ignores the relative runoff-producing capabilities of pervious and impervious areas and possible variations with rainfall intensity. Nevertheless, the current practice is to select the design storm duration as the catchment time of concentration or longer. In typical applications of design storms to small catchments (say less than 100 acres), the design storm duration of one hour should be adequate. For larger areas, the catchment time of concentration should be determined, and if greater than one hour, used as the design storm duration. Calculations of the time of concentration are discussed in detail in the section on the rational method.

d. Temporal rainfall distribution

The temporal rainfall distribution during the storm is an important factor which affects the timing as well as the magnitude of the resulting runoff peak. Realistic estimates of temporal distributions are best obtained by analysis of basic rainfall data from recording gage networks. Such an analysis may have to be done for several widely varying storm durations, in order to cover various types of storms and to produce distributions for various design problems. Where such analyses cannot be economically justified, the designer has to adopt one of the existing distributions. He should be aware of the fact that different distributions may apply to different climatic regions of the country. A brief discussion of several well-known temporal rainfall distributions follows. Four of such distributions are listed in Table 1.

The simplest temporal rainfall distribution is the uniform distribution. Although such a distribution does not describe well actual storms, it has been found appropriate for use with the rational method in which the losses are also uniformly distributed by specifying the runoff coefficient. Thus in this case, the design storm is fully determined by selecting the storm frequency and duration, and by determining the uniform rainfall intensity from the appropriate IDF curve.

Huff (7) produced temporal rainfall distributions from extensive rainfall data collected in Illinois. Heavy storms were divided into four groups according to the chronological location of the intensity peak. For each quartile, dimensionless temporal distributions (i.e., the percentage of total rainfall versus the percentage of total duration) were prepared for various probability levels. Short-duration, high-intensity storms, which are typically used in storm sewer design, were particularly common in the first quartile group. Consequently, Terstriep and Stall (14) recommended the first quartile median distribution for urban design storms in Illinois. Huff's analysis has been recently applied in Canada and temporal distributions have been produced for one-hour and twelve-hour durations. Such durations were selected to provide samples of both convective shower events as well as synoptic scale cyclonic circulation events (6).

Another temporal distribution has been proposed by Keifer and Chu (10) and incorporated in the so-called Chicago storm. This distribution is based on an assumption that, for a particular return period, the design storm should contain the corresponding maximum rainfalls for all the durations. The distribution of these rainfalls is generally skewed. This distribution can be readily derived from the local IDF curves and the analysis of skewness of actual storms. Although the Chicago storm distribution has been used extensively in practice, a word of caution is in order. Recent extensive analyses in Canada indicate (6) 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. Thus the designer should examine the applicability of this distribution in each case. A recently proposed modification of the Chicago distribution attempts to reduce the excessive sharpness of the storm hyetograph by averaging the storm segment which contains the intensity peak. By applying this procedure, the modified storm profile no longer contains all maximum rainfalls for some short durations.

Yen and Chow (18) developed a general nondimensional distribution by applying the method of statistical moments to describe the observed hyetographs. This procedure is being further refined (17).

For design storms of longer durations, temporal distributions have been developed by the Soil Conservations Service (SCS) (15) and by Hershfield (5).

The SCS distributions have been developed for 6-hour and 24-hour durations and the maximum probable precipitation and two climatic regions. The first region encompasses the coastal side of the Sierra Nevada Mountains and the interior regions of Alaska; the second region encompasses the remaining United States, Puerto Rico, and the Virgin Islands.

Hershfield's average time distributions are available for durations of 6, 12, 18 and 24 hours (5).

Finally, it should be emphasized that there are numerous other distributions which have been developed in various countries. For discussions of some of these distributions, consult references (12, 17).

e. Storm spatial characteristics

Storm spatial characteristics are important for larger drainage catchments. The average catchment rainfall depth may then be significantly lower than the point value, because of limited dimensions of storm cells and because of their movement and development. In such cases, rainfall data are obtained from a gage network and the catchment average rainfall depth is estimated from the point values. Alternatively, standard charts for reduction in point rainfall depth with the area are used. One of such charts, produced by the National Weather Service (13), is shown in Fig. 2.

In addition to areal reduction, storm movement and development and decay, manifesting themselves in time and space, may also affect the runoff peak magnitude and timing. Such considerations are particularly relevant to the case of operation and/or control of large systems of combined sewers. The required rainfall analysis then represents a fairly complicated and tedious task which is beyond the scope of this manual. Consequently, the reader is referred to appropriate references (1, 8, 9).

Finally, although the temporal distributions are often expressed as continuous functions, for the actual use in runoff models, they need to be discretized into time intervals generally coinciding with the computational time step.

2. Examples of Design Storms

Four design storms which are used extensively in engineering practice are described in this section. With the exception of the SCS design storm, all these storms are derived from point precipitation data and consequently are best applicable to relatively small catchments in which spatial effects may be neglected.

a. Uniform intensity storm

The uniform intensity storm is used exclusively with empirical formulas such as the rational method. Such a storm is fully defined by its return period, duration, and average rainfall intensity. In practice, the uniform storm is derived from the IDF curves for the selected return period and duration. The storm duration is typically chosen as the time of concentration of the catchment as defined earlier. For this duration, the corresponding intensity is read directly from the IDF curve as shown in Fig. 3.

b. Illinois State Water Survey (ISWS) storm

The ISWS design storm has been developed for the State of Illinois and described in the ILLUDAS manual (14). It is conceivable that the method used to develop this storm could be applied elsewhere with local IDF curves and temporal distributions.

The first step is to select the storm return period appropriate for the design problem at hand. The next step is to select the storm duration. In the original approach (14), such a duration is selected as one hour for catchment areas ranging from 0.5 to 8.4 square miles. As an alternative, it is possible to set the storm duration equal to the catchment time of concentration. For the selected storm return period and duration, the total storm rainfall depth is determined from the appropriate IDF curve. Finally, this rainfall depth is distributed in time. For Illinois, the standard distribution recommended by Terstriep and Stall (14) may be used as shown in Fig. 3. Elsewhere, the temporal distribution needs to be determined from local data. For this purpose, actual heavy storms are divided into four groups according to the chronological location of the peak intensity. It seems plausible to simplify the analysis by using only three groupings (11). For the predominant group, the median distribution of the percentage rainfall versus the percentage time is determined and applied to the rainfall depth to obtain the design hyetograph. An example of such a hyetograph is shown in Fig. 3.

c. Chicago storm

The Chicago storm method is applied in conjunction with local IDF curves and the locally derived skewness of the temporal distribution. For a selected return period, the storm duration is taken as the catchment time of concentration which is determined as in the rational method. In the original reference, a constant duration of three hours was recommended (10). The skewness of local storms is described by the dimensionless time to the intensity peak, r, which is defined as the time to peak divided by the storm duration. This parameter r is taken as the mean of r-values determined for heavy local storms. The IDF curve of the selected frequency is then expressed mathematically as:

$$i_{av} = a/(t_d^b + c)$$

where i_{av} is the average maximum intensity for duration t_d, and the constants a, b, and c satisfy the fit of data. Finally, the storm hyetograph is described by the following equation:

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

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

where t is the elapsed time from the onset of rainfall, and t_p is the time to peak. The Chicago storm hyetograph is plotted in Fig. 3.

d. Soil Conservation Service (SCS) storm

The SCS storm (15) has been developed for various storm types, storm durations, and climatic regions of the United States. For any location, the SCS storm can be derived from the maximum probable precipitation, areal reduction factors, and standard temporal distributions. The total storm rainfall depth is determined from maps of six-hour maximum probable precipitation for areas up to 10 square miles. The six-hour precipitation is then adjusted for various catchment areas and extrapolated to longer durations (up to 48 hours) using a set of graphs for nine geographical zones of the United States. Finally, a temporal distribution, which is given in graphs and tables (15), is applied to the 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.

The SCS storm method is sometimes modified by practicioners by replacing the maximum probable precipitation by the six-hour rainfall of a selected return period. An example of such a storm is given in Fig. 3.

It should be emphasized that the problem of urban design storms is currently studied by many researchers and further improvements can be expected. The examples given above just illustrate some methodologies which are currently used in engineering practice.

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Table 1. Temporal distributions for various design storms

	ISWS[1]	Chicago[2]	Hershfield[3]		SCS 6-hour[4]	
Minutes	P_t/P_T	P_t/P_T	t/T	P_t/P_T	Hours	P_t/P_T
0	0.00	0.00	0.00	0.00	0.00	0.00
5	0.21	0.03	0.10	0.06	0.60	0.04
10	0.44	0.07	0.20	0.12	1.20	0.10
15	0.59	0.13	0.30	0.20	1.50	0.14
20	0.68	0.23	0.40	0.29	1.80	0.19
25	0.75	0.51	0.45	0.34	2.10	0.31
30	0.80	0.67	0.50	0.45	2.28	0.44
35	0.84	0.77	0.55	0.63	2.40	0.53
40	0.87	0.84	0.60	0.73	2.52	0.60
45	0.90	0.89	0.65	0.81	2.64	0.63
50	0.94	0.94	0.70	0.86	2.76	0.66
55	0.97	0.97	0.80	0.94	3.00	0.70
60	1.00	1.00	0.90	0.99	3.30	0.75
			1.00	1.00	3.60	0.79
					3.90	0.83
					4.20	0.86
					4.50	0.89
					4.80	0.91
					5.40	0.96
					6.00	1.00

Legend:

[1] Illinois State Water Survey (Ref. 14), recommended for Illinois.
[2] Chicago design storm (Ref. 10), prepared for the following conditions: Return period 5 years, a=90, b=0.9, c=11.
[3] Applicable to durations 6 to 24 hours (Ref. 5).
[4] Soil Conservation Service, 6-hour design storm (Ref. 17).

t time lapsed from the storm start.
T total storm duration.
P cumulative rainfall depth.

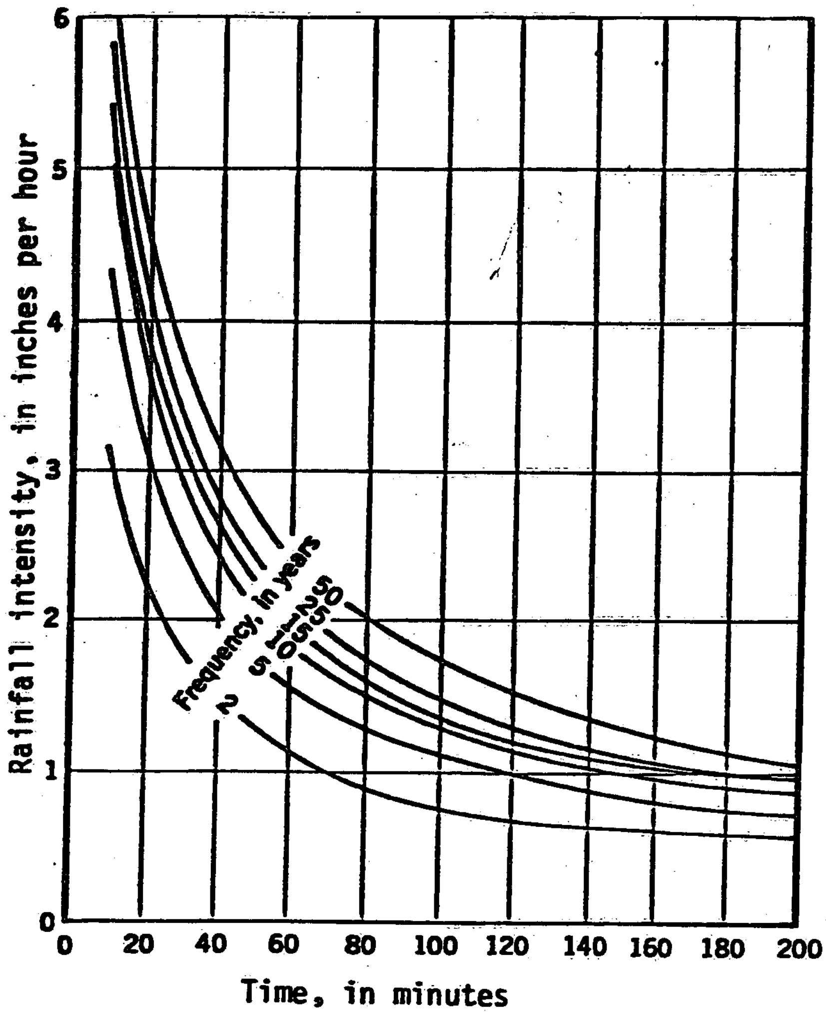

Fig.1. Rainfall intensity-duration-frequency curves, Boston, Mass.(2)

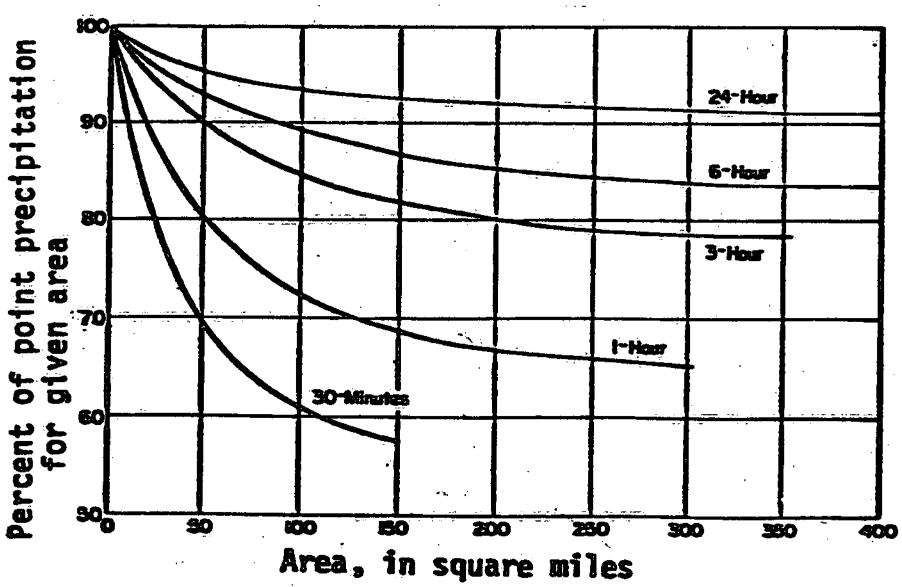

Fig.2. Reductions in point rainfall with area(after ref.13)

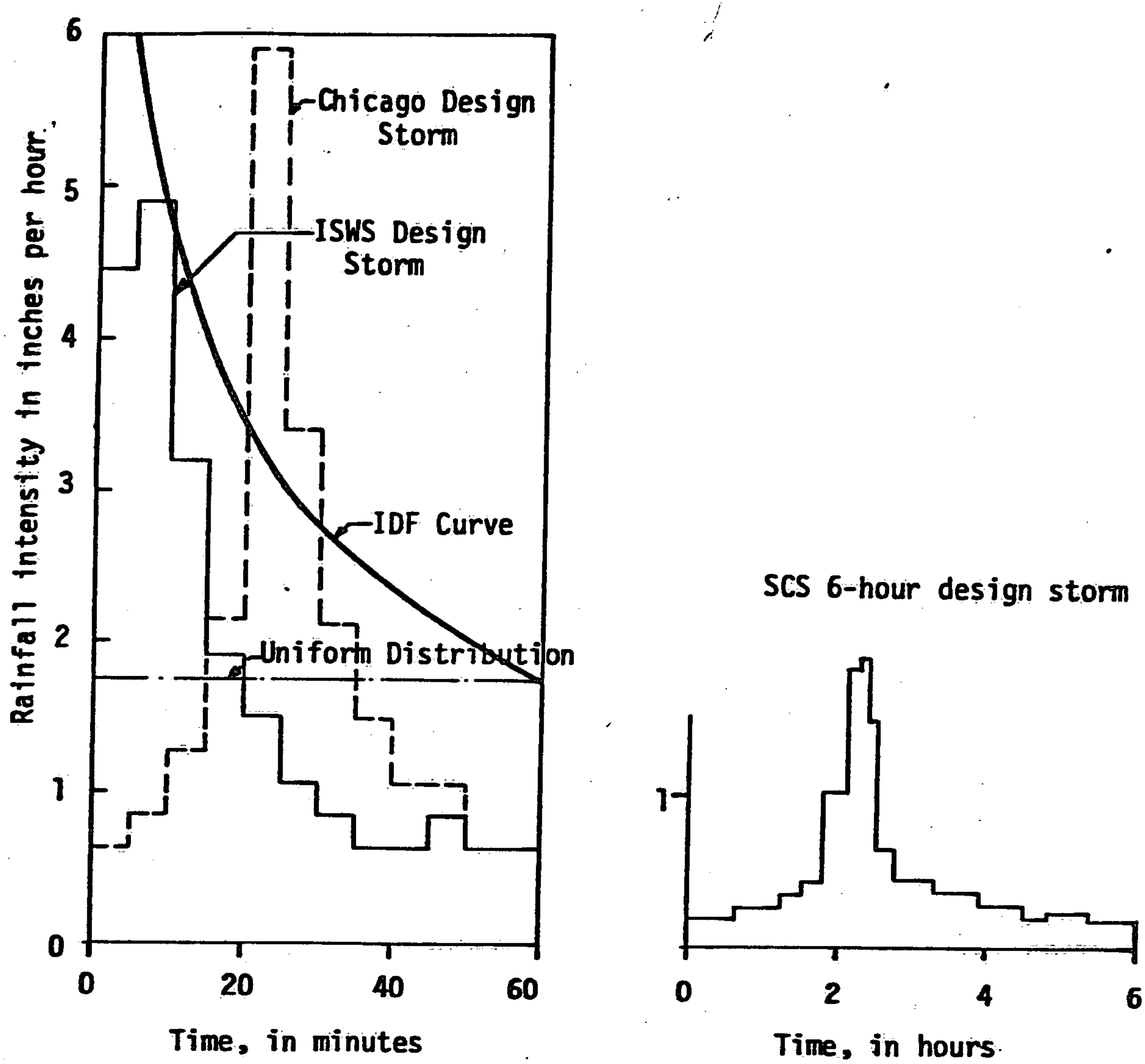

Fig.3. Examples of Design Storms

PART II INLET HYDROGRAPH

Whenever the rainfall rate exceeds the rate of water infiltration into the soil, water accumulates on the catchment surface and starts to fill surface depressions. After such depressions have been filled, water starts to flow across the catchment surface as surface runoff which is conveyed by various channels or conduits comprizing the catchment drainage system. Because different processes dominate various phases of surface runoff, it is customary to treat individual runoff phases separately.

The first runoff phase comprizes runoff formation and transport in the form of a thin-sheet flow on surface elements, usually referred to as overland flow planes (see Fig. 1). The outflow from these planes is then conveyed by minor conveyance elements, such as gutters or ditches, to inlets where it enters underground sewers. The variation of surface runoff discharge in time, at the inlet, is referred to as the inlet hydrograph. The second runoff phase comprizes runoff transport in major conveyance and storage elements and this phase is dealt with in the next chapter.

Various techniques for calculation of inlet hydrographs are described in this section. Such descriptions deal with catchment physiography, rainfall abstractions, and flow routing on overland flow planes and in gutters.

1. Catchment Physiography

The formation of surface runoff in the catchment depends on the rate of rainfall and on catchment physiography. Because catchment physiography varies in space, it is customary to subdivide (discretize) the catchment into a number of subcatchments of more or less homogeneous physiographic characteristics.

a. Catchment discretization

Catchment discretization is generally based on considerations of general drainage patterns, surface slopes, imperviousness distribution (usually given by land use), and major inlets to the transport system. On the basis of such considerations, subcatchments which have homogeneous characteristics and contribute to major transport elements are established. The size of subcatchments may vary from a few acres to tens of acres. While the objective of selecting homogeneous subcatchments may lead to a large number of small subcatchments, for practical reasons the number of subcatchments is kept as low as possible and the characteristics of such larger subcatchments are derived as spatial averages. The initial catchment discretization is sometimes reviewed and changed during the modelling process to improve the modelling results or to reduce computer costs. Further discussion of catchment discretization can be found in references 12 and 19.

b. Physiographic characteristics of catchments

After establishing catchment boundaries, physiographic characteristics of the catchment need to be established for the purpose of runoff computations. Where the catchment is further divided into subcatchments, such characteristics need to be determined for individual subcatchments. Although the list of characteristics which are of interest varies with the computational method used, a general list may be prepared and is given below.

Basic catchment physiographic characteristics

Contributing catchment area: Pervious

Impervious: Directly connected,

Non-connected

Overland flow plane geometry: Width and length

Slope

Surface cover roughness

Geometry of minor transport elements

A brief discussion of individual characteristics follows.

(1) Catchment area

The total catchment area is determined from topographic maps or aerial photographs. For runoff computations, the total catchment area is further subdivided into a number of categories. The first division is into contributing and non-contributing areas. Only the former type may contribute, under some circumstances, runoff to the inlet. The non-contributing area does not contribute runoff under any circumstances, e.g., it may be a natural depression without outflow.

The contributing area is further subdivided into pervious and impervious parts. The impervious part is generally characterized by small rainfall abstractions and fast response given by relatively high flow velocities in smooth impervious channels. The pervious part is characterized by greater abstractions and slow response.

The impervious area consists of two subareas - the directly connected impervious subarea (also referred to as the effective impervious area in ref. 1) and the non-connected impervious subarea. The former subarea drains directly into impervious transport elements, the latter subarea drains onto pervious areas. It should be stressed that the need to differentiate among various types of catchment surface elements follows from large differences in their contributions to catchment runoff.

The task to determine various types of surface elements in the catchment starts with determining the total impervious area. For this purpose, areas of individual impervious elements, such as roofs, roads, driveways, and sidewalks, are determined from maps and summed up. In the second stage, it is necessary to determine the fraction of the total impervious area which is directly connected. This is done by consideration of local drainage practices and ordinances, such as those controlling roof leaders discharge into storm sewers or onto property, etc. When analyzing existing drainage systems, additional information on connections of impervious areas can be obtained from field inspections, or from analysis of observed rainfall and runoff events. Further information on this subject can be found in refs. 1 and 15.

The catchment pervious area is then taken as the total contributing catchment area minus the total impervious area.

Urban areas are commonly characterized by their imperviousness which is defined as the total impervious area divided by the total catchment area. Typical values of imperviousness are listed below for various types of catchments.

Type of Catchment	Typical Imperviousness
Rural	~0%
Suburban (large lots)	5 - 15%
Urban residential	30 - 60%
Downtown districts	~100%

Experience with runoff computations indicates that in catchments with imperviousness greater than 15%, runoff peaks with relatively short return periods (1 to 10 years) are controlled by impervious surface elements.

(2) Geometry of overland flow planes

The geometry of surface elements in plan, their length and width, are also of interest in some computational methods. Particularly important is the length of the overland flow plane which in conjunction with some other parameters affects the time of concentration of the overland flow.

The surface slope is another important characteristic of the catchment and overland flow planes. Such a slope varies widely from one element to another in the range from less than 0.01, for roads or lawns, to values greater than 1.00 in the case of roofs. Because of such diversity, it is necessary to use the mean slope of the catchment surface in computations. To determine accurately such a slope, which may differ substantially from the slope of the undeveloped terrain, is rather time consuming. Consequently, the mean catchment slope is sometimes only estimated in the range from 0.01 to 0.05. Within such a range, runoff peaks simulated for urban areas have been found barely sensitive to variations in the surface slope (19).

(3) Roughness of surface cover

The roughness of surface cover is another important parameter of the overland flow planes and needs to be estimated for both impervious as well as pervious areas. Such a roughness can be described by various coefficients, among which the most common is the Manning's roughness coefficient. Typical values of the Manning's coefficient for various types of ground cover are listed in Table 1.

2. Rainfall Abstractions

Rainfall water reaching the catchment surface is partly converted into runoff and partly abstracted in the form of losses. The determination of such abstractions is one of the main problems of hydrology and perhaps the most uncertain part of runoff computations. Although the subject of rainfall abstractions is generally quite complex, considerable simplifications have been found acceptable in urban catchments. This follows from the fact that runoff processes in urban areas are generally controlled by impervious elements and abstractions on such elements are relatively simple.

The following rainfall abstractions are typically considered in urban areas:

Impervious Areas	Pervious areas
Surface depression storage	Surface depression storage
	Infiltration

a. Depression storage

Depression storage accounts for rainfall water, trapped on the catchment surface in minute depressions, that does not run off or infiltrate into the soil. Generally, the depression storage represents a combination of such phenomena as interception, surface wetting, surface ponding, and evaporation. A detailed discussion of depression storage can be found in refs. 16 and 26.

Depression storage is averaged over the catchment area and then expressed as the depth of storage. Various estimates of the depression storage depth can be found in the literature. Many of these have been derived from volumetric considerations of observed rainfall and runoff. In the absence of local data, it is recommended to use the values given in Table 2.

b. Infiltration

Infiltration of water into soil is a very complex process which is a function of many variables, including soil permeability, soil moisture, ground cover, depth of water on the surface, drainage conditions, and quantity of precipitation. The infiltration capacity, or the maximum rate at characteristics of the catchment (e.g., the soil type), as well as on variable characteristics, such as the soil moisture content. Thus a proper representation of infiltration in a catchment is a difficult task which often exceeds the scope of hydrological concepts and methods employed in drainage design. In most of such methods, infiltration considerations are fairly simplified. Such simplifications are supported by an argument that runoff generation in urban catchments is controlled by impervious elements and thus infiltration is of secondary importance. While this may be true for relatively frequent events in catchments with intermediate or high imperviousness, this argument is not necessarily valid in catchments with low imperviousness and proper attention should be paid to infiltration in those cases.

Over the years, many procedures for computation of infiltration have been developed. Among the most notable methods, one could name those proposed by Green and Ampt (7), Holtan (9), Horton (10, 11), Phillips (18), and the Soil Conservation Service (23-25). A proper treatment of all these approaches is beyond the scope of this manual. Consequently, the discussion is limited to three methods most frequently used in engineering practice - those proposed by Horton, SCS, and Holtan. Such methods are used in various hydrologic models and the user should be aware of underlying limitations. Finally, it should be emphasized that the best estimates of catchment infiltration are obtained by calibration of model infiltration parameters against field data. Only in the absence of such data, one should use the infiltration rates suggested here or in other references.

(1) Horton equation

One of the methods used most frequently in urban drainage design is the Horton equation which is usually presented in the following form (11):

$$f = f_c + (f_o - f_c)\, e^{-Kt}$$

where f is the infiltration capacity at time t, f_o and f_c are the initial and final infiltration rates (in inches per hour), respectively, and K is the exponential decay constant in hr^{-1}. It should be emphasized that the Horton equation is purely empirical, it is simply a convenient way to describe some early observations of infiltration (11).

A number of problems occur in the use of the Horton equation. In particular, many literature data do not refer to the soil type but the land use type, and do not reflect the soil moisture content or situations in which the rainfall intensity is less than the infiltration capacity (12). Some remedies to such problems follow.

The designer should identify the local soil type and define the parameters f_o, f_c, and K accordingly. For this purpose, it is recommended to use the "Soil Survey Interpretations" reports which are available from local Soil Conservation Service (SCS) offices (12). It is also required to establish typical soil moisture conditions at the start of the design event and to adjust the initial infiltration rates accordingly. Finally, since the Horton equation disregards the soil water storage available, it is sometimes applied in an integrated form (2). In that case, the infiltration capacity diminishes in time by the volume that is equivalent to the rainfall excess volume.

Typical values of parameters f_o, f_c, and K are given in Table 3. Infiltration curves for various dry soils with a dense turf cover are shown in Fig. 2 (21).

(2) SCS method

The Soil Conservation Service has developed a comprehensive method for runoff computations in rural as well as urbanized catchments (23-25). The SCS method has been used extensively in engineering practice and further refinements of the method and of its applications are underway.

In the SCS method, the soil water storage capacity, S(in.), is expressed as

$$S = \frac{1000}{CN} - 10$$

where CN is the soil cover complex number listed for various soils and land use or cover (23). The next step is to determine the initial abstraction as a fixed percentage of S. Once this abstraction is satisfied, infiltration takes place. Using the SCS method, Chen(4) and Aron et al.(3) expressed incremental infiltration ΔF, due to incremental amount of rainfall, ΔP, as

$$\Delta f = \frac{S^2}{(P_e + S)^2} \Delta P$$

where P_e is the cumulative excess rainfall (in inches) equal to the cumulative rainfall minus the initial abstraction.

(3) Holtan equation

The Holtan equation (9) is based on the water storage available within the soil mantle. The infiltration rate f at any time t can be expressed as

$$f = a(S - F)^n + f_c$$

where a is a vegetative basal factor reflecting the efficiency with which a crop root system utilizes the soil porosity for storing water (e.g. a = 1.0 for bluegrass turf, ref. 21); n is a constant equal to 1.4; S is the storage available in the soil mantle in inches (equals storage at the total soil porosity minus storage at the wilting point); F is the amount of water already stored in the soil at time t, in excess of the wilting point; and f_c is the final infiltration rate, in inches per hour.

3. Flow Routing on Overland Flow Planes and in Minor Coveyance Elements

The rainfall excess is defined as the rainfall depth minus the abstractions. Thus the rainfall excess represents directly surface runoff and is routed on the overland flow planes. Various techniques are used for this purpose. They are traditionally based on the kinematic wave approximation which assumes that the friction slope is equal to the channel bottom slope. Equations of uniform flow and of the continuity are solved simultaneously, at each time step, to determine the depth of flow and the outflow rate from the overland flow plane. In such calculations, all the overland flow plane characteristics need to be considered, including the plane dimensions, slope, and surface roughness. Further details of overland flow routing can be found in refs. 6 and 20.

The outflow from overland flow planes is typically collected in drainage channels (gutters) or in minor sewer pipes and conveyed to major inlets to the sewer system. The flow routing in such minor conveyance elements is accomplished in a similar way as for the overland flow - using the kinematic wave approach. To undertake such computations, the geometry of conveyance elements (the cross-section area, length, and the bottom slope) and their hydraulic roughness have to be characterized. Additional details of flow routing are given in the next chapter.

Overland flow routing yields an overland flow hydrograph. In practical applications, there may be several overland flow planes contributing either directly or indirectly, via drainage channels, to an inlet. Thus various overland flow hydrographs are added at the inlet to form the inlet hydrograph. This hydrograph is then used as an input to the sewer system and routed through the system. When inlet controls are applied to reduce the inflow to the sewer system, the inlet hydrograph has to be modified accordingly before being routed through the sewer system.

4. Examples of Inlet Hydrograph Computations

Computations of inlet hydrographs which were described in the preceding sections are performed by means of urban runoff models. Although the model user does not need a detailed understanding of such computations, he needs a general overview to understand possible limitations of obtained results. Two examples of inlet hydrograph computations, as used in two popular urban runoff models, are given below.

a. ILLUDAS Model (21)

Separate hydrographs are developed for impervious and pervious parts of the catchment and combined into a single inlet hydrograph. On the impervious part, the only rainfall abstraction considered is the initial loss which is a combination of initial surface wetting and depression storage. In the next step, the curve of the contributing area vs. the travel time to inlet is prepared. The runoff hydrograph is then produced by applying the rainfall excess pattern to the appropriate contributing area and summing up runoff contributions arriving at the inlet during each time step.

On pervious parts, the input represents the rainfall plus runoff from supplemental (not connected) impervious areas. The abstractions consist of the initial abstraction (surface wetting plus surface depression storage) and of infiltration which is computed from standard curves. Again, a curve of the contributing area vs. the inlet travel time is prepared and used to produce a runoff hydrograph. The inlet hydrograph is then obtained by adding hydrographs from impervious and pervious parts of the catchment.

b. Storm Water Management Model (12)

The plannar surface is divided into three planes - impervious elements with depression storage, impervious elements without depression storage, and pervious surfaces. For each type, different abstractions are considered.

In the case of impervious areas without depression storage, there are no abstractions and all rainfall is converted into runoff. On impervious areas with depression storage, the depression storage depth is subtracted from the rainfall depth to obtain the rainfall excess. Finally on pervious areas, depression storage as well as infiltration abstraction are considered. Infiltration is computed from the Horton or Green-Ampt equations.

The rainfall excess is routed, separately for each of the three surface types, using the kinematic wave approximation. The Manning flow equation and the equation of continuity are solved simultaneously at each time step to obtain overland flow hydrographs. All three hydrographs are added to obtain the inlet hydrograph.

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Table 1. Estimates of the Manning roughness coefficient for various types of ground cover (After ref. 5)

Ground Cover	Manning's n for Overland Flow
Smooth asphalt	0.012
Asphalt or concrete pavement	0.014
Packed clay	0.030
Light turf	0.200
Dense turf	0.350
Dense shrubber or forest litter	0.400

Table 2. Depression storage depth for various ground covers

Ground Cover	Depression Storage Depth (in)	Reference
Impervious Areas		
Impervious areas in general	0.0625	22
Impervious areas of various	$d = 0.03/\sqrt{S}$	14
slopes (S = 0.005 - 0.050)	0.05 - 0.15*	27
Large paved areas	0.10 - 0.30*	27
Roofs, flat	0.05 - 0.10*	27
Roofs, sloped		
Pervious Areas		
Urban pervious areas in general	0.250	22
Lawns, grass	0.20 - 0.50*	27
Sandy soils	0.20	8
Loan soils	0.15	8
Clay soils	0.10	8
Woods and open fields	0.20 - 0.60*	27

* Referred to as the depression and detention storage in the original reference

Table 3. Horton's parameters f_o, f_c, and K for various soils, ground cover, and moisture content

Soil Type	Ground Cover	Moisture Content	f_o (in/hr)	f_c (in/hr)	K hr^{-1}	Reference
All soils	-	-	-	-	3 - 6	12
A*	Turf	Dry	10.0	-		21
A*	Bare	-	-	0.30 - 0.45		17
B*	Turf	Dry	8.0	-		21
B*	Bare	-	-	0.15 - 0.30		17
C*	Turf	Dry	5.0	-		21
C*	Bare	-	-	0.05 - 0.15		17
D*	Turf	Dry	3.0	-		21
D*	Bare	-	-	0.00 - 0.05		17
Sandy soils	Dense vegetation	Dry	10.0	-	-	13
		Partly Drained	4.0 - 6.7	-	-	12
		Moist	3.3	-	-	12
	No vegetation	Dry	5.0	-	-	12
		Partly Drained	2.0 - 3.3	-	-	12
		Moist	1.7	-	-	12
Loam soils	Dense vegetation	Dry	6.0	-	-	13
		Partly Drained	2.4 - 4.0	-	-	12
		Moist	2.0	-	-	12
	No vegetation	Dry	3.0	-	-	12
		Partly Drained	1.2 - 2.0	-	-	12
		Moist	1.0	-	-	12
Clay soils	Dense vegetation	Dry	2.0	-	-	13
		Partly Drained	0.8 - 1.3	-	-	12
		Moist	0.7	-	-	12
	No vegetation	Dry	1.0	-	-	12
		Partly Drained	0.4 - 0.7	-	-	12
		Moist	0.3	-	-	12

* SCS soil classification:

A - Low runoff potential, high infiltration rates (deep sand, deep loess, aggregated silts).

B - Moderate infiltration rates and moderately well drained (shallow loess, sandy loam).

C - Slow infiltration rates (clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay).

D - High runoff potential, very slow infiltration rates (soils which swell significantly when wet, heavy plastic clays, certain saline soils).

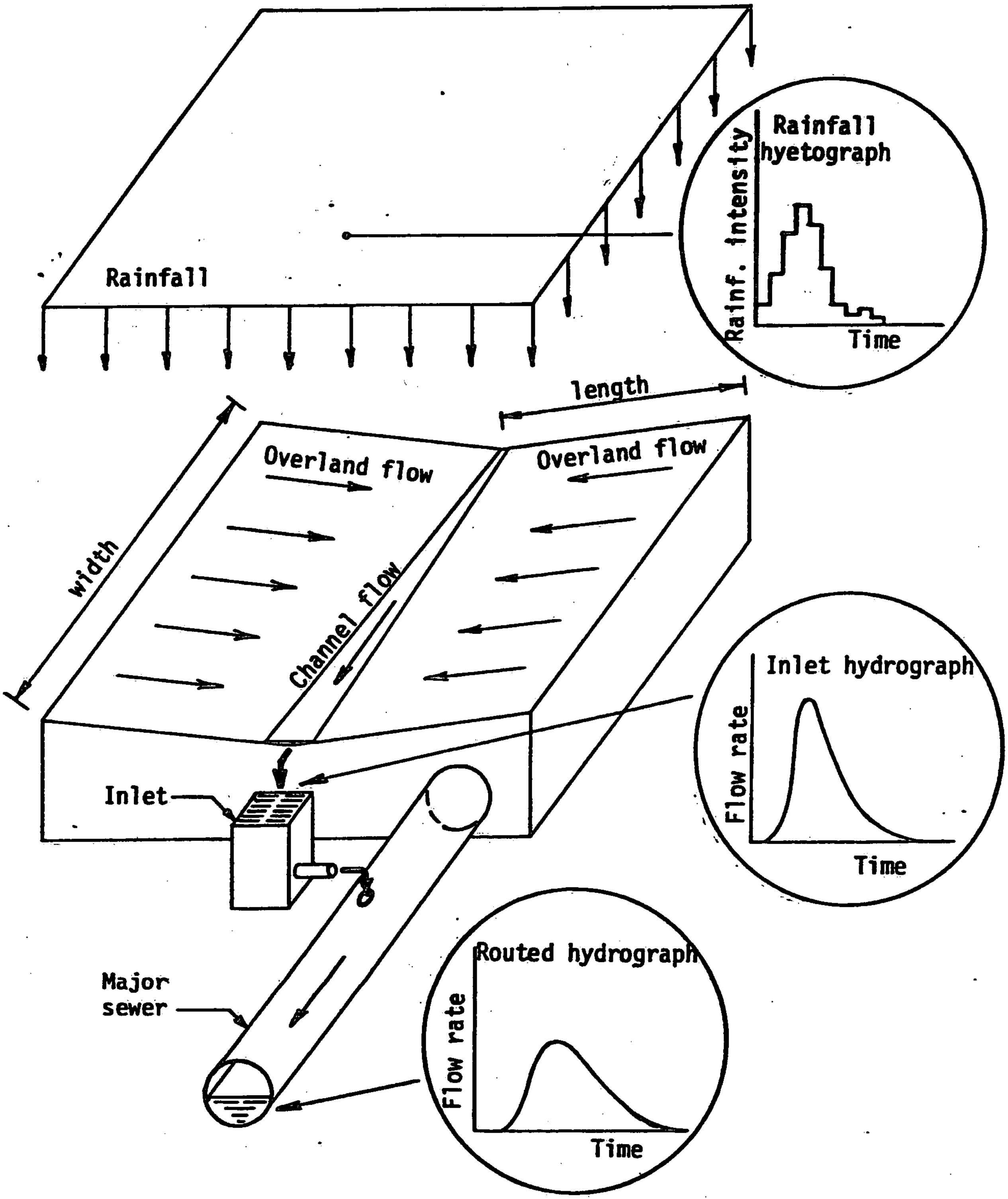

Fig.1. Simplified Urban Drainage System

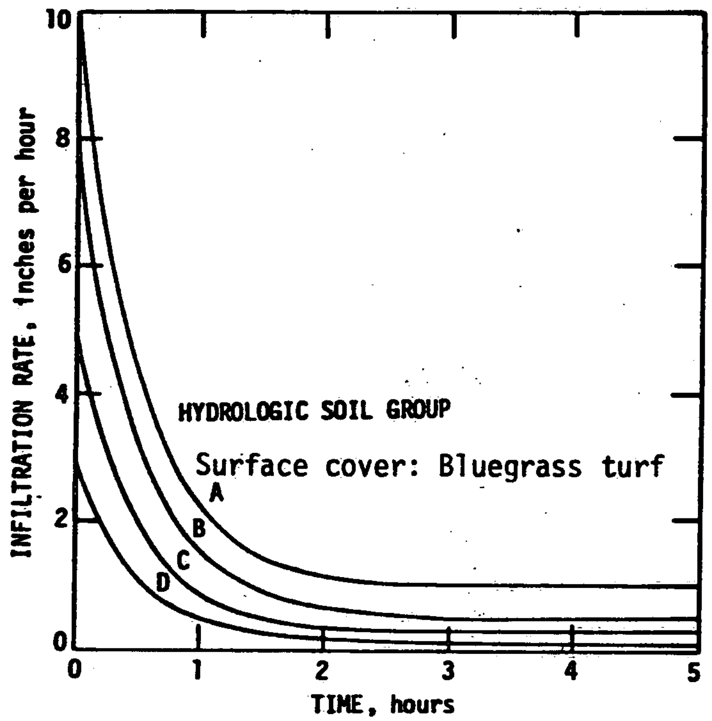

Fig.2. Examples of Infiltration Curves Used in the ILLUDAS Model(21)