URBAN STORM WATER RUNOFF
DETERMINATION OF VOLUMES AND FLOWRATES

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URBAN STORM WATER RUNOFF

DETERMINATION OF VOLUMES AND FLOWRATES

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Contract No. 68-03-0302
Program Element No. 1BB034

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FOREWORD

A prerequisite for effective control of storm runoff pollution is a reliable method to predict the quantity of the storm runoff. The time distribution of storm runoff from an urban drainage system depends on the areal and temporal distributions of the intensity of the rainfall, the frequency of the rainstorm, and the physical characteristics of the drainage system. Numerous methods have been proposed to evaluate urban runoff from rainfall. Many have been accepted for engineering applications while others need yet to be tested and verified. Therefore an investigation to evaluate the methods on a common basis would be a significant contribution to the recent efforts on pollution control.

ABSTRACT

An investigation is made to (a) develop a method of depth-duration-frequency analysis for precipitation events having short return period (high frequency) for urban storm water runoff management and control purposes; (b) develop a new high accuracy urban storm water runoff determination method which when verified, can be used for projects requiring high accuracy detailed runoff results and can also be used as the calibration scale for the less accurate urban runoff prediction methods; and (c) compare and evaluate selected urban storm water runoff prediction methods. The eight methods evaluated are the rational method, unit hydrograph method, Chicago hydrograph method, British Road Research Laboratory method, University of Cincinnati Urban Runoff method, EPA Storm Water Management Model, Dorsch Hydrograph Volume method, and Illinois Urban Storm Runoff method. The comparison and evaluation is done by using four recorded hyetographs of the Oakdale Avenue Drainage Basin in Chicago to produce the predicted hydrographs by the methods and the results are compared with recorded hydrographs. The relative merits of the methods are discussed and recommendations are made.

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ACKNOWLEDGMENTS

During the period of this project, many people have contributed to the progress of the work. Many students and assistants at the University of Illinois at Urbana-Champaign have helped in various phases of data collection, card punching, data analysis, and numerous other computations. Many engineers in various federal and local government agencies, consulting firms, research institutes and universities provided indispensable help in supplying data, information exchanges, and discussions, although only selected data are used in this report and not all the ideas provided by these people are presented here. The names of these people are too numerous to be listed here and their contributions are greatly appreciated. Particular thanks are due Professor Wayne Huber of the University of Florida, Dr. Albin Brandstetter of Battelle Pacific Northwest Laboratories, and Dr. P. Wisner of James F. MacLaren, Ltd. for their help.

Mr. A. Osman Akan, Graduate Research Assistant in the Department of Civil Engineering, University of Illinois at Urbana-Champaign, contributed most to the successfulness of this research project. He programmed the Illinois Surface Runoff Model and the Water Quality Model, provided the interface between these models and the Illinois Storm Sewer System Simulation Model, performed the tedious and painstaking testing and computer computations, participated in the field survey for data collection, and wrote the section on Program Description and Data Preparation for the Illinois Surface Runoff Model. Without his help, this report would not be materialized. Mr. T. A. Ula, Graduate Research Assistant, performed the frequency analysis in addition to his contributions in data collection and various aspects of computations.

Throughout this research project, Mr. Chi-Yuan Fan, Project Officer, and Mr. Richard Field, Chief, Storm and Combined Sewer Section, EPA Advanced Waste Treatment Research Laboratory, provided smooth and fruitful cooperation.

Their help in supplying information on data sources and coordination with other researches is indispensable and deeply appreciated.

Thanks are also due Mrs. Norma Barton for her typing of the report and other materials for the project.

I. SUMMARY AND CONCLUSIONS

The purpose of this report is to provide information which is useful for operation and management of urban storm and combined sewer runoff and also useful for design and operation of overflow treatment and other control facilities. This report is divided into three major parts. The first part contained in Chapter IV is precipitation analysis on depth-duration-frequency analysis of short return period (high frequency) rainstorms. Conditional probability is utilized and method of application on urban runoff problems is demonstrated using the hourly precipitation record available from the U.S. Environmental Data Service, National Climatic Center.

The second part of the report consists of Chapters VI, VII, and VIII describing the Illinois Urban Storm Runoff method. It includes the development of the Illinois Urban Surface Runoff model to couple with the existing Illinois Storm Sewer System Simulation Model and the formulation of a non-reactive water quality model to compute the concentration of the pollutants of urban runoff. In the surface runoff model, Horton's formula is used to evaluate infiltration. The overland flow is computed by using the kinematic wave method together with Darcy-Weisbach's formula to estimate the friction slope. The gutter flow is computed by using the kinematic wave method together with Manning's formula to estimate the friction slope. Inlets to catch basins are classified in types according to their geometry, and weir and orifice formulas are used to estimate the discharge. The sewer flow routing utilizes the complete St. Venant equations accounting for the junction backwater effects.

The third part of the report consisting of Chapters V, IX, and X is a comparative evaluation of eight selected urban runoff prediction methods using the data from the Oakdale Avenue Drainage Basin in Chicago. The methods evaluated are the rational method, unit hydrograph method, Chicago hydrograph method, British Road Research Laboratory method, University of Cincinnati Urban Runoff method, EPA Storm Water Management Model, Dorsch Hydrograph Volume Method, and the Illinois Urban Storm Runoff method. This part of study is believed to be of particular interest to practicing engineers. The evaluation is made by using the recorded hyetographs of four rainstorms applying the methods to compute the predicted runoff hydrographs and then compare the results with the recorded hydrographs. This comparative study suggests that the most suitable method to be used for an urban runoff problem depends on the accuracy required for the project. If only a quick simple approximate result of peak runoff rate is needed, the rational method is quite satisfactory, whereas for a project involving a large amount of money and high accuracy and details of the temporal and spatial distribution of the runoff are required, the Illinois Urban Storm Runoff method will be a suitable choice, and the Dorsch method and EPA SWMM may be the alternatives if backwater effects are not important. For in-between accuracy, the unit hydrograph method is recommended, if possible. When the unit hydrograph for the drainage area is not available, in most cases the Road Research Laboratory method appears to be superior to the Chicago and University of Cincinnati methods.

II. RECOMMENDATIONS

Based on the results of this investigation, the following recommendations related to the determination of flow rate of urban storm water runoff are made:

1. The most suitable method to be used for determination of urban storm runoff depends on the objective and size of the project, the required accuracy and detail of the flow rate determination, and the available data.
2. For the purpose of storm water runoff control and management, when required results are extensive (such as discharge and depth or velocity at different times and at many locations of the drainage basin) and the required accuracy is high such as in the case of a high-valued urban area or a high-cost project, and the detailed basin data is available, the Illinois Urban Runoff method appears to be a suitable choice. The Dorsch hydrograph-volume method may also be used. If the backwater effect of sewer junctions is not important, the EPA SWMM is a good alternative.
3. For a cheap, simple and quick estimation of the peak runoff rate without requiring the entire runoff hydrograph, the rational method can often be used satisfactorily. However, one should always bear in mind the limited accuracy the rational method can provide - a sacrifice of accuracy for the sake of simplicity.
4. For projects or problems requiring moderate accuracy and determination of the entire or part of the runoff hydrograph, the unit hydrograph method is the simplest

and cheapest to use if the unit hydrograph is available or can be reliably derived. In using the unit hydrograph, one should check that there should be no significant change in basin characteristics. If the unit hydrograph is not available, the order of choice would be the British Road Research Laboratory method, Chicago hydrograph method, and University of Cincinnati Urban Runoff Model.

5. The comparative study is made on only eight methods and is neither exhaustive nor exclusive. Other methods not mentioned here may also be useful.
6. Existing and available data on urban rainfall, runoff, and basin characteristics are generally inadequate for a reliable accurate evaluation of the methods. Neither are the data adequate for engineering operation, management and design purposes. This inadequacy is in both details and accuracy. For example, the best readily available rainfall data, the hourly data from the Environmental Data Service, National Climatic Center, is inadequate in view of many urban drainage basins having a time of travel much shorter than an hour.
7. More study on urban infiltration would be desirable. This includes the determination and listing of infiltration parameters such as f_c and k in Horton's formula for typical urban surface conditions, and data on information such as anticedent moisture condition related to infiltration.
8. A detailed accurate evaluation of the surface runoff part of the methods is desirable. Because of the lack

of sufficient detailed geometric (percentage and distribution of roofs, lawns, sidewalks, driveways, etc.) and infiltration data, it is believed that the effects of the surface runoff on the methods have not been adequately evaluated. In addition, surface runoff hydrographs at certain urban locations will also be useful for engineering purposes.

9. Further study on hydraulic characteristics of inlets is necessary. There is no use to have a highly sophisticated surface runoff model if its downstream control, the inlet, cannot be accurately modeled. Existing information is inadequate to represent the large number of types of inlets now being used.

10. Improvement on the manner to handle surcharge and supercritical flow in sewers would be useful and desirable.

11. Further study on differences between design and flow prediction purposes on modeling and data requirement may be useful for urban storm runoff management.

12. In view of the probabilistic nature of the physical conditions of the drainage system, such as clogging of inlets and gutters, interference of parked vehicles on street and gutter flows, change of roughness of sewers, etc., development of a probabilistic method to account for such uncertainties will be useful for both design and operation purposes.

13. It is of course possible to further improve the hydraulic aspects of the existing sophisticated methods. For instance, the kinematic wave routing of the overland and

gutter flows can be replaced by solving the more accurate St. Venant equations. However, at present such improvement appears to be unfruitful and immature because of the uncertainties involved in the basin physical properties and the detail and reliability of the data. Furthermore, such improvement would require considerably more computer time, thus making the method impractical.

14. An advanced stochastic approach to analysis and predicted high frequency rainfall as an alternative to the method proposed in Sec. IV-1 may be useful.

III. INTRODUCTION

III-1. Problems of Urban Storm Water Runoff

Metropolitan areas, in the United States as well as elsewhere in the world, are growing at an unprecedented rate. One result often associated with urbanization is the deterioration of the living environment due to either lack of comprehensive planning or incapability of the cities to keep pace with the growth. Among the vital facilities in preserving the living environment, urban sewer drainage systems affect directly the quantity and quality in disposing urban waste water. Large amounts of money and resources are involved in the design, construction, modification, operation and maintenance of urban sewer systems. There are two major types of problems related to urban storm water runoff: (1) flooding due to inadequate sewer capacity causing damage of properties and disruption of traffic and other human activities; and (2) pollution due to storm runoff. The flooding problem is a design problem involving design rainstorms having return period of once in several years. Many methods have been developed for sewer design purpose in the past 130 years (Chow, 1962, 1964). The common objective of these methods is to provide a design flow for sizing the sewers of a new storm drainage system or an existing system with adequate capacities to dispose of this once-in-many-year design flow without flooding.

Conversely, the storm runoff pollution problem is an operation and management problem. It involves consideration of rainstorms with frequencies of several times in a year. For an existing drainage system the problem involves the management of the time distribution of the quantity and quality of storm water so that the runoff would not overload the treatment facilities or unacceptably pollute the receiving water bodies. This may require some modification of the drainage facilities. For new drainage systems a balanced design considering both the pollution control and adequate capacity to avoid flooding would require an optimization analysis.

During the past decade the public as well as the engineering profession has been greatly alarmed by the pollution aspects of urban storm runoff. Many investigations have been sponsored by EPA and by other agencies on the extent of urban runoff pollution (e.g., see American Public Works Association, 1967, 1969; Engineering-Science, Inc., 1967; Envirogenics Company, 1971; Hawkins and Judd, 1972; Sartor and Boyd, 1972; Weibel et al., 1964). Recent studies on urban storm runoff pollution and efforts on its control and management have been summarized in two excellent literature reviews (Field and Weigel, 1973; Field and Szeeley, 1974).

One fundamental prerequisite for an efficient and economic design and operation of urban sewer systems is a reliable method to predict the quantity and quality of water handled by the system, particularly the time distribution of runoff due to rainfall at important locations such as junctions and overflow facilities. Numerous methods have been proposed to estimate rainstorm runoff. Many of these methods are of regional nature whereas many others are applicable only to rural or similar natural drainage basins. For those methods applicable to urban areas, some treat the drainage area as a "black box" without considering the time or space distribution of the runoff in the drainage system; others treat the drainage system as sequential overland and channel flows without considering the detention storage in the drainage system due to backwater effects of the junctions. Furthermore, many of the improved methods which have been proposed recently are not yet widely accepted mainly because their relative merits have not been assessed, nor have they been compared in a comprehensive manner on a common basis to the previously developed methods.

However, in order to comply with recent pollution control laws, attempts have been made to utilize the storage capacity of a sewer system to detent storm runoff and to control overflow in order to reduce the cost in handling urban waste water problems (Poertner, 1972; Anderson et al., 1972; Field and Struzeski, 1972). Examples of such attempts are the automatic control systems built or proposed in Minneapolis-St. Paul (Anderson, 1970; Tucker, 1971), Seattle, San Francisco, and Detroit (Field and Struzeski, 1972). Obviously, a reliable storm runoff prediction method would be particularly useful and beneficial for such efforts. In fact, without a reliable runoff quantity prediction it is most unlikely that a satisfactory runoff quality prediction can be achieved.

III-2. Objectives and Scope of Study

As discussed in the preceding section, a "demonstration" type investigation to evaluate quantitatively on a common basis the relative merits of the conventional as well as recently developed runoff prediction methods is needed for improvement in urban storm water management and pollution control. Since many of the urban storm runoff models are being compared quantitatively in an EPA project at Battelle Pacific Northwest Laboratories using hypothetical data, it is not the intention of this investigation to evaluate all of the existing urban storm runoff models. Accordingly, the major objectives of the present study are: (a) to develop a surface runoff model, coupled with an existing sewer routing model previously developed at the University of Illinois, the Illinois Storm Sewer System Simulation Model, to form an urban storm runoff method; and (b) to evaluate this new method using actual field data and also compare the results quantitatively with other relatively popular methods using the same field data from the Oakdale Avenue Drainage Basin in Chicago. The methods compared in this study include the rational method, unit hydrograph method, Chicago hydrograph method, British Road Research Laboratory method, EPA Storm Water Management Model, University of Cincinnati method, Dorsch Hydrograph-Volume method, and the Illinois Urban Storm Runoff method. The description of the last method is given in Chapters VI and VII. Brief descriptions of the other seven methods are given in Chapter IX.

In addition to the Oakdale Drainage Basin (0.052 km^2 or 12.9 ac) which was chosen to test the eight chosen methods because of its available data and previous studies, a much bigger basin, the Boneyard Creek Drainage Basin in Champaign-Urbana, Illinois (9.3 km^2 or 3.0 sq mi) was also used to test the applicability of the Illinois Urban Storm Runoff method on large basins. However, because of the enormous cost, time, and manpower that would be involved if the other methods were also tested on the Boneyard Basin, and the result would probably produce no additional information than that from the Oakdale Basin, the other seven methods were not tested on the Boneyard Basin. Furthermore, because of the large amount of data for the Boneyard Basin due to its size, its inclusion here would make this report voluminous. Therefore, its physical properties together with the results to demonstrate the applicability of the Illinois Urban Storm Runoff method on large basins may later be presented as a separate supplementary report.

Also, an auxiliary objective to objectives (a) and (b) mentioned above is (c) to develop a method of rainfall depth-duration-frequency analysis suitable for urban storm runoff management purposes.

IV. PRECIPITATION ANALYSIS

Not all the rainwater falling on an urban area becomes runoff. There are infiltration and other losses called abstractions. The total precipitation subtracted by the abstractions is called precipitation excess. Since both precipitation and abstractions are statistical quantities, precipitation excess is also of statistical nature. In this chapter precipitation and abstractions are discussed in view of urban storm runoff.

IV-1. Rainfall Depth-Duration-Frequency Analysis for Urban Runoff

Natural rainfalls have finite duration and areal and temporal variation of their intensities. It has yet to be found that two rainfalls are identical. Thus rainfall data is analyzed statistically to be useful for engineering purposes. The intensity of rainfall is a function of its duration, frequency, and area, which has been discussed extensively elsewhere (e.g., U.S. National Weather Service, 1961; Chow, 1964). There are two types of rainfall information needed for urban storm runoff management purposes. The first is the duration and maximum intensity for rainfalls having long return periods of a number of years to be used for design and safety considerations. The other is the information on high frequency rainstorms with return periods less than a year to be used for operation and pollution control purposes.

Because of the large number of rainfalls involved for a given location, conventionally in engineering hydrology as well as in meteorology only the maximum values in the form of partial duration series or annual maximum series are analyzed to establish the rainfall intensity-duration-frequency relationship for long return period events. This is done so because usually for the purpose of design of drainage facilities based on catastrophic failure concept (Yen and Ang, 1971) rainfall of small return period is usually of no significance. However, this is not the case from an operational viewpoint for the control of pollution due to urban storm runoff for which every rainstorm contributes to the problem. Because of the lack of dilution effect due to their small volume, the low return period rainstorms probably contribute considerably more to pollution per unit volume of water than the long return period ones. Treatment plants, detention storages, overflows and other sewer facilities designed for small capacities corresponding to most frequent rainfalls would not be able to control the runoff from less frequent rainstorms. Contrarily, such facilities designed to operate at full capacity only once in every several years would be costly and unjustified if the safety consideration does not require it.

Obviously, the volume and time distribution of storm runoff quantity and quality from an urban drainage system depend on the areal and temporal distributions of the rain falling on the urban basin. Consequently, the depth, duration, and frequency of the rainfall and other parameters defining the internal pattern of the rainstorm should all be considered. In fact, the time elapsed between successive rainfalls is also an important factor in determining the quantity and especially the quality of storm runoff. Conceivably, most of the water from a rainstorm followed soon after an earlier heavy rainfall would become runoff and the quality of the water would be relatively better.

Information on long return period rainfall useful for urban runoff studies have been well established and can readily be found (e.g., Chow, 1953; U.S. Weather Service, 1961). Unfortunately, analytical information on high frequency rainstorms with return periods less than a year which is useful for urban engineering purposes is practically nonexistant. This lack of information is due mainly to the large amount of data needed to be analyzed and to the difficulty in defining precisely the duration and intensity of a rainstorm. The purpose of this part of the study is to provide a practical and realistic model on a probablistic basis for rainfall input for urban storm runoff studies and operations.

The frequency analysis methods for rainfalls of long return period can be extended to short return period rainfalls. However, unlike the long return period case for which data can be selected to form a partial series for the analysis, for short return period analysis the entire set of data, i.e., all the rainstorms recorded, are utilized. Grayman and Eagleson (1969) applied this concept on hourly rainfall data for 546 rainstorms in a 5-year period at Boston.

Ideally, the precipitation data used for short return period frequency analysis should be a continuous record of hyetograph (rainfall-time curve). For most precipitation recording-gaging stations in the United States, the most detailed precipitation data readily available from the Environmental Data Service, National Climatic Center* are hourly records. Since many urban drainage basins have the time of travel of surface runoff less than an hour, the hourly rainfall data is obviously inadequate and unsatisfactory. Data of 5- or 10-min intervals would be much more satisfactory. Unfortunately, rainfall records having time intervals shorter than one hour may only be obtained by the user from the original record, if available, which is a very time consuming process and most unlikely to be undertaken by

*Hourly rainfall data for recording raingages in the U.S. Weather Service system are available at cost on punched cards from U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Environmental Data Service, National Climatic Center, Federal Building, Asheville, N.C. 28801.

practicing engineers. Therefore, from a practical viewpoint, this part of research on rainfall frequency analysis method is developed by using hourly rainfall records. Nevertheless, the methodology is equally applicable to data of other time intervals.

The method of analysis which has been written in a computer program (Appendix A) will be described in the following in this section. The data used to demonstrate the application of the method are the point hourly precipitation data on punched cards as provided by the National Climatic Center for the Morrow Plot raingage at Urbana, Illinois, (11-8740 Urbana), covering 14 years from 1959 to 1972. Because of the seasonal characteristics of rainstorms, the analysis is carried out separately for different seasons. The analysis for the record of June, July, and August consisting of 455 rainstorms is presented here as an example.

(A) Identification of rainstorms. - A rainstorm is defined here as a period of continuous non-zero rainfall. However, since the input data provided on punched cards do not identify traces (rainfall less than 0.01 in./hr or 0.25 mm/hr), there is no way to differentiate traces from hours of no rainfall in the data. The duration of a rainstorm, t_d in hours, is defined as the length of time between the beginning and the end of a continuous non-zero rainfall. The total volume of a rainstorm as customarily expressed in depth, D in in. or mm covering the entire area considered, is equal to the sum of the depth for each time interval within the duration of the rainstorm, i.e.,

$$D = \sum_{j=1}^{n} d_j \tag{1}$$

in which d_j is the depth for the j-th time interval and n is the number of time intervals of the rainstorm. In engineering practice, as a matter of convenience, equal time interval Δt is usually used and the standard time interval used here is one hour according to the standard U.S. Weather Service data, although other time intervals can also be specified. The average intensity of the rainstorm, i, is defined as equal to D/t_d, and usually expressed in in./hr or mm/hr. The elapse time between successive rainstorms, t_b, is defined as the time between the end of a rainstorm and the beginning of the next rainstorm. The computer program traces the entire record, identifies the rainstorms and determines the values of D, i, t_d, and t_b.

(B) Calculation of rainstorm parameters. - A schematic drawing of an example hyetograph is shown in Fig. 1. The other parameters of the rainstorms essential for a statistical analysis are computed as follows. The standard deviation of the rainstorm depth, σ_d in in. or mm, is computed as

$$\sigma_d = \left[\frac{\sum_{j=1}^{n} (d_j - \bar{d})^2}{n}\right]^{1/2} \tag{2}$$

where the average depth per time interval, $\bar{d}$ in in. or mm, is

$$\bar{d} = \frac{\sum_{j=1}^{n} d_j}{n} \tag{3}$$

The first moment arm of the hyetograph with respect to the beginning time of the rainstorm, $\bar{t}$ in hr, is

$$\bar{t} = \Delta t \, [\sum_{j=1}^{n} (j-0.5) \, d_j] / \sum_{j=1}^{n} d_j \tag{4}$$

and the corresponding second moment arm, G in hr^2, is

$$G = (\Delta t)^2 \, [\sum_{j=1}^{n} (j-0.5)^2 d_j + \frac{1}{12} \sum_{j=1}^{n} d_j] / \sum_{j=1}^{n} d_j \tag{5}$$

(C) Nondimensional hyetograph. - In order to describe in more general terms the time distribution of the rainfall of a rainstorm, the hyetograph is nondimensionalized by using the rainstorm depth D and duration t_d as the nondimensionalizing parameters. Therefore, for a time interval, the nondimensional depth $d_j^o = d_j/D$ where the superscript (o) represents the nondimensional quantity. Accordingly, $D^o = \sum_{j=1}^{n} d_j^o = 1$ and $t_d^o = 1$. The nondimensional average intensity $i^o = D^o/t_d^o = 1$. Similarly, the nondimensional average depth is

$$\bar{d}^o = \frac{\sum_{j=1}^{n} d_j^o}{n} = \frac{1}{n} \tag{6}$$

The nondimensional standard deviation of the depth is

$$\sigma_d^o = \left[\frac{\sum_{j=1}^{n} (d_j^o - \bar{d}^o)^2}{n}\right]^{1/2} = \frac{\sigma_d}{D} \tag{7}$$

The first moment arm of the nondimensional hyetograph is

$$\bar{t}^o = \frac{\bar{t}}{t_d} \tag{8}$$

The second moment arm of the nondimensional hyetograph is

$$G^o = \frac{G}{t_d^2} \tag{9}$$

(D) Shape of hyetographs. - The shape of the hyetographs may be approximated by some simple geometric figures. Assuming that the hyetographs can be represented by trapezoids shown in Fig. 2,

$$t_d = a + b + c \tag{10}$$

$$\bar{d} = \frac{h}{2} \left(1 + \frac{c}{t_d}\right) \tag{11}$$

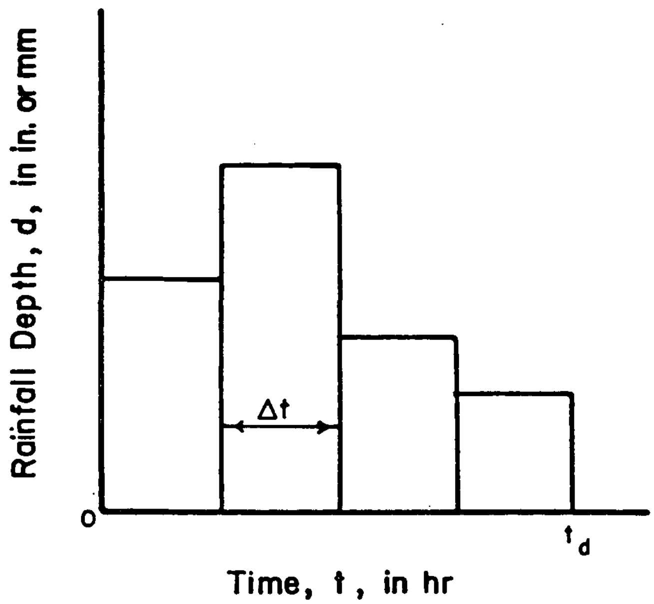


Fig. 1. Example hyetograph

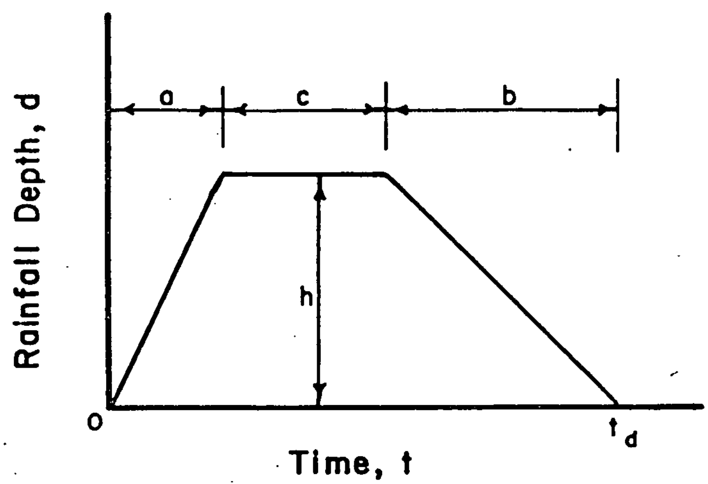


Fig. 2. Trapezoidal representation of hyetograph

$$\bar{t} = \frac{t_d(t_d + a + c) + c(2a + c)}{3(t_d + c)} \tag{12}$$

and

$$G = \frac{t_d^3 + (a+c)\ t_d^2 + (a+c)^2\ (t_d+c) + ca(2a+c)}{6(t_d + c)} \tag{13}$$

Equations 10 through 13 can be solved simultaneously for a, b, c, and h so that the trapezoid representing the hyetograph can be determined. To demonstrate the methodology in application, it is assumed that a special case of the trapezoidal shape with c = 0, i.e. triangles, can be used to approximate the Urbana rainfall data. For this triangular case, solving Eqs. 10, 11, and 12 yields

$$a = 3\bar{t} - t_d \tag{14}$$

$$b = 2t_d - 3\bar{t} \tag{15}$$

and

$$h = 2\bar{d} \tag{16}$$

For nondimensional hyetograph,

$$a^o = a/t_d = 3\bar{t}^o - 1 \tag{17}$$

$$b^o = b/t_d = 2 - 3\bar{t}^o \tag{18}$$

and

$$h^o = h/D = 2\bar{d}/D = 2/n \tag{19}$$

The statistics of the parameters for the 455 summer rainstorms at Urbana, Illinois are given in Table 1.

(E) Frequency analysis of rainstorm parameters. - With the rainstorm parameters computed for every rainstorm in the record, a one-way frequency analysis can subsequently be made for each parameter. The computer

Table 1. STATISTICS OF PARAMETERS FOR SUMMER RAINSTORMS AT URBANA, ILLINOIS

Parameter		Mean	Standard Deviation	Min	Max
t_b,	hr	59.4	81.9	1	744
t_d,	hr	2.45	2.08	1	14
D,	in.	0.31	0.51	0.01	3.52
	mm	7.87	13.0	0.25	89.4
i,	in./hr	0.11	0.15	0.01	1.05
	mm/hr	2.79	3.81	0.25	26.7
σ_d,	in.	0.06	0.11	0	0.87
	mm	1.52	2.79	0	22.1
$\bar{t}$,	hr	1.14	1.01	0.50	7.68
G,	hr^2	2.90	7.10	0.33	71.2
a,	hr	0.95	1.39	-5.00	10.0
b,	hr	1.50	1.80	-2.10	13.8
h,	in.	0.21	0.30	0.02	2.09
	mm	5.33	7.62	0.51	53.1
σ_d^o		0.13	0.15	0	0.48
$\bar{t}^o$		0.47	0.11	0.13	0.80
G^o		0.30	0.11	0.05	0.68
a^o		0.42	0.32	-0.62	1.40
b^o		0.58	0.32	-0.40	1.62
h^o		1.23	0.65	0.14	2.00

Number of Rainstorms = 455

program (Appendix A) has a one-way frequency analysis subroutine which tabulates for any given parameter the frequencies (number of observations over given intervals), relative frequencies (frequency divided by the total number of observations), probability densities (relative frequency divided by the interval size) and non-exceedance probabilities (cumulative relative frequencies). The mean and standard deviation of the parameter are also calculated and the maximum and minimum values are found (Table 1). Histograms of the probability densities for the rainstorm parameters can then be plotted as shown in Figs. 3, 4, 5, and 6 for the rainstorm depth, duration, intensity, and elapse time between rainstorms, respectively.

(F) Fitting of probability density functions. - The next step is to fit some probability density functions to the histograms of the rainstorm parameters. Exponential distribution and gamma distribution have a non-negative range and have been applied in the analysis of non-negative valued rainstorm parameters. The probability density function for the exponential distribution is

$$f(x) = \frac{1}{B} e^{-x/B} \qquad x \geq 0 \text{ and } B > 0 \qquad (20)$$

where both the expected value and the standard deviation of the distribution are given by B. This single-parameter distribution can easily be fitted to data (Grayman and Eagleson, 1969) but the fact that it has the same expected value and standard deviation makes its use difficult to justify in most applications. The exponential density functions fitted in Figs. 3 through 6 all assume a value of B equal to the observed mean. Use of the mean rather than standard deviation for B implicitly implies that the mean is more "meaningful" than the standard deviation. The probability density function for the gamma distribution is

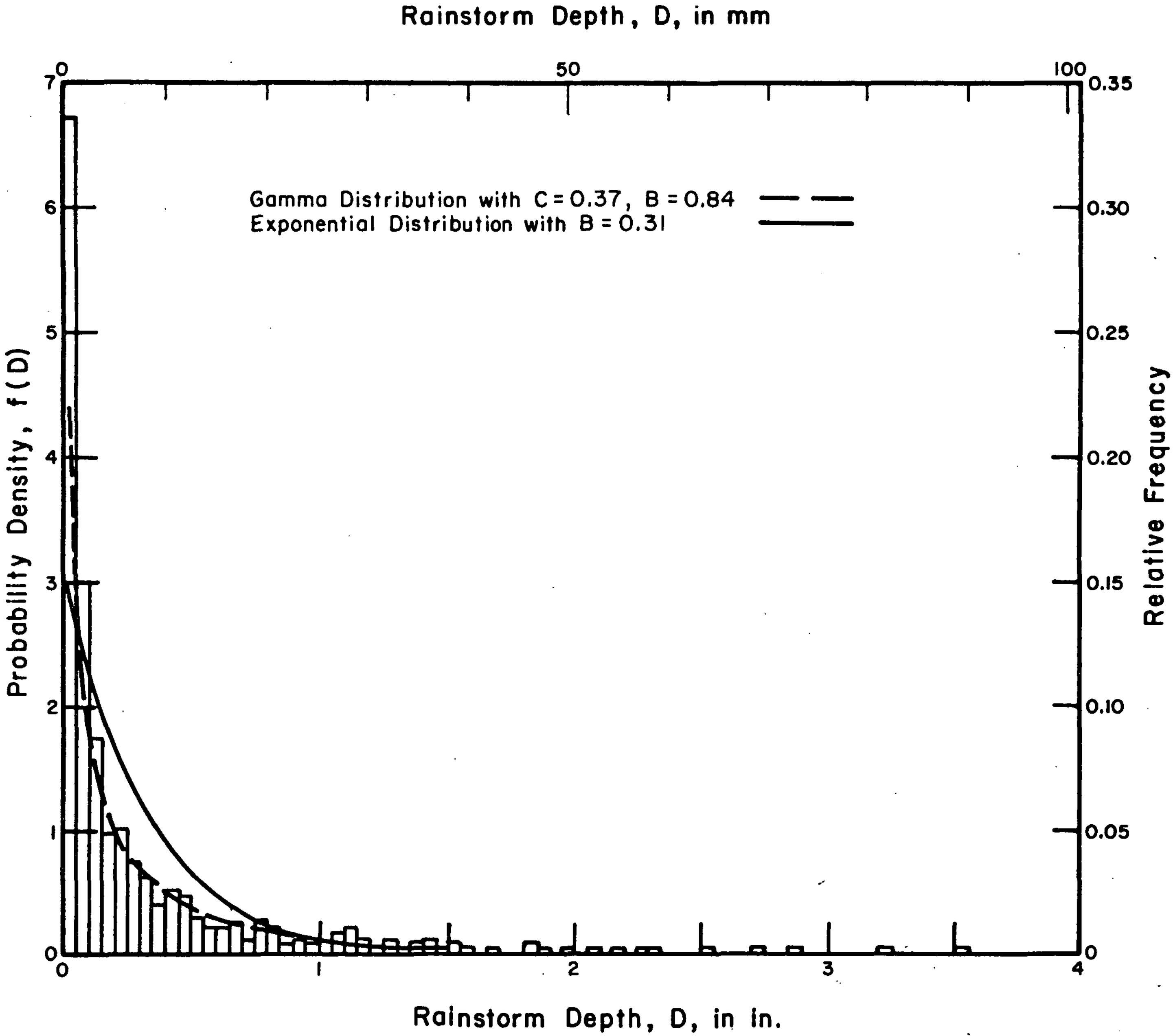


Fig. 3. Probability distribution of rainstorm depth

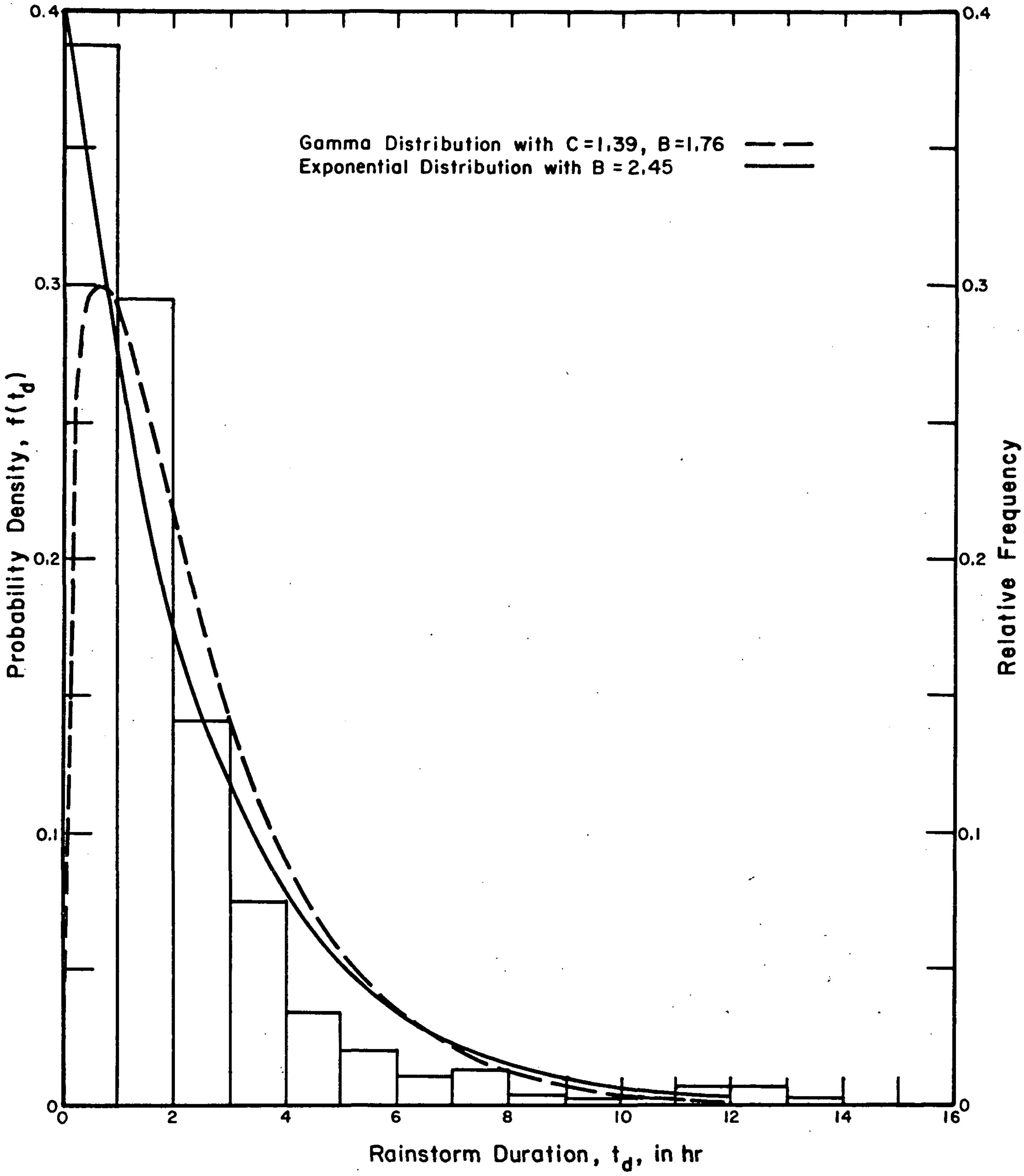


Fig. 4. Probability distribution of rainstorm duration

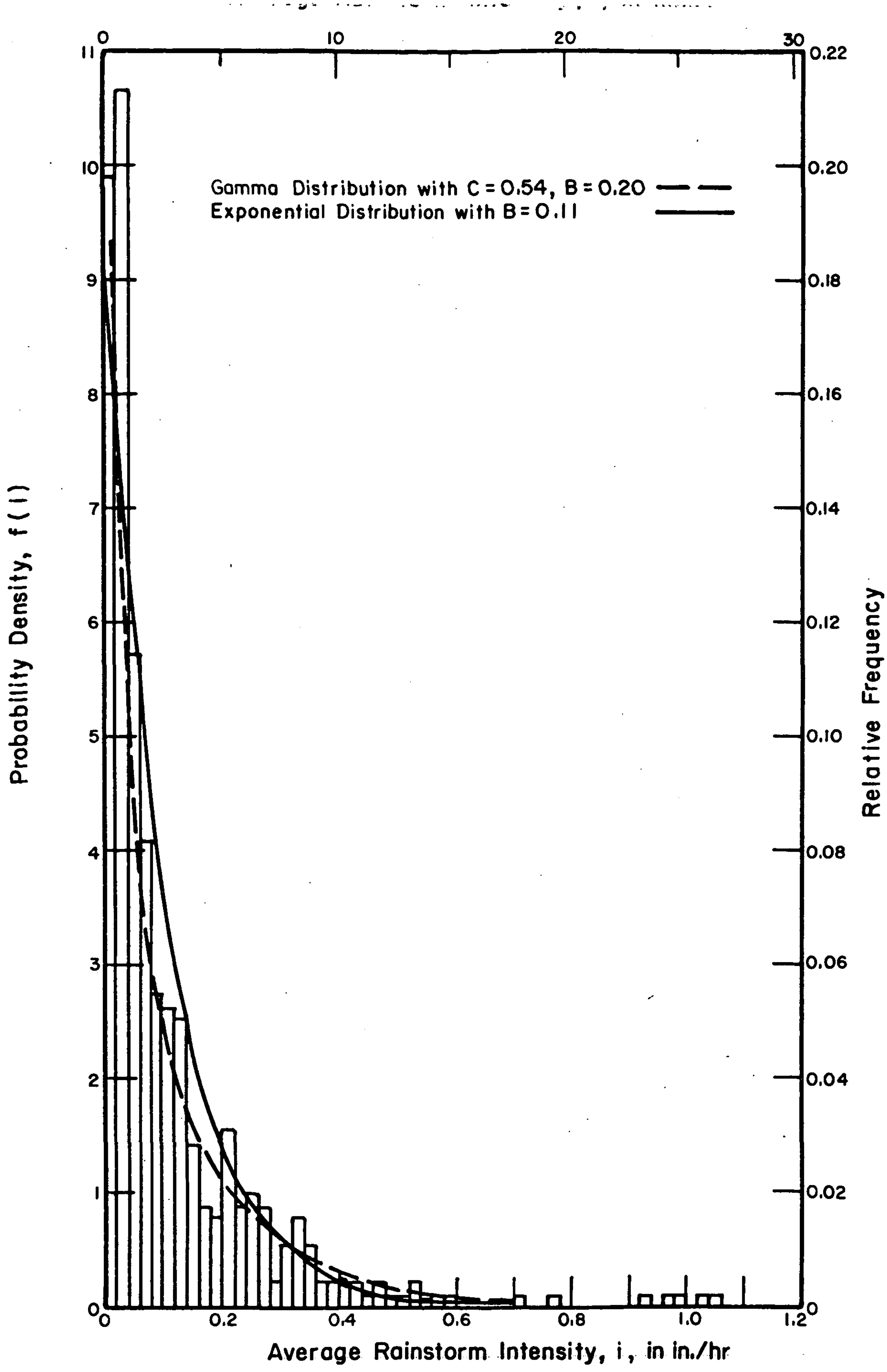


Fig. 5. Probability distribution of average rainstorm intensity

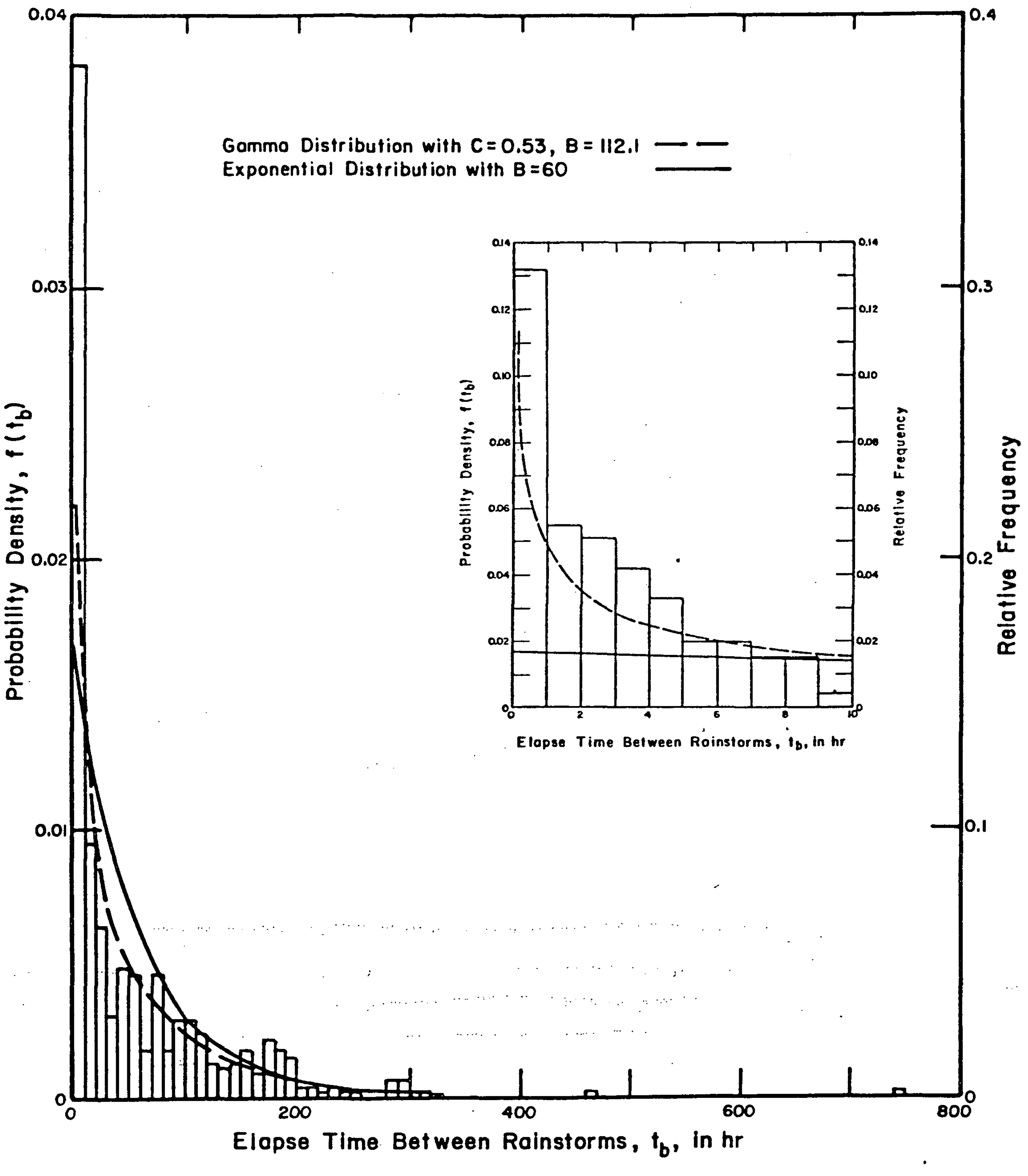


Fig. 6. Probability distribution of elapse time between rainstorms

$$f(x) = \frac{1}{\Gamma(C)B^C} x^{C-1} e^{-x/B} \qquad x \geqslant 0 \text{ and } B,\ C > 0 \tag{21}$$

where the gamma function

$$\Gamma(C) = \int_0^\infty z^{C-1} e^{-z} dz \qquad \text{for all } C > 0 \tag{22}$$

The expected value

$$E(x) = CB \tag{23}$$

and the variance

$$V(x) = CB^2 \tag{24}$$

The exponential distribution is actually a special case of gamma distribution with C = 1. The gamma distribution being a two-parameter distribution provides an extra degree of freedom compared to the exponential distribution in fitting data, and both mean and standard deviation of the observed data can be preserved by adjusting its parameters C and B. In Figs. 3 through 6, the values of C and B for the fitted gamma probabity density function are computed by using the observed values of mean and standard deviation:

$$C = \frac{E^2(x)}{V(x)} \tag{25}$$

$$B = \frac{E(x)}{C} \tag{26}$$

It can be seen from Figs. 3 through 6 that in general the gamma distribution provides a better fit to the data than the exponential distribution.

(G) Conditional frequency analysis of rainstorm parameters. - As the rainstorm parameters are not truly independent, conditional probabilistic analysis is included to provide information useful for solving urban storm runoff problems. The computer program has a two-way frequency analysis subroutine which tabulates, for any two given parameters, a two-dimensional table of frequencies, relative frequencies, and probability densities. The results can then be used to plot three-dimensional histograms of joint probability densities for pairs of rainstorm parameters. However, it is a formidable task to fit bivariate joint density functions to three-dimensional histograms. Mainly for this reason and also from a practical viewpoint, in this study joint distributions are dealt with through the use of conditional distributions. The computer program has a sorting subroutine which can sort the values of a rainstorm parameter in an ascending order and can rearrange simultaneously the corresponding values of another parameter. By using this subroutine together with the one-way frequency analysis subroutine, conditional frequency analysis can be carried out for pairs of rainstorm parameters.

The example described here is to find the conditional distributions of average rainstorm intensity, i, for different rainstorm durations. The same procedure can be applied to any other pair of dependent rainstorm parameters. By using the sorting subroutine, values of t_d are sorted in an ascending order (here they vary from 1 to 14 hr), and corresponding values of i are rearranged simultaneously. For a given value or a given range of values of t_d, the corresponding i values are picked up for a one-way frequency analysis as described in (E). By repeating this one-way frequency analysis of i for different values of t_d, a set of histograms of conditional probability densities of i are obtained as shown in Figs. 7a to 7e. Exponential and gamma density functions were then fitted to these histograms with parameters evaluated based on the observed conditional means and standard deviations. Tables 2 (a) to (e) give the conditional means, standard deviations, minimum and maximum values of each rainstorm parameter for different values of t_d.

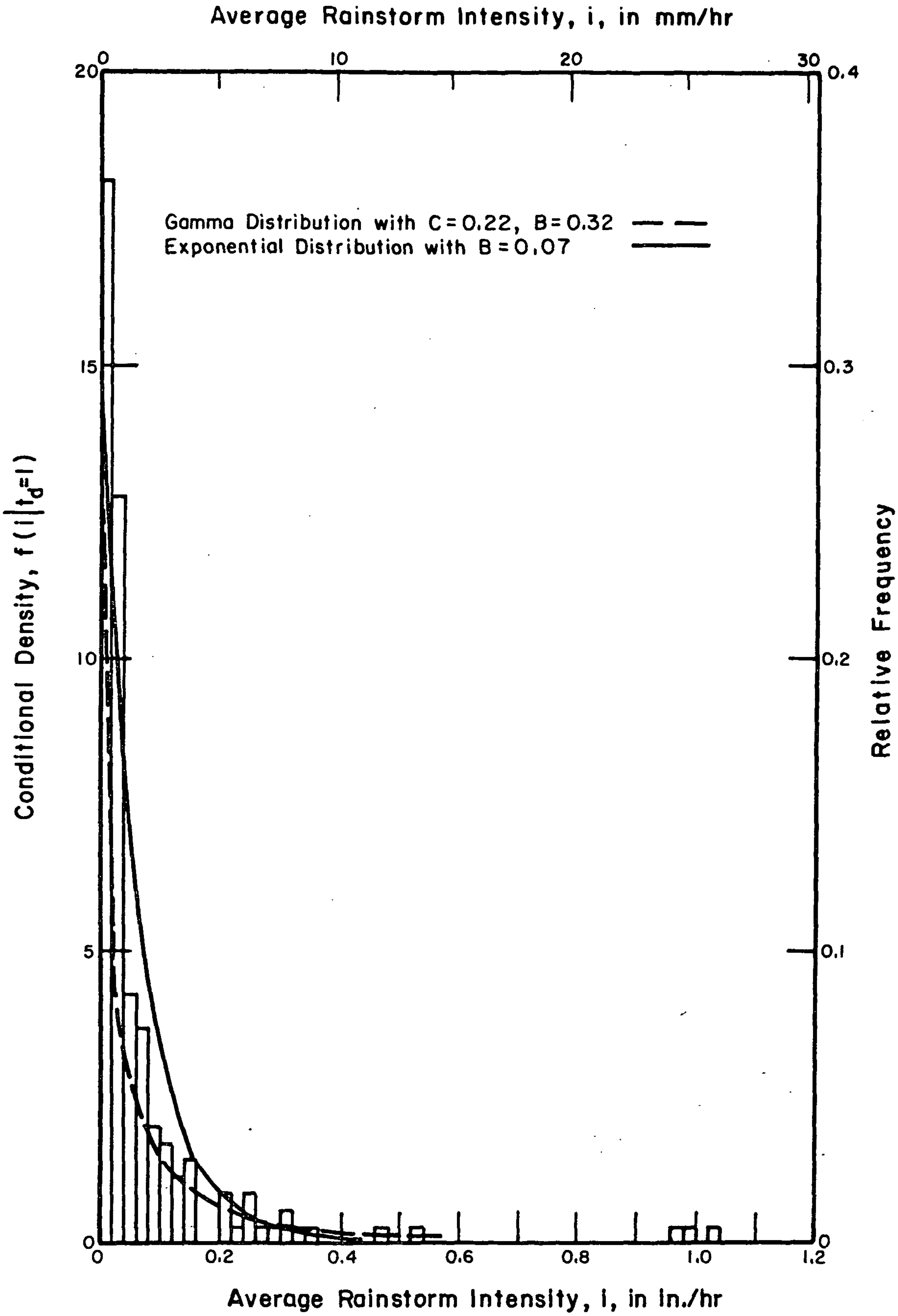


Fig. 7. Conditional distributions of average rainstorm intensity
(a) t_d = 1 hr

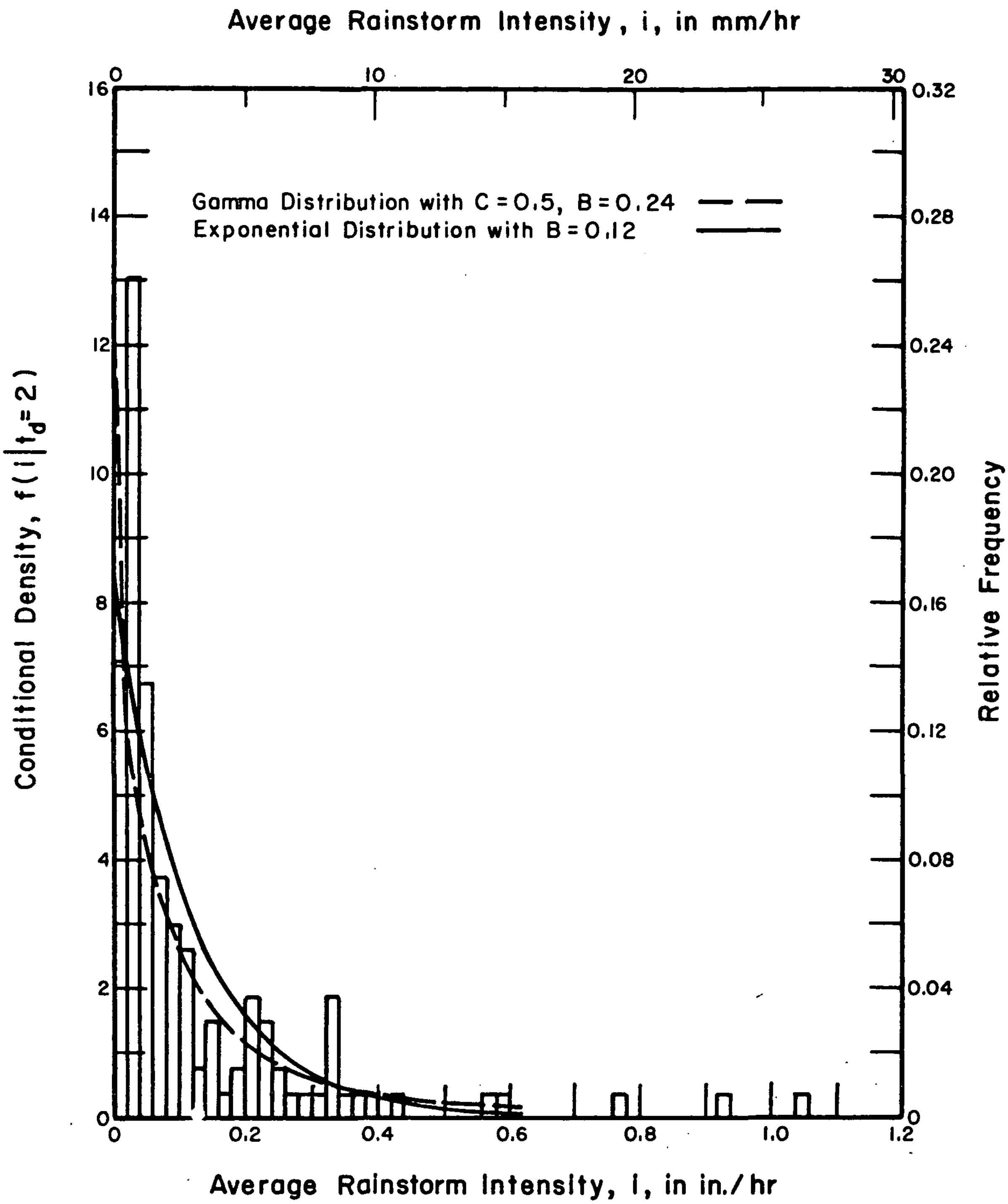

Average Rainstorm Intensity, i, in mm/hr
0
10
20
30
Gamma Distribution with C = 0.5, B = 0.24
Exponential Distribution with B = 0.12
Conditional Density, f(i|t_d = 2)
16
14
12
10
8
6
4
2
0
Relative Frequency
0.32
0.28
0.24
0.20
0.16
0.12
0.08
0.04
0
0
0.2
0.4
0.6
0.8
1.0
1.2
Average Rainstorm Intensity, i, in in./hr

Fig. 7. (b) t_d = 2 hr

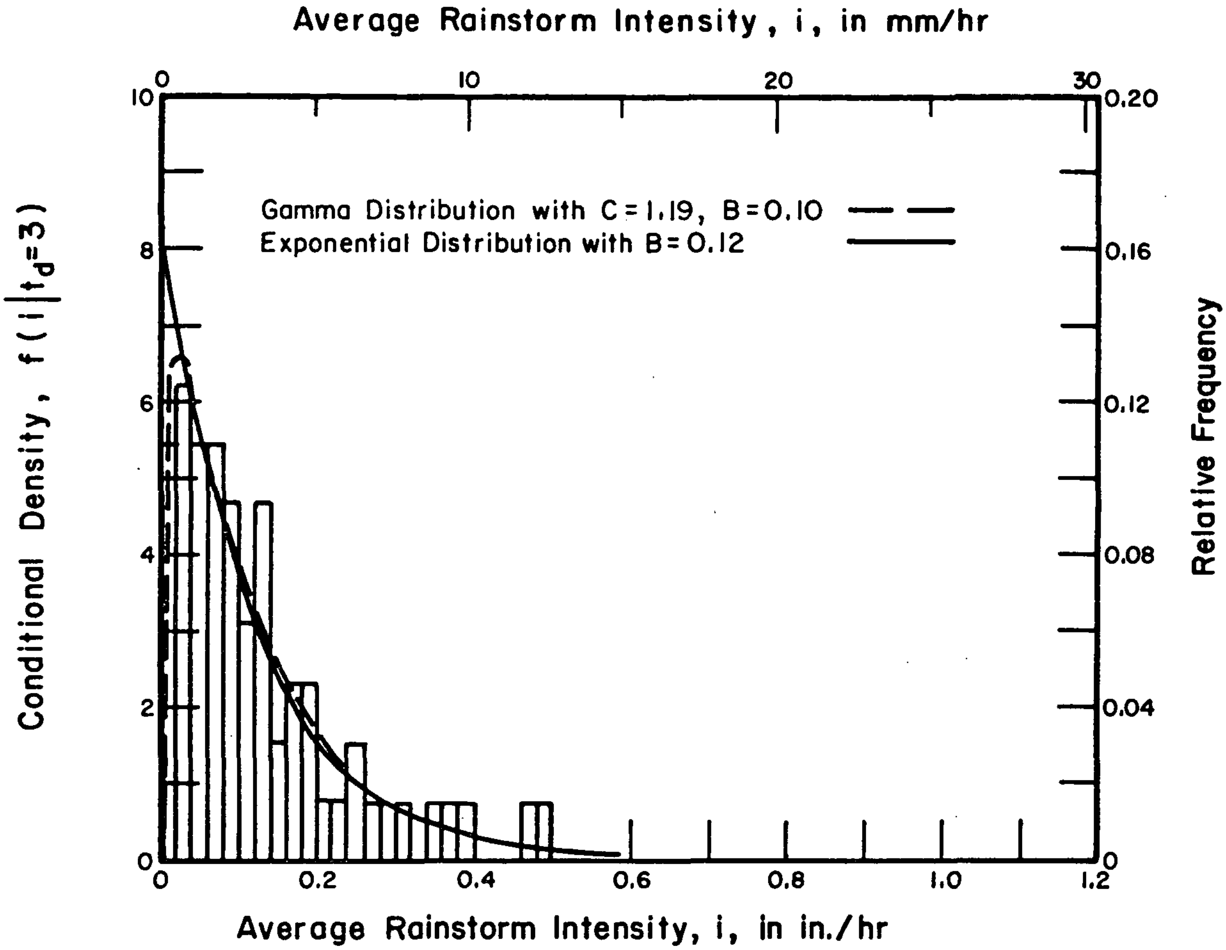

Average Rainstorm Intensity, i, in mm/hr
0
10
20
30
Gamma Distribution with C = 1.19, B = 0.10
Exponential Distribution with B = 0.12
Conditional Density, f(i|t_d=3)
Relative Frequency
10
8
6
4
2
0
0.20
0.16
0.12
0.08
0.04
0
0
0.2
0.4
0.6
0.8
1.0
1.2
Average Rainstorm Intensity, i, in in./hr

Fig. 7. (c) t_d = 3 hr

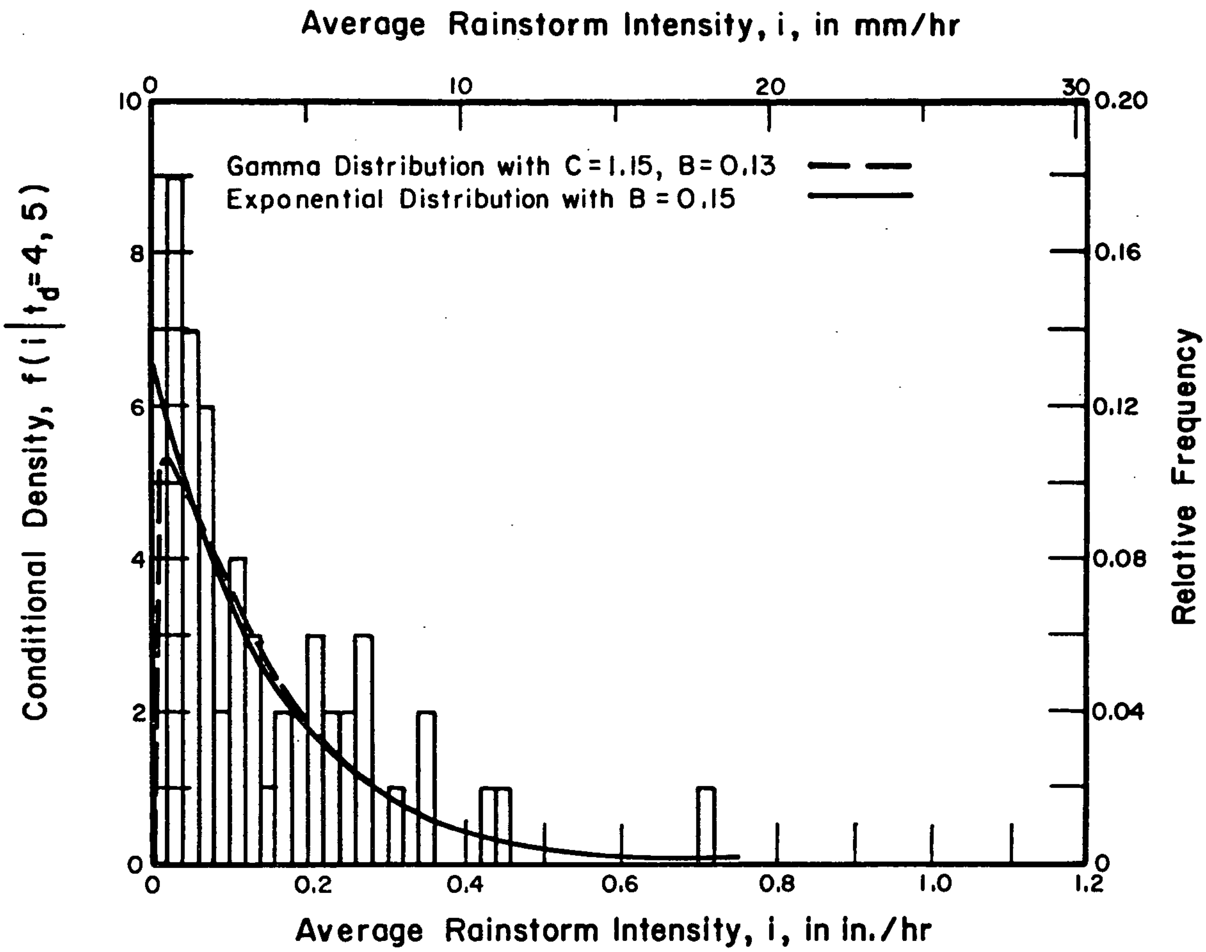


Fig. 7. (d) t_d = 4, 5 hr

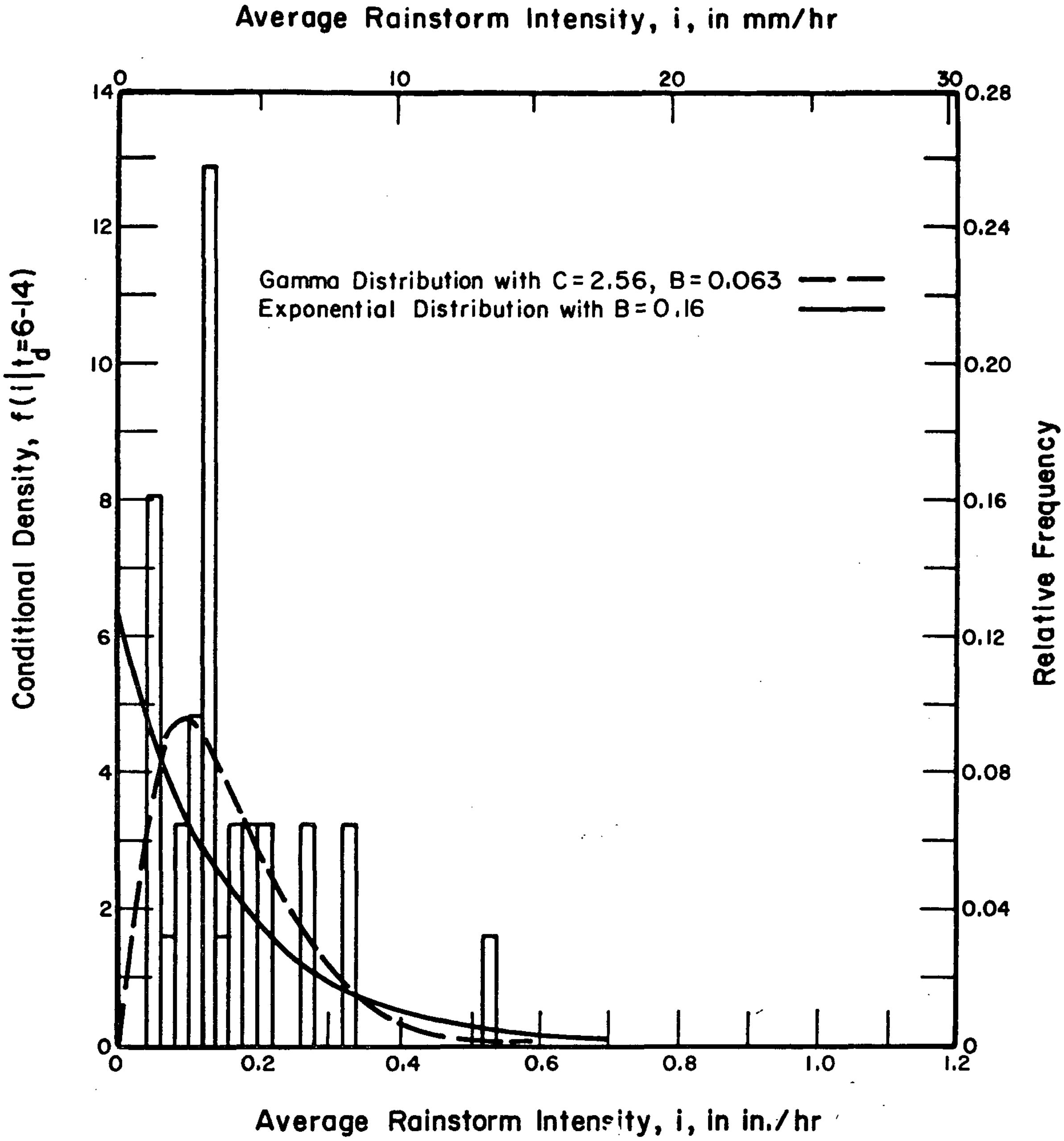


Fig. 7. (e) t_d = 6-14 hr

Table 2. CONDITIONAL STATISTICS OF RAINSTORM PARAMETERS FOR DIFFERENT DURATIONS
(a) t_d = 1 hr

Parameter		Mean	Standard Deviation	Min	Max
t_b	, hr	55.6	94.5	1	744
t_d	, hr	1	0	1	1
D ,	in.	0.07	0.15	0.01	1.03
	mm	1.78	3.81	0.25	26.2
$\bar{i}$,	in./hr	0.07	0.15	0.01	1.03
	mm/hr	1.78	3.81	0.25	26.2
σ_d ,	in.	0	0	0	0
	mm	0	0	0	0
$\bar{t}$	, hr	0.50	0	0.50	0.50
G	, hr^2	0.33	0	0.33	0.33
a	, hr	0.50	0	0.50	0.50
b	, hr	0.50	0	0.50	0.50
h ,	in.	0.14	0.29	0.02	2.06
	mm	3.56	7.37	0.51	52.3
σ_d^o		0	0	0	0
$\bar{t}^o$		0.50	0	0.50	0.50
G^o		0.33	0	0.33	0.33
a^o		0.50	0	0.50	0.50
b^o		0.50	0	0.50	0.50
h^o		2	0	2	2

Number of Rainstorms = 176

Table 2. (b) t_d = 2 hr

Parameter		Mean	Standard Deviation	Min	Max
t_b	, hr	67.8	76.1	1	327
t_d	, hr	2	0	2	2
D ,	in.	0.24	0.34	0.02	2.09
	mm	6.10	8.64	0.51	53.1
i ,	in./hr	0.12	0.17	0.01	1.05
	mm/hr	3.05	4.32	0.25	26.7
σ_d ,	in.	0.07	0.12	0	0.87
	mm	1.78	3.05	0	22.1
$\bar{t}$	, hr	0.92	0.27	0.53	1.48
G	, hr^2	1.18	0.54	0.40	2.29
a	, hr	0.76	0.81	-0.41	2.43
b	, hr	1.24	0.81	-0.43	2.41
h ,	in.	0.24	0.34	0.02	2.09
	mm	6.10	8.64	0.51	53.1
σ_d^o		0.23	0.15	0	0.48
$\bar{t}^o$		0.46	0.13	0.27	0.74
G^o		0.29	0.13	0.10	0.57
a^o		0.38	0.40	-0.20	1.22
b^o		0.62	0.40	-0.22	1.20
h^o		1	0	1	1

Number of Rainstorms = 134

Table 2. (c) t_d = 3 hr

Parameter		Mean	Standard Deviation	Min	Max
t_b	, hr	54.3	70.2	1	304
t_d	, hr	3	0	3	3
D	, in.	0.36	0.33	0.03	1.44
	mm	9.14	8.38	0.76	36.6
i	, in./hr	0.12	0.11	0.01	0.48
	mm/hr	3.05	2.79	0.25	12.2
σ_d	, in.	0.09	0.10	0	0.53
	mm	2.29	2.54	0	13.5
$\bar{t}$	, hr	1.38	0.43	0.56	2.40
G	, hr^2	2.58	1.28	0.48	5.97
a	, hr	1.14	1.29	-1.34	4.18
b	, hr	1.86	1.29	-1.18	4.34
h	, in.	0.24	0.22	0.02	0.96
	mm	6.10	5.59	0.51	24.4
σ_d^o		0.23	0.11	0	0.45
$\bar{t}^o$		0.46	0.14	0.19	0.80
G^o		0.29	0.14	0.05	0.66
a^o		0.38	0.43	-0.45	1.40
b^o		0.62	0.43	-0.40	1.45
h^o		0.67	0	0.67	0.67

Number of Rainstorms = 64

Table 2. (d) t_d = 4 and 5 hr

Parameter		Mean	Standard Deviation	Min	Max
t_b	, hr	55.7	64.9	1	292
t_d	, hr	4.32	0.47	4	5
D	, in.	0.65	0.61	0.08	2.86
	mm	16.5	15.5	2.03	72.6
i	, in./hr	0.15	0.14	0.02	0.72
	mm/hr	3.81	3.56	0.51	18.3
σ_d	, in.	0.15	0.15	0.01	0.53
	mm	3.81	3.81	0.25	13.5
$\bar{t}$	, hr	1.97	0.60	0.66	3.42
G	, hr^2	5.17	2.57	0.75	12.4
a	, hr	1.58	1.73	-2.02	5.30
b	, hr	2.74	1.77	-1.30	6.29
h	, in.	0.30	0.28	0.04	1.43
	mm	7.62	7.11	1.02	36.3
σ_d^o		0.20	0.08	0.06	0.39
$\bar{t}^o$		0.46	0.13	0.17	0.78
G^o		0.28	0.13	0.05	0.65
a^o		0.37	0.40	-0.51	1.33
b^o		0.63	0.40	-0.33	1.51
h^o		0.47	0.05	0.4	0.5

Number of Rainstorms = 50

Table 2. (e) t_d = 6-14 hr

Parameter	Mean	Standard Deviation	Min	Max
t_b , hr	61.5	77.8	1	324
t_d , hr	8.55	2.62	6	14
D , in.	1.35	0.85	0.30	3.52
D , mm	34.3	21.6	7.62	89.4
i , in./hr	0.16	0.10	0.05	0.53
i , mm/hr	4.06	2.54	1.27	13.5
σ_d , in.	0.15	0.13	0.03	0.74
σ_d , mm	3.81	3.30	0.76	18.8
$\bar{t}$, hr	3.83	1.70	1.00	7.68
G , hr^2	22.0	17.6	2.54	71.2
a , hr	2.93	3.42	-4.99	10.0
b , hr	5.61	3.31	-2.10	13.8
h , in.	0.32	0.21	0.10	1.07
h , mm	8.13	5.33	2.54	27.2
σ_d^o	0.12	0.06	0.04	0.27
$\bar{t}^o$	0.44	0.13	0.13	0.77
G^o	0.27	0.13	0.05	0.68
a^o	0.32	0.39	-0.62	1.30
b^o	0.68	0.39	-0.30	1.62
h^o	0.25	0.07	0.14	0.33

Number of Rainstorms = 31

The above procedure gives a set of fitted conditional density functions for i corresponding to different values of t_d. Although it is not necessary from a practical viewpoint, it is often convenient for easy application to obtain a single expression as the conditional density function of i given t_d; i.e. $f(i|t_d)$. For the present example, this is done by using the exponential and gamma density functions and by expressing the parameters of these density functions as functions of the storm duration, using the values fitted in Figs. 7a to 7e. For the gamma distribution, as shown in Fig. 8, assuming an exponential relationship between B or C and t_d, the fitted expressions are

$$B = 0.33t_d^{-0.79} \tag{27}$$

and

$$C = 0.31t_d \tag{28}$$

Thus, from Eqs. 21, 27 and 28, the conditional gamma density function is

$$f(i|t_d) = \frac{1}{\Gamma(0.31t_d)}(3t_d^{0.79})^{0.31t_d} i^{0.31t_d-1} \exp(-3it_d^{0.79}) \tag{29}$$

Likewise, for the exponential distribution, a plot of B against t_d (Fig. 9) yields

$$B = 0.08t_d^{0.37} \tag{30}$$

and from Eqs. 20 and 30, the conditional exponential density function is

$$f(i|t_d) = 12.5t_d^{-0.37} \exp(-12.5it_d^{-0.37}) \tag{31}$$

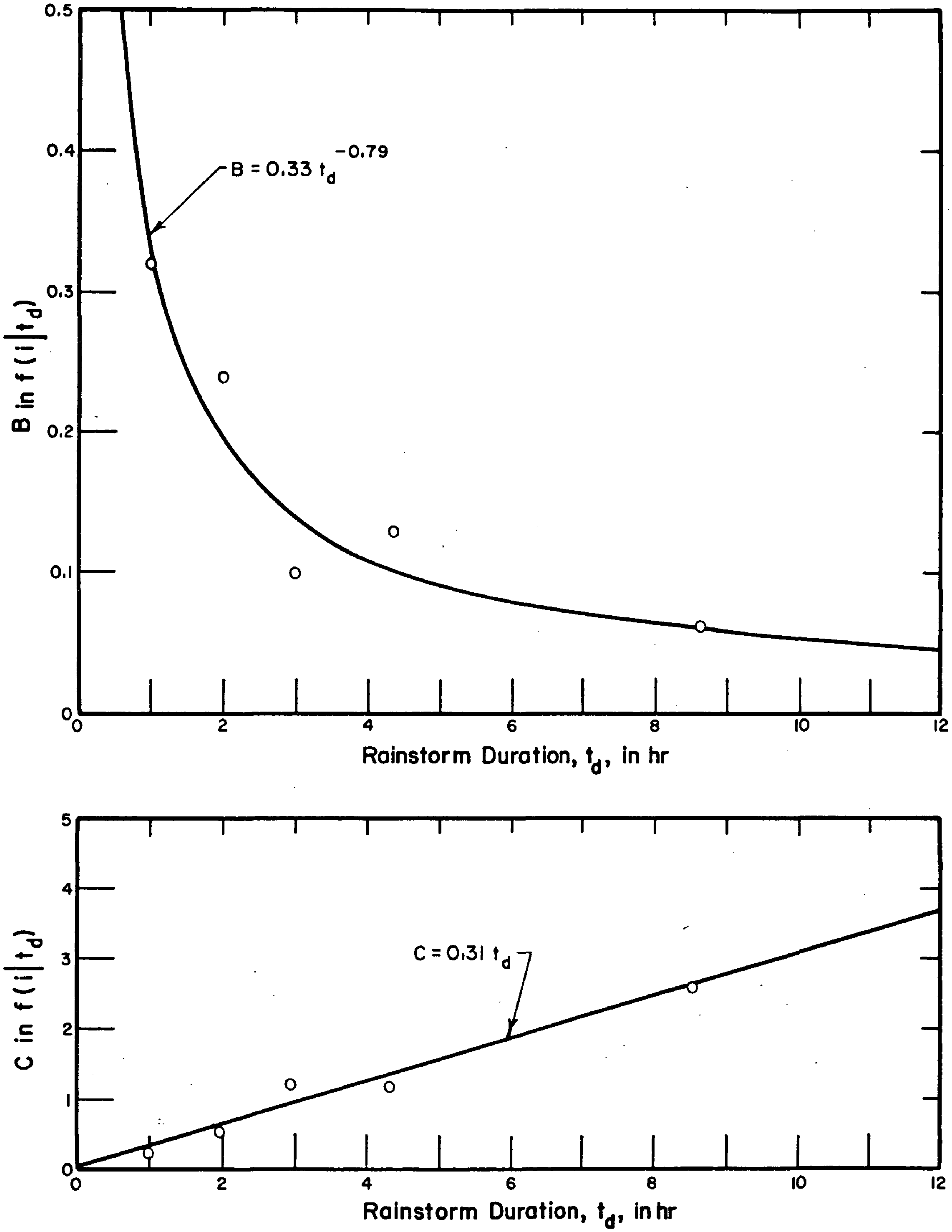


Fig. 8. Gamma density function parameters as functions of rainstorm duration

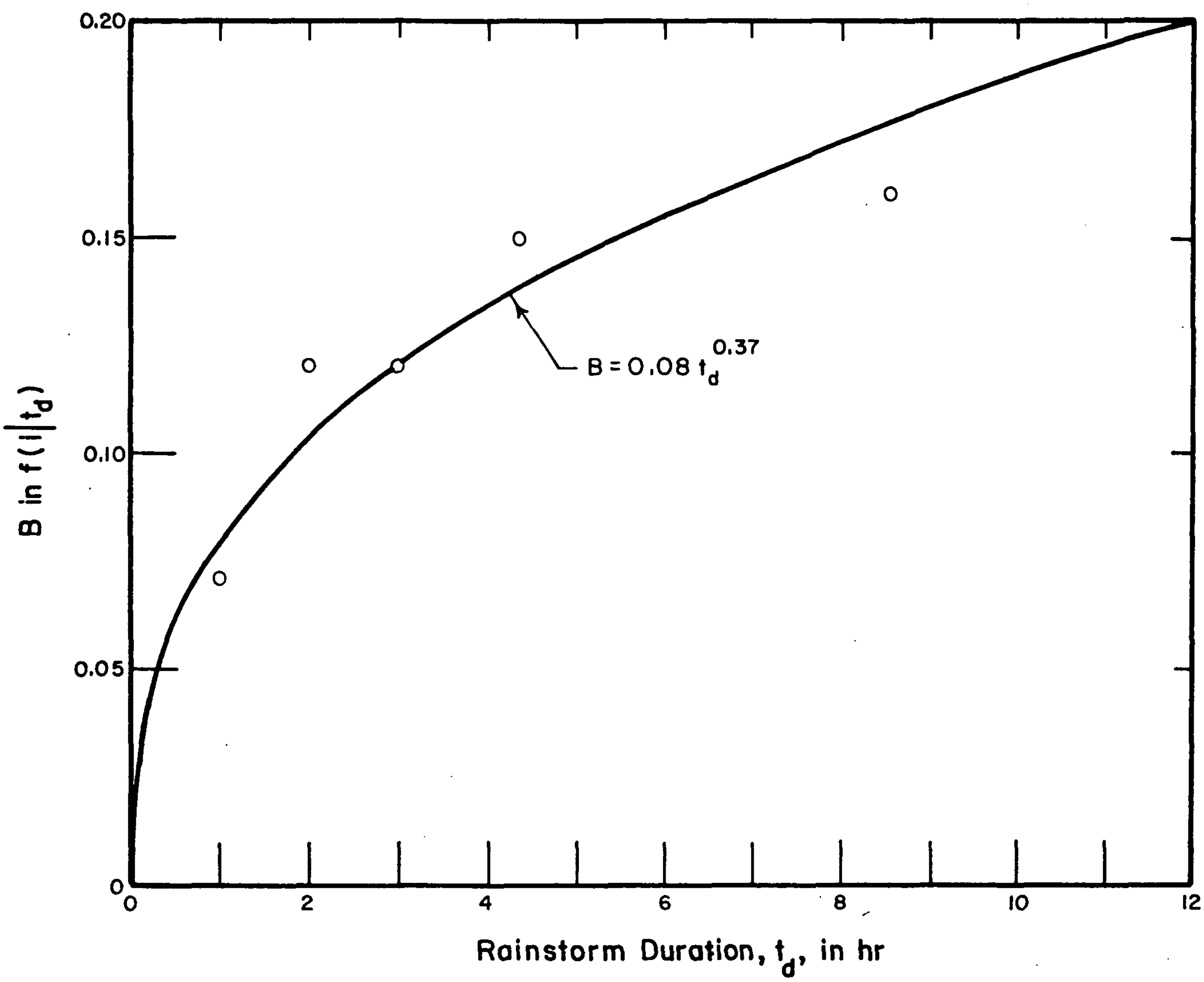


Fig. 9. Exponential density function parameter as function of rainstorm duration

The conditional probability analysis can be extended to obtain joint density functions for practical uses. For instance,

$$f(i,\ t_d) = f(i|t_d)\ f(t_d) \tag{32}$$

By using the exponential density functions, Eqs. 31 and 32 together with Eq. 20 (with value of B = 2.45 for $f(t_d)$ given in Fig. 4) yield

$$f(i,\ t_d) = 5.1t_d^{-0.37} \exp(-12.5it_d^{-0.37} - 0.41t_d) \tag{33}$$

Similar analyses can be performed on other parameters, and the procedure can be extended to trivariate cases, e.g., $f(\bar{t}|D,\ t_d)$. However, trivariate frequency analysis is rather tedius, requiring large amount of data, and at present is unlikely to be undertaken by most engineers.

(H) Application procedure. - In urban storm runoff problems, two types of application often arise in connection with the statistical analysis just described. The first is for design of certain facilities such as treatment plants and overflow devices. The second is for operational purposes involving the prediction of the time of occurrence, depth and duration of the next rainstorm after a rainstorm of a given depth and duration has just occurred.

To illustrate the application to design, assume that the storm runoff quantity is the controlling factor in determining the capacity of a waste water treatment plant and that the plant is to be designed with a capacity which will be exceeded on the average at most twice a month. For the 3-month summer rainstorm data over the 14-year period at Urbana, the average number of rainstorms for a 3-month summer period is 455/14 = 32.5. Assuming that the Urbana data is applicable for the design under consideration, the given exceedance frequency of twice a month (6 times in 3 months) corresponds to an exceedance probability of 6/32.5 = 0.185

and a non-exceedance probability of 0.815. It is obvious that among rainstorms of the given frequency those with large depth of rainfall and short duration are most critical to the design. Assuming that depth is the most significant one among all the rainstorm parameters considered, from the non-exceedance probability curve shown in Fig. 10a that for the given design frequency the design rainstorm depth is 12.9 mm (0.51 in.). Subsequently, from the conditional probability density function of t_d, $f(t_d|D)$, the most frequently occurred duration for this depth, i.e., the mode of $f(t_d|D)$, can be found and used as the design duration. For the Urbana summer data for D = 12.9 mm (0.51 in.), based on the 49 rainstorms with D between 8.9 and 16.5 mm (0.35 and 0.65 in.), the mode of t_d is 2 hr (Fig. 11a). Likewise, the mode of $\bar{t}$ can be found from the conditional probability density function $f(\bar{t}|D)$, or alternatively, $f(\bar{t}|t_d)$ or $f(\bar{t}|D, t_d)$. Because $\bar{t}$ appears to be more sensitive to t_d than to D, and because the trivariate conditional probability density function is difficult to obtain, the mode of $f(\bar{t}|t_d)$ is adopted as the design value of $\bar{t}$. For the example Urbana data (Fig. 11b) the value of $\bar{t}$ used for the design is 1.025 hr. Substituting the values of D, t_d, and $\bar{t}$ into Eqs. 14, 15, and 16 yields the design values of the triangular hyetograph shape factors a, b, and h, being 1.075 hr, 0.925 hr, and 12.9 mm (0.51 in.), respectively. The design hyetograph thus determined can subsequently be routed through the surfaces and sewers of the drainage basin using the routing methods that will be described later to give the design discharge or hydrograph for the treatment plant.

For the case of application to storm water runoff control, the problem is of the nature of flow prediction for management purposes. For instance, for an existing drainage system, when a rainstorm comes,

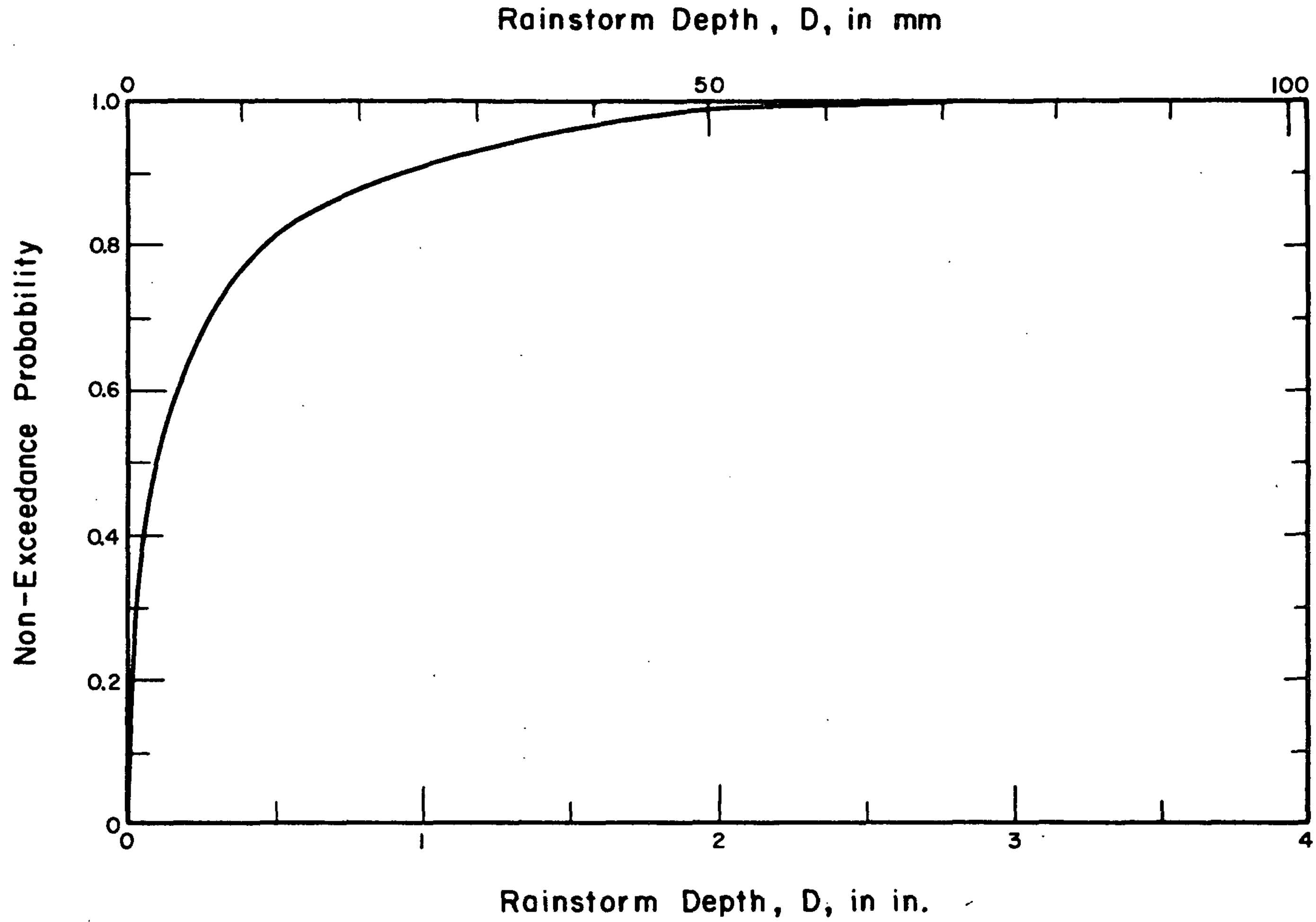


Fig. 10. Non-exceedance probabilities for rainstorm depth and elapse time between rainstorms
(a) Rainstorm depth

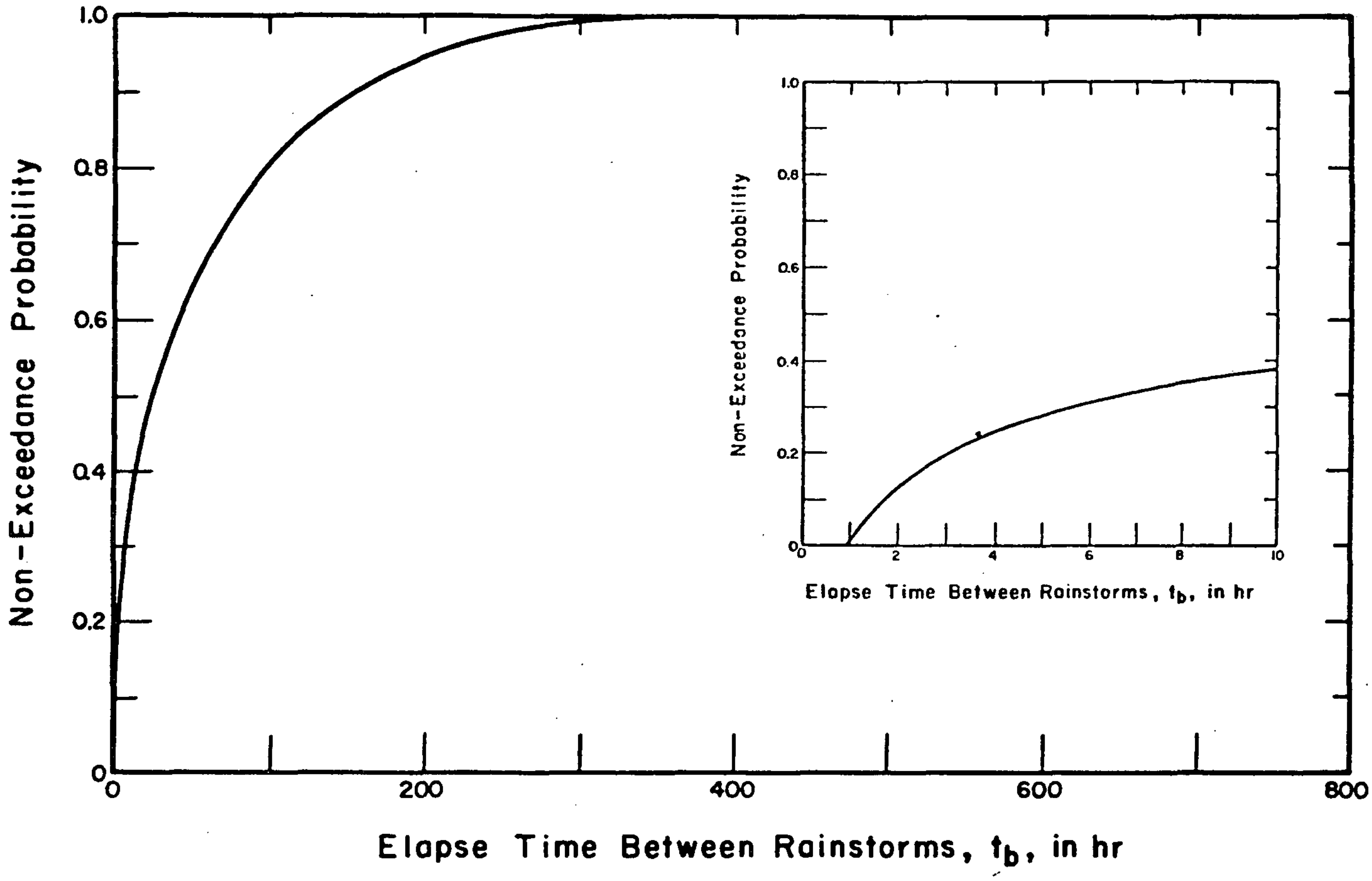


Fig. 10. (b) Elapse time between rainstorms

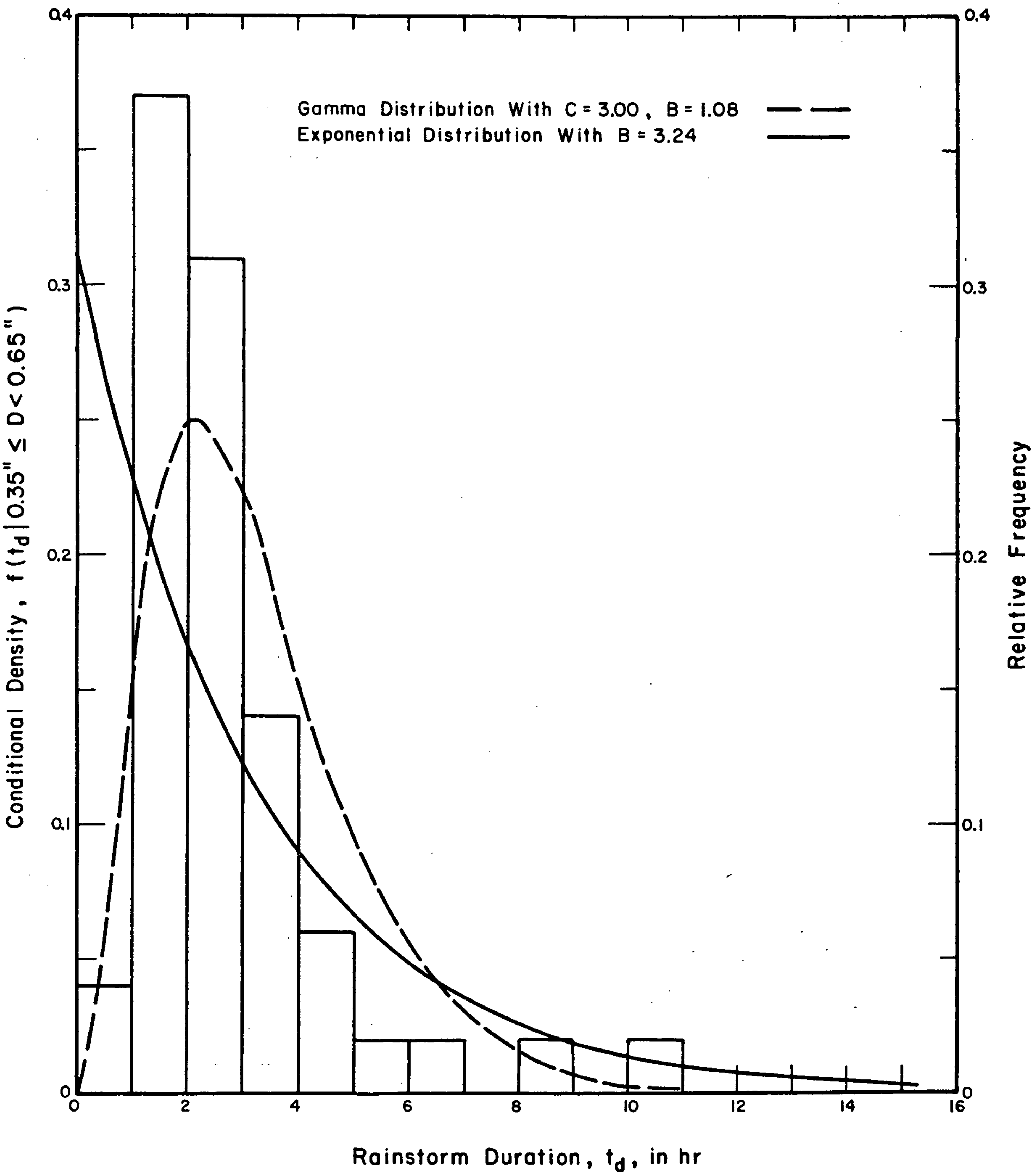


Fig. 11. Conditional distributions of rainstorm duration and hyetograph first moment arm
(a) Rainstorm duration for 0.35" ≤ D < 0.65"

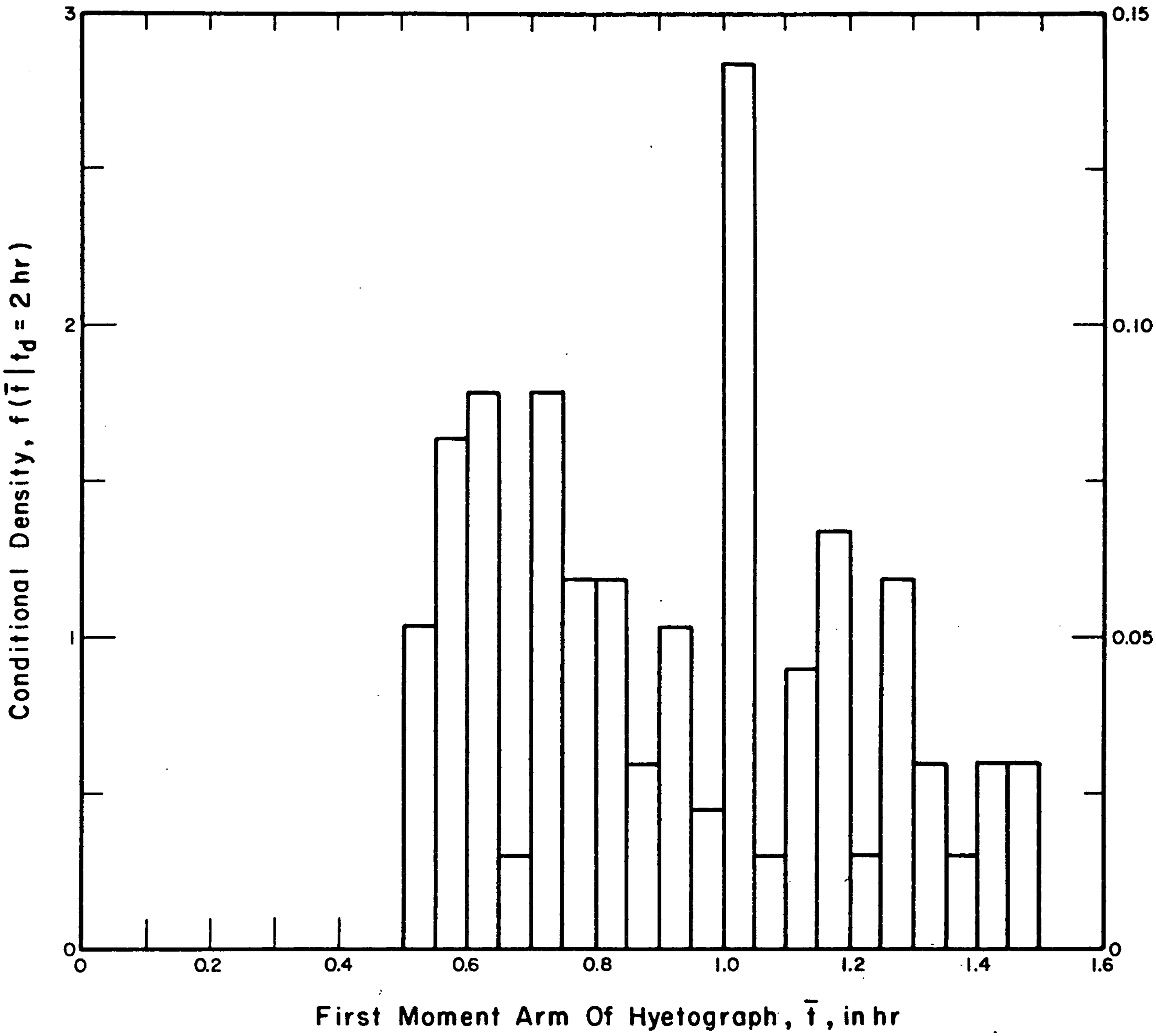


Fig. 11. (b) First moment arm of hyetograph for t_d = 2 hr and for all D

it is desirable for operational purposes to know the time of occurrence, depth and duration of the next rainstorm so that decision can be made on the utilization of in-line storage and other control facilities. Consider the case when a rainstorm of depth D and duration t_d has just occurred. Data from Urbana and other locations indicate that the elapse time between rainstorms is nearly independent of the depth and duration of the preceding rainstorm. Therefore, assuming that t_b is independent of D and t_d, the most probable time of occurrence of the next rainstorm can be estimated as the mode of the probability density function such as the one shown in Fig. 6. The depth of the next rainstorm can be evaluated as the mode of the conditional distribution of the depth of a rainstorm given the depth of the previous rainstorm, $f(D_2|D_1)$. Subsequently the duration of the next rainstorm can be determined from the mode of the conditional probability density function $f(t_d|D_2)$ and the shape of the hyetograph for the next rainstorm from $f(\bar{t}|t_{d2})$ as described in the design application.

Of course, many refinements and improvements can be made on the procedures for runoff control and for design just described. For example, in the flow prediction for storm water runoff control, the elapse time t_b can be estimated by using a non-exceedance probability function for t_b (Fig. 10b) with an assumed or selected non-exceedance probability. The depth of the next rainstorm can be estimated by using $f(D_2|D_1, t_{d1})$ instead of $f(D_2|D_1)$, and $\bar{t}$ by $f(\bar{t}|t_{d2}, D_2)$ instead of $f(t|t_{d2})$. The corresponding risks of the prediction can be evaluated accordingly using the joint density functions of the parameters involved. However, such refined methods are rather tedious and complicated, and available data are often inadequate to establish the needed conditional probabilities. Therefore, they are suggested to be considered only in the future after significant information based on simpler procedures are obtained and when adequate data are available.

IV-2. Infiltration and Other Abstractions

Not all of the rainfall produces runoffs. In hydrology the losses that do not produce surface runoff are called abstractions. These losses consist of interception, evaporation, transpiration, infiltration, and depression storage. Interception is the amount of rainwater being intercepted by trees, vegetation, posts and buildings that never reach the ground surface. For urban areas the rainfall on roofs which is drained to the surface or directly to the sewers is not considered an interception. The relative importance of interception on runoff depends on the intensity and duration of rainfall. In urban areas usually there is no dense woods or vegetation, the amount of interception is no more than a fraction of an inch (a few mm) and mostly occurs during the beginning of the rainfall. Therefore, for relative heavy rainfall of short duration, which would be of importance for design or pollution control because of overflow, the amount of interception is less than a few percent of the runoff volume and can be neglected without causing serious accuracy problems.

Evaporation and transpiration are often considered simultaneously for obvious practical reasons. Evapotranspiration may be important when the water balance over a long period is considered. But as shown by Shen et al. (1974), it is negligible when heavy rainfall over short duration is considered, particularly in view of the vegetation and tree situation in most urban areas. Actually, there is no difficulty to include interception and evapotranspiration losses when reliable formulas become available. At present the amount of these losses is assumed negligible.

Infiltration loss is a major factor affecting surface runoff. Infiltration is defined as the process of water flowing through the ground surface, i.e., the interface between the fluid environment and the soil environment below. Various theoretical approaches and empirical formulas have been proposed to estimate infiltration. At the beginning of this research project a study was made to use Philip's (1969) theory to derive a four-parameter method to account for infiltration. Unfortunately, available field data are inadequate to substantiate this approach and hence the simpler and popular Horton's formula is used. In fact, even for Horton's formula which is a three-parameter function, there are difficulties to establish the values of the parameters based on existing data. Philip (1969) also proposed a two-parameter approximate infiltration equation. However, this equation has not been adequately tested nor has it been widely accepted by Civil Engineers.

Horton's formula is

$$f = f_c + (f_o - f_c)\, e^{-kt} \tag{34}$$

in which f is the instantenous infiltration capacity; f_o and f_c are the initial and final infiltration capacities, respectively; t is time; and k is an exponent accounting for the decay rate of infiltration. For Horton's equation to apply, the water supply rate (rainfall and water stored on land surface) must be equal to or greater than the infiltration capacity at that instant. Otherwise, the entire amount of water is assumed infiltrated.

The difficulty in applying Horton's formula to actual drainage basins arises partly from the fact that in experimental and field rainfall-runoff studies, the soil properties and surface moisture conditions are often inadequately recorded, and partly because of a natural drainage basin, the soil condition is inevitably nonhomogeneous and the values of f_c, k and f_o are different for different areas within the drainage basin. The measured basin runoff hydrograph merely reflects the integrated effects of infiltration and other factors, and there is actually no single set of representative values of f_o, f_c, and k for the entire basin.

Theoretically, strictly speaking, for a given soil none of the values of k, f_c, and f_o is constant. They depend on the soil type, fluid properties, moisture condition of the soil, and water pressure (usually depth) on the ground surface. The initial infiltration capacity, f_o, obviously depends heavily on the initial soil moisture condition. The final infiltration rate, f_c, and the exponent expressing the decay rate, k, are the soil properties and should be constant if secondary effects such as those due to changes in soil flow potential near the ground surface, in water depth, in fluid properties and seasonal effects are neglected. Philip (1969, Figs. 16 and 17) has shown that for a given soil and liquid, f_c is essentially constant but the decay rate decreases with decreasing initial moisture and with increasing overlaying water depth, i.e., k decreases with decreasing initial moisture content or increasing water depth. However, for a given surface in the practical range of conditions the variation of k is relatively small. In other words, from a purely theoretical viewpoint considering a liquid entering a porous medium, the values of f_c and k are not constant, but from a practical viewpoint in using Horton's formula, the values of f_c and k can be treated as essentially constants. The approximate constancy of f_c for a given soil surface has generally been accepted. The relative constancy of k is still a matter of debate. Many researchers using experimental data tried to show that k varies considerably with initial soil moisture condition and other factors. However, a small error in measurement in f_c would easily give a varying k for the same soil. Of course, the differences may also reflect the seasonal effects. Unfortunately, for the purpose of the present study on storm runoffs this uncertainty on infiltration imposes a serious problem on the accuracy of the results, making a reliable comparison of the runoff prediction methods difficult. Should Horton's formula be used with f_c and k treated as constants, it is suggested that different sets of values be used for different seasons.

A simple one-parameter approach, the ϕ-index method has also been used for rainfall-runoff studies. The method assumes a constant infiltration rate over the period of rainfall. This method is compatible with the requirement for the more sophisticated urban runoff methods evaluated in this study.

It should be noted here that not all the infiltrated water is necessarily lost because some water may find its way through subsurface flow to contribute to the basin runoff. In urban basins this may occur as infiltrated water entering sewers through joints and other leakages. However, such a case is not considered in this study.

The amount of loss due to depression storage depends on how the term is defined. Loosely it is usually defined as the water to fill the ground depressions before surface runoff starts. The amount, obviously, is a function of the surface texture. Actually, as it is commonly defined, the depression storage includes a thin layer of water held by surface tension before surface runoff starts. Expectedly the depression storage is of statistical nature. The most commonly used formula for depression storage supply rate (in./hr or mm/hr) is (Linsley et al., 1949)

$$s = (i - f) \exp \left[\frac{-(P-F)}{s_c}\right] \tag{35}$$

in which s_c is the depression storage capacity expressed in depth (in. or mm); P is the cumulative rainfall in depth; and F is the cumulative infiltration in depth.

IV-3. Snow Melt

Runoff from snow melt in urban areas differs from that of rural areas in two major aspects. First there is more heat available for snow melting in urban areas than in rural. Second and most important, the intense human activities and interferences in urban areas would hasten the melting process. Although runoff from snow melt is never a problem in urban storm sewer design because its magnitude is smaller than the flash flood of urban runoff due to heavy rainstorms, it is of considerable importance in pollution control because of the quality of the melted water, particularly when de-icing additives are used to speed up the melting process.

The energy needed for snow melting comes mainly from three sources: the radiant heat from the sun, the conduction heat from the environment, and the latent heat of vaporization released by the condensation of water vapor. The first two are the major ones to be considered for urban snow melting. Many empirical and semi-empirical studies have been made on snow melt in rural areas (Chow, 1964, Section 10). However, these studies consider a time of melting usually much longer than that which would be interesting and useful for urban settings, and the interferences of human activities are not included. Snow melt in urban areas is a topic practically untouched. A possible approach is to consider the energy budget of snow melting and the thermodynamic processes involved, such as the idea outlined by Eagleson (1970, Chap. 13). But such an idea has not been extended or developed into any form nearly adoptable in practice, and to undertake such a research is beyond the scope of this study.

From an engineering viewpoint the worst condition in terms of both runoff quantity and quality for snow melt is melting of snow under warm rain. For this situation the following daily snow melt formula recommended by the U.S. Army Corps of Engineers (1960) is tentatively adopted in this study:

$$M = 0.007\ P_r(T_a - 32) \tag{36}$$

in which M is the daily snow melt in in., P_r is the daily rainfall in in., and T_a is the mean daily temperature of saturated air at 10-ft above ground in ^oF. If M and P_r are in mm and T_a in ^oC, the formula becomes

$$M = 0.013\ P_r\ T_a \tag{37}$$

V. DRAINAGE SYSTEM CHARACTERISTICS OF OAKDALE AVENUE BASIN

The Oakdale Avenue Drainage Basin in Chicago was selected to verify and evaluate the urban storm runoff models. It is one of the very few drainage basins for which relatively compatible and reliable data are available. The basin is located in a residential section in the city of Chicago (Fig. 12). It is approximately 2 1/2 block long by 1 block wide and has a drainage area of 0.052 km^2 or 12.9 acres. The basin consists entirely of residential dwellings, and the drainage characteristics are relatively uniform. The street pattern may be considered as being typical of many cities in Illinois and the United States.

V-1. Surface Drainage Pattern

The drainage pattern of the land surface of the Oakdale Basin is shown in Fig. 13. There are 30 inlet catch basins, each delivering its water to a sewer junction and each receiving water from a gutter except inlets 7, 14, 15, 21, 29, and 30 (Fig. 13) which receive water from two gutters for each of these 6 inlets. Each of the 36 gutters is contributed by water from one or more subcatchments as shown by the dashed lines in Fig. 13. The subcatchment area of the Oakdale Basin consists of four types of surfaces: roofs, lawns, paved sidewalks, and street pavements. The relative percentages of size of these four types of surfaces vary from subcatchment to subcatchment, and as one would expect, also change with time as a result of change of land uses and structures. Detailed data on the distribution of these four types of surfaces for each of the subcatchments in the Oakdale Basin is not available.

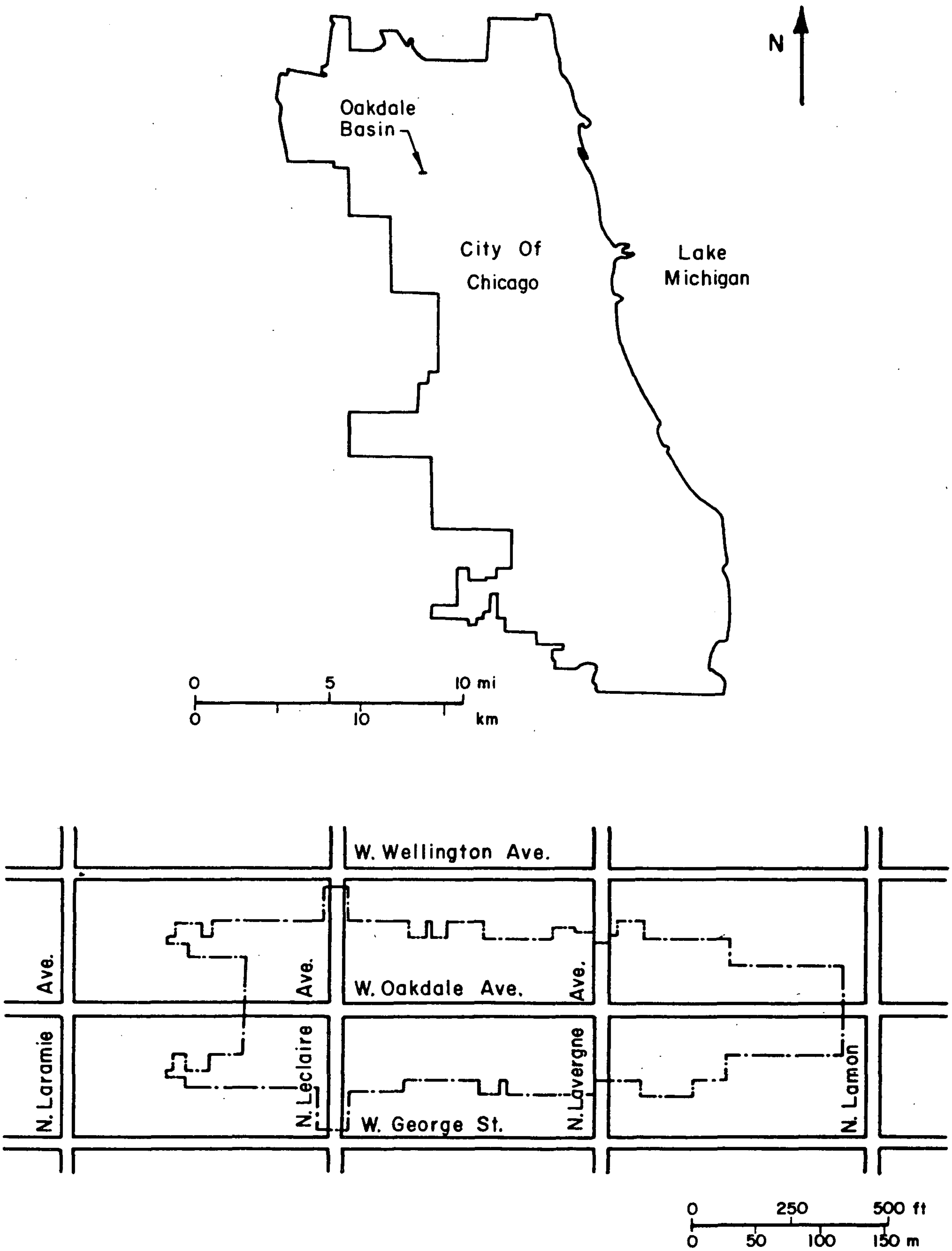


Fig. 12. Oakdale Basin location map

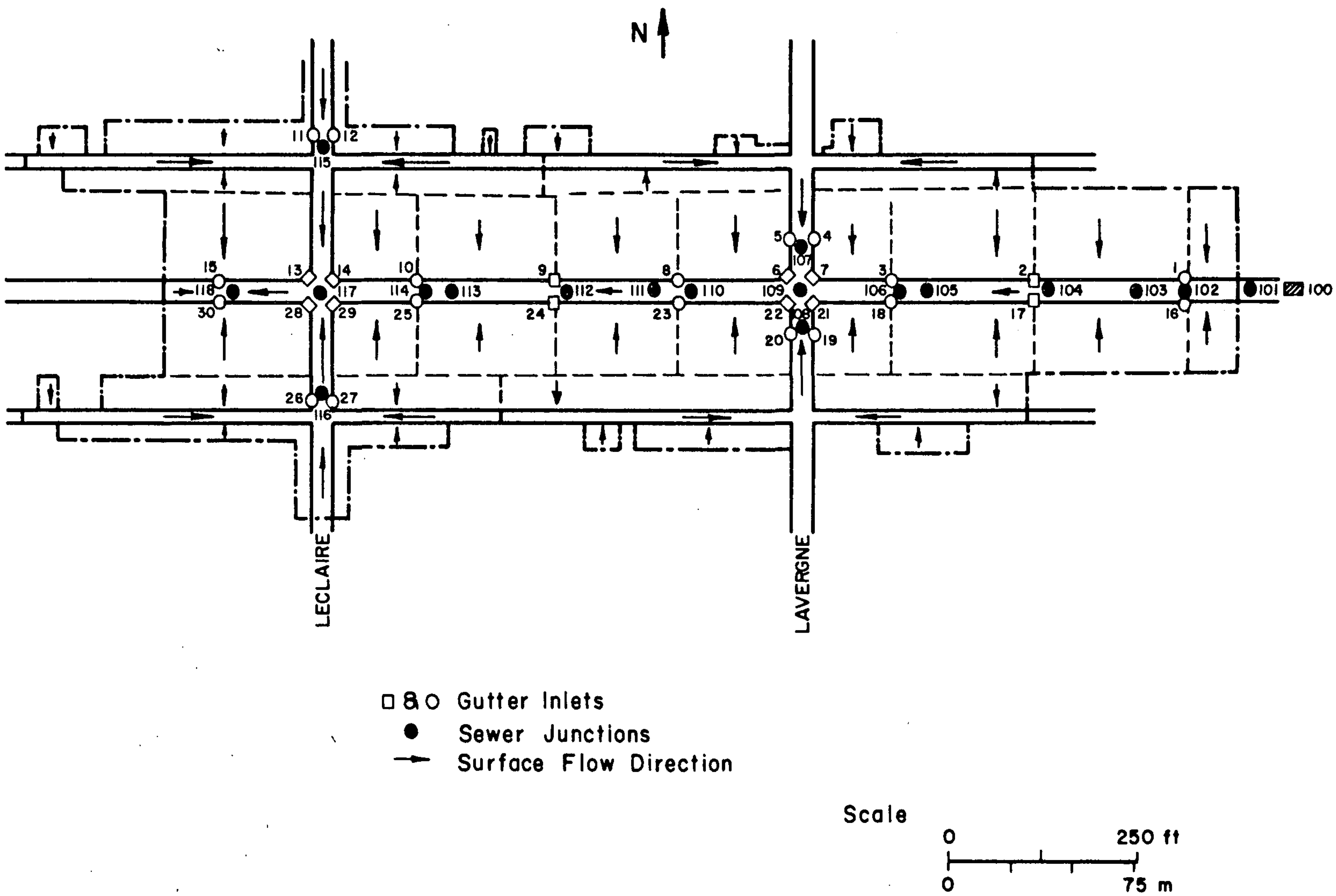


Fig. 13. Drainage pattern of Oakdale Basin

In most metropolitan areas, data on such detailed land surface uses are generally nonexistent. For a small drainage basin like the Oakdale Basin, it is possible to conduct a detailed survey to actually measure the physical properties of each of the subcatchments. However, even for such a small basin, at least several man-months of work is needed to obtain the data. Atop of this difficulty there is always the problem of getting permission to survey in private properties. One simpler, faster, and less costly method is to use aero-photos or satellite pictures. Because of the time limitation of the research project, this photogrammetric method was not undertaken. Nevertheless, even if such detailed surface use data is available, it would be extremely tedious and costly to actually use it to route the rainwater through all the surfaces to the gutters. In view of the practical consideration of the costs involved in obtaining and using the detailed subcatchment surface data and the seasonal changes of the surface characteristics, it appears most unlikely that any practical urban storm runoff simulation model would require to use such detailed subcatchment information. For the case of the Oakdale Basin used in this research, although the instrumented survey was not extended to cover the details of the subcatchments, a thorough visual survey was conducted in order to provide reliable information for the evaluation of the urban storm runoff methods.

The streets in the Oakdale Basin are 8.5 m (28 ft) wide, paved with asphalt having a cross-slope of 2.4 to 3.0% on both sides of the street crown. The longitudinal slope of the streets varies as listed in Table 3. The street slopes are typically flat as for most of the Midwest cities. Between Leclaire Street and the outlet of the basin Oakdale Avenue is actually sloping down towards west although the sewer line underneath has its slope in the opposite direction.

The street gutters are all triangular in cross section formed by cast-in-place concrete. The curbs are essentially vertical and the curb height varies along the gutters and often interrupted by driveways. The most frequently observed curb height is 0.25 m (10 in.). The lateral angle, θ, between the gutter bottom and the vertical is 1.54 radians. There is a break of lateral slope where the gutter bottom joins the street pavement. Consequently, assuming that the water surface of the gutter flow is horizontal along the lateral direction, the relationship between the flow cross sectional area, A, and depth measured from the apex of the gutter, h, (Fig. 14) is

$$A = \frac{h^2}{2 \cot\theta} \qquad \text{for } h \leq W \cot\theta \tag{38a}$$

$$A = Wh - \frac{W^2 \cot\theta}{2} \qquad \text{for } h > W \cot\theta \tag{38b}$$

in which W is the gutter width. The hydraulic radius R is

$$R = \frac{h \sin\theta}{2(1 + \cos\theta)} \tag{39a}$$

$$R = \frac{Wh - \dfrac{W^2 \cot\theta}{2}}{h + \dfrac{W}{\sin\theta}} \tag{39b}$$

and the water surface width b is

$$b = h/\cot\theta \text{ for } h \leq W \cot\theta \tag{40a}$$

$$b = W \text{ for } h > W \cot\theta \tag{40b}$$

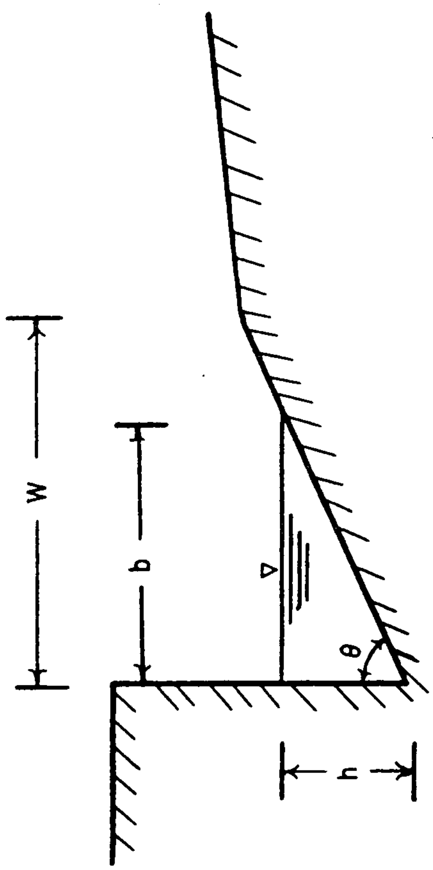


Fig. 14. Schematic drawing of gutter cross section

The longitudinal slopes of the gutters are the same as those for the streets. The gutter width measured horizontally is 0.3 m (1 ft). The geometric dimensions of the 36 gutters are listed in Table 3. In most of the time the gutters are kept reasonably clean although some debris have been observed. Accordingly Manning's roughness factor n is estimated to be 0.013 for the gutters.

A certain amount of rainstorm water is discharged directly from subcatchments into the alleys between the streets as shown in Fig. 13. Hydraulically these alleys act like wide shallow channels to transport the water into inlets or gutters. Most of the alleys have concrete surface with uneven joints and cracks and their estimated Manning's roughness factor n is 0.016. The length, width, and slope of the alleys are listed in Table 4.

The 30 inlets in the basin are grate inlets either circular or rectangular in shape as listed in Table 3. The details for the circular grate inlets are shown in Fig. 15a and those for the rectangular in Fig. 15b. The approximate locations of the inlets are identified by the inlet numbers in Fig. 13. The distances between the inlets are given in Table 3. Some of the inlets do not start from the curb line but offset slightly and extend beyond the gutter proper into the street pavement. Such irregularity occurs mostly for replacing inlets with clogged inlet cátch basins. Apparently some of the inlet catch basins have the clogging problem. There is no record to identify whether the inlet catch basins surveyed now in 1973-74 are the same as those a decade ago.

V-2. Sewer System

The combined sewer system of the Oakdale Avenue Drainage Basin consists of 18 circular sewer pipes and 18 junctions or manholes plus the sewer system outlet. The diameter of the concrete pipes ranges from 0.25 m

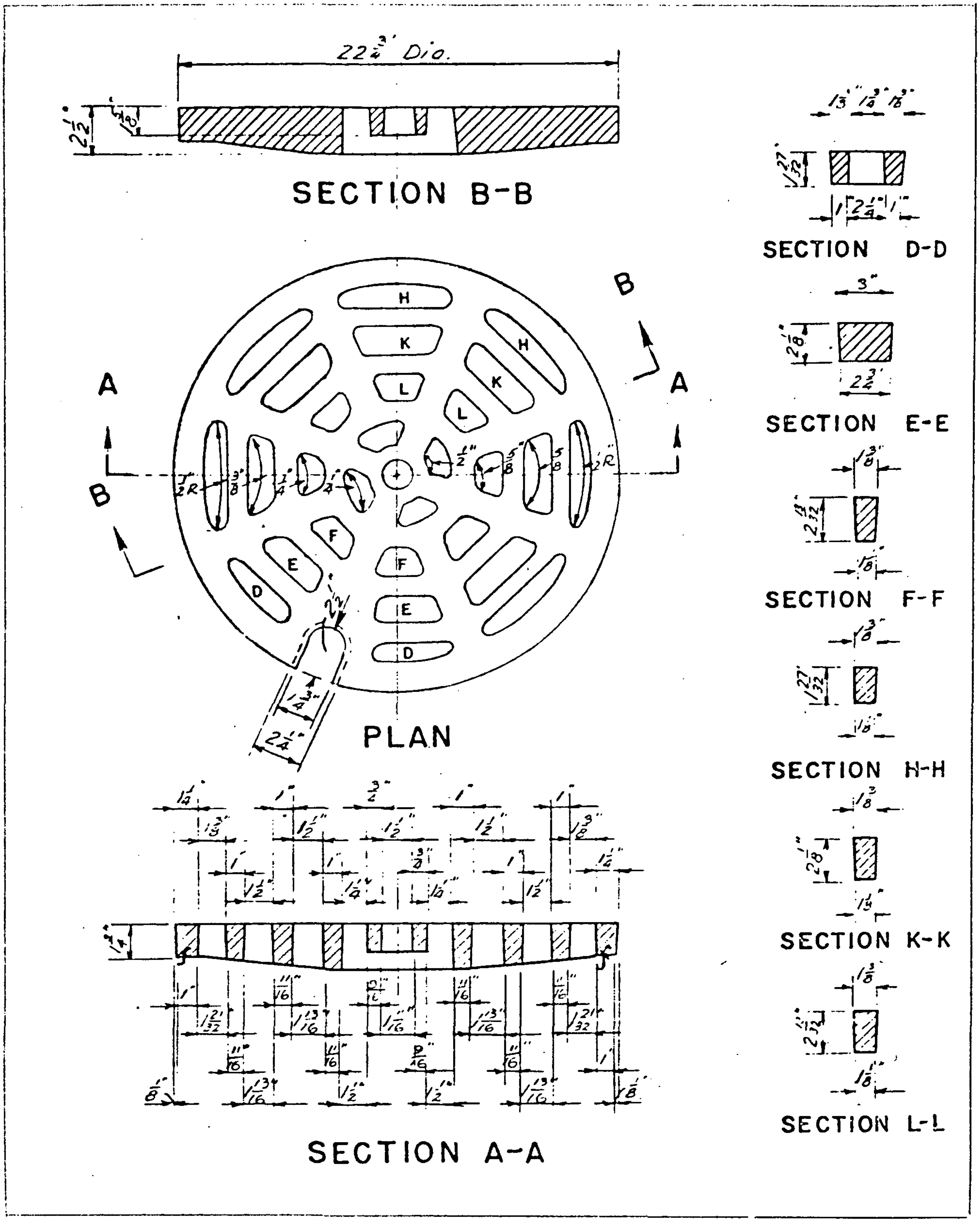


Fig. 15. Details of grate inlets
(a) Circular grate inlets

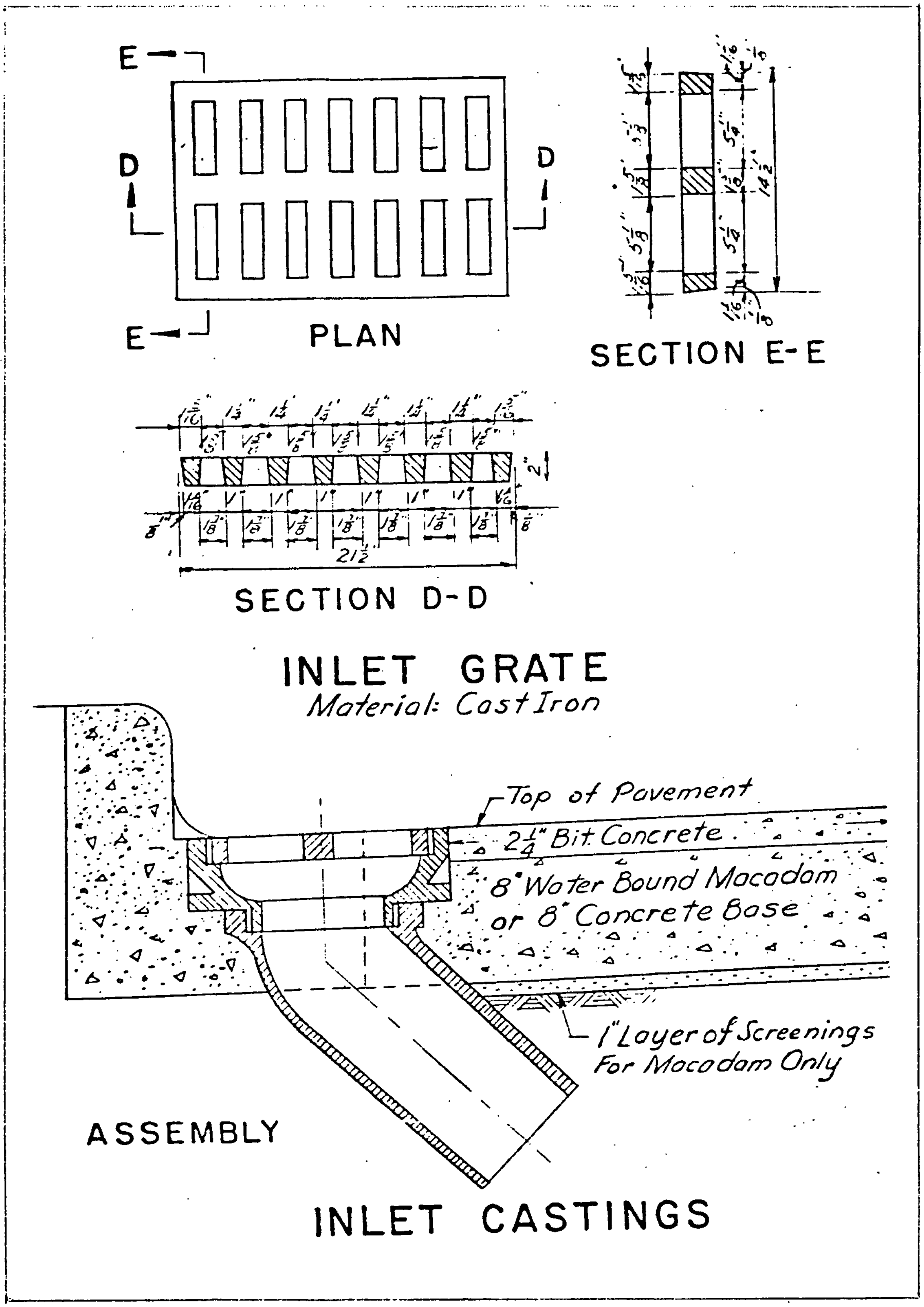


Fig. 15. (b) Rectangular grate inlets

(10 in.) to 0.76 m (30 in.) and the pipe roughness is estimated to be 0.01 ft or 3 mm. The junctions or manholes are marked as 3-digit numbers (e.g., 101 to 118) in Fig. 13 and the dimensions of the sewers are listed in Table 5. Most of the manholes are of flow-through type with a half-cut pipe embedded at the manhole bottom connecting the upstream and downstream sewers to induce smooth flow at low discharge. None of the junctions or manholes has a horizontal cross sectional area bigger than 2 m^2 (20 sq ft) and hence their storage capacity is relatively small.

For circular sewers such as those used in the Oakdale Basin, when the pipe is flowing partially filled with a depth h and central angle $\theta = \cos^{-1}[1 - (2h/D)]$ as shown in Fig. 16, the flow area A and the corresponding hydraulic radius R and water surface width b are

$$A = \frac{D^2}{8} (\theta - \sin\theta) \tag{41a}$$

$$R = \frac{D}{4} (1 - \frac{\sin\theta}{\theta}) \tag{41b}$$

$$b = \frac{D}{\sin\frac{\theta}{2}} \tag{41c}$$

for $0 \leq \theta \leq 2\pi$ and D is the pipe diameter.

At the outlet of the Oakdale Avenue Drainage Basin a Simplex 0.76 m (30 in.) Type "S" parabolic flume is placed in a vault at the corner of Oakdale and Lamon Avenues to measure and record the basin runoff. This runoff measurement and recording system together with a tipping bucket recording rain gauge located at one block north of the basin has been in operation since 1959 measuring rainfalls and runoffs. Details of these measuring devices and the data collected can be found elsewhere (Tucker, 1968) and are not presented here.

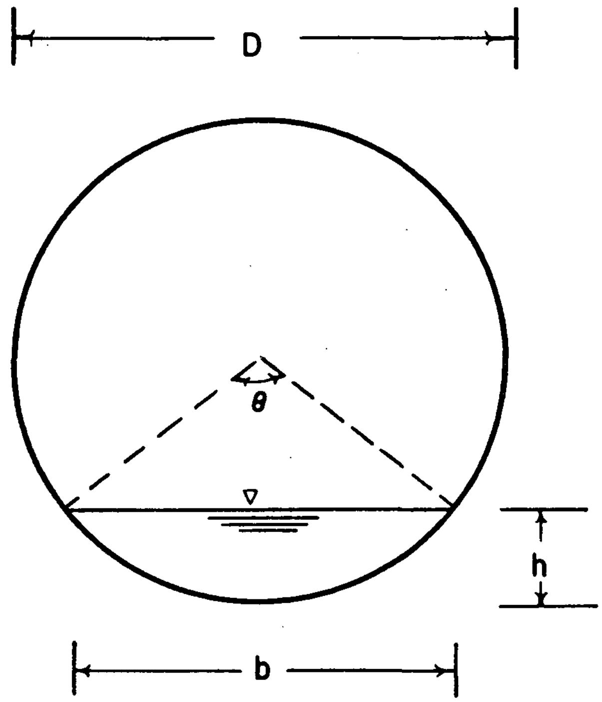


Fig. 16. Circular sewer flow cross section

Table 3. DIMENSIONS OF GUTTERS OF OAKDALE AVENUE DRAINAGE BASIN

Gutter From Inlet	Gutter To Inlet	Gutter Length ft	Gutter Length m	Size of Contributing Subcatchments ac	Size of Contributing Subcatchments m^2	Longitudinal Slope	Type of Grate Inlet at Down-stream End of Gutter*	Contribution to Sewer Junction
-	1	58	18	0.17	690	0.0012	C	102
1	2	201	61	0.53	2140	0.0012	R	104
2	3	192	59	0.51	2060	0.0012	C	106
3	7	120	37	0.32	1290	0.0012	R	109
-	4	104	32	0.04 (0.54)#	160 (2190)#	0.0010	C	107
-	5	104	32	0.04 (0.64)#	160 (2590)#	0.0010	C	107
4	7	46	14	0.03	120	0.0010	R	109
5	6	42	13	0.02	80	0.0010	R	109
6	8	117	36	0.32	1290	0.0027	R	110
8	9	194	59	0.51	2060	0.0027	R	112
9	10	200	61	0.53	2140	0.0027	C	114
10	14	114	35	0.31	1250	0.0027	R	117
-	11	100	30	0.02	80	0.0010	C	115
-	12	100	30	0.02	80	0.0010	C	115
11	13	192	59	0.07 (0.50)#	280 (2020)#	0.0010	R	117
12	14	192	59	0.07 (0.68)#	280 (2750)#	0.0010	R	117
13	15	96	29	0.26	1050	0.0010	C	118
-	15	96	29	0.26	1050	0.0010	C	118
-	16	58	18	0.17	690	0.0012	C	102
16	17	201	61	0.53	2140	0.0012	R	104
17	18	192	59	0.51	2060	0.0012	C	106
18	21	120	37	0.32	1290	0.0012	C	109
-	19	100	30	0.01 (0.54)#	400 (2190)#	0.0010	C	108
-	20	100	30	0.01 (0.78)#	400 (3160)#	0.0010	C	108
19	21	42	13	0.02	80	0.0010	R	109
20	22	42	13	0.02	80	0.0010	C	109
22	23	117	36	0.32	1290	0.0027	C	110
23	24	194	59	0.51	2060	0.0027	R	112
24	25	200	61	0.53	2140	0.0027	C	114
25	29	114	35	0.31	1250	0.0027	R	117
-	26	151	46	0.13 (0.64)#	530 (2590)#	0.0010	C	116
-	27	151	46	0.13 (0.41)#	530 (1660)#	0.0010	C	116
26	28	128	39	0.05	200	0.0010	R	117
27	29	128	39	0.05	200	0.0010	R	117
28	30	96	29	0.26	1050	0.0010	C	118
-	30	96	29	0.26	1050	0.0010	C	118

*Type of grate inlets: C = circular, R = rectangular
#Contribution by alleys
Gutter width W = 1.0 ft (0.30m)
Gutter bottom inclined 88^{o} 15' from vertical curb
Manning's n for gutters = 0.013

Table 4. DIMENSIONS OF ALLEYS OF OAKDALE AVENUE DRAINAGE BASIN

Location		Length ft	Length m	Width ft	Width m	Slope	Contributing to inlet
Alleys between Wellington and Oakdale	West of Leclaire	395	120.4	15.5	4.7	0.0047	13
	East of Leclaire	295	89.9	15.5	4.7	0.0049	14
	West of Leclaire	335	102.1	15.5	4.7	0.0042	5
	East of Lavergne	295	89.9	15.5	4.7	0.0053	4
Alleys between Oakdale and George	West of Leclaire	396	120.7	15.5	4.7	0.0043	26
	East of Leclaire	236	71.9	15.5	4.7	0.0053	27
	West of Lavergne	380	115.8	15.5	4.7	0.0040	20
	East of Lavergne	290	88.4	15.5	4.7	0.0054	19

Table 5. DIMENSIONS OF SEWERS OF OAKDALE AVENUE DRAINAGE BASIN

Sewer From Node	Sewer To Node	Length ft	Length m	Slope %	Diameter ft	Diameter m
118	117	108	32.9	0.72	1.00	0.30
115	117	170	51.8	0.71	0.83	0.25
116	117	105	32.0	1.08	0.83	0.25
117	114	134	40.8	0.45	1.25	0.38
114	113	34	10.4	0.45	1.25	0.38
113	112	168	51.2	0.45	1.25	0.38
112	111	158	48.2	0.40	1.50	0.46
111	110	38	11.6	0.40	1.50	0.46
110	109	131	39.9	0.40	1.50	0.46
107	109	50	15.2	3.78	0.83	0.25
108	109	45	13.7	4.20	0.83	0.25
109	106	153	46.6	0.35	1.75	0.53
106	105	39	11.9	0.35	1.75	0.53
105	104	156	47.6	0.35	1.75	0.53
104	103	156	47.6	0.30	2.00	0.61
103	102	61	18.6	0.30	2.00	0.61
102	101	73	22.3	0.30	2.00	0.61
101	100	32	9.8	0.30	2.50	0.76

VI. ILLINOIS SURFACE RUNOFF MODEL

The Illinois Urban Storm Runoff method actually consists of two parts: the surface runoff model and the sewer system routing model. The input into the surface runoff model is the hyetograph and the output is the inlet hydrographs which constitute the input into the sewer system routing model. The sewer routing model is the Illinois Storm Sewer System Simulation Model (Sevuk et. al., 1973) and will be described briefly in the following chapter. The Illinois surface runoff model is a recent development and will be discussed in this chapter. It should be noted here that improvement and refinements are continuously being made on both surface and sewer models and those reported here are the most up-to-date versions at the time of writing this report.

VI-1. Runoff in Subcatchments

The surface runoff is subdivided into two subsequent parts: the subcatchments which consists of only strips of overland flows receiving rainfall as the input; and the gutters which receive water from the subcatchments as well as from direct rainfall and deliver the water into inlet catch basins to produce inlet hydrographs. The overland surface of a drainage basin can be approximated by a number of equal-width rectangular strips of different lengths. A large number of such strips of narrow width will closely approximate the actual overland surface. But this will require a large amount of computations without significant improvement in accuracy. Contrarily, too few strips would approximate the actual geometry poorly.

Time varying free-surface flow including overland flows can be described mathematically by a pair of partial differential equations called the St. Venant equations (Chow, 1959; Yen, 1973a, 1973b; Sevuk et al., 1973)

$$\frac{\partial h}{\partial t} + D\frac{\partial V}{\partial x} + V\frac{\partial h}{\partial x} = \frac{1}{b}\int_{\sigma} q \ d\sigma \tag{42}$$

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g \cos\theta \ \frac{\partial h}{\partial x} = g(S_o - S_f) + \frac{1}{A}\int_{\sigma} (U_1 - V)q \ d\sigma \tag{43}$$

in which x is the direction of the flow measured along the bed; t is time; A is the flow cross sectional area; b is the width of the free surface; $D = A/b$ is the hydraulic depth; V is the cross sectional average flow velocity; h is the depth of the flow above the invert; θ is the angle between the channel bed and the horizontal; $S_o = \sin\theta$ is the bed slope; S_f is the friction slope; σ is the perimeter bounding A; q is the lateral discharge per unit length of σ having a velocity component U_1 along the x-direction when joining or leaving the flow; and g is the gravitational acceleration. The first equation is the equation of continuity and the second the momentum equation.

With the appropriate initial and boundary conditions, these two equations can be used to solve numerically using digital computer for the overland flow on subcatchments. However, the solution requires considerable amount of computer time and in view of the usually large number of subcatchments (overland strips) such an approach is feasible but impractical. In field conditions, the accuracy of data on overland geometry and rainfall input usually does not render the accuracy that the St. Venant equations can provide. Several approximations of Eqs. 42 and 43 are possible. Often the overland flow is approximated by using the Manning's formula or the Izzard's method which essentailly assumes the flow or the rainfall to be steady. Other approximations of the St. Venant equations include the kinematic wave model and diffusion wave model (Yen, 1973a). From past experience the non-linear kinematic wave model was found to be most suitable for solving overland flows because it does not require a downstream boundary condition and hence considerably reduces the computer time and difficulties and yet its accuracy is substantially better than that given by Manning's formula. Those who are interested in the relative accuracy of the different approximate models can refer elsewhere (Sevuk, 1973, Yen, 1973a).

For the kinematic wave approximation, the inertia and pressure terms of the momentum equation (Eq. 43) are neglected; thus,

$$S_o = S_f \tag{44}$$

The friction slope S_f can be estimated by using the Darcy-Weisbach formula

$$S_f = f \frac{1}{4R} \frac{V^2}{2g} \tag{45}$$

in which f is the Weisbach resistance coefficient given by the Moody diagram and R is the hydraulic radius, by the Manning formula

$$S_f = \frac{n^2}{2.22} V^2 R^{-4/3} \tag{46}$$

where n is the Manning's roughness factor, or by the Chezy formula

$$S_f = \frac{V^2}{c^2 R} \tag{47}$$

in which c is the Chezy factor. The continuity equation (Eq. 42) and Eq. 44, together with the initial condition and one upstream boundary condition, can be solved numerically for the unsteady flow. Equation 46 is for V in fps and R in ft; if V is in m/sec and R in m, the coefficient is unity instead of 2.22.

In selecting the resistance formula to approximate the friction slope, the Weisbach coefficient has the advantage of being dimensionless and having better theoretical justification, whereas Manning's n has the advantage of being nearly constant independent of flow depth for flows over rough boundaries with sufficiently high Reynolds number. However, for overland flows the depth is usually so shallow and the Reynolds number of the flow not sufficiently high that it would be erroneous to consider n to be constant (Chen and Chow, 1968; Yen, 1975). Therefore, the Weisbach formula (Eq. 45) is adopted in this model to evaluate the overland flow.

In Eq. 45, the value of f is given by the Moody diagram which can be found in standard hydraulics reference books (e.g., Rouse, 1950; Chow, 1959). For the case of overland flow under rainfall, limited information was given by Yen et al. (1972) and Shen and Li (1973). Based on the available information, the Weisbach f is computed as

$$f = \frac{C}{\mathbb{R}} \tag{48}$$

for laminar flow, in which $\mathbb{R} = VR/\nu$ is the Reynolds number of the flow where ν is the kinematic viscosity; and the coefficient C is

$$C = 24 + 101\ i^{0.4} \tag{49a}$$

for i in mm/hr, or

$$C = 24 + 27\ i^{0.4} \tag{49b}$$

for i in in./hr. Since the surface of natural overland is inevitably rough, for turbulent flow, f is constant as

$$\frac{1}{\sqrt{f}} = 2\ \log \frac{2R}{k} + 1.74 \tag{50}$$

where k is a length measure of surface roughness. The transition between Eqs. 48 and 50 is shown schematically in Fig. 17. The critical Reynolds number $\mathbb{R}_c$ determining which equation should be used is

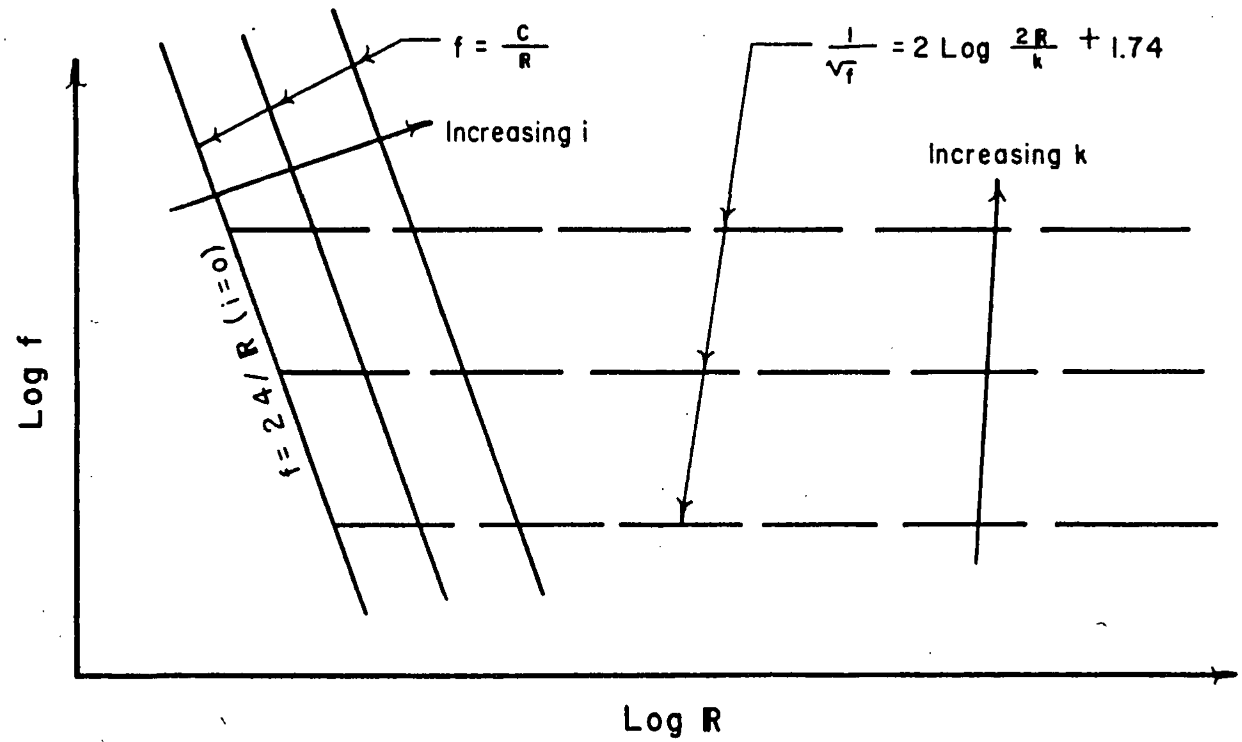


Fig. 17. Evaluation of Weisbach resistance coefficient

$$\mathbb{R}_c = C\ (2\ \log \frac{2R}{k} + 1.74)^2 \tag{51}$$

When the Reynolds number of the flow $\mathbb{R} < \mathbb{R}_c$, Eq. 48 applies. Otherwise, Eq. 50 is used. It should be noted that actually there is a transition between laminar (Eq. 48) and fully developed rough turbulent flow (Eq. 50). This transition is neglected here and the steady uniform flow values of f are used for unsteady cases. This approximation can of course be improved when more information on f becomes available.

The water input onto the subcatchment surface to produce the flow is the lateral flow q in Eqs. 42 and 43. The value of q is equal to the rainfall minus infiltration. The infiltration is estimated by using Horton's formula (Eq. 34). When the rainfall rate is smaller than the infiltration capacity, the deficiency is supplemented by the water on the surface, if any.

In solving Eqs. 42, 44 and 45 numerically, the initial condition to start the solution cannot be zero depth and zero velocity because this condition will impose a mathematical singularity. In reality, when rain falls on a dry overland surface, there is indeed an initial wetting process before runoff starts. The surface tension will hold a small amount of water without producing runoff. Therefore, the initial condition for the overland runoff from the subcatchments can be assumed as a small finite depth with zero velocity. In other words, immediately following the commencement of rainfall, after infiltration and other losses are subtracted, the water left on the overland surface simply accumulated without producing runoff until the initial depth is reached. This initial depth depends on the slope and nature of the overland surface. Future studies will provide more information on this initial depth. It suffices at present to assume the initial depth to be 0.0012 in. or 0.03 mm. It has been found that the final solution is practically unaffected by the value of the initial depth so long as it is assumed within a reasonable realistic range.

Several numerical schemes can be used to obtain the solution (Sevuk and Yen, 1973). A 4-point, noncentral, semi-implicit scheme is used to solve the equations because of its independent selection of the time and space increments (Δt and Δx) in the computations without stability problems and consequently saves computer time.

A more accurate and convenient form of Eq. 42 for the purpose of numerical solution is

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q_\ell \tag{52}$$

where Q is the flow rate at any flow section, A is the flow cross-sectional area and $q_\ell = \int_\sigma q d\sigma$ is the lateral flow per unit length of flow in x-direction, being positive for inflow. Applying the chain rule of differentiation to Eq. 52, and letting $G(h) = \partial Q/\partial h$ and $b(h) = \partial A/\partial h$, one obtains

$$G\frac{\partial h}{\partial x} + b\frac{\partial h}{\partial t} = q_\ell \tag{53}$$

For the semi-implicit four-point backward difference scheme adopted, referring to the fixed rectangular grid in Fig. 18, the coefficients and partial differential terms of Eq. 53 may be approximated by the following expressions

$$G = \frac{1}{2} (G_D + G_C) \tag{54a}$$

$$b = \frac{1}{2} (b_D + b_C) \tag{54b}$$

$$\frac{\partial h}{\partial x} = \frac{1}{\Delta x} (h_C - h_D) \tag{55a}$$

$$\frac{\partial h}{\partial t} = \frac{1}{2\Delta t} (h_D + h_C - h_A - h_B) \tag{55b}$$

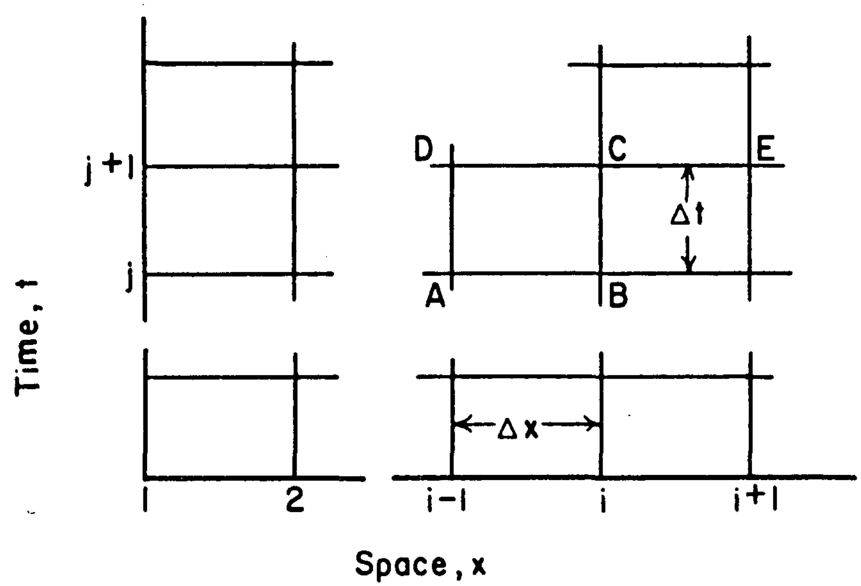


Fig. 18. Computational grid for semi-implicit four-point backward difference scheme

Substitution of Eqs. 54 and 55 into Eq. 53 yields

$$\frac{1}{2\Delta x}(G_C + G_D)(h_C - h_D) + \frac{1}{4\Delta t}(b_D + b_C)(h_D + h_C - h_A - h_B) = q_\ell \qquad (56)$$

The flow parameters at grid points A and B are known either from the initial conditions or from previous time step computations, and the flow parameters at point D are known either from the upstream boundary condition or from previous computations. Therefore, with G_C and b_C being specified functions of h_C, the only unknown in Eq. 56 is h_C.

Surface runoff on subcatchments usually occurs in the form of open-channel flow in wide channels. Consequently the runoff problem can be simplified by solving for the discharge per unit overland width, Q_u. Hence, in Eq. 56, $b_A = b_B = b_C = b_D = 1$. For laminar flow in a wide rectangular channel, combining Eqs. 44, 45 and noting that $Q_u = VR$ and $R = h$, one obtains

$$Q_u = \frac{8gS_o}{C\nu} h^3 \qquad (57)$$

Hence

$$G = \frac{\partial Q_u}{\partial h} = \frac{24gS_o}{C\nu} h^2 \qquad (58)$$

For the case of turbulent flow, Eqs. 45 and 50 yield

$$Q_u = \sqrt{8gS_o}\,(2 \log \frac{2h}{k} + 1.74)\, h^{3/2} \qquad (59)$$

and

$$G = \sqrt{8gS_o}\,(3 \log \frac{2h}{k} + 3.47)\, h^{1/2} \qquad (60)$$

The depth of water h at any subcatchment flow section is obtained by using Eqs. 56 and 58 for laminar flow, and Eqs. 56 and 60 for turbulent flow. Newton's iteration technique is used for the numerical processes. Knowing the flow depth, the discharge per unit width is evaluated by using Eq. 57 and Eq. 59 respectively, for laminar and turbulent flows. After the flow parameters at grid point C is computed, the computations for the next downstream station at the same time level (grid point E in Fig. 18) can be performed. After the flow parameters at all the stations at a given time level are evaluated, the computations is advanced to the next time level starting from the upstream end.

VI-2. Gutter Flow Routing

Street gutters and surface runoff in defined channels receive water from overland runoff of subcatchments, from upstream water sources, if any, and directly from rainfall. The input water is transported through the gutter or channel into the inlet catch basin to produce the inlet hydrographs for the sewer runoffs. Horton's formula (Eq. 34) is assumed applicable to account for infiltration. Theoretically, the gutter flow is also described mathematically by the St. Venant equations (Eqs. 42 and 43). Again, using these equations to solve for gutter flows is feasible but impractical in view of the large number of gutters for a drainage basin and the accuracy of the input data. In field conditions gutters are rarely prismatic channels because of poor control in construction and interruption of local facilities such as driveways and other intersections. Furthermore, in actual operation, gutters are often obstructed by debris, parked cars and the like. Such obstructions are time varying and random in nature. It is possible to describe the boundary condition of the gutters precisely for any particular runoff considered. Therefore, solving the gutter runoff by using the St. Venant equations, which require large amount of computer time and detailed geometry data, cannot be justified. Consequently the kinematic wave approximation is adopted for gutter flow routing as for the case of overland flow in subcatchments.

The differential equations of the kinematic-wave model for gutter flows are the same as those for overland flows, i.e., Eqs. 42, and 44. Therefore, the same solution technique can be used for both cases, and Eq. 56 is the finite difference equation representing also the gutter flow. However, the flow conditions in gutters are often within the range where Manning's formula (Eq. 46) can be used to approximate the friction slope, S_f. Use of Manning's formula instead of Darcy-Weisbach's (Eq. 45) simplifies the computation as it is no longer needed to check the instantaneous flow Reynolds number in order to estimate S_f. Thus, from Eqs. 44 and 46,

$$Q = \frac{C_n}{n} S_o^{1/2} AR^{2/3} \tag{61}$$

and

$$G = \frac{C_n}{n} S_o^{1/2} \left(\frac{2}{3} AR^{-1/3} \frac{\partial R}{\partial h} + R^{2/3} \frac{\partial A}{\partial h}\right) \tag{62}$$

where $C_n = 1$ in SI system and 1.49 in English system. The terms A, R, $\partial A/\partial h$ and $\partial R/\partial h$ in Eq. 62 should be evaluated from the cross-sectional shape of the gutter under consideration. For instance, for the triangular gutters in the Oakdale Avenue Drainage Basin these terms can be evaluated by using Eqs. 38 and 39. Specifying b in Eq. 56 as a function of h from the gutter geometry, it is possible to solve Eqs. 56 and 62 simultaneously for the flow depth h_C at grid point C by use of Newton's iteration technique. The discharge then is evaluated from

Eq. 61. The computation progresses in the downstream direction for each time level as described for subcatchment runoffs.

The initial condition for the gutter flow routing is essentially the same as that for runoff in subcatchments. As to the boundary conditions, kinematic wave model requires only the upstream conditions be provided. In the present gutter routing model the upstream boundary condition is provided as specified flow depths at the upstream end of the gutter at each time level. These flow depths are evaluated within the model using Manning's formula (Eq. 46) and they correspond to the carry-over of water from the upstream inlets. When there is no such carry-over at the upstream end of a gutter, the depth of water at the upstream flow section is assumed to be always equal to the initial depth.

VI-3. Inlets

Inlets are one of the most important components of urban drainage systems to determine the time distribution of urban storm runoffs. They control the amount of water to flow from gutters into sewers. The hydraulic characteristics of an inlet depend on the geometric properties of the inlet. Unfortunately, despite the large number of inlets used in streets and highways, the geometries of inlets have never been standardized. Furthermore, in their operation, inlets are seldom kept clean to be free from foreign materials partially clogging the inlet.

Inlets can be classified as curb type and grate type. A combination of the two is also used. Hydraulically they can be described by the weir formula

$$Q = C_d b H^{3/2} \tag{63}$$

or the orifice formula

$$Q = C_d A H^{1/2} \tag{64}$$

in which C_d is a discharge coefficient; A is the cross-sectional area of the orifice opening; b is the length of the weir; and H is the available head. The value of H depends on the gutter or surface flow depth near the inlet. The difficulty in using Eqs. 63 and 64 for inlet flow computation is the wide range of variations of the values of C_d and A, particularly for unclean inlets. Also, the determination of the range of application of the weir and orifice formulas is a matter of debate.

The inlet imposes a backwater effect on the gutter flow. For a supercritical flow in the gutter, disturbance waves cannot propagate upstream and hence the numerical solution of the St. Venant equations or its nonlinear kinematic-wave approximation can proceed forward from upstream without depending on the downstream boundary conditions.

For a subcritical flow the gutter flow is directly affected by the hydraulic conditions at its downstream end, i.e., the inlet. Consequently the inlet flow condition, which is by itself unknown and yet to be solved, becomes the necessary downstream boundary condition for the numerical solution of the St. Venant equations. Contrarily, for the kinematic-wave approximation, no downstream boundary condition is required. Consequently, the gutter flow can be solved without requiring simultaneous solution of the yet unknown inlet flow conditions, and hence the solution technique can be simplified and the required computer time greatly reduced. The inlet flow can subsequently be computed as will be described later. Such an approximation neglecting the backwater effect due to the inlet of course differs from the reality. However, in view of the uncertainties on the physical conditions of the gutters and inlets, it appears to be justified from a practical viewpoint that the gutter flow is computed by using the nonlinear kinematic-wave approximation and the inlet flow is computed independently.

A more accurate and potentially practical approach to gutter-inlet flow solution is to use generalized nondimensional curves describing inlet runoff hydrographs for different input and geometry conditions (Akan, 1973). However, this approach requires at least a certain degree of standardization tion of the inlets in order to avoid a large number of nondimensional graphs and hence it is not adopted here, although it may be used in the future for the refinement of the surface runoff model.

In the Illinois surface runoff model, the average depth plus the velocity head at the end of the gutter is used as the value of H in Eqs. 63 and 64 for the calculation of the inlet discharge. The discharge coefficient C_d in these equations is assigned different values according to the type of inlet under consideration. For instance, in Eq. 63, C_d is assumed to be equal to 3.0 for grate inlets with longitudinal bars and for combined inlets, 2.4 for grate inlets with diagonal bars, 2.7 for grate inlets with cross bars, and 1.2 for curb openings. The corresponding C_d values in Eq. 64 are 0.60, 0.48, 0.54, and 0.30, respectively. The inlet discharge is first computed by using Eq. 63 until this equation gives a discharge greater than the discharge of the approaching gutter flow. From then on, it is assumed that the flow around the inlet has the characteristics of orifice flow, and the inlet discharge is computed by using Eq. 64. During the recession of the gutter runoff, when the computed inlet discharge using the weir formula is smaller than the approaching gutter flow, the inlet discharge is assumed to be computed again by using the weir formula, Eq. 63.

When the approaching gutter flow is greater than the inlet discharge, the excessive water is assumed carried over the inlet to continue on as the input flow into the next gutter immediately following. Should there exist more than one downstream gutter such as at an intersection, the model allows a distribution of the carry-over flow among these downstream gutters. However, the distribution factors should be provided on the program data cards. This carry-over of excessive flow from inlet is assumed to continue until the flow reaches a low point such as Junctions 109 and 117 in Fig. 13 where no further carry-over can reasonably be assumed and a reservoir storage routing is performed for the discharge through the last inlet and the storage around it.

The assumptions on the distribution of carry-over flow, on the transition between the weir and orifice flows, and on the values of C_d are not precise as the reality. Improvement and refinements on these aspects can be made in the future when more reliable and useful laboratory and field data become available.

VI-4. Program Description and Data Preparation

The Illinois Surface Runoff Model is programmed in Fortran IV language for computer solutions. The input into the computer program consists of the geometric characteristics of subcatchments, gutters, inlets and the identification of the sewers joining the inlet catch basins, and also the rainfall hyetographs. The output is the inlet hydrographs which serve as the input into the sewer system. The program also performs water quality computations of the runoff to produce inlet pollutographs. The formulation and details of the water quality model will be given in Chapter VIII.

(A) Program Description. - The computer program of the Illinois Surface Runoff Model as listed in Appendix B allows the consideration of a maximum number of 100 gutters at a time. Along each gutter, the subcatchments can be approximated by as many as 10 rectangular strips. These strips of overland areas may have different lengths, slopes, and surface and infiltration properties, but should be equal in width. Two different pollutants are considered at a time for each gutter and subcatchment strip. The program allows for the entire basin a maximum of five zones of rainfall with different hyetographs. The computational logic is shown schematically in Fig. 19. The computer storage requirement for the program in its present form is about 400K. If more storage is available, the program can easily be modified to consider larger basins. This modification can be achieved by simply changing the arrays in the dimension statements.

The computer program consists of one main program and six subroutines. The relationship between the main program and the subroutines is shown schematically in Fig. 20. A brief description is as follows:

MAIN PROGRAM: It reads and stores data for the entire basin. It performs

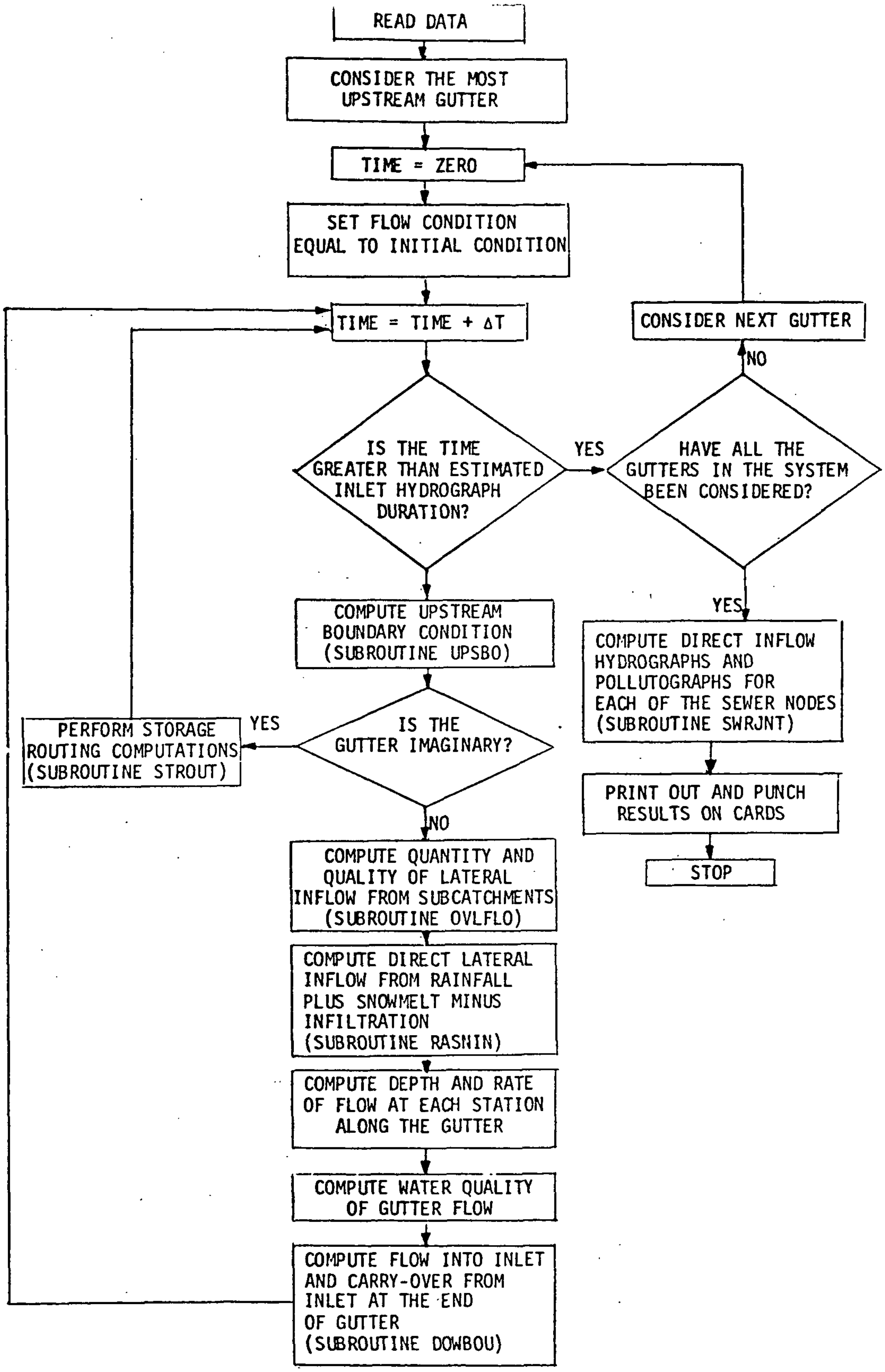


Fig. 19. Flow chart for Illinois surface runoff model computer program

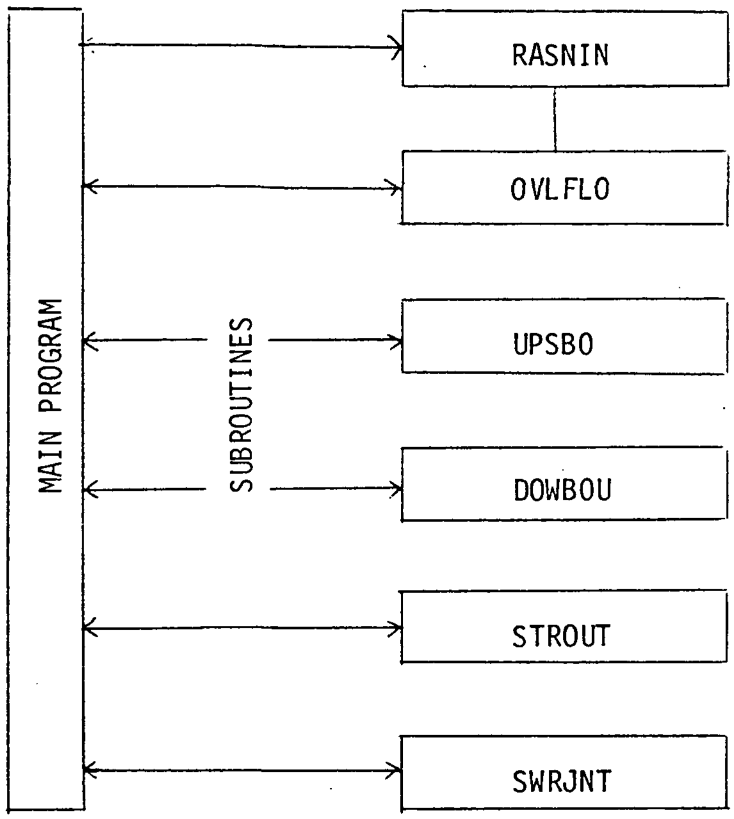


Fig. 20. Composition of Illinois surface runoff model computer program

the gutter routing computations based on nonlinear kinematic wave and Manning's equations. Newton's iteration technique is used to solve Eqs. 56 and 62, and discharge is computed by using Eq. 61. The computations are made starting from the most upstream gutters and proceeding towards downstream. A mechanism is built in the program to decide which gutters should be considered first. This allows the gutters in the drainage basin to be numbered arbitrarily from 1 to 100 while preparing data. However, the user should specify the flow direction in each gutter. The water quality computations for gutter flows are also performed in the main program.

SUBROUTINE OVLFLO: Flow in subcatchment strips is computed in this subroutine. Newton's iteration technique is used to solve the nonlinear kinematic wave equations with Darcy-Weisbach's formula; i.e. Eqs. 56 to 61. An approximate form of the Moody diagram (Fig. 17) is built in the subroutine to estimate the resistance coefficient. This subroutine is called from the main program while the computations are being done for each grid point along the gutter, unless the strip characteristics are identical for which the preceding values can be used. The water quality computations for each subcatchment strip are also performed in this subroutine as will be described in Chapter VIII. The output from subroutine OVLFLO provides part of the lateral inflow for the gutter routing in the main program.

SUBROUTINE RASNIN: This subroutine computes from rainfall the rate of lateral inflow for the subcatchments and part of the lateral inflow for the gutters. The inflow is evaluated as rainfall plus snowmelt minus infiltration. This subroutine is called from the main program and subroutine OVLFLO.

SUBROUTINE UPSBO: This subroutine computes the upstream boundary condition for each gutter. It is called by the main program when computations are made for the upstream end of each gutter at each time level. The carry-over water from all the immediately upstream inlets are summed up and a corresponding flow depth is computed using Manning's formula. This computed depth is accepted as the upstream flow depth for the gutter being examined at the time level being considered.

SUBROUTINE DOWBOU: Knowing the gutter outflow as computed by the main program at each time level, this subroutine is called upon to evaluate the inflow into the inlets and the carry-overs. The flow into the inlets is computed by using the weir or orifice formulas (Eqs. 63 and 64) as explained in Sec. VI-3.

SUBROUTINE STROUT: This subroutine provides a storage routing procedure around the inlets where there exist no immediate downstream gutters and hence there is no carry-over. The inflow to the storage area consists of the outflow from the upstream gutters. The outflow from the storage area is the flow into the inlet computed by using the orifice formula (Eq. 64).

SUBROUTINE SWRJNT: This subroutine prepares the output (inlet hydrographs) in data cards and provides the link between the Illinois Surface Runoff Model and the Illinois Storm Sewer System Simulation Model. After all the flow hydrographs into the inlets are computed, this subroutine is called by the main program. For the sake of convenience in linking the surface runoff and sewer routing models, the output hydrographs of the surface runoff model are identified by the sewer nodes, i.e., sewer manholes or junctions, instead of the corresponding inlets if they carry different identification numbers. When there are more than one gutter inlets discharging into the same catch basin or sewer node, the ordinates of those inlet hydrographs are summed up. The computed hydrographs for each of the sewer nodes are provided on computer cards as a part of the surface runoff program output. These cards are in a format compatible to the input data card requirements for the Illinois Storm Sewer System Simulation Model and can be used as part of the data deck for sewer routing. The ordinates of the sewer node inflow hydrographs and pollutographs are also printed out from this subroutine.

(B) Data Preparation. - Detailed information on basin characteristics is needed for the Illinois Surface Runoff Model and hence a number of data cards are required for the model. The data deck consists of the following sets in the order of presentation:

(1) General description of drainage basin: This set consists of two cards. The first card of the first set in the data deck specifies whether the data is provided in English or metric system of units. If the English system is used, the integer number 1 should be punched in the first column of the card. If the metric system is used, the integer number 2 should be punched in the first column. The second card of the set specifies the following information: the total number of gutters in the system; the total number of sewer nodes in the system; the total number of rain-zones considered; an integer number that indicates the frequency of printed output (e.g. when the number is equal to 1, the output is printed out at every time level); the time interval of computation in min; time in min when the execution should stop for each gutter corresponding to the estimated duration of the gutter outflow hydrographs; the gravitational acceleration = 32.2 ft/sec^2 or 9.81 m/sec^2; the average daily temperature on the day of rainstorm (in^{o}F or ^{o}C); the kinematic viscosity of water in gutter (in ft^2/sec or mm^2/sec); and the constant C in Eq. 48. The first four quantities must be punched in I5 formats and the remaining *six quantities must* be punched in I5 format. The time interval and the stop execution time values should be

selected such that the latter must be an exact multiple of the former, and there should be no more than 100 time steps of computation. When no snowmelt is involved the space for the daily temperature is left blank.

(2) Hyetographs: This set of data consists of several subsets of cards. Each subset corresponds to a rainfall zone. The first data card in each subset gives time in min at which the rainstorm starts (F10.0); time in min the rainstorm stops (F10.0); the total daily rainfall on the day of rainstorm in in. or mm (F10.0); and an integer number to specify the number pairs of time and rainfall intensity values used to describe the hyetograph (I5). The other cards following in the same subset give the hyetograph ordinates. There should be no more than 8 pairs of values on each card corresponding respectively to time in min (F5.0) and rainfall in in./hr or mm/hr (F5.0). There should be no more than 100 pairs of time-intensity values to describe a hyetograph.

(3) Gutter, inlet and subcatchment descriptions: This set contains a number of subsets. Each subset corresponds to a gutter considered in the system. Each gutter is given the same number as the inlet at its downstream end. When there exists no inlet, an imaginary inlet should be assigned. The types of gutters and inlets considered in the program are represented by a number as shown in Fig. 21. When an imaginary inlet (type = 0.0) is introduced, the flow into the inlet is always zero, and the gutter outflow is equal to the carry-over from the imaginary inlet. When storage routing is required for an inlet, it is necessary to use the concept of imaginary gutter (type = 0.0). When there is an imaginary gutter, the program does not perform any gutter routing but it calls the subroutine STROUT to perform storage routing. The data cards required for each subset contains the following four or more cards: (a) The first card of the subset specifies the gutter number (I5); number of grid points to be considered for gutter routing (I5); rain-zone number to

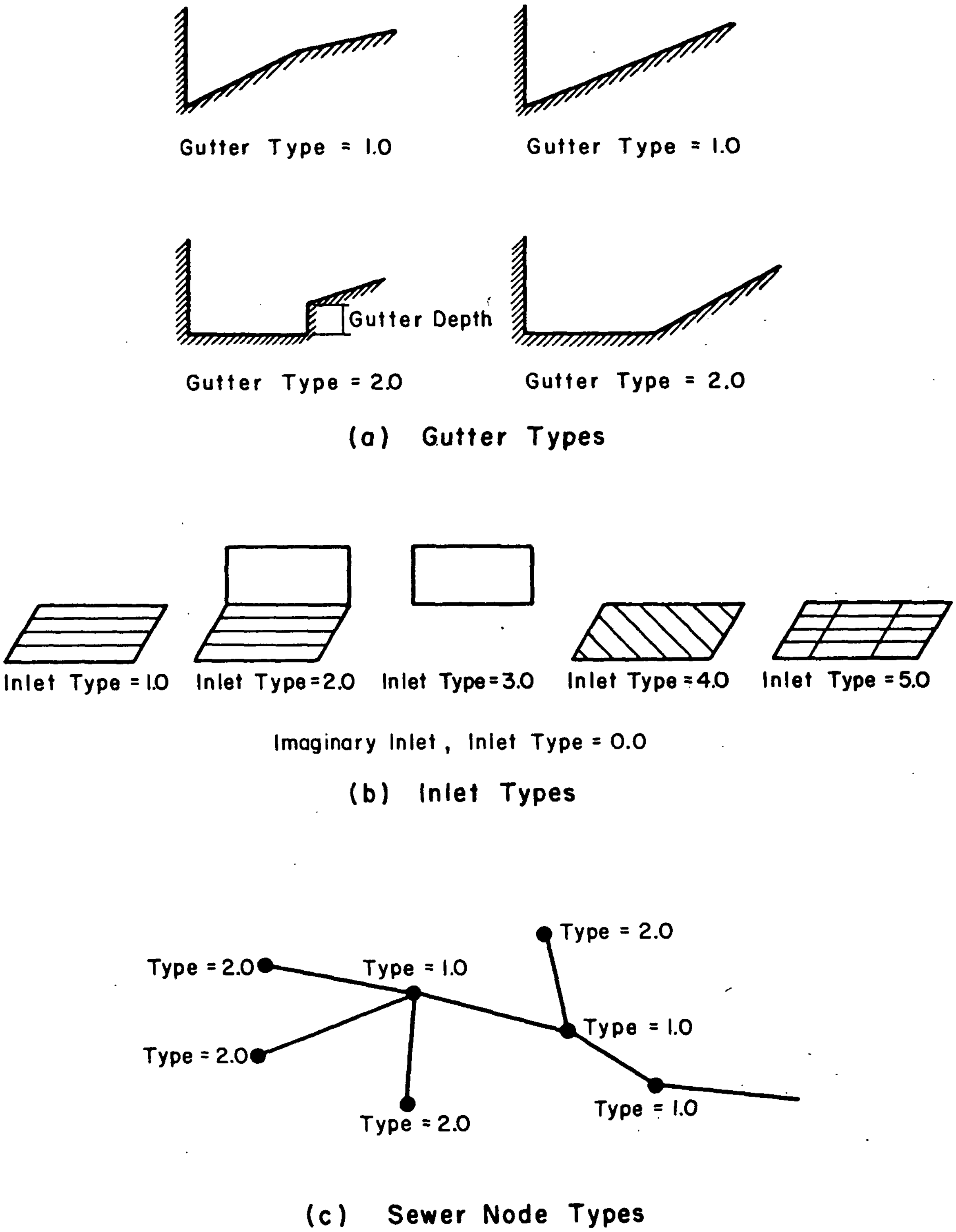


Fig. 21. Identification of types of gutter and inlet

which the gutter belongs (I5); type of gutter (F5.0); length of gutter in ft or m (F5.0); width of gutter in ft or m (F5.0); longitudinal slope of gutter (F5.0); depth of gutter in ft or m for rectangular gutters (F5.0); the angle between the gutter plane and the vertical in radians (F5.0); inlet type at the end of the gutter (F5.0); width of the inlet in ft or m (F5.0); length of the inlet in ft or m (F5.0); ratio of total area of the openings to the total area of the inlet (F5.0); width of street pavement measured from the crown to the gutter in ft or m (F5.0); the uniform initial depth of flow along the gutter in ft or m (F5.0); and Manning's roughness factor for the gutter (F5.0). (b) On the second card of the subset are the initial infiltration capacity of gutter surface in in./hr or mm/hr (F5.0); the final infiltration capacity of gutter surface in in./hr or mm/hr (F5.0); Horton's constant of decay rate of infiltration, k, for gutter surface in hr^{-1} (F5.0); initial infiltration capacity for street pavement in in./hr or mm/hr (F5.0); final infiltration capacity for street pavement in in./hr or mm/hr (F5.0); Horton's constant of decay rate of infiltration, k, for street pavement in hr^{-1} (F5.0); initial concentration of the first pollutant associated with gutter flow in ppm or mg/l (F5.0); and initial concentration of the second pollutant associated with gutter flow in ppm or mg/l (F5.0). (c) On the third card of the subset are the numbers of six immediately upstream inlets (6I5); and the proportions of carry-over from these inlets that go into the gutter (6F5.0). Note that when there are no upstream inlets, this card should still be there but with nothing punched on. (d) On the fourth card of the subset are the length of a subcatchment strip in ft or m (F5.0); the slope of the strip (F5.0); surface roughness in ft or m (F5.0); initial capacity of infiltration in in./hr or mm/hr (F5.0); final infiltration capacity in in./hr or mm/hr (F5.0); Horton's k for infiltration in hr^{-1} (F5.0); uniform initial depth of flow along the strip in ft or m (F5.0); initial concentration of the first pollutant associated with the subcatchment flow in the strip in ppm or mg/l (F5.0); initial concentration of the second pollutant in ppm or mg/l (F5.0); and number of computation grid points along the strip (I5). This card (d) should be repeated for each of the subcatchment strips starting from the one at the upstream end of the gutter. The number of subcatchment strips is equal to the number of computation grid points along the gutter minus one.

When considering an imaginary gutter, the area of the storage surface is punched as the fifth quantity on card (a) in ft^2 or m^2 instead of the gutter length. The other gutter properties can be assigned any values since they will not be used. The number of grid points should be assigned the value 2. Then the second (c) and the fourth (d) cards each can be replaced by a blank data card.

(4) Sewer node description: In this set of data one card is needed to describe each sewer node. There are two different types of sewer nodes to be considered. The type 1.0 represents the junctions of sewers in the layout. The upstream nodes without any incoming sewer pipes are classified as type 2.0. On each card of this data set the following information is required: the sewer node number (I5); the type of sewer node (F5.0); the base flow for the sewer node in cfs or m^3/sec (F5.0); the concentration of the first pollutant associated with the base flow in ppm or mg/l (F5.0); the concentration of the second pollutant in ppm or mg/l (F5.0) and the inlet identification numbers of up to ten gutter inlets discharging into the sewer node under consideration (10I5). When there are less than ten gutter inlets discharging into the sewer node, the excess space should be left blank.

A conceptual simple drainage system and the corresponding data representation is shown in Fig. 22 as an example.

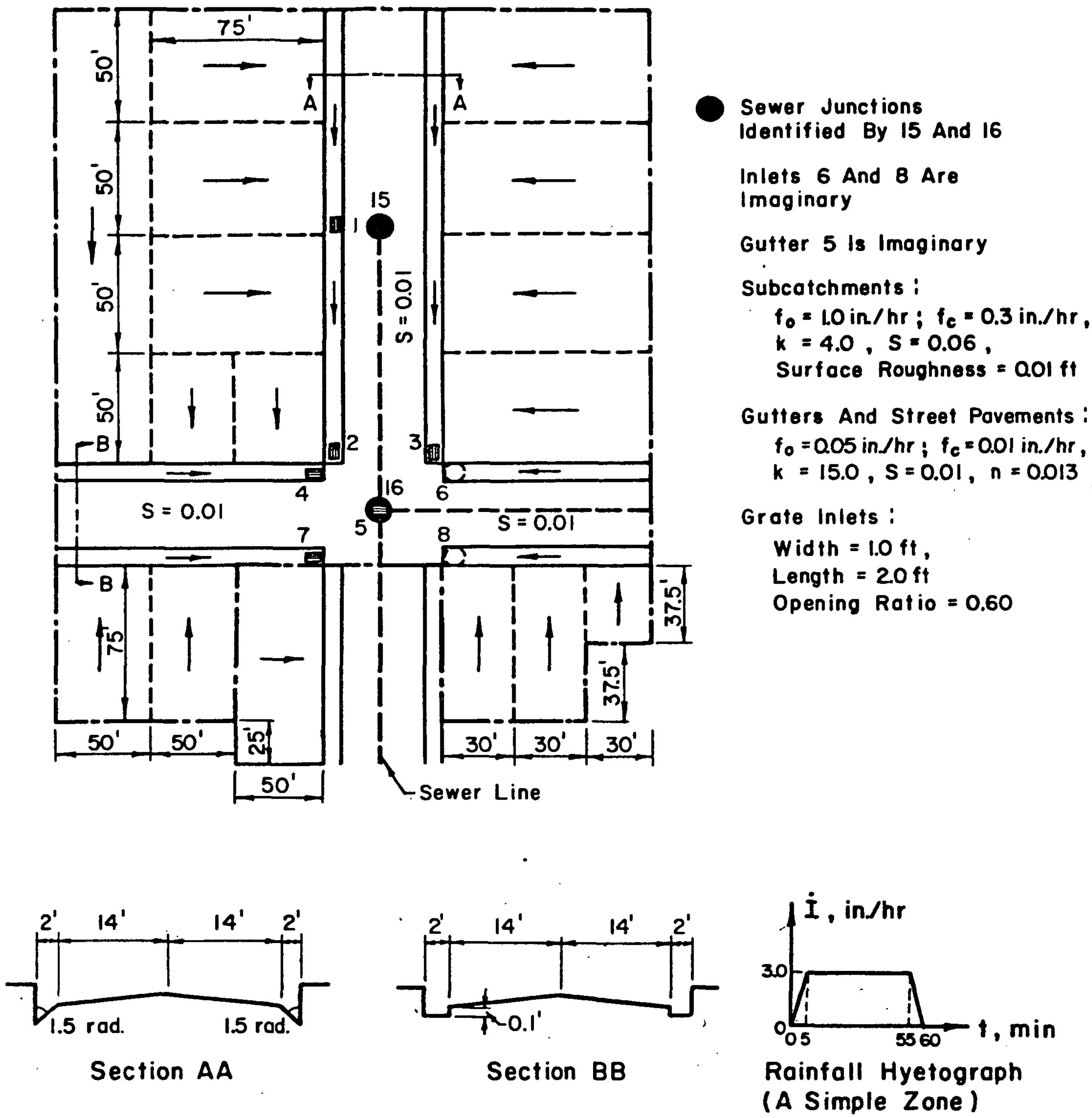


Fig. 22. Example of Illinois surface runoff model data preparation
(a) Layout of conceptual basin

```
SET 1  1
          8     2     1     2     1.0        90.0         32.2        34.0   0.000011     24.0

SET 2     0.        60.0          3.5           9
       0.0   0.0  5.0   3.0  10.0   3.0  20.0   3.0   30.0  3.0  40.0  3.0   50.0  3.0  55.0  3.0
       60.0  0.0

          2     3     1  1.0  100.  2.0  0.01  0.0   1.5   1.0   1.0   2.0   0.6  14.0.00010.013
       0.05  0.01 15.0  0.05  0.01 15.0  0.5   0.1
          1                               1.0
        75.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
         0.    0.    0.    0.   0.    0.    0.    0.    0.
          1     3     1  1.0  100.  2.0  0.01  0.0   1.5   1.0   1.0   2.0   0.6  14.0.00010.013
       0.05  0.01 15.0  0.05  0.01 15.0  0.5   0.1

        75.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
        75.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
          3     5     1  1.0  200.  2.0  0.01  0.0   1.5   1.0   1.0   2.0   0.6  14.0.00010.013
       0.05  0.01 15.0  0.05  0.01 15.0  0.5   0.1

        90.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
        90.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
        90.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
        90.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
          4     4     1  2.0  150.  2.0  0.01  0.1   0.    1.0   1.0   2.0   0.6  14.0.00010.013
       0.05  0.01 15.0  0.05  0.01 15.0  0.5   0.1

SET 3  200.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      6
        50.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
        50.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
          7     4     1  2.0  150.  2.0  0.01  0.1   0.    1.0   1.0   2.0   0.6  14.0.00010.013
       0.05  0.01 15.0  0.05  0.01 15.0  0.5   0.1

        75.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
        75.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
       100.  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      4
          6     3     1  2.0  90.   2.0  0.01  0.1   0.    0.0   0.    0.0   0.   14.0.00010.013
       0.05  0.01 15.0  0.05  0.01 15.0  0.5   0.1

         0.    0.    0.    0.   0.    0.    0.    0.    0.
         0.    0.    0.    0.   0.    0.    0.    0.    0.
          8     4     1  2.0  90.   2.0  0.01  0.1   0.    0.0   0.    0.    0.   14.0.00010.013
       0.05  0.01 15.0  0.05  0.01 15.0  0.5   0.1

       37.5  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
       75.0  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
       75.0  0.06  0.01  1.0  0.3   4.0  .0001  0.5   0.1      3
          5     2     1  0.0  324.                           1.0   1.0   2.0   0.6

          4     7     2     3     6     8  1.0   1.0   1.0   1.0   1.0   1.0

SET 4    15    2.   0.2   0.5   0.1     1
         16    1.   0.2   0.5   0.1     2     3     4     6     7     8
```

Fig. 22. (b) Data representation

VII. SEWER SYSTEM ROUTING MODEL

As mentioned previously, the sewer routing model of the Illinois Urban Storm Runoff Method is the Illinois Storm Sewer System Simulation Model (ISS Model). The ISS Model actually consists of two options: the design option for which the size of the sewers are to be determined, and the flow prediction option for which the size of the sewers and junctions are known and the objective is to compute the runoff hydrographs for given inputs. Since the ISS Model has been reported in detail elsewhere (Sevuk et al., 1973), and the objective of this research is to investigate methods of prediction for urban storm runoffs for the purposes of pollution control and management, only the flow prediction option of the ISS Model is briefly summarized in this chapter. Those interested in design of storm sewer networks are recommended to refer to a comparative study on using the ISS and other methods (Yen and Sevuk, 1975).

VII-1. Sewer Network Representation

One of the most important aspects in solving sewer flow problems in a network of sewers is to properly and systematically represent the geometric sequences of the sewers. This is particularly important if computer solution is used for which a logical means of selecting the proper order of sewers is a prerequisite of solution. For a small network consisting of a few sewers, it is not difficult to assign specifically the sequence that the computation should follow. For a large system consisting of many sewers, and particularly with the possibility of alternation of the sewer connecting pattern, it is more desirable and practical to set up some rules that the computer can follow to select the sequences. A computer program written for specific sequence can be used only for those networks with patterns following that sequence, and it is necessary to number the sewers precisely as the sequence requires. Contrarily, a computer program written for a certain sequence selection rule allows arbitrary patterns. In view of the variability of urban sewer systems and the large number of sewers involved in each of the system, the latter approach of setting a rule for sequencing to allow arbitrary numbering of the sewers is adopted in the ISS Model.

In this approach the members of a sewer system are represented by a node-link system commonly used in network analyses. The nodes are the junctions or manholes that join the sewers and each is assigned arbitrarily a number, as shown in Fig. 13 and Table 5 for the Oakdale Avenue Drainage Basin. The sewers are the links in the network and they are represented by two numbers, the first being the number of the upstream node and the second the downstream node. The outlet of sewer system is the root node and is joined by only one sewer. Thus, two sewers that join at the same node each will have one of its two identification numbers identical to the others. The sequence becomes a systematic search of marked numbers, and the computer can easily determine the connectivity, i.e., the pattern, of the network.

In solving for the flow in a sewer network, the solution obviously proceeds from upstream sewers toward downstream, no matter if the backwater effect is accounted for or not. To determine which upstream sewers should be solved first and the sequence of the sewers to be solved, a systematic searching method is adopted. The search starts from the root node, i.e., the outlet of the sewer system, to detect if the sewer connected to this node has already been solved. If this connected sewer is not yet solved, then the search moves to its upstream node and the process is repeated again. For nodes joining two or more branches, the process can be proceeded one by one following certain order, e.g., following the relative order of the branches stored in the computer, or following the order, say, from left to right of the branches connected to the node. For example, if the latter rule of left to right is used, the search will start from the root node. Since the flow for the last sewer connecting to the root node has not yet been solved, the search will move to the upstream node of the last sewer. Assuming that there are two other sewers joining to this node, the search will first look if the left one was solved. If it has not been solved, the search will automatically move up to the upstream node of this left sewer and repeat the entire process again. If the left sewer has already been solved, the search will then move to the sewer at the right. If this right sewer has not yet been solved, the search will move to its upstream node. If the right sewer has already been solved, this implies that the sewer with its upstream end joining the node is the only one to be solved. After the solution for this sewer is obtained, the search returns to its downstream end node. In this manner, the solution is obtained systematically, branch by branch, from upstream towards downstream.

VII-2. Method of Solution

In the ISS Model, the flow in each of the sewers is determined by solving the St. Venant equations (Eqs. 42 and 43). The friction slope, S_f, is evaluated by using the Darcy-Weisbach formula (Eq. 45). The Weisbach resistance coefficient f is estimated by using a simplified form of the Moody diagram. Since laminar flow rarely occurs in sewers, only turbulent flow is considered. For fully developed turbulent flow in hydraulically rough conduits, Eq. 50 applies. For hydraulically smooth conduits, the Blasius formula is

$$f = \frac{0.223}{\mathbb{R}^{0.25}} \tag{65}$$

for $\mathbb{R} < 4 \times 10^5$. In sewers hydraulically smooth boundary flow seldom occurs with $\mathbb{R} > 10^6$. Hence if the Blasius formula is assumed to apply up to slightly higher Reynolds number and the transition between smooth- and rough-surface flows is neglected, the threshold Reynolds number, $\mathbb{R}^*$, is

$$\mathbb{R}^* = 0.633\ (\log \frac{2R}{k} + 0.87)^8 \tag{66}$$

The value of f is computed from Eq. 65 or Eq. 50 depending on whether $\mathbb{R}$ is less or greater than $\mathbb{R}^*$.

One initial condition and two boundary conditions are needed to solve the St. Venant equations. For supercritical flow, the two boundary conditions are furnished by the flow conditions at the upstream node of the sewer. This imposes no computational problem since the solution is proceeding towards downstream. However for subcritical flow, one boundary condition is furnished from the upstream node and the other should be from the downstream node. This downstream boundary condition physically represents the backwater effect from the junction to the sewer and usually it is an unknown to be solved. The junction flow condition, in turn, is determined by not only its physical properties but also the flow conditions of all the sewers joining to the junction. Therefore, to solve for the flow in a sewer network, it is necessary either to solve simultaneously all the equations describing mathematically the flows in all the sewers and junctions of the network, or to subdivide the network into components for solution by successive approximations. The first approach is possible and practical for small systems consisting of a few sewers and junctions. For large systems this simultaneous solution method would easily become immanageable. Therefore the second approach is adopted for the ISS Model using a technique called overlapping Y-segments.

In the ISS Model the sewer systems are considered as a tree type network, each consisting of branches formed by a number of connected Y-segments. Each Y-segment contains three sewers joined by a common junction. The hydraulic condition of a junction is accounted for by a dynamic equation in addition to the continuity equation commonly used. The continuity equation is

$$Q_1 + Q_2 + Q_j - Q_3 = \frac{ds}{dt} \tag{67}$$

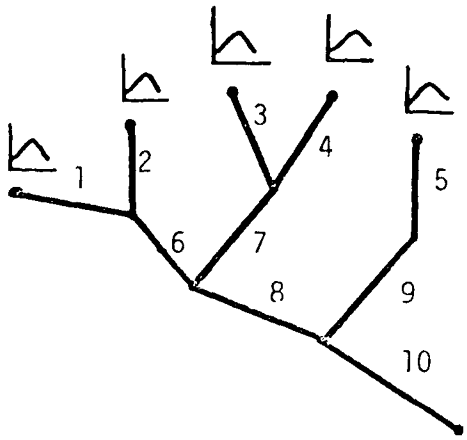
in which s is the storage in the junction; Q_j is the direct inflow into the junction; and the subscripts 1 and 2 represent the inflow sewers and 3 the outflow sewer from the junction. If the storage of the junction is negligible, the right-hand side of Eq. 67 is equal to zero.

The dynamic equation for a junction with large storage, i.e., reservoir type junction, is

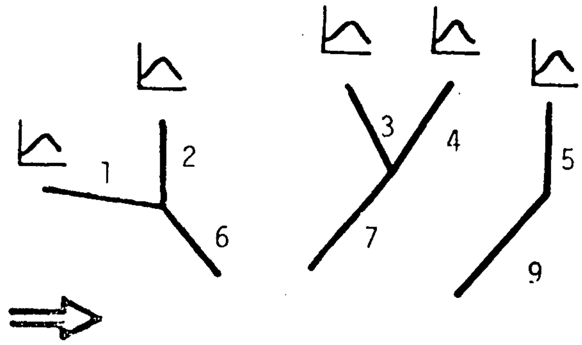
$$z_1 + h_1 = z_2 + h_2 = z_3 + h_3 + \frac{V_3^2}{2g} \tag{68}$$

in which h is the depth of sewer flow at the junction and z is the elevation of the sewer invert above a reference horizontal datum. For a point-type junction with negligible storage, the velocity head term ($V_3^2/2g$) in Eq. 68 is assumed equal to zero. Furthermore, if the inflowing sewer has a drop producing a free-fall of the flow, the depth for that sewer is equal to the critical flow depth corresponding to the instantaneous discharge of the sewer.

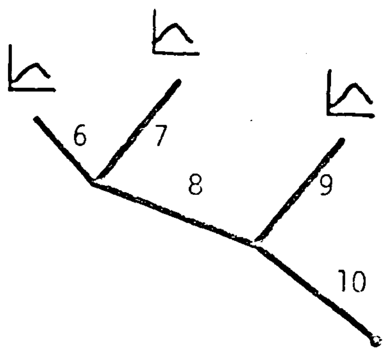
The St. Venant equations (Eqs. 42 and 43) and the junction equations (Eqs. 67 and 68) can be solved simultaneously for a Y-segment with known upstream boundary conditions of the inflowing sewers and assumed (if unknown) downstream boundary condition for the outflowing sewer. Since the downstream boundary condition is assumed, the solution is only approximate and a successive overlapping Y-segment technique is adopted to improve the accuracy of the solution. The technique is shown schematically in Fig. 23. Numerical solutions are first obtained, branch by branch, for those Y-segments whose inflowing sewers are connected to the inlet catch basins. The prescribed inlet hydrographs (or the outflow hydrographs from the Illinois Surface Runoff Model) and the compatibility conditions at the junction are used as the boundary conditions. In addition, if the downstream condition of the Y-segment is unknown, the forward differences are used as a substitution for the downstream boundary conditon. The solution is obtained by applying the St. Venant equations to each of the three sewers and Eqs. 67 and 68 to the junction and solving these equations simultaneously for the Y-segment using a first-order characteristic method (Sevuk et al., 1973). After the computation is completed, the first trial solution for the outflowing sewer is discarded but the "true" solution for the inflowing sewers is retained. Thus the inflow hydrograph into the junction of the current Y-segment is obtained. This junction will serve as one of the two inlets of the next Y-segment and the original outflowing sewer will become an inflowing sewer for the advanced new Y-segment. This procedure is repeated until the entire network is solved. For the last segment of the system, the prescribed boundary condition at its downstream end, the outlet of the system, is used and thus the numerical solution over the entire network is completed.



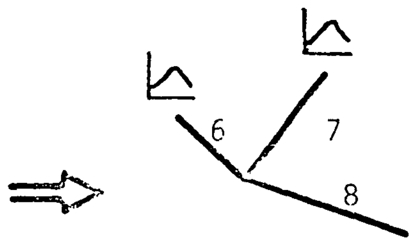
a. Complete solution domain



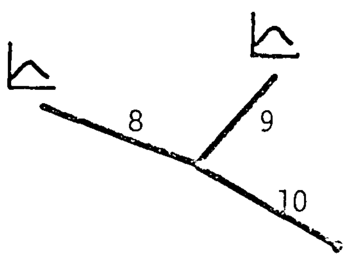
b. Solution domains for first-order sewers (1 through 5)



c. Reduced solution domain



d. Solution domain for second-order sewers (6 and 7)



e. Final solution domain for the sewers 8, 9, and 10

Fig. 23. Solution by method of overlapping y-segments

The ISS Model, in its present form, can simulate the flow with regulating and operational devices if they are located at the system outlet. From the hydraulics viewpoint, such control facilities can be expressed mathematically as stage-time relationship $h = f(t)$; velocity-depth relationship $V = f(h)$; discharge-depth relationship $Q = f(h)$; and discharge-time relationship $Q = f(t)$.

Similar to the case of solving numerically the overland and gutter flows, the initial condition for the sewer routing cannot be dry-bed; i.e., zero depth and zero velocity will impose a computational singularity. For combined sewers, the initial condition can be evaluated from dry-weather flow. For storm sewers without initial base flow, a small and negligible base flow is assumed to start the computation.

In using the overlapping Y-segment technique, only three sewers are considered at a junction. For a junction with more than three joining sewers, only three can be considered for direct backwater effects. Others, preferably those with small backwater effects from the junction, can be treated as direct inflows, i.e., as Q_j in Eq. 67. For a junction joined by only two sewers, the third sewer of the Y-segment can be considered as imaginary with zero length.

VII-3. Computer Program Description

The ISS Model just described has been programmed for computer solution. A macro flow chart showing the logic of solution for the flow prediction option of the computer program is given in Fig. 24. The program begins its execution by reading a set of control and data cards which describes the run control specifications (user commands), sewer system layout and physical characteristics of system components and

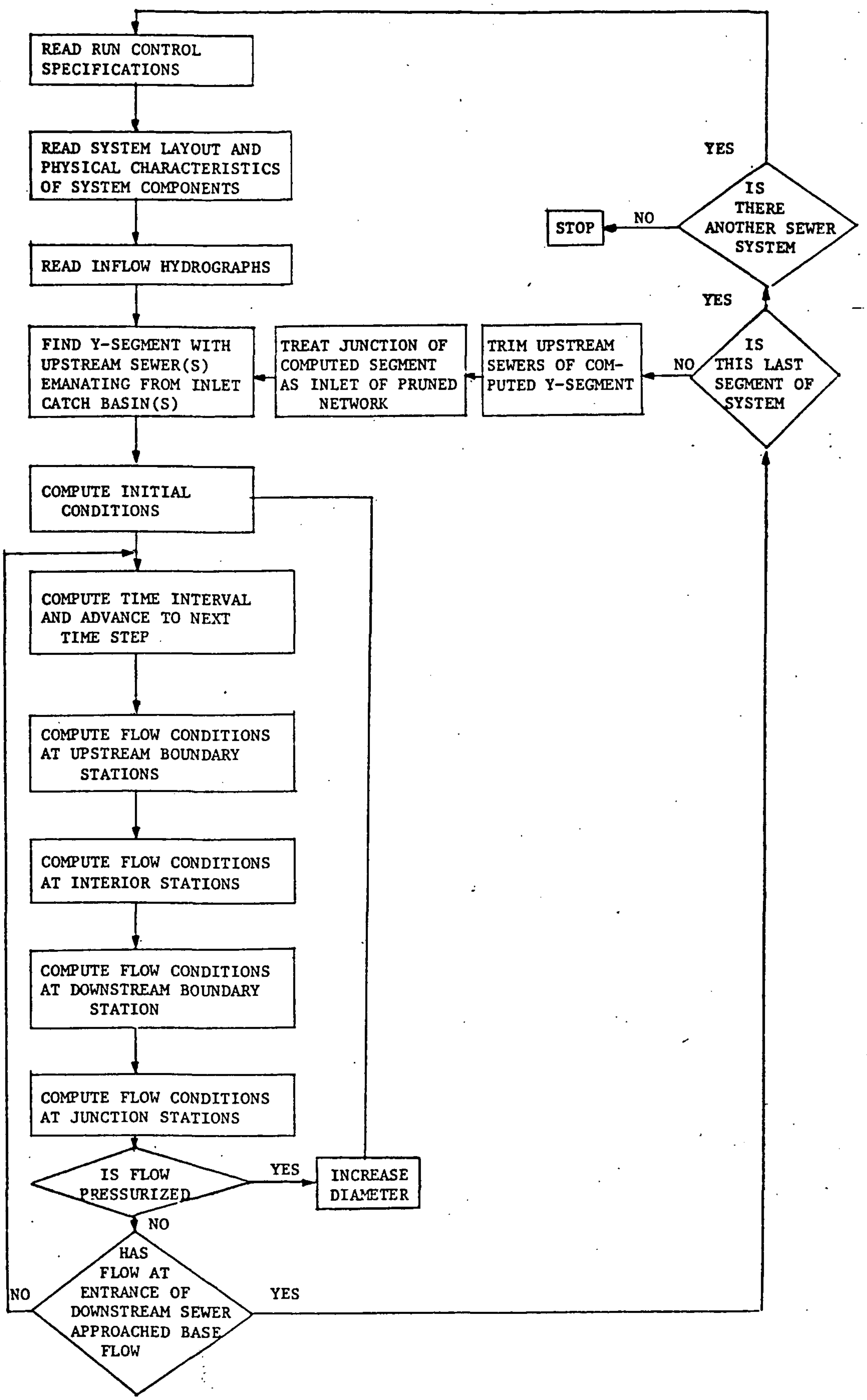


Fig. 24. Flow chart for ISS model computer program flow prediction option

inflow hydrographs. After verifying the accuracy and completeness of this information, the program starts to execute the numerical simulation phase. Output from the numerical simulation consists of a tabular print out of the computed or specified sewer diameters, time variations of flow rate, velocity, and depth at sewer entrances, and space variations of flow rate, velocity, and depth along the sewers at specific time intervals. In addition to this default print out (the amount of which is under the control of user through various control card options), optional data capture facilities are provided which allow the user to produce his own plots or graphic displays of flow behavior at the inlets, junctions, and outlets of the network. After completion of the flow simulation for one sewer network, the program proceeds on to the next network, if any, until all such networks are processed.

At its present form, the computer program of the ISS Model cannot account for moving hydraulic jumps and surges within the sewers and only circular sewers can be considered. The latter restriction may be removed, since for sewers with noncircular cross sections, simulation can still be made by using equivalent diameters of the conduits based on best fit of hydraulic radius or cross sectional area.

The computer program of the ISS Model consists of around 3,000 statements written in PL/1-source code, as well as an assembler language subroutine. The PL/1 portion includes several short external subroutines, a main controlling section with network control card input routines, and a large set of numerical computation routines. The program is written to be composed of modules, whenever possible, so that most of the routines are separately compiled and then linkage edited together with the assembler language routine to form the executable load module.

The program can be adapted for execution on most large IBM 360 or 370 models running under OS/360 (preferably OS/MVT) operating system or its VS equivalent. A memory storage of 300K bytes is desirable although under certain conditions 100K bytes may be acceptable. The machine must have the floating point instruction set. In implementation, the program resides on a direct-access storage device, such as a disk, a drum, or a data cell drive. This enables the program to be loaded quickly and efficiently into the main storage without entailing the added overhead of recompilation each time the program is executed.

Despite the sophistications of the theory and programming techniques of the ISS Model, the computer program was written for easy adoption by users having only elementary computer knowledge. Programming experiences in PL/1 language will be helpful but not essential for the implementation of the program. The program can be implemented through the use of either a distribution tape or the program listing. The former approach is recommended, particularly for those users who are not technically oriented on computer operations. Prospective users may obtain the distribution tapes at cost of the magnetic tape, handling, postage, and computer time for duplication.

Standard distribution tapes are provided on 1600 bytes per inch, 2400 ft magnetic tapes. When requesting for a distribution tape the following information should be provided:

(a) Model of computer system, e.g., IBM System 370, model 168 MP.

(b) Operating system in use, e.g., OS/MVT with HASP.

(c) Region size available for the program, e.g., 220K.

(d) Type of disk or other direct-access device to which the program will be transferred from the distribution tape, e.g., 2314, 2319, 3330.

(e) Compilers available at the installation, e.g., either, neither or both of the OS PL/1 (F) and PL/1 Optimizing compilers.

However, users who prefer to use the program listing for implementation or modification should have adequate understanding of the following IBM systems 360 and 370 concepts:

(a) creation, use, and modification of partitioned data sets for storage of source and object decks, as well as load modules;

(b) use of the linkage editor to produce executable load modules, and method of executing the load modules thus created;

(c) use of other IBM system utilities, such as IEHMOVE, IEBUPDTE, etc;

(d) use of tapes, disks, and other direct access devices; and

(e) use of IBM job control language (commonly referred to as JCL).

Those who need to use the program listing and yet unfamiliar with computer applications are advised to seek assistance from experienced programmers.

The listing of the computer program, input and output data format, program structures, operational procedure and the finite differences equations used for numerical solution have been reported in details by Sevuk et al. (1973) and are not repeated here for brevity. Those interested in details should refer to that report.

VIII. WATER QUALITY MODEL

As mentioned in Chapter III, Introduction, storm runoff has been known as a significant source of pollution because of its capability to wash and carry pollutants on its course of flow. With recent stringent requirements on water quality for the control of pollution, the necessity, desirability, and economical feasibility of treatment of storm runoff is often an unavoidable concern. For combined sewer systems, the problem is further complicated by the variable quality as well as the quantity of the dry weather flow.

For pollution control purposes, ideally the time and spatial distributions of the quality of runoff should be known, at least at certain key locations like sewer outlets and overflows. This knowledge is particularly useful, for example, for a selective withdraw and treatment of storm runoff. However, such detailed information requires, from an experimental viewpoint, the time-consuming, tedious and expensive measurements and laborous analyses of the data, and from a theoretical viewpoint, the precise theories concerning diffusion and dispersion of pollutants and the chemical and thermal processes involved. Furthermore, a prerequisite for an accurate water quality analysis is a reliable quantitative prediction of water.

The water quality model introduced in this chapter is an attempt to provide a means to evaluate water quality of storm runoff as a supplement to the quantitative evaluations of the Illinois Surface Runoff Model and the ISS Model. The model is a relatively simple one by using a one-dimensional approach considering the time and spatial variations of the pollutant expressed in terms of cross-sectional averaged concentration. Only the equation of conservation of mass is used.

Implicitly this involves the assumption of instant mixing within the flow cross section. The dynamic equation expressing the gravity effect (buoyancy or settling) is not used in the model. Neither are the effects of biological and chemical reactions accounted for. Improvements on these aspects, of course, can and should be made in the future. However, this is to be done after the simple model has been adequately tested with data. At present available field measurements are mostly for limited number of locations in a drainage basin and do not provide enough data on spatial and temporal variations of storm runoff water quality to verify the model sufficiently.

VIII-1. Water Quality Model Formulation

The water quality model described here utilizes the computed discharge hydrographs and flow area-(or depth-) time relationship at desired locations of the drainage system from the surface runoff and ISS Models together with the initial pollutant distribution to evaluate the transport of pollutant by storm runoff. The output is a set of pollutographs (pollutant concentration-time graphs) at the desired locations.

The mass conservation equation of a pollutant expressed in concentration c for the flow with a discharge Q and flow cross sectional area A

$$\frac{\partial cA}{\partial t} + \frac{\partial cQ}{\partial x} = c_\ell q_\ell \qquad (69)$$

in which c_ℓ is the concentration for the lateral flow q_ℓ. The term $c_\ell q_\ell$ includes concentrated pollutant dosage in which case the value of $c_\ell q_\ell$ is simply replaced by the value of the dosage. Substitution of the continuity equation (Eq. 52) into Eq. 69 yields

$$A\frac{\partial c}{\partial t} + Q\frac{\partial c}{\partial x} = q_\ell(c_\ell - c) \qquad (70)$$

By using the computational grid shown in Fig. 18, Eq. 70 can be written in finite difference form as

$$A_C \frac{c_C - c_B}{\Delta t} + Q_C \frac{c_C - c_D}{\Delta x} = q_\ell \; (c_\ell - c) \tag{71}$$

However, because of the details of the output of the Illinois Surface Runoff Model and the ISS Model, the computational procedure for water quality for these two parts of runoff are different. For the surface runoff part, the water quality computation is programmed in the Illinois Surface Runoff Model and the quantity and quality computations are done concurrently. In the sewer system runoff part, the water quality model is programmed separately using the output from the ISS Model as the input.

(A) Water quality computation for surface runoff. - Water quality for surface runoff is computed within the Illinois Surface Runoff Model. In Eq. 71, if the computational grid points D and C represent respectively the upstream and downstream ends of a gutter or subcatchment strip, the flow parameters for D at each time level are known from the upstream boundary conditions. The values of A_C and Q_C are supplied from the water quantity computation of the model. For gutter flows, the values of c_ℓ and q_ℓ are known from the subcatchment runoff. For subcatchment flows, the values of c_ℓ and q_ℓ are known from rainfall (usually with $c_\ell = 0$) and other known lateral flows, if any. Hence, from Eq. 71 the concentration c_C can be solved explicitly for each time level as

$$c_C = \frac{q_\ell c_\ell + \frac{c_B A_C}{\Delta t} + \frac{c_D Q_C}{\Delta x}}{\frac{A_C}{\Delta t} + \frac{Q_C}{\Delta x} + q_\ell} \tag{72}$$

(B) Water quality model for sewer system runoff. - In view of water quality routing, a sewer system can be considered to consist of two different types of elements; namely, the sewer conduits and the junctions with substantial storage capacities. For a sewer with no lateral flow, Eq. 71 yields for the downstream end C

$$\left(\frac{A_C}{\Delta t} + \frac{Q_C}{\Delta x}\right) c_C - \frac{Q_C}{\Delta x} c_D = \frac{c_B A_C}{\Delta t} \tag{73}$$

and for the upstream end

$$\frac{Q_D}{\Delta x} c_C + \left(\frac{A_D}{\Delta t} - \frac{Q_D}{\Delta x}\right) c_D = \frac{c_A A_D}{\Delta t} \tag{74}$$

With the discharge and flow cross-sectional area given from the output of the ISS Model, and c_B and c_A known from the initial condition or previous time computations, Eqs. 73 and 74 can be solved for c_C and c_D at each time level using Cramer's rule. This procedure provides the pollutographs at both ends of sewers which can also be used for water quality computation at sewer junctions.

For a sewer junction, conservation of mass of the pollutant gives

$$\frac{d(cs)}{dt} = \sum_i c_i Q_i + \sum_j c_j Q_j \tag{75}$$

in which c is the pollutant concentration for the volume of water s in the junction; c_i is the concentration of discharge Q_i from the i-th sewer into the junction, Q being positive for inflow and negative for outflow; and c_j is the concentration for the j-th direct inflow Q_j. Writing Eq. 75 in finite difference form and solving for the concentration at the present time level one obtains

$$c_p = \frac{1}{h_p} \left[c_o h_o + \frac{\Delta t}{A} \left(\sum_i c_i Q_i + \sum_j c_j Q_j\right)\right] \tag{76}$$

in which A is the constant horizontal cross sectional area of the junction being considered; h is the depth of water in the junction; and the subscripts (p) and (o) denote respectively the present and previous time levels.

VIII-2. Program Description and Data Preparation

As mentioned in the preceding section, the water quality computations for the surface runoff and sewer system are handled separately. The computation for the surface runoff quality is done as a part of the surface runoff model whereas that for the sewer is done through a program supplement to the ISS Model. Since the Illinois Surface Runoff Model Program has been described in Sec. VI-4, only the quality part of the program is described here.

(A) Program description and data preparation for surface runoff quality computation. - Because the discharges and flow cross-sectional areas of the gutter and subcatchment flows required by Eq. 72 for quality computation are computed in the surface runoff model but not provided as a part of the surface runoff output, it is more advantageous to integrate the quality computation into the surface runoff program than to separate it. The computer program for the surface runoff allows the consideration of two pollutants at a time. The quality of subcatchment flow is computed in subroutine OVLFLO using Eq. 71. At the upstream end of a subcatchment strip, the concentration of each pollutant is assumed to remain the same as the initial concentration. Knowing the flow from the subcatchments, the water quality computations for gutter flow are made in the MAIN program. In Eq. 71, the total lateral flow into a gutter from all the components (subcatchments, direct rainfall etc.) is computed at every time level and averaged over the length of the gutter. The concentration in lateral inflow is evaluated as the average of component flow concentrations weighed with respect to flow rate. The upstream boundary condition for a gutter is computed in subroutine UPSBO. The concentration of each pollutant at the upstream end of the gutter is evaluated at each time level as the average of concentrations weighed over the flow rate of the carry-overs from the immediately upstream inlets. When there is no upstream inlets, the concentration at the upstream end is equal to the initial value at all time levels of computation. The quality of flow into an inlet is assumed to be the same as the corresponding gutter outflow. When there are more than one gutter inlets discharging into the same catch basin or sewer node, an average pollutograph for each pollutant weighed with respect to the inlet flow rates is computed for the sewer node in subroutine SWRJNT.

The data required for the quality part of the surface runoff model consists of the initial concentration of each pollutant in subcatchment strips and gutters. The input format and the preparation of data cards has been described in Sec. VI-4. The output from the computer program includes the ordinates of direct inflow hydrographs and the corresponding pollutographs for all the sewer nodes printed out at equal time intervals.

(B) Description of sewer system water quality model. - The sewer system water quality model is programmed in Fortran IV language as a supplement to the ISS Model. The input to the computer program includes the data on the depth and discharge at the entrance and exit of each sewer and the volume of water at each storage junction at given times as provided by the output of the ISS Model. In addition, the pollutographs as obtained from the surface runoff quality computation, and the direct inflow hydrographs, if any, and the sewer system layout are also input into the program for runoff quality routing in the sewer system, using Eqs. 73, 74, and 76. The output from the computer program consists of the pollutographs representing the time variation of pollutant concentration at the sewer system outlet, at the storage junctions, and at the entrance and exit of all sewers.

The computer program of the Illinois Sewer System Water Quality Model allows the consideration of two different pollutants at a time. The sewer system may consist of as many as 100 sewers. Arbitrary identification numbers can be assigned to the sewer nodes (junctions and manholes). Because the backwater effect is already accounted for in the quantity computation, the sewer system is not restricted to tree-type networks. The computer program consists of approximately 200 statements and the storage requirement is 300 K. When a sewer system consisting of more than 100 sewers is to be considered, the program should be modified by simply changing the DIMENSION statements. This may cause an increase in storage requirement. The computer program is listed in Appendix C and the computational logic is shown schematically in Fig. 25.

(C) Data preparation for sewer system water quality model computer program. - The data deck for the sewer system water quality model consists of three sets of cards. The first set is a simple card describing the general information on the sewer system and input data. The order and format of this information are as follows: The total number of sewers in the system (I5); total number of reservoir-type junctions in the system (I5); number of points used to describe each of the discharge and stage hydrographs at the entrance and exit of each sewer (I5); number of points used to describe direct inflow hydrographs at each of the sewer nodes (I5); identification number of the

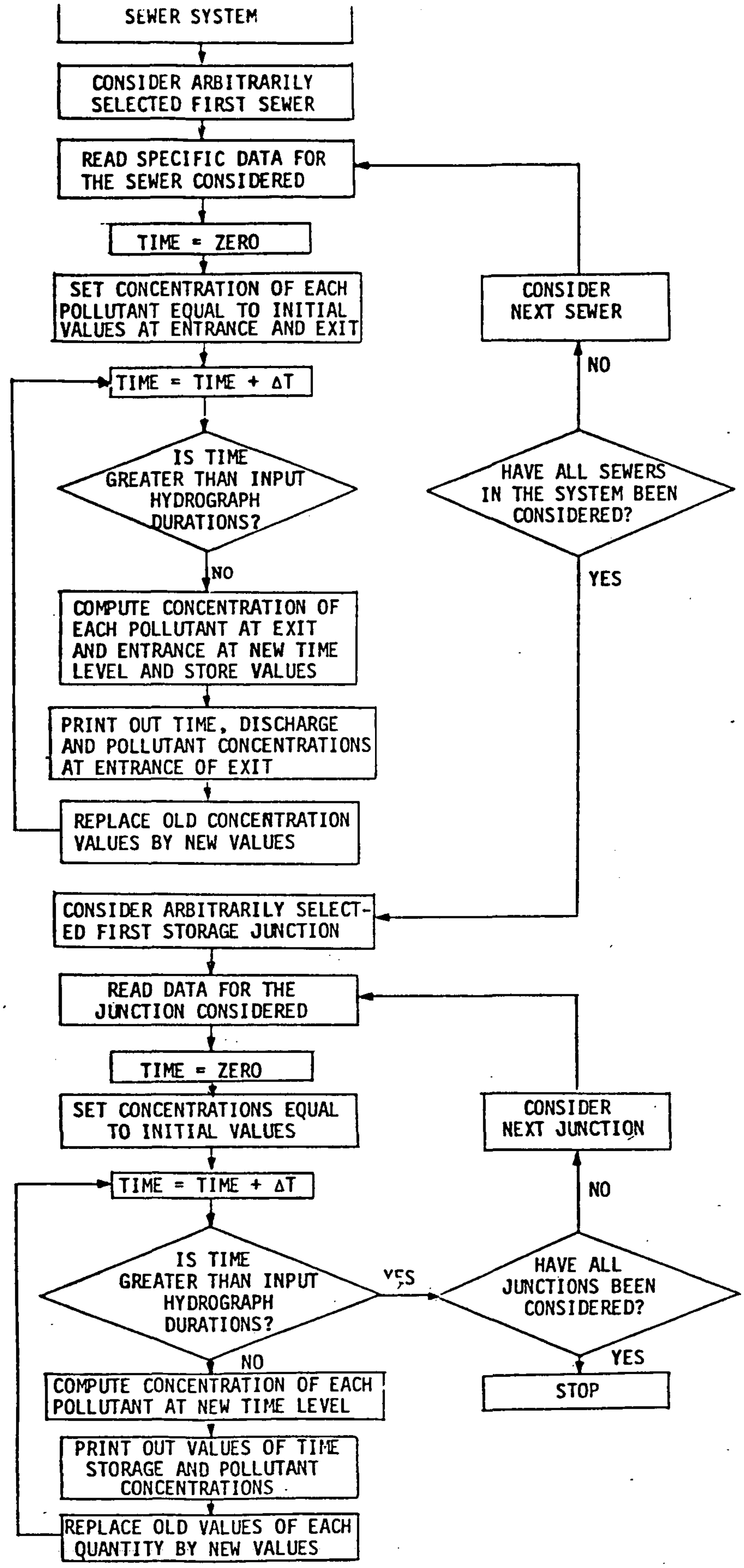


Fig. 25. Flow chart for Illinois sewer system water quality model computer program

sewer node representing the system outlet (I5); and the time interval at which ordinates of the direct inflow hydrographs at the sewer nodes are provided as input (F5.0).

The second set of cards are for sewer data. This set consists of as many subsets as the total number of sewers, each subset corresponding to a sewer. The order of these subsets is arbitrary within the set. Each subset contains the following information: (a) The first card in each subset provides the description of the corresponding sewer: The identification number of the junction node at the upstream end of the sewer (I5); the identification number of the junction node at the downstream end of the sewer (I5); length of the sewer (F5.0): diameter of the sewer (F5.0); initial concentration of the first pollutant at the upstream end (F5.0) and at the downstream end (F5.0); initial concentration of the second pollutant at the upstream end (F5.0) and at the downstream end (F5.0). (b) The data cards following the first one in each subset contain the ordinates of the storage and flow hydrographs at the entrance and exit of the sewer. These values need not be provided at equal time intervals because the output from the ISS Model on the depth and discharge at the entrance and exit of each of the sewers are at irregular time intervals.* Each card contains three groups of data punched in sequence and each group consists of the time (F6.0), discharge at the entrance (F5.0), flow depth at the entrance (F5.0), discharge at the exit (F5.0), and flow depth at the exit (F5.0).

The third set of cards are for the data for reservoir-type junctions, containing as many subsets as the total number of

*However, the direct inflow into the sewer junctions and the water stage in the junctions can be obtained respectively from the surface runoff and sewer routing models at constant time intervals. Therefore these values are provided as input at regular time intervals.

reservoir-type junctions in the sewer system. In each subset representing a junction, (a) the first card contains in sequence the information on the identification number of the junction node (I5), cross sectional area of the junction (F5.0), and the initial concentration of the first pollutant (F5.0) and second pollutant (F5.0) in the junction; (b) the data cards following the first card of the subset contain the ordinates of direct inflow hydrographs into the junction, the ordinates of the corresponding pollutographs and the water stage of the junction. All values should be given at equal time intervals.* Each card contains three groups of data punched in sequence and each group consists of: time (F6.0), rate of direct inflow (F4.0), concentration of first pollutant of the direct inflow (F4.0), that of the second pollutant (F4.0); and the water stage in the junction (F4.0).

The output of the sewer quality model includes the time, discharge, and concentration of the two pollutants at the sewer outlet and at the entrance and exit of all the sewers, printed out at equal time intervals under the appropriate headings, as well as the storage and concentration of the two pollutants in each of the reservoir-type junctions of the system at the same constant time intervals.

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*See footnote in the preceding page.

IX. GENERAL DESCRIPTION OF OTHER METHODS EVALUATED

Numerous rainfall-runoff "models" have been proposed by previous investigators. Most of these models were developed for rural areas. These include the Burkli-Ziegler and other similar formulas (Chow, 1962), the monograph methods such as the methods of ARS-SCS (U.S. Soil Conservation Service, 1971), BPR (Potter, 1961), California (1953), Chow (1962), and Cook (Hamilton and Jepson, 1940), and many of the hydrograph methods.

Among those models applicable to urban areas, some are strictly for overland runoff prediction. These include Izzard's (1946) and Horton's (1938) methods. The models which consider both overland and sewer flows can be divided into two groups. The lumped system group, including the rational method and the unit hydrograph method, treats the drainage basin as a black box producing output (basin runoff) from given input without considering what is happening to the flowing water within the basin. The distributed system approach which includes most other urban runoff models, routes the rainfall excess through the overland surface and sewers to produce the runoff hydrographs.

It would be tedious to list and costly to compare in this study all the methods applicable to urban drainage systems. Therefore, only eight methods which are either most well known and widely adopted or having great application potentials are evaluated in this report. The Illinois Urban Storm Runoff method is described in Chapters VI and VII. The other seven methods are briefly described in this chapter, following roughly their relative order-of-complexity. Major features of these eight methods are listed in Table 6. Presumably the most authoritive description of the procedures in using a method is that by the original method developer. Therefore, the steps for application of these methods are not repeated here.

Table 6. URBAN RUNOFF PREDICTION METHODS EVALUATED

Model	Input data			Runoff routing			Output results	Selected references
	Rainfall	Abstractions	Basin properties	Overland	Gutters	Sewers		
Rational	Average intensity over the duration	Accounted by runoff coefficient	Basin size	——	——	——	Peak discharge	Chow, 1962; ASCE and WPCF, 1969
Unit hydrograph	Hyetograph	Infiltration by ϕ index or Horton's formula; other abstractions, if accountable	Base flow	——	——	——	Basin runoff hydrograph	Chow, 1964
Chicago hydrograph	Hyetograph	Infiltration by Horton's formula and depression storage by an exponential function	Overland surfaces; lengths, slope, cross-sectional dimensions and roughness of gutters and sewers	Izzard's method	Linear kinematic wave storage routing with Manning's formula	Linear kinematic wave storage routing with Manning's formula or time offset method	Basin runoff hydrograph	Tholin and Keifer, 1960
RRL	Hyetograph	Pervious areas produce no runoff and all rainfall on impervious areas becomes runoff	Areas of directly contributing impervious surfaces; time of travel of impervious areas	Flow time - area method		Reservoir routing, lagged by time of travel in sewers	Basin runoff hydrograph	Watkins, 1962; Terstriep and Stall, 1969
UCUR	Hyetograph	Infiltration from rainfall only by Horton's formula and depression storage by an exponential function	Length, slope, and n for overland surfaces; length of gutters; diameter, slope and n of sewers	Manning's formula and empirical detention storage function	Continuity equation of steady spatially varied flow	No routing, lagged by time of travel in sewers	Runoff hydrograph	Papadakis and Preul, 1972; Univ. of Cincinnati, 1970

Table 6. (continued)

Model	Input data			Runoff routing			Output results	Selected references
	Rainfall	Abstractions	Basin properties	Overland	Gutters	Sewers		
SWMM	Hyetographs, allows areal variation	Infiltration by Horton's formula and depression storage values	Overland surface length, width, roughness n, slope and percent imperviousness; length, slope, cross-sectional dimensions and roughness of gutters and sewers	Manning's formula with uniform depth	Linear kinematic wave model, storage routing with Manning's formula and continuity equation	Improved nonlinear kinematic wave model	Hydrographs of runoff quantity and quality, also depth of flow	Metcalf & Eddy, Inc et al., 1971 Heaney et al., 1973
Dorsch	Hyetographs	Infiltration by Horton's formula and depression storage	Overland surface and gutter length, slope, and roughness; sewer size, length, slope and roughness	Kinematic wave model	Kinematic wave model	St. Venant eqs. with partial backwater effects	Runoff hydrographs and depth	Klym et al., 1972
Illinois	Hyetographs, allows areal variation	Infiltration by Horton's formula and initial detention storage	Length, width, slope, and roughness of overland surface elements; length, roughness, cross-sectional dimensions and slope of gutters; type and dimensions of inlets; length, slope, roughness and diameter of sewers, size of manholes and junctions	Nonlinear kinematic wave with Darcy-Weisbach's eq.	Nonlinear kinematic wave with Manning's formula	St. Venant eqs. with back water effects	Runoff hydrographs, also depth and velocity	Sevuk et al., 1973

Those who want to use the particular methods should refer to the original reports for detailed procedures.

IX-1. The Rational Method

The rational method is the oldest, simplest, and most widely adopted method for storm runoff estimation (Chow, 1962). In the rational method, the peak rate of storm runoff, Q_p, is estimated as

$$Q_p = CiA \tag{91}$$

in which C is a dimensionless runoff coefficient; i is the rainfall intensity and A is the size of the drainage area. The infiltration and other abstractions from the rainfall is implicitly accounted for by the runoff coefficient. The value of i is equal to the average rainfall intensity over a duration equal to the so-called time of concentration. Details of application of the rational method to urban storm sewer design can be found elsewhere (e.g. ASCE and WPCF, 1969; Yen et al., 1974).

The drawbacks of the rational method have been discussed by many investigators (e.g., see Chow, 1964; McPherson, 1969). For urban storm runoff quantity and quality control, the most serious drawback of the rational formula is that it gives only the peak discharge, Q_p, and provides no information on the time distribution of the storm runoff.

IX-2. Unit Hydrograph Method

Since Sherman (1932) proposed the concept of unit hydrograph, it has been used to study the rainfall-runoff relationship for rural as well as for urban areas. A unit hydrograph for a drainage basin is defined as the discharge-time graph (hydrograph) of a unit volume of direct runoff (usually expressed as unit depth) from the basin produced by an areally and temporally uniformly distributed effective rainfall of a specified unit duration. The unit hydrograph for a drainage basin is obtained by reduction from previous rainfall and runoff data, by synthetic means, or by transpose of the unit hydrograph from a neighboring basin of similar physical characteristics. The unit hydrographs for different durations can be obtained by direct derivation or by the S-hydrograph method. The procedures to derive the unit hydrograph for a drainage area and to apply it can be found in standard hydrology reference books (e.g., Chow, 1964).

With the unit hydrographs of different durations for a given drainage basin known, the procedure to produce runoff hydrographs for given rainstorms is as follows:

(a) The rainfall excess is first computed by subtracting abstractions from the total rainfall. For urban storm runoff studies among the different abstractions usually only infiltration is considered although other abstractions can also be included without causing much difficulty. Often the ϕ-index method of constant infiltration rate is used because of its simplicity to estimate the infiltration, although Horton's formula (Eq. 34) and other methods can also be used.

(b) Subdivide the rainfall excess obtained in (a) into a number of small rainstorms of various durations each of which has nearly uniform distribution of intensity of rainfall excess over its duration. Determine the amount (depth) of rainfall excess, duration, and the beginning time for each of these subdivided rainstorms.

(c) For each of these subdivided rainstorms, apply the unit hydrograph with a duration closest to the rainstorm's duration. The ordinates of the unit hydrograph are multiplied by the depth of the rainfall excess to give the runoff hydrograph for that component rainstorm. This procedure is repeated in sequence for all the rainfall excesses of different depths and durations occurring at different times. The resulted runoff hydrographs for all the component rainstorms are then added linearly with appropriate time shifting (to account for the different times for different rainfall excesses) to give the combined runoff hydrograph due to the rainstorm.

(d) A base flow, such as dry weather flow, is added to the combined runoff hydrograph obtained in (c) to give the total runoff hydrograph.

For urban drainage basins which are often of the size smaller than several km^2 (several sq mi), reliable unit hydrograph usually cannot be established because of lack of data. Also, for the unit hydrographs obtained based on past records to be applicable, the drainage basin characteristics must be time invariant, i.e., no significant changes of the physical properties of the basin in time. Furthermore, from the fluid mechanics viewpoint the unit hydrograph theory may not be reliable for small drainage areas of a few acres (hecters) or smaller.

IX-3. Chicago Hydrograph Method

The Chicago method is a steady-flow hydrograph routing method to determine the time distribution of the quantity of storm runoff (Tholin and Keifer, 1960). The method takes into consideration storages in gutters and sewers. For a given drainage area and rainstorm, the infiltration is computed by using Horton's formula (Eq. 34) and the surface depression storage is computed by using an empirical function (Eq. 35). The computed rainfall excess is then routed through the overland surface using a modified Izzard's method. The gutter flow is routed using a storage routing method with Manning's formula. The same routing method is applied to the sewer laterals and mains. To simplify the computation, a time-offset method to subdivide the hydrographs for sewer routing was also proposed. However, this time-offset method has no theoretical basis and is not used in the present study.

Detail procedure of the Chicago method is described in a paper by Tholin and Keifer (1960). Hand computation of the method is tedious and time consuming. The Department of Public Works of the City of Chicago has a computer program of the Chicago method.

IX-4. Road Research Laboratory Method

The British Road Research Laboratory (RRL) method is another hydrograph routing method developed specifically for urban areas (Watkins, 1962; Terstriep and Stall, 1969). The method was developed for the purpose of determination of design runoff hydrographs although it can also be used for flow prediction purpose. The method assumes that pervious areas and impervious areas not directly connected to the drainage system produce no runoff, and all the rainfall on impervious areas directly connected to the drainage system becomes runoff. A linear flow time-area concept similar to that adopted in the development of Chow's method (1962) is used to establish the hydrographs for the impervious areas. A time of entry, similar to the time of concentration for the rational method, is estimated by experience as the time required for the directly connected impervious area to contribute to the flow into the inlet catch basin. Terstriep and Stall (1969) proposed to use a formula suggested by Hicks to compute the time of entry for overland flow and Izzard's modification of Manning's formula for gutter flow. Thus, in the original version the required overland and gutter data consist only of the area of the directly connected impervious subcatchments and their time of entry, whereas for the Terstriep and Stall version the slope and length of the overland surfaces and the slope, shape, and Manning's n for the gutters are required. The inflows from different contributing areas of an inlet are combined linearly with appropriate time lag to give the inlet inflow hydrograph.

The inlet hydrographs are then routed through the sewers using a reservoir routing technique. The time of travel in a sewer is computed as $t = L/V$, where L is the length of the sewer and V is the full-pipe flow velocity computed by using the Chezy formula (Eq. 47) in Colebrook-White form with c in $m^{0.5}$/sec given by

$$c = 17.72 \log \left[\frac{\frac{k}{148000} + \frac{8.04 \times 10^{-8}}{\sqrt{RS}}}{R} \right] \tag{92}$$

where the surface roughness k is in mm and hydraulic radius R in m.

The inflow hydrograph of a sewer is the combination of the outflow hydrographs, with appropriate time lag as computed by the time of travel, of the inlets and sewers joining at the upstream end of the sewer being considered. This inflow hydrograph is then routed through the sewer by using the continuity equation

$$\frac{I_1 + I_2}{2} - \frac{Q_1 + Q_2}{2} = \frac{s_2 - s_1}{\Delta t} \tag{93}$$

in which I is the inflow rate as given by the inflow hydrograph, Q is the outflow rate of the outflow hydrograph, s is the storage in the sewer, and the subscripts 1 and 2 refer to the beginning and end of the time interval Δt, respectively. A storage-discharge relationship is then supplemented to Eq. 93 to give the outflow rate. Originally, Watkins suggested to use the recession part of recorded runoff hydrograph to establish the storage-discharge relationship. In a later version, it was suggested to approximate this relationship using Chezy's formula (Eq. 47) with c given by Eq. 92 assuming instantaneously the sewer flow is steady and uniform with a slope equal to the sewer slope.

A linear interpolation between the values of hydraulic radius and flow area was suggested to avoid time consuming iterative solution.

Obviously, the RRL method would be most successful for drainage basins of nearly equal in sizes of impervious and pervious areas and for rainstorms of moderate intensities and durations. From the theoretical viewpoint, the criticisms on the RRL method are numerous (Heeps and Mein, 1973). For instance, in considering the sewer flow, the velocity is computed using Eqs. 47 and 92 assuming a full-pipe flow, whereas in computing the sewer storage, the sewer is assumed to have unlimited storage capacity as required by Eq. 93. In general, the method does not take into account the actual physics of storm water flow on the land surfaces and in sewers. The method should be considered as empirical rather than theoretical.

IX-5. Cincinnati Urban Runoff Model

The University of Cincinnati Urban Runoff (UCUR) Model is basically a hydrograph routing model developed under a grant from the U.S. Environmental Protection Agency (University of Cincinnati, 1970; Papadakis and Preul, 1972). The model has many similarities to the Chicago method and hydraulically it is essentially a linear kinematic-wave model. As in the Chicago method, infiltration is subtracted from rainfall using Horton's formula (Eq. 34) (Eq. 34) with the aid of Jens' (1948) curves, and the depression storage by using the same exponential function (Eq. 35) as in the Chicago method. No infiltration is allowed from water stored on the land surface. Interception and evapotranspiration are neglected. The resulted hyetograph of rainfall excess can then be used for routing. The basin is subdivided into a number of subcatchments each having homogeneous infiltration characteristics. At any instant the rainfall is assumed uniformly distributed over the entire basin but the abstractions are different for different subcatchments.

In routing the storm water, the overland flow is computed by using Manning's formula (Eq. 46) coupled with an empirical detention storage function. Assuming uniform flow with depth equal to the hydraulic radius, Manning's formula combined with the continuity equation gives a relationship for detention storage per unit width of the overland strip under equilibrium condition, d_e, as

$$d_e = C_e \frac{(i_e nL)^{0.6} L}{S^{0.3}} \tag{94}$$

in which i_e is the overland flow supply rate which is equal to the intensity of rainfall excess; n is the Manning roughness factor; L and S are the length and slope of the overland strip, respectively; and C_e is a dimensional coefficient, equal to 0.625 sec/m$^{0.3}$ in SI system with length in m and time in sec, and equal to 0.437 sec/ft$^{0.3}$ in English system or 0.00073 if i_e is in in./hr. The empirical detention storage function to be used together with Eq. 46 to solve for the overland flow hydrograph is

$$h = d + 0.6d \left(\frac{d}{d_e}\right)^{0.3} \tag{95}$$

in which d is the detention storage and h is the flow depth at the exit of the overland strip. During recession the ratio d/d_e is assumed to be unity. It should be noted here that in UCUR method the overland flow depth does not include the depth of water for depression storage.

The gutter outflow is assumed equal to the sum of the upstream inflow, Q_u, and lateral inflow from overland at the same time increment; i.e.,

$$Q = Q_u + q_\ell L \tag{96}$$

in which q_ℓ is the overland flow supply per unit length of gutter and L is the gutter length. No time lag of gutter flow is considered. Thus, despite the required detailed data for overland surfaces (length, slope, and Manning's n of the overland surfaces), the required gutter data is rather simple: only the length of the gutter. Neither the slope nor the cross section of the gutter is needed. In Eq. 96 it is implicitly assumed that the overland surface connected to the gutter is approximately rectangular in shape. Because the effects of the slope, shape, and surface roughness of the gutter are not accounted for, it can be expected that the UCUR method is not reliable when the gutter storage is important, such as the cases with small gutter slope and street cross slope and with heavy rainstorms.

For sewer flows, no pipe storage is considered. The hydrographs are simply lagged, without changing shape, by an average flow velocity, V_{av}, which is the weighed velocity of the flow velocity of the sewers computed by using Manning's formula; i.e.,

$$V_{av} = \frac{\Sigma V_j Q_j}{\Sigma Q_j} \tag{97}$$

The procedure has been programmed for digital computer solutions. The method is rather tedious and complicated in view of the assumptions made to simplify the hydraulic routing of storm runoffs and the accuracy that the method can provide. Heeps and Mein (1973) modified the program to reduce the computer time and to allow defined sewer networks. They used Newton-Raphson's iteration instead of trial-and-error to solve the overland and sewer flow equations and omitted the weighed average velocity computation (Eq. 97). Their modifications substantially improve the applicability of the method.

IX-6. EPA Storm Water Management Model

The Storm Water Management Model (SWMM) was first developed jointly by Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc. (1971) under the sponsorship of the U.S. Environmental Protection Agency. Subsequent separate modifications and improvement of SWMM by the University of Florida researchers and Water Resources Engineers resulted in the EPA SWMM version and the WRE SWMM version, respectively. The version compared in this report is the nonproprietary EPA SWMM (Heaney et al., 1973).

SWMM is a comprehensive urban storm water quantity and quality runoff prediction and management simulation model. The surface runoff is routed by using a linear kinematic approximation and the sewer routing is an improved nonlinear kinematic wave model incorporating some features of the nonlinear quasi-steady dynamic-wave model. Because SWMM is probably the best documented one among the recently developed models, details of the EPA SWMM can easily be found in reports by Metcalf & Eddy, Inc. et al. (1971) and Heaney et al. (1973).

In the five methods previously described in this chapter, at a given time only one hyetograph is permitted for a drainage basin. In other words, the rainfall is assumed uniformly distributed over the entire basin and no areal variation is permitted. In SWMM several hyetographs applied to different subcatchments at any given instant can be specified. Like other methods, interception and evapotranspiration are neglected.

Infiltration is accounted for by using Horton's formula (Eq. 34). Different degrees of permeability and different infiltration parameters for Horton's formula may be applied to different subcatchments. Infiltration is allowed for water stored on the surface in addition to rainfall. Overland flow is assumed not to occur until depression storage is filled. Also, if not specified, by default one-quarter of the impervious surface area is assumed to be of zero depression storage.

Overland flow is assumed to be one-dimensional with its depth being constant along its length at any given instant. A quasi-steady flow routing is applied to the overland flow using the continuity equation (Eq. 93) and Manning's formula (Eq. 46) with depth equal to hydraulic radius. Within each time interval the flow is assumed steady and uniform. The subcatchment data required include the length, width, slope, surface roughness, and infiltration parameters and percent imperviousness.

The gutter flow is routed using the quasi-steady storage routing approach with the storage continuity equation and Manning's formula. Each gutter is assumed to be fed uniformly along its length by the lateral flow from overland surface.

The sewer routing is by a modified nonlinear kinematic-wave scheme. The continuity equation (Eq. 42) and Manning's formula (Eq. 46) are used with the slope assumed equal to friction slope, and the flow is assumed to be steady within each time interval. The sewer can be of any cross sectional shape. The continuity equation is put into a finite difference form as follows

$$\frac{(1-W_t)(A_{u2}-A_{u1}) + W_t(A_{d2}-A_{d1})}{\Delta t} + \frac{(1-W_x)(Q_{d1}-Q_{u1}) + W_x(Q_{d2}-Q_{u2})}{L} = 0 \qquad (98)$$

in which Q is the discharge; A is the flow cross sectional area; L is the sewer length; the subscripts u and d denote respectively the upstream and downstream conditions; and the subscripts 1 and 2 represent respectively the conditions at the beginning and end of the time interval Δt. The time derivative is weighted W_t at the downstream station and the spatial derivative is weighted W_x at the end of Δt. The backwater effect is considered by including the convective acceleration term in the equation of motion and the friction slope is computed by

$$S_f = S_o - \frac{h_{ul} - h_{dl}}{L} + \frac{V_{ul}^2 - V_{dl}^2}{2gL} \tag{99}$$

To simplify the computations for various conditions, these equations are normalized using values of flow rate and area for conditions of the conduit flowing full.

If the flow is supercritical, no routing is attempted and the sewer outflow is assumed equal to its inflow. If the backwater effect is expected to be small and the sewer is circular in cross section, the gutter flow routing method can be used as approximation to the nonlinear kinematic wave routing.

IX-7. Dorsch Hydrograph-Volume-Method

The Dorsch Hydrograph-Volume-Method (Klym et al., 1972 relatively unknown in the U.S. until recently. It is a flow prediction model and in most aspects it is more sophisticated than any one of the preceding six models briefly described in this chapter. It takes hyetograph as its rainfall input and a statistical method can be adopted for determination of frequent urban rainstorms. Interception and evapotranspiration are neglected and infiltration is computed by using Horton's formula (Eq. 34), allowing different parameters for different subcatchments.

The surface flow is routed by a kinematic-wave scheme (Eqs. 42 and 44) similar to the Illinois surface runoff model but Manning's formula (Eq. 46) instead of Darcy-Weisbach's formula (Eq. 45) is used to give the friction slope. No consideration is given on changes of n for shallow depth and due to rainfall. Sewer flows are routed by using the St. Venant equations (Eqs. 42 and 43) with Manning's formula (Eq. 46) to give the friction slope. Backwater effect of junctions are partially accounted for. The large number of the sewer and junction flow equations are solved simultaneously using an implicit finite difference scheme. Because the Dorsch HVM is a proprietary model, users should refer to the model developer for procedural details. Because of its relative sophistication and presumably better accuracy, the data requirements for the Dorsch method are considerably more extensive than those for the preceding six models.

X. EVALUATION OF MODELS

The seven urban storm runoff simulation models described in Chapter IX and the Illinois Urban Storm Runoff Model are compared using four rainstorms on the Chicago Oakdale Avenue Drainage Basin. Based on the measured hyetographs the runoff hydrographs are predicted by using the eight simulation models (the rational method gives only the peak discharge) and the results are compared with the measured hydrographs. Such an evaluation using only a few measured rainstorms of course is limited in scope and one should be cautioned on the generalization of the conclusions.

Theoretically, it would be desirable also to compare the different models on drainage basins having their sizes an order-of-magnitude bigger than that of the Oakdale Basin. However, in view of the requirements of data details and accuracy for the more sophisticated models and the computer time required for large basins, comparison using many rainstorms on large basins would be extremely costly and time consuming. For applications some errors on the data and results may be acceptable, whereas for the purpose of model evaluation and comparison excessive errors are intolerable. Furthermore, from the hydrodynamic viewpoint it is more desirable to evaluate the models with smaller basin size with sufficient and accurate details than to larger basins, because for the larger basins the local effects tend to be averaged out and not as clearly reflected in the basin outflow hydrographs as for the smaller basins. For the Oakdale Avenue Basin, because of its small size and proximity, the writers were able to check in detail the field situation needed for the more complex models within the time and budget allowance. With the presently available data, comparison using a larger basin would not provide additional new conclusions. In fact, as will be discussed later, the testing on the Oakdale Basin suggests that a testing of the surface runoff part of the models on a smaller area with accurate data is highly desirable.

X-1. Hydrographs for the Eight Methods

The four rainstorms together with their respective measured runoff hydrographs for the Oakdale Avenue Drainage Basin in Chicago selected for the comparison study of the eight methods are the rainstorms of May 19, 1959, July 2, 1960, April 29, 1963 and July 7, 1964. These four rainstorms are chosen based on the following reasons: (a) The measurements of rainfall and runoff are supposedly reliable as recommended by Tucker (1968). (b) The rainfall is relatively heavy and hence presumably the measurements are relatively accurate. (c) The rainstorms have been used by previous investigators for establishment of their methods or for testing. The measured hyetographs and hydrographs are taken from a report by Tucker (1968) and reproduced in Figs. 26 to 29. The physiographic characteristics of the Oakdale Drainage Basin have been described in Chapter V and supplementary information can be found elsewhere (e.g. Tucker 1968).

(A) Rational method. - The computation for the rational method is shown in Table 7. The time of concentration, t_c, for the 0.052 km^2 (12.9 ac) drainage basin is 23 min, determined using the monograph by Kerby (1959) for the flow on the subcatchment draining into the sewer Junction 117 (Fig. 13) plus the sewer flow time from Junction 117 to the basin outlet which is computed by using Manning's formula assuming barely filled gravity pipe flow. The actual time of concentration is probably shorter as indicated by the recorded hyetographs and hydrographs. Therefore, a time of concentration of 20 min is adopted here. The rainfall intensity i used in the rational formula is the average intensity of the recorded rainfall over a duration equal to t_c. The runoff coefficient C used is 0.60 which is the value one would adopt from standard tables (Chow, 1962, 1964; ASCE and WPCF, 1969) corresponding to the surface condition of the Oakdale Basin. As customarily done for the rational method, no abstraction is made from the rainfall and no adjustment of the C

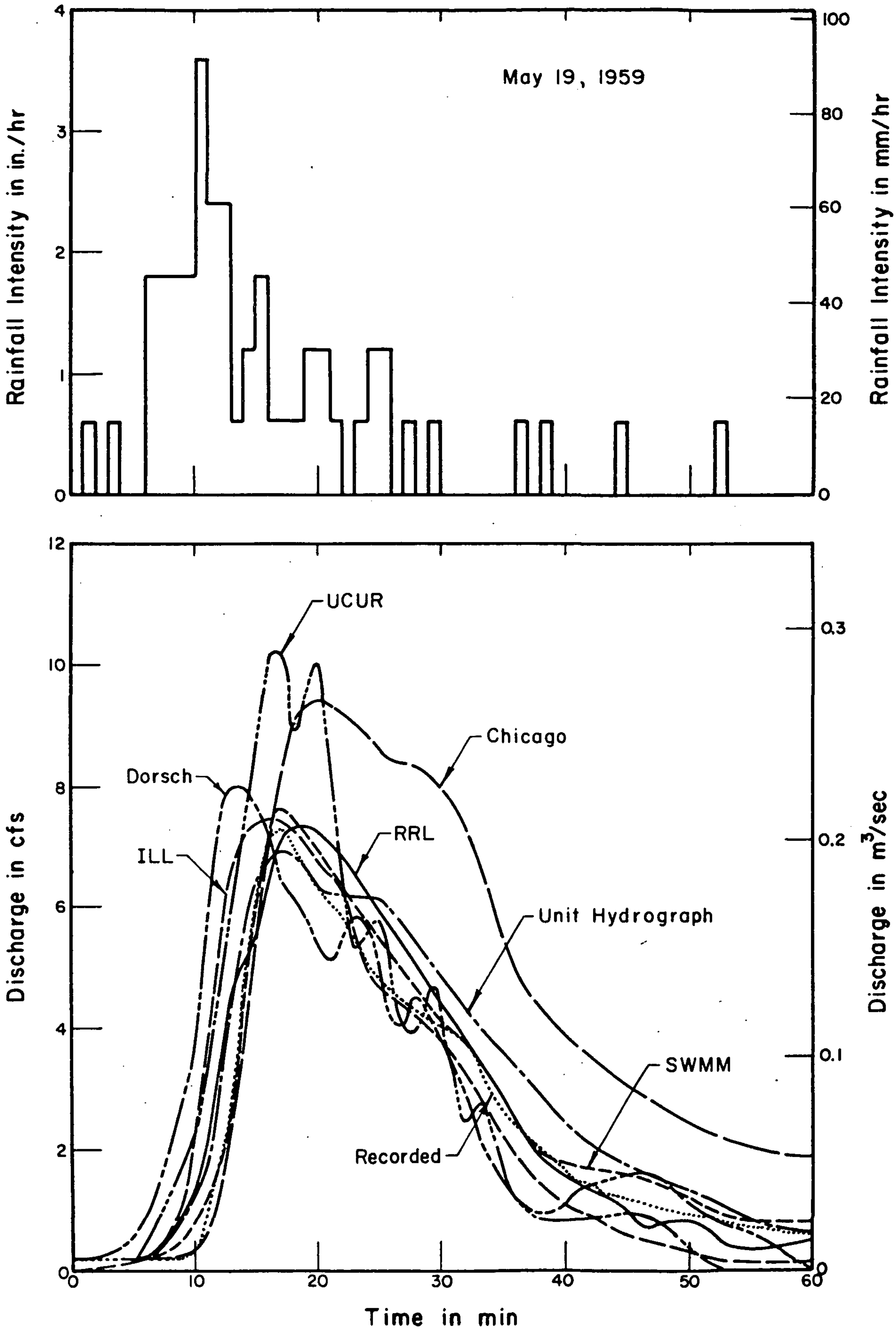

Fig. 26. Hyetograph and hydrographs for May 19, 1959 rainstorm

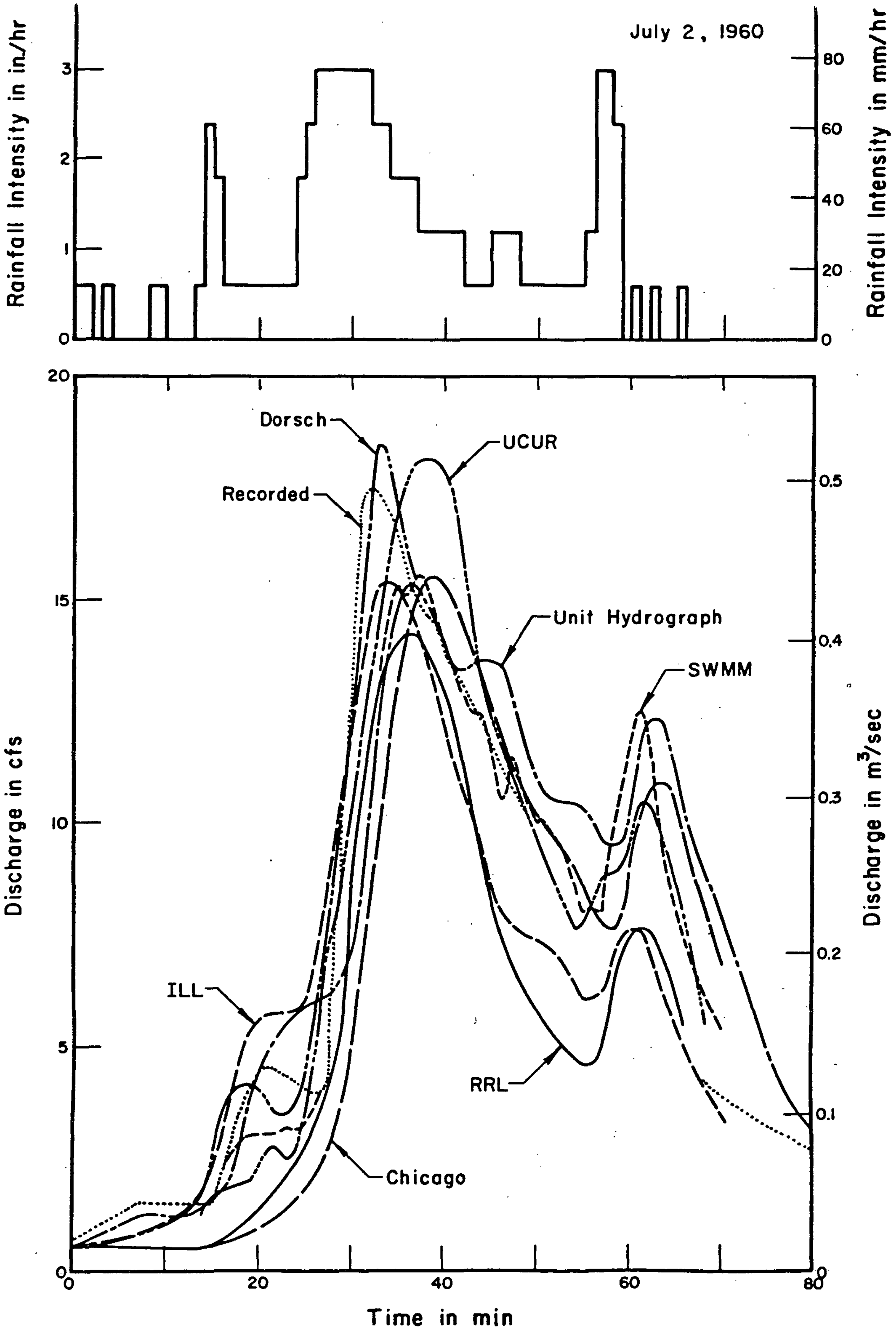

Fig. 27. Hyetograph and hydrographs for July 2, 1960 rainstorm

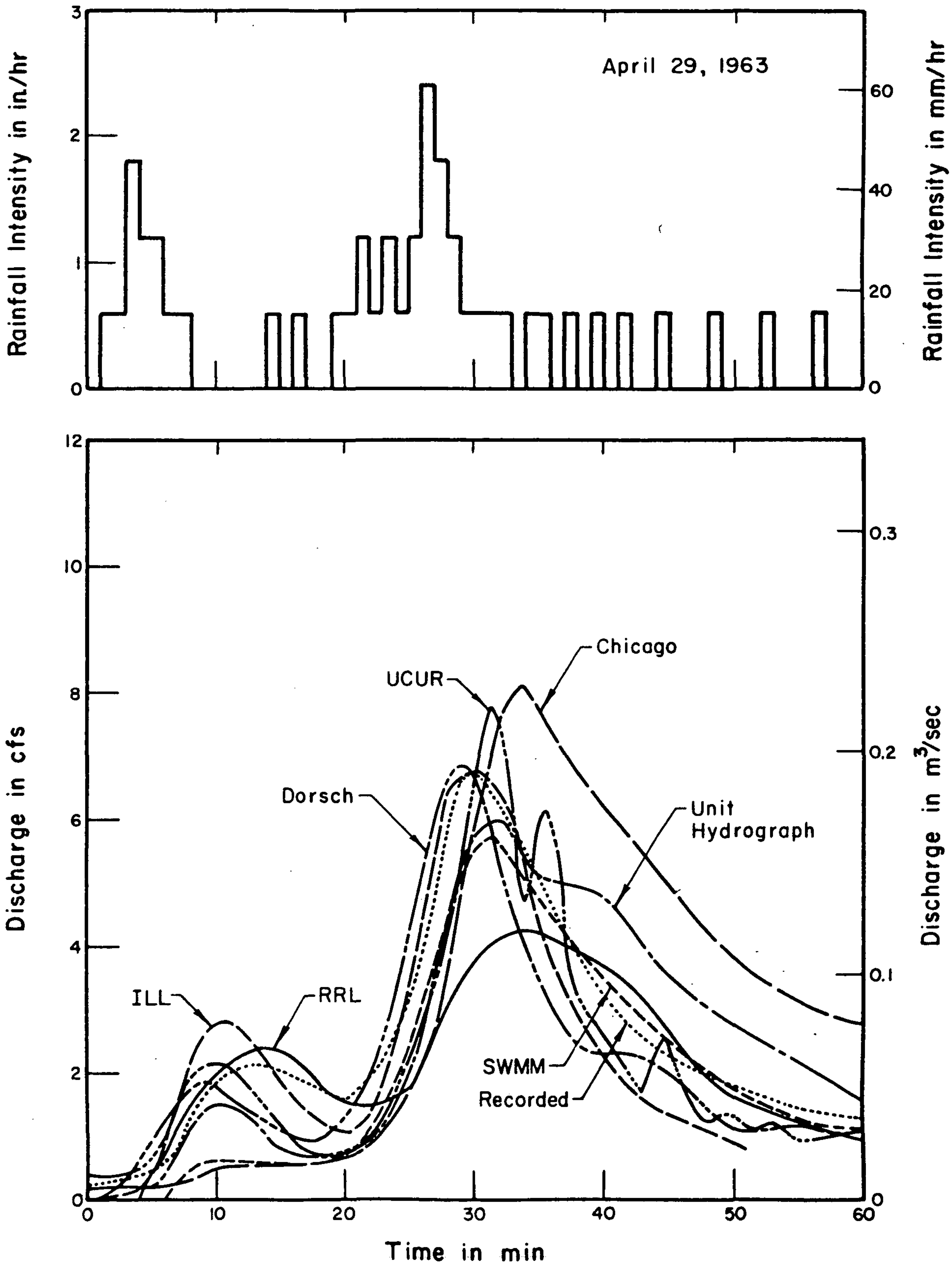

Fig. 28. Hyetograph and hydrographs for April 29, 1963 rainstorm

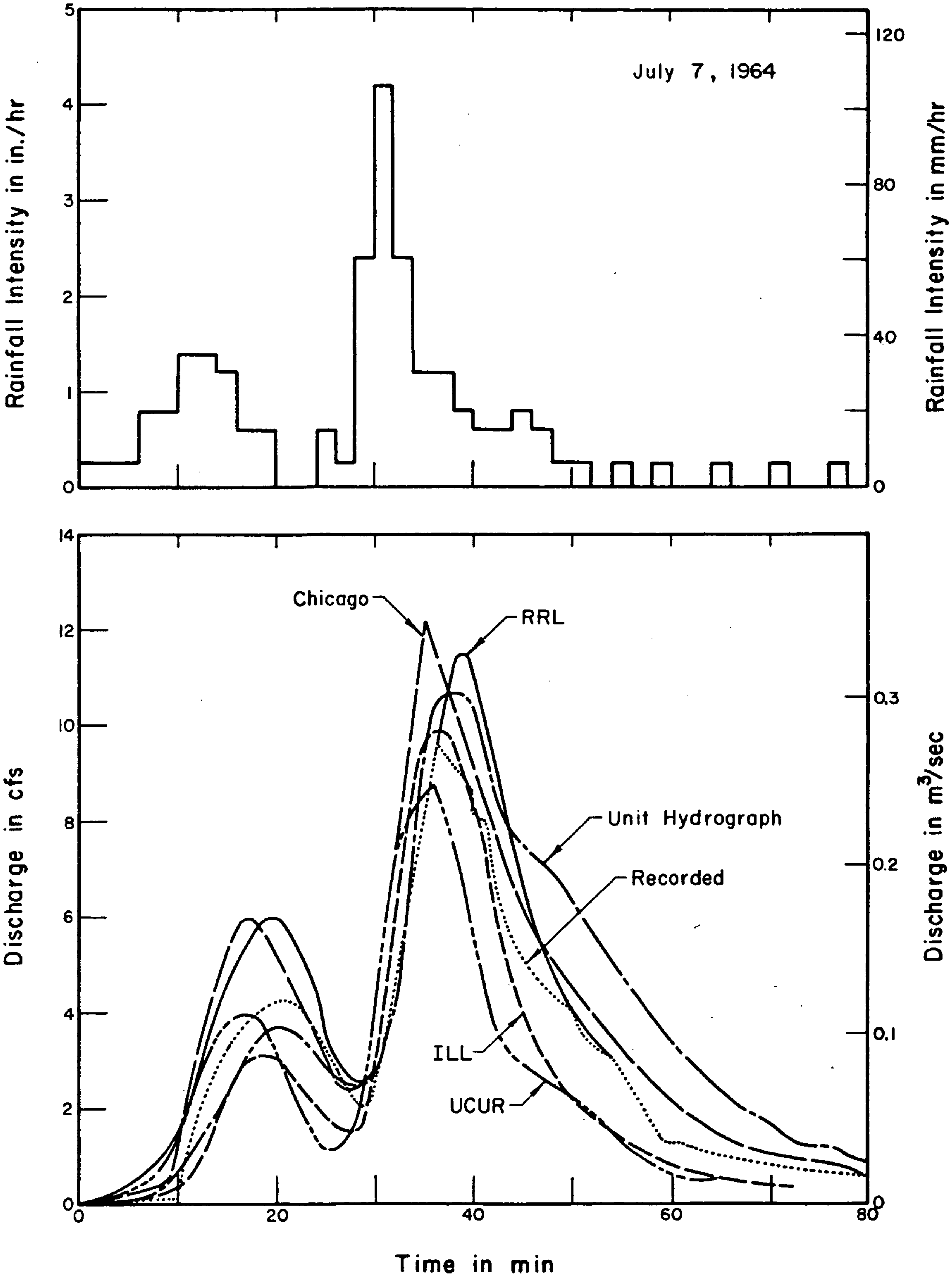

Fig. 29. Hyetograph and hydrographs for July 7, 1964 rainstorm

Table 7. RATIONAL METHOD COMPUTATION

Area = 12.9 ac or 0.052 km^2, Runoff Coefficient C = 0.60

Rainstorm		May 19, 1959	July 2, 1960	April 29, 1963	July 7, 1964
Period in min for i, from		0	15	13	18
to		20	35	33	38
Duration, min		20	20	20	20
i	in./hr	1.17	1.77	0.75	1.29
	mm/hr	29.7	44.9	19.0	32.8
Q_p	cfs	9.1	13.7	5.8	10.0
	m^3/sec	0.26	0.39	0.16	0.28

Table 8. COMPARISON OF URBAN STORM RUNOFF METHODS

Method \ Rainstorm	May 19, 1959			July 2, 1960			April 29, 1963			July 7, 1964		
	Q_p		t_p	Q_p		t_p	Q_p		t_p	Q_p		t_p
	cfs	$\frac{m^3}{sec}$	min	cfs	$\frac{m^3}{sec}$	min	cfs	$\frac{m^3}{sec}$	min	cfs	$\frac{m^3}{sec}$	min
Rational	9.1	0.26	---	13.7	0.39	---	5.8	0.16	---	10.0	0.28	---
Unit hydrograph	6.9	0.20	17.0	15.3	0.43	36.2	6.0	0.17	31.7	10.7	0.30	38.0
Chicago	9.3	0.26	20.0	15.5	0.44	38.8	8.2	0.23	34.0	12.2	0.35	35.2
RRL	7.3	0.21	18.7	14.2	0.40	36.2	4.4	0.12	33.8	11.5	0.33	38.8
UCUR	10.2	0.29	16.5	18.2	0.52	38.0	7.8	0.22	31.5	8.8	0.25	36.0
EPA SWMM	7.6	0.22	17.0	15.6	0.44	37.2	5.7	0.16	31.5	---	---	---
DORSCH HVM	8.0	0.23	13.5	18.5	0.53	32.0	6.8	0.19	29.0	---	---	---
Illinois	7.4	0.21	16.0	15.4	0.44	33.9	6.7	0.19	30.3	9.9	0.28	36.6
Recorded	7.2	0.20	16.9	17.5	0.49	32.2	6.7	0.19	30.0	9.6	0.27	36.5

value is made to account for the preceding rainfall or antecedent surface wetting conditions. As shown in Table 8, the computed peak discharges by the rational method can be considerably different from the measured values.

(B) Unit hydrograph method. - As discussed in the preceding chapter, only in rare cases that sufficient data are available for urban drainage basins to establish their respective unit hydrographs. Fortunately, the Oakdale Basin is one of those few that its unit hydrographs can be obtained. From the recorded data recommended by Tucker (1968), rainfalls of 10-min duration were selected together with the corresponding hydrographs to establish the 10-min unit hydrographs. The eight 10-min unit hydrographs so obtained are shown in Fig. 30. Based on these eight unit hydrographs the average 10-min unit hydrograph is plotted as shown in Fig. 30 and is used in this study for runoff prediction. The 1-min unit hydrograph is subsequently obtained by using the S-hydrograph method (Chow, 1964) as shown in Fig. 31 and checked against recorded data. The 2-min unit hydrograph can then be derived by adding one 1-min unit hydrograph to another 1-min unit hydrograph with a 1-min time lag. It should be mentioned here that in Fig. 30 the deviation among the eight hydrographs for the eight recorded 10-min rainfalls may reflect, in addition to data accuracy and linear approximation of a nonlinear physical phenomenon, the effect of changes in basin characteristics with season and time.

In applying the unit hydrographs of different durations to the four rainstorms to obtain the runoff hydrographs, the abstraction made from the total rainfall to give the rainfall excess is a rather subjective matter. As discussed in Sec. IX-2, in this investigation for the sake of consistency, Horton's formula (Eq. 34) is used to estimate the infiltration with the values of the parameters roughly the same as those used for the Illinois surface

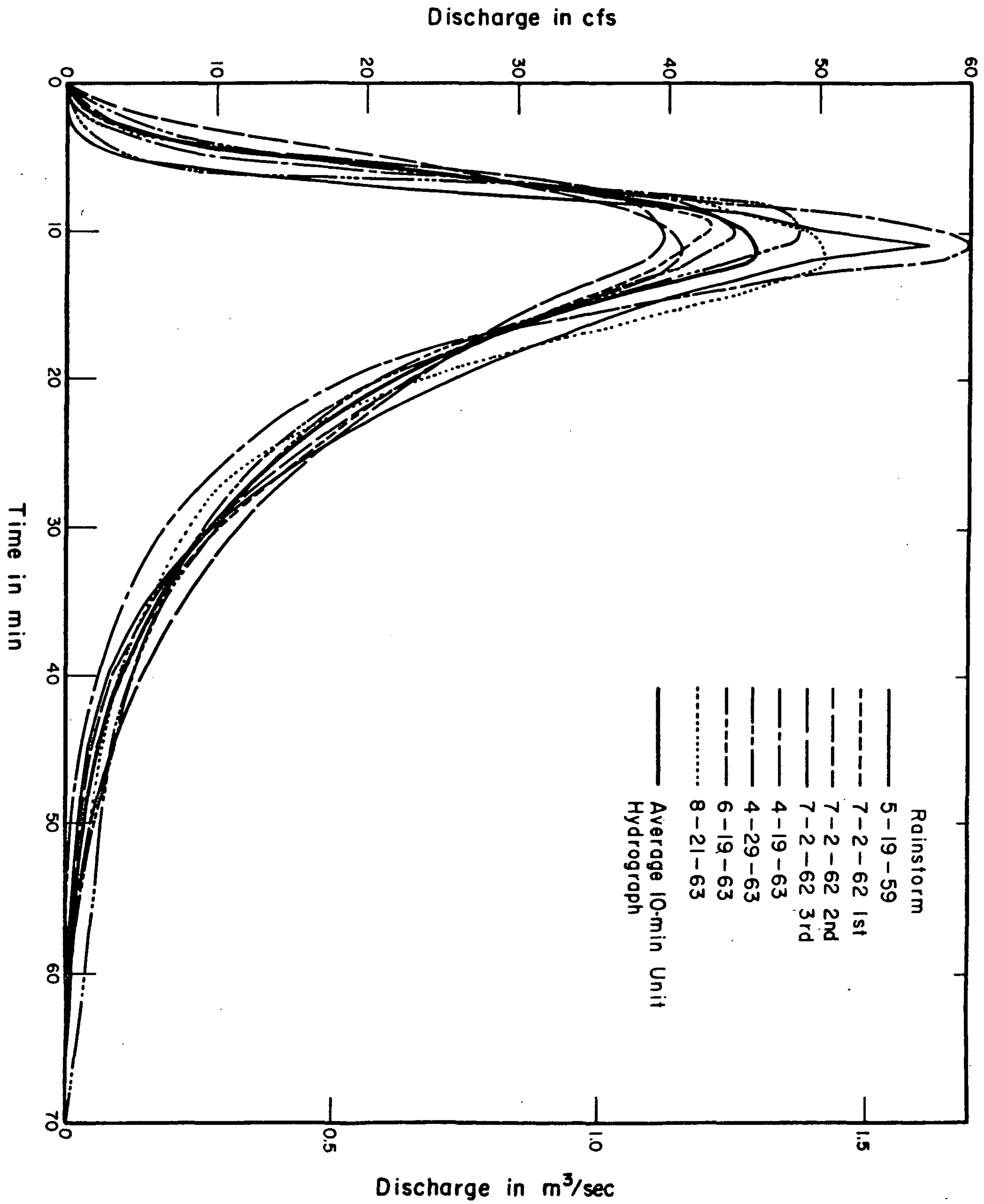

Fig. 30. Ten-min unit hydrograph for Oakdale Basin

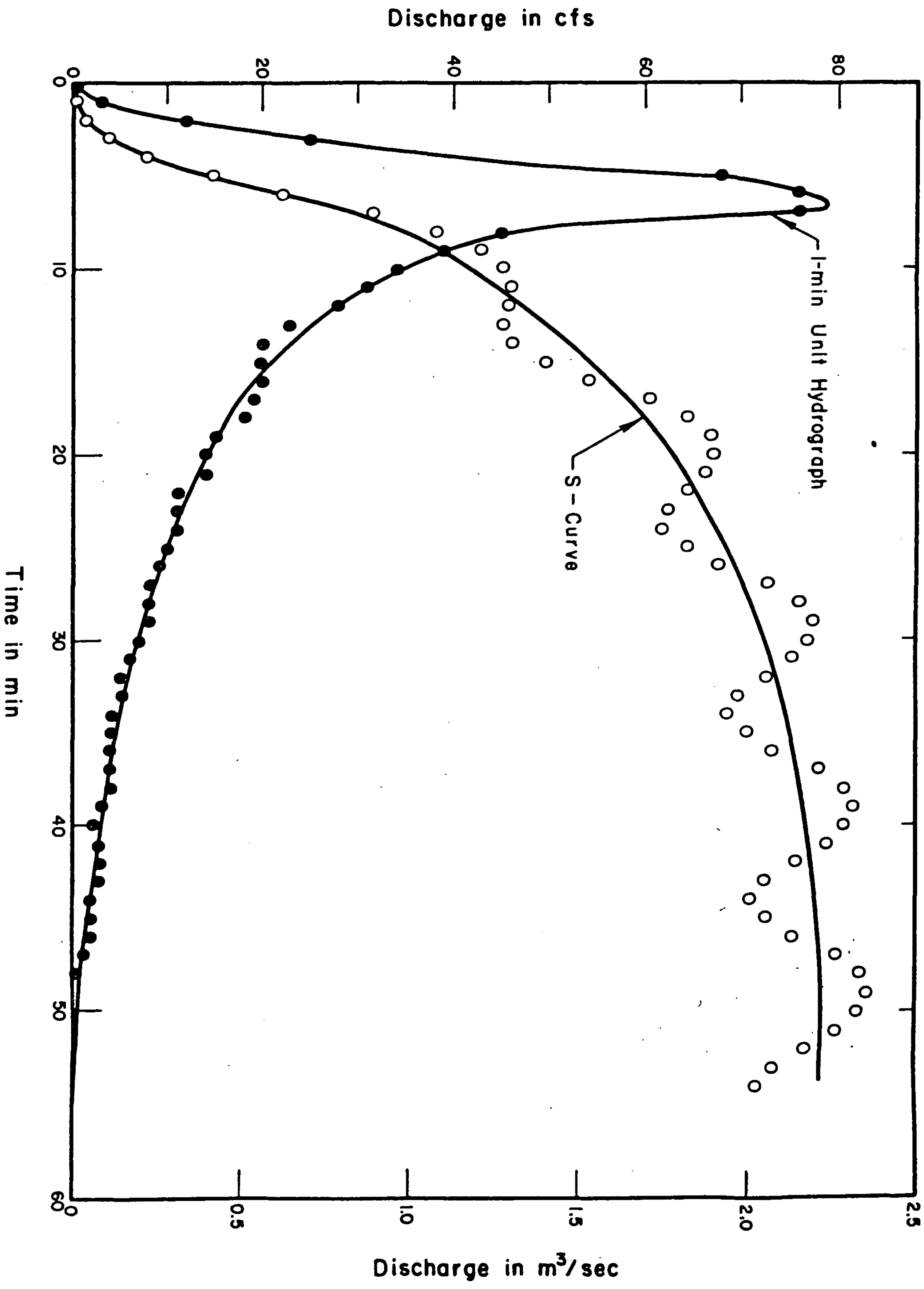

Fig. 31. S-Curve and one-min unit hydrograph for Oakdale Basin

runoff model. Also, the same base flow is added to both methods.

With the rainfall excess and base flow known, the standard procedure for using unit hydrographs is applied. The headings of the table used for the computations is shown in Table 9 and the results are shown in Figs. 26 to 29 respectively for the four rainstorms tested.

(C) Chicago hydrograph method. - As mentioned in Sec. IX-3, the Chicago method actually has two versions. The time-offset version is not adopted in this study because the sewer layout of the Oakdale Basin is not favorable for using this version. Therefore, the storage routing version is used. The hydrograph for the July 7, 1964 rainstorm by the Chicago method shown in Fig. 29 was taken from the University of Cincinnati (1970) report. Because the computer program for Chicago method could not be released by the city of Chicago and only three other rainstorms were to be evaluated, it is not worthwhile in this study to write a computer program and hence the hydrographs for the other three rainstorms were computed by hand calculations. The computed results are plotted in Figs. 26 to 28 for comparison.

(D) Road Research Laboratory method. - The hydrographs by the British Road Research Laboratory method for the four rainstorms tested have been computed by other investigators. The July 7, 1964 rainstorm was taken from Terstriep and Stall (1969) and plotted in Fig. 29. The July 2, 1960 rainstorm was taken from the University of Cincinnati (1970) report and shown in Fig. 27. For the other two, the May 19, 1959 and April 29, 1963 rainstorms, the hydrographs were taken from James F. MacLaren (1974) and replotted in Figs. 26 and 28. Of course the results would be different if the directly contributing impervious areas are taken differently from the values assumed by those investigators.

Table 9. HEADINGS FOR COMPUTATION OF UNIT HYDROGRAPH METHOD

Time	Recorded Rainfall	Infiltration		Rainfall Excess	Component Rainstorm		Runoff = Rainfall Excess x Unit Hydrograph Ordinates						Base Flow	Total Runoff
		Rate	Average Rate During Period		Duration	Excess	Component Rainstorm 1	Component Rainstorm 2	Component Rainstorm 3					

(E) University of Cincinnati Urban Runoff Model. - The hydrographs of rainstorms of July 2, 1960 and July 7, 1964 for the UCUR model were originally computed by the model developers (University of Cincinnati, 1970) and replotted here in Figs. 27 and 29. The hydrographs for the other two rainstorms, May 19, 1959 and April 23, 1963, were computed by James F. MacLaren (1974) for another comparative study and borrowed for this study as shown in Figs. 26 and 28.

(F) EPA Storm Water Management Model. - The hydrograph of the July 2, 1960 rainstorm for SWMM shown in Fig. 27 was taken from the report by the model developers (Metcalf & Eddy, Inc. et al., 1971). It was calculated using the original version of SWMM. However, the quantity part of the original SWMM is essentially the same as the later modified version called EPA SWMM. Hence this hydrograph can be used without recalculation. The hydrographs for the rainstorms of May 19, 1959 and April 29, 1963 were taken from the study by James F. MacLaren (1974) and replotted in Figs. 26 and 28.

(G) Dorsch Hydrograph Volume Method. - The Dorsch HVM is the only proprietary model compared in this study because the hydrographs for the May 19, 1959, July 2, 1960, and April 29, 1963 rainstorms are readily available (James F. MacLaren, 1974) and replotted in Figs. 26 to 28. For the same reason of proprietary no attempt was made to use the Dorsch HVM to calculate the hydrograph for the July 7, 1964 rainstorm.

(H) Illinois Urban Storm Runoff Method. - The Illinois Urban Storm Runoff Method was used to compute the hydrographs for all the four rainstorms tested and the results are plotted in Figs. 26 to 29. Since details of this method are not available elsewhere they are described as follows.

The measured rainfall recorded as the hyetograph is subtracted by infiltration to give the rainfall excess. The rainfall excess is then routed through the subcatchments using the nonlinear kinematic-wave approximation. When the rainfall rate recorded on the hyetograph is smaller than the infiltration capacity, the depth of water detented on the surface supplements the supply to infiltration.

As mentioned in Chapter V, the subcatchments of the Oakdale Basin consists of four types; roofs, lawns, sidewalks (including drive-ways), and street pavements. The asphalt street pavements are 4.27-m (14-ft) wide (one-half of street width) having an average cross slope of 2.7%. The infiltration through the street pavement is rather small and the values of f_c and k for Horton's formula (Eq. 34) used are 0.01 in./hr or 0.25 mm/hr and 10 hr^{-1}, respectively. Without adequate information on the initial wetting conditions the value of the initial infiltration f_o for the four rainstorms is assumed to be 0.02 in./hr or 0.5 mm/hr. It was found that because of the small infiltration through the pavement, the results are not sensitive to the values of f_o and k used.

Theoretically, the rainwater falling on the street pavements can be routed through the length of the pavement strips as described in Sec. VI-1. However, because of the short pavement length perpendicular to the gutter (4.3 m or 14-ft), the pavement flow time is less than half a minute and much less than the time intervals used for gutter flow computations. Consequently, such routing of flow through the street pavement would substantially increase the computer time and cost with essentially no improvement in the accuracy of the results. Furthermore, the kinematic wave routing method described in Sec. VI-1 does not account for the backwater effect from the gutter and hence the solution is only approximate. Therefore, it was decided not to perform the street pavement flow routing for the Oakdale Basin and the computed rainfall excess falling on the pavement is assumed to be transformed into lateral flow to the gutter directly.

As discussed in Sec. V-I, it is possible but impractical to consider in urban runoff prediction the details of distributions of the areas for roofs, lawns, and sidewalks separately and accurately. Therefore, in the present sutdy for the Oakdale Basin for the purpose of flow routing these three types of surfaces are treated together as homogeneous overland surfaces and an average slope of 6% and hydraulic surface roughness of 0.01 ft or 3 mm is assumed.

As discussed in Sec. IV-2, theoretically, the final infiltration capacity f_c and the exponent k in Horton's formula (Eq. 34) should be nearly constant for the pavements as well as for the roofs, lawns and sidewalks. The only reasons that these values would change are the seasonal changes and changes of the drainage basin physical characteristics in the span of 5 years. However, sufficient data were not available to establish the values of f_c and k for the Oakdale Basin. Consequently a costly and time consuming trial-and-error process based on the recorded hydrographs was adopted. This approach is neither accurate nor foolproof and it is highly undesirable. This difficulty necessarily points out the importance of the need for detailed accurate data on land surface conditions in storm runoff prediction. For the four rainstorms tested, no suitable common values of f_c and k were found. In fact, the computed runoff hydrograph is sensitive to the values of f_c, k, and f_o assumed as shown in Fig. 32 for rainstorms of July 2, 1960 and July 7, 1964. Values of Horton's infiltration parameters used for the computed hydrographs of the Illinois Urban Storm Runoff Method shown in Figs. 26 to 29 are listed in Table 10.

Table 10. VALUES OF INFILTRATION PARAMETERS USED FOR ILLINOIS URBAN STORM RUNOFF METHOD SHOWN IN FIGS. 26 TO 29

Parameter \ Rainstorm		May 19, 1959	July 2, 1960	April 29, 1963	July 7, 1964
f_o	in./hr	1.00	0.55	0.80	1.00
f_o	mm/hr	25	14	20	25
f_c	in./hr	0.15	0.05	0.1	0.5
f_c	mm/hr	3.8	1.3	2.5	12.7
k	hr^{-1}	10	15	15	6

The variations of f_c and k estimated from recorded hydrographs for the four rainstorms tested due in addition to the error of using average values to represent the integrated effect over the entire basin of infiltration of individual component areas of different types, may also actually reflect the effects of movement of the rainstorms and nonuniform areal distribution of the rainfall intensity, as well as differences in roof-top and other surface retentions.

In routing the runoff, the direction of the subcatchment flow is assumed perpendicular to the gutter. The overland surface (particularly the street pavements) of the Oakdale Basin, as in many American cities, is reasonably homogeneous and that only limited number of subcatchment strips need to be computed and other strips can simply use the result obtained. This simplification considerably reduces the computer time and costs without sacrificing the accuracy of the results. The initial condition for the overland flow as well as for gutter flow is a depth of 0.0012 in. or 0.03 mm of water with zero velocity.

The subcatchment runoffs together with the direct rainfall constitute the input into the gutter flow. The gutter infiltration is evaluated using Horton's formula (Eq. 34) with the same values of f_c, k and f_o as for the street pavement since no better information is available, although the model allows different sets of values for the pavement and gutter. The gutter flows are routed from the upstream, i.e., from the gutter with the highest elevation of the basin towards downstream, as shown by the arrows in Fig. 13. The westward flow direction of Oakdale Avenue east of Junction 118 is opposite to the direction of the slope of the sewers underneath. At the downstream end of a gutter, water flows into the inlet with a capacity given by Eqs. 63 and 64, in which the discharge coefficients C_d used are 3.0 and 0.6, respectively. Not all of the inlets are placed immediately from the curb line and some may extend beyond the gutter proper at the pavement side. Such irregularities are not accounted for in the model. Also, the circular grate inlets are approximated by rectangular inlets of equivalent area in the computation. Moreover, as mentioned in Sec. VI-3, should the inlet discharge be smaller than the gutter discharge, the excessive flow is assumed carried through the inlet continuing into the next gutter immediately following.

The outflow hydrographs obtained from the surface runoff model are then used as the input into the sewer system for routing using the ISS Model to give the drainage basin runoff hydrographs. Since the sewer junctions in the Oakdale Basin have small cross-sectional areas, they are considered as point-type junctions in the ISS Model. Also, the flow measurement flume located at the outlet of the Oakdale Basin is approximated by a 0.76-m (30-in.) diameter pipe in calculations. The computed hydrographs for the four rainstorms tested using the Illinois Urban Storm Runoff Method with the values of infiltration parameters for subcatchments listed in Table 10 are plotted in Figs. 26 to 29 for comparison with the recorded hydrographs and computed hydrographs by other methods.

X-2. Comparison of the Methods

The evaluation of the eight methods are made against the recorded hydrographs for the four rainstorms tested. Of course there is no guarantee on the measurement accuracy of the recorded results. In fact, the accuracy of rainfall and runoff measurements for the Oakdale Basin data has not been adequately established. For example, the sharp drop at the peak discharge of the July 7, 1964 data and the strange shape of the recorded hydrograph in the next five min appear to be somewhat suspicious. For the July 2, 1960 rainstorm the peak discharge occurred at the same instant as the end of the maximum rainfall intensity, leaving no time lag between the two, is also questionable. Nevertheless, in view of the general situation of poor quality of field data for urban storm runoffs, the Oakdale data should be considered as one of the best sets that one can utilize.

The comparison of the methods can be made from two aspects. First is the ability of the methods to reproduce the recorded hydrographs based on the recorded rainfalls. Second is the computational time and cost required for the methods. For the first, in addition to the hydrographs shown in Figs. 26 to 29, the peak discharge Q_p and its time of occurrence, t_p, for the four rainstorms tested are listed in Table 8.

As mentioned in Chapter IX the rational method gives only the peak discharge and hence its comparison with other methods is limited. The rational method is extremely simple and gives a reasonable accurate estimation of Q_p if the runoff coefficient C and the time of concentration, t_c, used for rainfall intensity determination are properly chosen. However, the choice of C and t_c is more an art of judgement than a scientific precision determination, and as customarily used neither C nor t_c takes into account the preceding surface moisture condition or the intensity and areal distribution of rainfall.

The unit hydrograph method provides reasonable though not very accurate runoff hydrographs, and it is simple, fast, and straight forward if the unit hydrograph for the drainage basin is available. Because the unit hydrograph theory itself involves the assumption of linearity (Yen et al., 1969, Yen et al., 1973), one cannot expect high accuracy prediction from unit hydrographs, particularly for small urban drainage basins of a few acres (hectares). From the practical viewpoint, the biggest problem of using unit hydrograph is its availability. For most urban drainage basins there are no data to establish the unit hydrographs. Occasionally, transposing the known unit hydrograph of a neighboring drainage basin of similar nature with appropriate size and other adjustments may be used. This is indeed a practical approach but one should not expect high accuracy from it.

None of the methods evaluated consistently reproduces the recorded runoff hydrographs faithfully. In general the three most sophisticated methods, namely the Illinois Urban Storm Runoff Method, the Dorsch HVM, and the EPA SWMM usually give better results than the other methods. The Chicago method, RRL method, and UCUR method may give results considerably different from the recorded hydrographs and from those predicted by the more sophisticated methods. However, a more precise, detailed, and meaningful comparison of accuracy is not possible at present because of the uncertainties involved in the amount and area and time distributions of infiltration as discussed earlier. As shown in Fig. 32, the predicted runoff hydrographs are quite sensitive to the values of infiltration used. Although attempts had been made in this study to use the same infiltration function as much as possible, differences still exist for different rainstorms and for different methods. As pointed out by Torno (1974), infiltration of an amount different from that for other methods was used for the UCUR method for the July 2, 1960 and July 7, 1964 rainstorms. Presumably the agreement of the UCUR predictions with the recorded hydrographs would be poorer if the same infiltration is used.

Furthermore, at least for the Illinois Urban Storm Runoff Method and presumably also for other sophisticated methods, the accuracy on the details of the basin geometry have profound influence on the shape of the hydrograph as shown in Fig. 32a. During the investigation of the effect of infiltration on hydrographs, it was thought that since the Oakdale Basin is reasonably symmetric with respect to Oakdale Avenue, computer time and cost can be saved by computing the surface runoff for one half of the basin and then double the result for sewer routing as a first approximation before further refinement is made. The rainstorm of July 2, 1960 was tested with the actual geometry and symmetric approximation for infiltration k = 20,

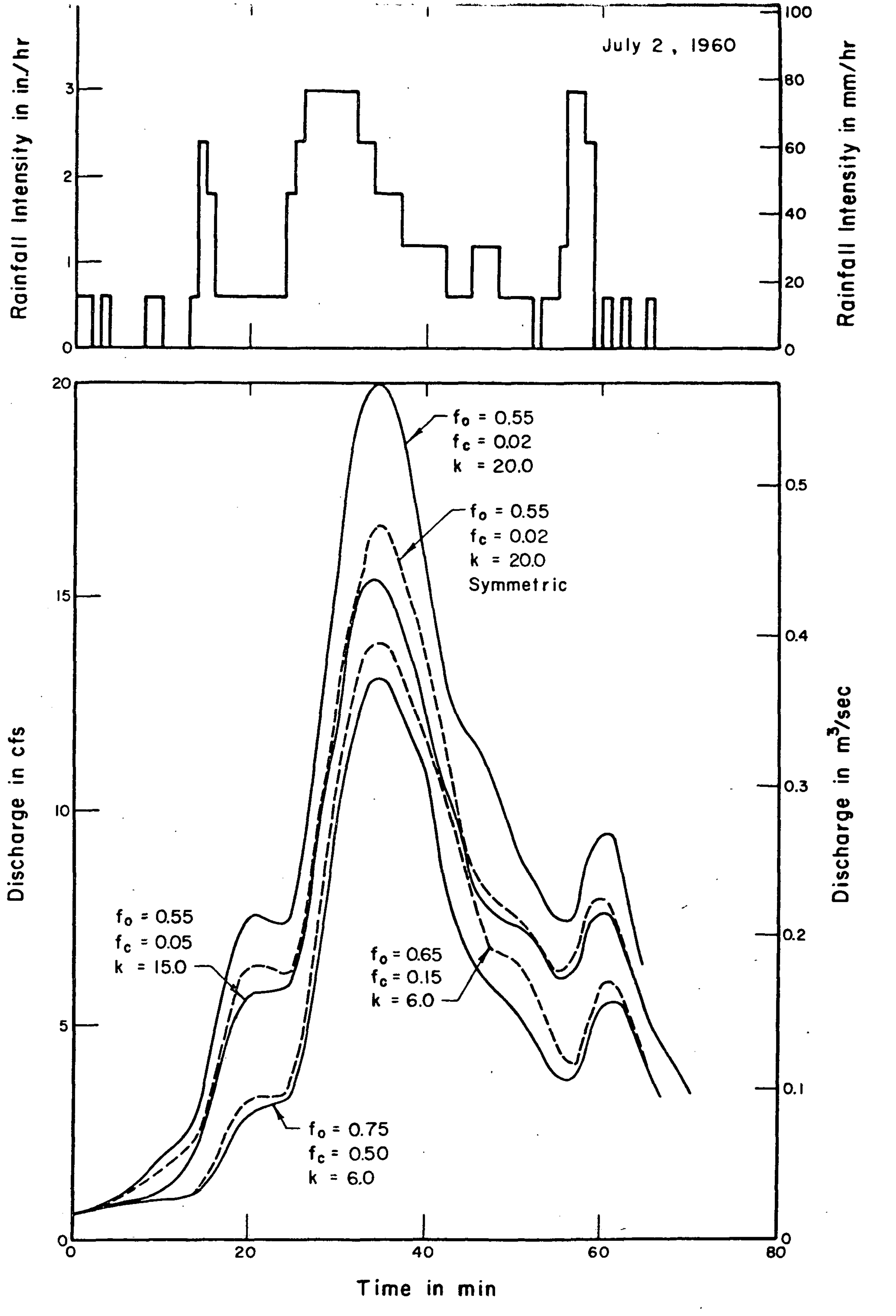

Fig. 32. Sensitivity of computed hydrograph to Horton's infiltration parameters (a) Rainstorm of July 2, 1960

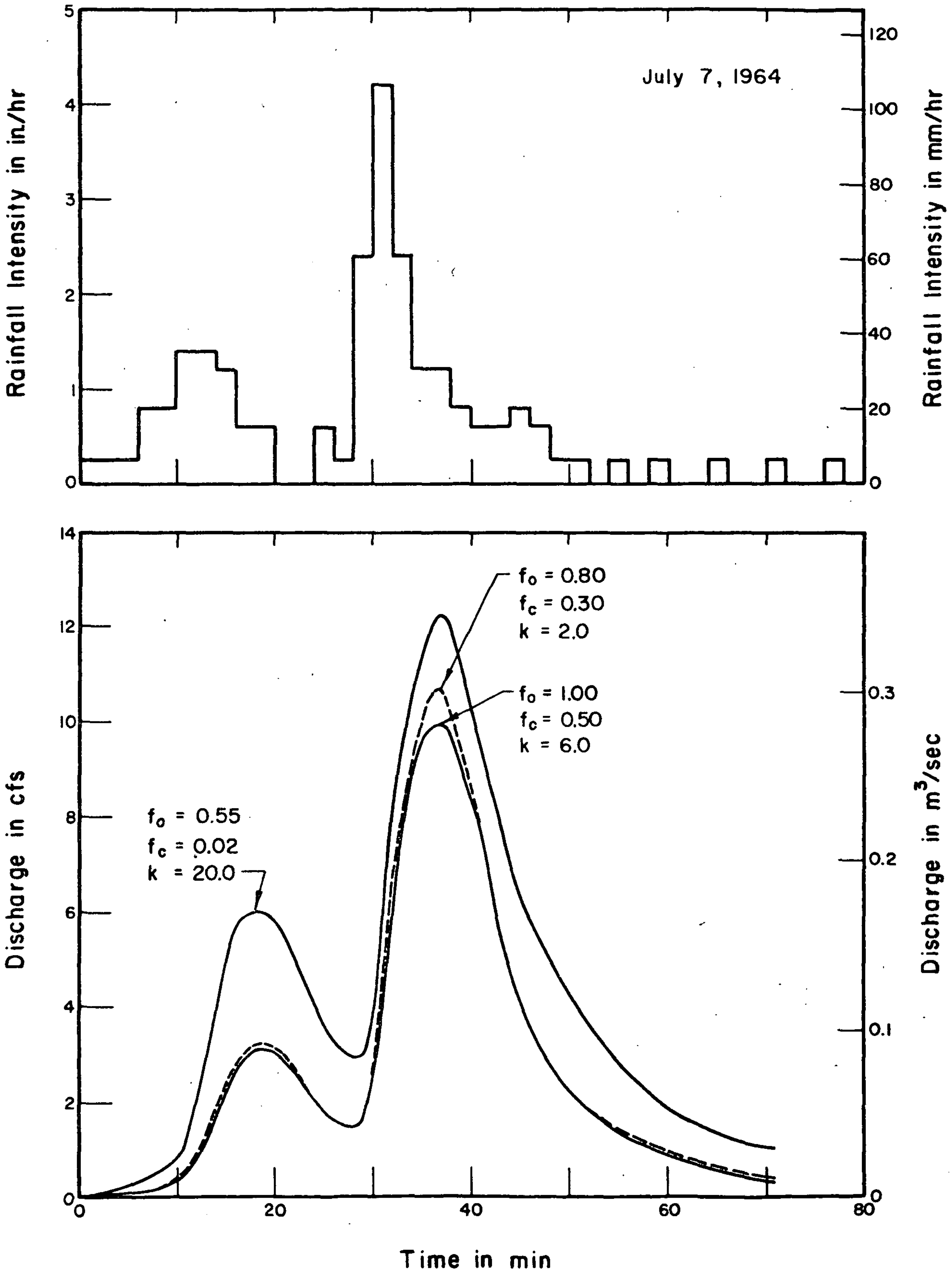

Fig. 32. (b) Rainstorm of July 7, 1964

f_c = 0.5 mm/hr (0.02 in./hr) and f_o = 14 mm/hr (0.55 in./hr). The difference between the two resulted hydrographs of assumed symmetric and nonsymmetric cases are surprisingly large, 17% difference in the peak discharge. Presumably when the amount of infiltration increases the difference would decrease. Nevertheless this result indicates that for the sophisticated models for overland flow routing the accuracy of the basin geometry is one important factor to produce reliable hydrographs.

The difficulty in obtaining reliable comparison for the different methods because of the uncertainties in infiltration suggests that further study should be done on this line and that an even smaller drainage basin than the Oakdale Basin with accurate detailed data should be used for testing, and perhaps a separate testing on the surface runoff and sewer runoff be conducted. This separate testing of surface and sewer runoffs would more positively identify the relative merit of the different methods. At present, it is intended in this report to provide from a practical viewpoint some useful information for an engineer to decide which method he should choose to use for his particular project.

As mentioned earlier, the computation time and effort for the rational method is minimal, and that for the unit hydrograph method is also small provided the unit hydrograph is available. For the other six methods use of computer is highly recommended, and in fact the three most sophisticated methods cannot be done without using a computer. The relative amount of computer time for the six methods varies depending on the rainstorm and drainage basin. In addition to the experience gained in this study, some information on computer time for certain methods have been reported by Heeps and Mein (1973) and James F. MacLaren, Ltd. (1974). Roughly for a drainage basin like the Oakdale with the rainstorms similar to the four tested, the process time for RRL and Chicago methods are of the order of a tenth of a min on the University of Illinois IBM 360/75 system, with the latter slightly longer. The UCUR method requires a computer process time more than one order of magnitude longer than the RRL method with no obvious improvement in accuracy. The computer time for EPA SWMM is about the same as that for UCUR method and the Dorsch HVM is slightly more. The Illinois Urban Storm Runoff Method requires the longest computer time among the eight methods evaluated, being about twice as much as that for the EPA SWMM, or somewhat more than 20 min for each of the four Oakdale rainstorms tested.

The evaluation made in this as well as other similar studies on urban runoff methods is definitely of limited scope and not exhaustive, and one should interpret the results with caution. Nevertheless, it can be concluded that if only the peak discharge is required and a quick result is expected without high accuracy, the rational method is the most suitable one to use. However, if the runoff hydrograph is needed such as for flow regulation and storm runoff pollution control purposes the rational method is unacceptable and other methods should be sought. Among the seven methods evaluated that produce runoff hydrographs, the Illinois Urban Storm Runoff Method most likely will give the most accurate result and is recommended if high accuracy is required and no restrictions on computational costs. The drawbacks of the Illinois method are that (a) it requires a large amount of computer time and hence costs; (b) it requires detailed data on basin physical characteristics that are not required by the other methods; and (c) at present the sewer routing model (ISS Model) allows only circular pipes and for sewers having other cross sectional shapes the equivalent pipe diameter giving similar depth-area or depth-hydraulic radius relationship should be first determined.

The Dorsch HVM, though perhaps less accurate than the Illinois method, also requires less computer time, so it can be used if the program is readily available. Nevertheless, its data requirement is also as detailed and demanding as the Illinois method.

The EPA SWMM can be operated with much less required data on basin physical characteristics. Although it may not produce consistently relatively reliable hydrographs as the Illinois or Dorsch methods, it is much cheaper to use and is well documented and relatively most well known. It also has many other features that the Illinois and Dorsch methods do not have. It is recommended as another useful practical method.

When less accurate result is acceptable but the entire hydrograph is required, the unit hydrograph method should be used whenever it is possible. It is relatively simple, cheap, and fast, and it offers an accuracy at least comparable, if not better, than the Chicago, UCUR, or RRL methods. It does not require the use of computer to give the hydrograph, although using computer would save time and programming is rather simple. However, if synthetic means or basin transposition has to obtain the unit hydrograph, one should be careful on the reliability and accuracy.

Should the unit hydrograph not be available for the drainage basin of interest and the required accuracy of the hydrograph is not very high, the RRL method appears to be the next choice because its data requirement is not very high and the computer time is rather short. The Chicago method does not seem to offer anything better than the RRL method, in fact, it often over-predicts the peak discharge, and it requires more basin data and computer time. The UCUR method differs little from the Chicago method and yet requires considerably more data and longer computer time. So it should be considered only as the last resort if other methods cannot be used.

Finally, it should be mentioned here that both the ISS Model and EPA SWMM have built-in mechanism to consider the effectiveness of using in-line storage for runoff quantity and quality control purpose should it be desired to do so. Presumably the Dorsch HVM can also consider this storage effect. The total runoff volume of course simply corresponds to the total area under the hydrograph. The shape of the hydrograph, i.e., the time distribution of the runoff, will be altered with different in-line storage. An example of alternating a storm sewer system design by including in-line storage utilizing the ISS model has been reported by Yen and Sevuk (1975).

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XII. NOTATION

A = area;

B = parameter for exponential density function (Eq. 20) or gamma density function (Eq. 21);

b = water surface width;

C = parameter for gamma density function (Eq. 21); also, constant; also, runoff coefficient;

C_d = discharge coefficient;

c = Chezy's roughness factor; also, concentration;

D = rainfall depth; also, diameter; also, hydraulic depth = A/b;

d = detention storage;

$\overline{d}$ = average rainfall depth per time interval (Eq. 3);

d_j = depth of rainfall of the j-th time interval;

$E(x)$ = expected value of x

F = cumulative infiltration expressed in depth;

f = infiltration capacity; also, Weisbach resistance coefficient;

f_c = final infiltration capacity;

f_o = initial infiltration capacity;

G = second moment arm of hyetograph (Eq. 5); also, a function of flow depth, = $\partial Q/\partial h$;

g = gravitational acceleration;

H = available head of flow;

h = flow depth;

I = inflow rate;

i = rainfall intensity;

j = index number;

k = decay rate of infiltration in Horton's formula; also, surface roughness;

L = length

M = daily snow melt;

n = Manning's roughness factor; also, a number;

P = cumulative rainfall in depth;

P_r = daily rainfall;

Q = discharge;

Q_p = peak discharge;

q = lateral inflow per unit length of σ;

$q_\ell = \int_\sigma q d\sigma$, lateral discharge per unit length of flow;

R = hydraulic radius;

$\mathbb{R} = VR/\nu$, Reynolds number;

S = slope;

S_f = friction slope;

$S_o = \sin\theta$, bed slope;

s = depression storage supply rate; also, storage;

s_c = depression storage capacity expressed in depth;

T_a = mean daily temperature of saturated air at 10-ft level;

t = time;

$\bar{t}$ = first moment arm of hyetograph (Eq. 4);

t_b = elapsed time between the end of a rainstorm and the beginning of the following rainstorm;

t_d = rainfall duration;

U_1 = x-component of velocity of lateral flow;

V = flow velocity;

$V(x)$ = variance of x;

W = gutter width;

x = longitudinal direction;

z = elevation above horizontal reference datum;

θ = angle;

ν = kinematic viscosity of water;

σ = perimeter bounding flow area A; and

σ_d = standard deviation

APPENDIX A

LISTING OF COMPUTER PROGRAM FOR FREQUENCY ANALYSIS OF HOURLY PRECIPITATION DATA

The frequency analysis program is written in Fortran IV language. The program in its present form requires about 160K bytes of storage. The computer time required for the analyses described in Section IV-1 of this report is about 90 seconds. Storage and time requirements may vary considerably depending on the particular application.

Program descriptions, input and dimension requirements are given at the beginning of the program as comment statements.

The listing of the computer program is given below.

```
C*********************************************************************
C*********************************************************************
C
C      PROGRAM FOR FREQUENCY ANALYSIS OF HOURLY PRECIPITATION DATA.
C
C    * THE FREQUENCY ANALYSIS PROGRAM IS COMPOSED OF FOUR INTERRELATED
C      PROGRAMS AND THREE SUBROUTINES.
C
C    * PROGRAM DESCRIPTIONS,DIMENSION AND INPUT REQUIREMENTS
C      ARE GIVEN BELOW.
C
C    * ALL PROGRAMS CAN HANDLE BOTH ENGLISH AND SI (INTERNATIONAL SYSTEM)
C      UNITS.ALL INPUTS ARE EITHER IN ENGLISH UNITS OR IN SI UNITS.COMBI-
C      NATIONS ARE NOT ALLOWED.IF INPUTS ARE IN ENGLISH UNITS,THE RESULTS
C      ARE PRINTED IN ENGLISH UNITS AND,IF DESIRED,ALSO IN SI UNITS.
C      IF INPUTS ARE IN SI UNITS,THE RESULTS ARE PRINTED IN SI UNITS.
C      SPECIFICATIONS FOR UNITS ARE READ IN PROGRAM 1 AND APPLY TO ALL
C      PROGRAMS.
C
C*********************************************************************
C
C      PROGRAM DESCRIPTIONS.
C
C---------------------------------------------------------------------
C
C      PROGRAM 1 ... IDENTIFICATION OF RAINSTORMS AND CALCULATION OF
C                    RAINSTORM PARAMETERS.
C
C    * THIS PROGRAM TRACES THE GIVEN HOURLY PRECIPITATION DATA,IDENTIFIES
C      THE RAINSTORMS AND CALCULATES THE VALUES OF THE RAINSTORM
C      PARAMETERS.THE VALUES OF THE RAINSTORM PARAMETERS ARE THEN STORED
C      FOR USE IN THE SUBSEQUENT PROGRAMS.
C      IF A SEASON BEGINS OR ENDS WITH A RAINSTORM,THESE RAINSTORMS
C      ARE NOT CONSIDERED BECAUSE THEY MIGHT EXTEND OUTSIDE THE SEASON.
C      IF A SEASON BEGINS WITH A DRY HOUR,THE FIRST RAINSTORM IN THE
C      SEASON IS NOT CONSIDERED BECAUSE TIME BETWEEN THIS RAINSTORM AND
C      THE PREVIOUS ONE IS NOT DETERMINED FOR THE SAME REASON AS ABOVE.
C
C    * THE OUTPUTS FROM THIS PROGRAM ARE THE PRINTOUTS OF HOURLY
C      PRECIPITATION VALUES AND THE VALUES OF THE RAINSTORM PARAMETERS.
C      FOR EACH SEASON,HOURLY PRECIPITATION VALUES ARE PRINTED FOR EACH
C      DAY AND HOUR OF THE SEASON TOGETHER WITH THE VALUES OF THE
C      RAINSTORM PARAMETERS FOR ALL RAINSTORMS IN THE SAME SEASON.
C
C    * THE PRINTOUTS ARE OPTIONAL.
C
C---------------------------------------------------------------------
C
```

```
C      PROGRAM 2 ... SORTING VALUES OF RAINSTORM PARAMETERS.
C
C    * THIS PROGRAM PERFORMS THE FOLLOWING OPERATIONS
C
C      OPERATION 2-1 ...
C
C            PRINTS THE VALUES OF THE RAINSTORM PARAMETERS
C            FOR ALL RAINSTORMS IN THE RECORD.
C
C      OPERATION 2-2 ...
C
C            SORTS THE VALUES OF THE RAINSTORM PARAMETERS IN ASCENDING
C            ORDER AND PRINTS THE RESULTS.
C
C      OPERATION 2-3 ...
C
C            FOR ANY GIVEN RAINSTORM PARAMETER,SORTS THE VALUES OF THAT
C            PARAMETER AND PRINTS THE RESULTS TOGETHER WITH THE
C            CORRESPONDING VALUES OF THE OTHER PARAMETERS.
C
C      OPERATION 2-4 ...
C
C            FOR ANY GIVEN RAINSTORM PARAMETER,SORTS A GIVEN SUBSET
C            OF THE VALUES OF THAT PARAMETER AND PRINTS THE RESULTS
C            TOGETHER WITH THE CORRESPONDING VALUES OF THE OTHER
C            PARAMETERS.
C
C    * ALL OPERATIONS ARE OPTIONAL.
C
C-----------------------------------------------------------------------
C
C      PROGRAM 3 ... ONE-WAY FREQUENCY ANALYSIS OF THE VALUES OF
C                    RAINSTORM PARAMETERS.
C
C    * THIS PROGRAM PERFORMS ONE-WAY FREQUENCY ANALYSIS ON ALL VALUES
C      OR ON A SUBSET OF VALUES (CORRESPONDING TO A GIVEN SUBSET OF
C      VALUES OF ANOTHER PARAMETER) OF ANY GIVEN RAINSTORM PARAMETER.
C
C      OPERATION 3-1 ...
C            ONE-WAY FREQUENCY ANALYSIS OF ALL VALUES OF A GIVEN
C            RAINSTORM PARAMETER.
C
C      OPERATION 3-2 ...
C            ONE-WAY FREQUENCY ANALYSIS OF A SUBSET OF VALUES
C            (CORRESPONDING TO A GIVEN SUBSET OF VALUES OF ANOTHER
C            PARAMETER) OF A GIVEN RAINSTORM PARAMETER.
C
C    * THE OUTPUTS FROM THIS PROGRAM ARE THE TABLES OF FREQUENCIES(NUMBER
```

```
C      OF OBSERVATIONS OVER GIVEN CLASS INTERVALS), RELATIVE FREQUENCIES
C      (FREQUENCY DIVIDED BY THE TOTAL NUMBER OF OBSERVATIONS),PROBABI-
C      LITY DENSITIES(RELATIVE FREQUENCY DIVIDED BY THE INTERVAL SIZE)
C      AND NON-EXCEEDANCE PROBABILITIES(CUMULATIVE RELATIVE FREQUENCY).
C      IN ADDITION,MINIMUM,MAXIMUM,MEAN AND STANDARD DEVIATION OF THE
C      VALUES CONSIDERED ARE PRINTED.
C
C    * ALL OPERATIONS ARE OPTIONAL.
C
C-----------------------------------------------------------------------
C
C      PROGRAM 4 ... TWO-WAY FREQUENCY ANALYSIS OF THE VALUES OF PAIRS OF
C                    RAINSTORM PARAMETERS.
C
C    * THIS PROGRAM PERFORMS TWO-WAY FREQUENCY ANALSIS ON THE VALUES OF
C      ANY GIVEN TWO RAINSTORM PARAMETERS.
C
C    * THE OUTPUTS FROM THIS PROGRAM ARE THE TWO-WAY TABLES OF
C      FREQUENCIES(NUMBER OF OBSERVATIONS OVER GIVEN CLASS INTERVALS),
C      RELATIVE FREQUENCIES(FREQUENCY DIVIDED BY THE TOTAL NUMBER OF
C      OBSERVATIONS),AND PROBABILITY DENSITIES(RELATIVE FREQUENCY DIVIDED
C      BY THE INTERVAL AREA).
C
C    * THE EXECUTION OF PROGRAM 4 IS OPTIONAL.
C
C-----------------------------------------------------------------------
C
C      SUBROUTINES ...
C
C    * THE FOLLOWING SUBROUTINES ARE USED IN THE PROGRAMS
C
C           SORT
C           TAB1
C           TAB2
C
C    * SEE SUBROUTINE LISTINGS FOR PROGRAM DESCRIPTIONS.
C
C***********************************************************************
C
C      DIMENSION REQUIREMENTS.
C
C    * X( ) IS USED TO STORE THE HOURLY PRECIPITATION VALUES FOR ALL
C      HOURS OF A SEASON.ITS DIMENSION SHOULD BE EQUAL TO OR GREATER
C      THAN THE NUMBER OF HOURS WITHIN THE SEASON.
C
C    * D( ) IS USED TO STORE THE HOURLY PRECIPITATION VALUES FOR A RAIN-
C      STORM.ITS DIMENSION SHOULD BE EQUAL TO OR GREATER THAN THE NUMBER
C      OF HOURS FOR THE LONGEST DURATION RAINSTORM IN THE RECORD.
```

```
C
C   * P( ) IS USED TO STORE THE VALUES OF THE RAINSTORM PARAMETERS
C     FOR ALL RAINSTORMS IN A SEASON.THE ROW DIMENSION SHOULD BE EQUAL
C     TO OR GREATER THAN THE MAXIMUM NUMBER OF RAINSTORMS EXPECTED IN
C     ANY ONE SEASON.
C
C   * PA( ) IS USED TO STORE THE VALUES OF THE RAINSTORM PARAMETERS
C     FOR ALL RAINSTORMS IN THE RECORD.THE ROW DIMENSION SHOULD BE EQUAL
C     TO OR GREATER THAN THE MAXIMUM NUMBER OF RAINSTORMS EXPECTED FOR
C     THE WHOLE RECORD.
C
C   * PA1( ) SHOULD HAVE THE SAME DIMENSIONS AS PA( ).
C
C   * A( ) AND B( ) SHOULD BOTH HAVE THE SAME DIMENSION AS THE ROW
C     DIMENSION OF PA( ).
C
C   * FREQ( ),PCT( ),DEN( ),CDEN( ) AND XX( ) SHOULD ALL HAVE THE SAME
C     DIMENSION AND IT SHOULD BE AT LEAST ONE MORE THAN THE MAXIMUM
C     NUMBER OF CLASS INTERVALS THAT WILL BE USED FOR ANY ONE OF THE
C     ONE-WAY FREQUENCY ANALYSES.
C
C***********************************************************************
C
C     INPUT REQUIREMENTS.
C
C   * INPUT STATEMENTS TOGETHER WITH THE DESCRIPTIONS OF THE INPUT
C     PARAMETERS ARE LISTED BELOW IN THE ORDER THEY APPEAR IN THE
C     PROGRAMS.INPUT DATA SHOULD BE READ IN THE SAME ORDER.
C
C-----------------------------------------------------------------------
C
C     PROGRAM 1 ...
C
C *** INPUT 1-1
C     READ(5,57) IDIMX,IDIMPA
C  57 FORMAT(2I4)
C     IDIMX=DIMENSION OF VECTOR X( ).
C *** IDIMPA=ROW DIMENSION OF MATRIX PA( ).
C
C *** INPUT 1-2
C     READ(5,1) IUNIT
C     READ(5,1) IPRSI
C   1 FORMAT(I1)
C     IUNIT=1 IF ALL INPUTS ARE IN ENGLISH UNITS.
C     IUNIT=2 IF ALL INPUTS ARE IN SI UNITS.
C     IPRSI=0 IF ALL INPUTS ARE IN SI UNITS.
C     IPRSI=0 IF ALL INPUTS ARE IN ENGLISH UNITS AND IF THE RESULTS ARE
C     TO BE PRINTED ONLY IN ENGLISH UNITS.
```

```
C      IPRSI=1 IF ALL INPUTS ARE IN ENGLISH UNITS AND IF THE RESULTS ARE
C      TO BE PRINTED BOTH IN ENGLISH AND SI UNITS.
C ***  THE ABOVE APPLY TO ALL PROGRAMS.
C
C ***  INPUT 1-3
C      READ(5,2) NUMMON
C    2 FORMAT(I2)
C      NUMMON=NUMBER OF MONTHS IN THE SEASON (01-12).
C      READ(5,3) (MONTH(I),I=1,NUMMON)
C    3 FORMAT(I2)
C      MONTH( )=MONTH NUMBERS IN ORDER (01-12=JAN-DEC).
C ***  ONE CARD FOR EACH MONTH.
C
C ***  INPUT 1-4
C      READ(5,4) NUMYR,LASTYR
C    4 FORMAT(2I2)
C      NUMYR=NUMBER OF YEARS IN THE RECORD
C ***  LASTYR=LAST TWO DIGITS OF THE LAST YEAR IN THE RECORD.
C
C ***  INPUT 1-5
C      READ(5,5) IPR1
C    5 FORMAT(I1)
C      IPR1=1 IF,FOR EACH SEASON,HOURLY PRECIPITATION VALUES ARE TO BE
C      PRINTED FOR EACH DAY AND HOUR OF THE SEASON TOGETHER WITH THE
C      VALUES OF THE RAINSTORM PARAMETERS FOR ALL RAINSTORMS IN THE SAME
C ***  SEASON.OTHERWISE,IPR1=0
C
C ***  INPUT 1-6
C      READ(5,13) IYR,IMO,IDY,(X(I+I2),I=1,24),INXDY
C   13 FORMAT(6X,3I2,1X,12F3.2/13X,12F3.2,29X,I2)
C      THIS INPUT STATEMENT READS THE HOURLY PRECIPITATION DATA IF DATA
C      ARE IN ENGLISH UNITS.
C
C   12 READ(5,15) IYR,IMO,IDY,(X(I+I2),I=1,24),INXDY
C   15 FORMAT(6X,3I2,1X,12F3.1/13X,12F3.1,29X,I2)
C      THIS INPUT STATEMENT READS THE HOURLY PRECIPITATION DATA IF DATA
C      ARE IN SI UNITS.
C
C      THE INPUT IS THE CARD DECK OF HOURLY PRECIPITATION DATA.
C      THE PREPERATION OF THE DECK IS DESCRIBED AT THE END
C ***  OF THIS SECTION.
C
C-----------------------------------------------------------------------
C
C      PROGRAM 2 ...
C
C ***  INPUT 2-1
C      READ(5,80) IPR2
```

```
C  80 FORMAT (I1)
C       IPR2=1 IF EXECUTION OF PROGRAM 2 IS DESIRED.OTHERWISE,IPR2=0
C *** IF IPR2=0 NO OTHER INPUT CARDS SHOULD BE USED FOR PROGRAM 2.
C
C       OPERATION 2-1 ...
C
C *** INPUT 2-2
C       READ(5,81) IPR21
C  81 FORMAT(I1)
C *** IPR21=1 IF OPERATION 2-1 IS TO BE PERFORMED.OTHERWISE,IPR21=0
C
C       OPERATION 2-2 ...
C
C *** INPUT 2-3
C  90 READ(5,82) IPR22
C  82 FORMAT(I1)
C *** IPR22=1 IF OPERATION 2-2 IS TO BE PERFORMED.OTHERWISE,IPR22=0
C
C       OPERATION 2-3 ...
C
C *** INPUT 2-4
C  98 READ(5,85) IPR23
C  85 FORMAT(I1)
C *** IPR23=1 IF OPERATION 2-3 IS TO BE PERFORMED.OTHERWISE,IPR23=0
C
C *** INPUT 2-5
C       READ(5,99) NUMT
C  99 FORMAT(I2)
C       NUMT=NUMBER OF TIMES OPERATION 2-3 IS TO BE REPEATED.
C *** IF IPR23=0 DISREGARD THIS INPUT.
C
C *** INPUT 2-6
C       READ(5,94) NOPAR
C  94 FORMAT(I2)
C       NOPAR=THE NUMBER OF THE RAINSTORM PARAMETER FOR WHICH OPERATION
C       2-3 WILL BE CARRIED OUT.
C       (NUMBERING OF RAINSTORM PARAMETERS IS DESCRIBED AT THE END OF
C       THIS SECTION.)
C       ONE CARD FOR EACH REPETITION.
C       THE TOTAL NUMBER OF CARDS IS EQUAL TO NUMT.
C *** IF IPR23=0 DISREGARD THIS INPUT.
C
C       OPERATION 2-4 ...
C
C *** INPUT 2-7
C 104 READ(5,126) IPR24
C 126 FORMAT(I1)
C *** IPR24=1 IF OPERATION 2-4 IS TO BE PERFORMED.OTHERWISE,IPR24=0
```

```
C
C *** INPJT 2-8
C       READ(5,127) NUMT
C 127 FORMAT(I2)
C       NUMT=NUMBER OF TIMES OPERATION 2-4 IS TO BE REPEATED.
C *** IF IPR24=0 DISREGARD THIS INPUT.
C
C *** INPUT 2-9
C       READ(5,107) NOPAR,XLL,XUL
C 107 FORMAT(I2,8X,2F10.0)
C       NOPAR=THE NUMBER OF THE RAINSTORM PARAMETER FOR WHICH OPERATION
C       2-4 WILL BE CARRIED OUT.
C       XLL=LOWER LIMIT FOR THE VALUES OF THE PARAMETER CONSIDERED.
C       XUL=UPPER LIMIT FOR THE VALUES OF THE PARAMETER CONSIDERED.
C       ONLY THOSE VALUES .GE. XLL AND .LT. XUL ARE COSIDERED.
C       ONE CARD FOR EACH REPETITION.
C       THE TOTAL NUMBER OF CARDS IS EQUAL TO NUMT.
C *** IF IPR24=0 DISREGARD THIS INPUT.
C
C-----------------------------------------------------------------
C
C       PROGRAM 3 ...
C
C *** INPJT 3-1
C       READ(5,150) IPR3
C 150 FORMAT(I1)
C       IPR3=1 IF EXECUTION OF PROGRAM 3 IS DESIRED.OTHERWISE,IPR3=0
C *** IF IPR3=0 NO OTHER INPUT CARDS SHOULD BE USED FOR PROGRAM 3
C
C       OPERATION 3-1 ...
C
C *** INPUT 3-2
C       READ(5,152) IPR31
C 152 FORMAT(I1)
C *** IPR31=1 IF OPERATION 3-1 IS TO BE PERFORMED.OTHERWISE,IPR31=0
C
C *** INPUT 3-3
C       READ(5,154) NUMT
C 154 FORMAT(I2)
C       NUMT=NUMBER OF TIMES OPERATION 3-1 IS TO BE REPEATED.
C *** IF IPR31=0 DISREGARD THIS INPUT.
C
C *** INPUT 3-4
C       READ(5,155) NOPAR,UBO(1),UBO(3),UBO(2)
C 155 FORMAT(I2,8X,3F10.0)
C       NOPAR=THE NUMBER OF THE RAINSTORM PARAMETER FOR WHICH OPERATION
C       3-1 WILL BE CARRIED OUT.
C       UBO(1)=LOWER LIMIT OF THE FIRST CLASS INTERVAL.
```

```
C     UBO(3)=UPPER LIMIT [illegible]ST CLASS INTERVAL.
C     UBO(2)=NUMBER OF CLASS INTERVALS.
C     IF UBO(1)=UBO(3),THE PROGRAM USES MINIMUM AND MAXIMUM VALUES
C     OF THE RAINSTORM PARAMETER AS UBO(1) AND UBO(3),RESPECTIVELY.
C     UBO(2) MUST INCLUDE TWO CLASS INTERVALS FOR THE VALUES UNDER
C     AND ABOVE LIMITS.
C     INTERVAL SIZE IS COMPUTED AS FOLLOWS
C
C               XINTSZ=(UBO(3)-UBO(1))/(UBO(2)-2.)
C
C     A COUNT IS CLASSIFIED INTO A PARTICULAR INTERVAL IF THE VALUE IS
C     .GE. THE LOWER LIMIT OF THAT INTERVAL BUT .LT. THE UPPER LIMIT
C     OF THE SAME INTERVAL.
C     ONE CARD FOR EACH REPETITION.
C     THE TOTAL NUMBER OF CARDS IS EQUAL TO NUMT.
C *** IF IPR31=0 DISREGARD THIS INPUT.
C
C     OPERATION 3-2 ...
C
C *** INPUT 3-5
C 153 READ(5,184) IPR32
C 184 FORMAT(I1)
C *** IPR32=1 IF OPERATION 3-2 IS TO BE PERFORMED.OTHERWISE,IPR32=0
C
C *** INPUT 3-6
C     READ(5,185) NUMT
C 185 FORMAT(I2)
C     NUMT=NUMBER OF TIMES OPERATION 3-2 IS TO BE REPEATED.
C *** IF IPR32=0 DISREGARD THIS INPUT.
C
C *** INPUT 3-7
C     READ(5,187) NOPARB,NOPAR,XLL,XUL,UBO(1),UBO(3),UBO(2)
C 187 FORMAT(2I2,6X,5F10.0)
C     ONE-WAY FREQUENCY ANALYSIS WILL BE PERFORMED ON A SUBSET OF VALUES
C     OF RAINSTORM PARAMETER NOPAR CORRESPONDING TO THE SUBSET OF VALUES
C     OF RAINSTORM PARAMETER NOPARB WHICH ARE .GE. XLL AND .LT. XUL
C     UBO(1),UBO(2),UBO(3) ARE AS DEFINED FOR INPUT 3-4
C     ONE CARD FOR EACH REPETITION.
C     THE TOTAL NUMBER OF CARDS IS EQUAL TO NUMT.
C *** IF IPR32=0 DISREGARD THIS INPUT.
C
C-----------------------------------------------------------------
C
C     PROGRAM 4 ...
C
C *** INPUT 4-1
C     READ(5,200) IPR4
C 200 FORMAT(I1)
```

```
C     IPR4=1 IF EXECUTION OF PROGRAM 4 IS DESIRED.OTHERWISE,IPR4=0
C *** IF IPR4=0 NO OTHER INPUT CARDS SHOULD BE USED FOR PROGRAM 4
C
C *** INPUT 4-2
C     READ(5,203) NUMT
C 203 FORMAT(I2)
C *** NUMT=NUMBER OF TIMES PROGRAM 4 IS TO BE EXECUTED.
C
C *** INPUT 4-3
C     READ(5,205) NOV(1),UBO2(1,1),UBO2(3,1)
C     READ(5,205) NOV(2),UBO2(1,2),UBO2(3,2)
C 205 FORMAT(I2,8X,2F10.0)
C     NOV(1)=THE NUMBER OF THE FIRST RAINSTORM PARAMETER TO BE
C            CROSS-TABULATED.
C     NOV(2)=THE NUMBER OF THE SECOND RAINSTORM PARAMETER TO BE
C            CROSS-TABULATED.
C     UBO2(1,J)=LOWER LIMIT OF THE FIRST CLASS INTERVAL FOR THE
C               J TH VARIABLE, J=1,2
C     UBO2(3,J)=UPPER LIMIT OF THE LAST CLASS INTERVAL FOR THE
C               J TH VARIABLE, J=1,2
C     IF UOB2(1,J)=UBO2(3,J), THE PROGRAM USES MINIMUM AND MAXIMUM
C     VALUES OF THE VARIABLE J AS UBO2(1,J) AND UBO2(3,J),RESPECTIVELY.
C
C     UBO2(2,J)=NUMBER OF CLASS INTERVALS FOR THE J TH VARIABLE, J=1,2
C     UBO2(2,J) MUST INCLUDE FOR EACH VARIABLE TWO CLASS INTERVALS FOR
C     THE VALUES UNDER AND ABOVE LIMITS.
C     IN THIS PROGRAM,NUMBER OF CLASS INTERVALS FOR BOTH VARIABLES IS
C     EQUAL TO 20.THAT IS,UBO2(2,1)=UBO2(2,2)=20.HOWEVER,DESIRED
C     INTERVAL SIZES FOR VARIABLES CAN BE OBTAINED BY PROPER
C     CHOICE OF CORRESPONDING UBO2(1,J) AND UBO2(3,J) VALUES.
C     INTERVAL SIZE FOR EACH VARIABLE IS COMPUTED AS FOLLOWS
C
C                (UBO2(3,J)-UBO2(1,J))/(UBO2(2,J)-2.)     J=1,2
C
C     FOR EACH VARIABLE,A COUNT IS CLASSIFIED INTO A PARTICULAR INTERVAL
C     IF THE VALUE IS .GE. THE LOWER LIMIT OF THAT INTERVAL BUT .LT. THE
C     UPPER LIMIT OF THE SAME INTERVAL.
C     TWO CARDS FOR EACH REPETITION.
C *** THE TOTAL NUMBER OF CARDS IS EQUAL TO 2*NUMT.
C
C-----------------------------------------------------------------------
C
C     PREPERATION OF THE CARD DECK OF HOURLY PRECIPITATION DATA.
C
C   * THE MAIN INPUT TO THE FREQUENCY ANALYSIS PROGRAM IS THE CARD DECK
C     OF HOURLY PRECIPITATION DATA.
C
C   * THE DECK SHOULD BE PREPARED AS FOLLOWS
```

```
C
C     (1)   FOR EACH DAY WITH PRECIPITATION,2 CARDS ARE PUNCHED
C           AS FOLLOWS
C
C     CARD NO. 1
C     COLUMNS
C
C      1-6  LEAVE BLANK OR USE FOR STATION IDENTIFICATION PURPOSES.
C           IN U.S.A. NATIONAL CLIMATIC CENTER FORMAT,COLS.1-2 ARE USED
C           TO PUNCH STATE CODE AND COLS.3-6 ARE USED TO PUNCH STATION
C           NUMBER.
C      7-8  LAST TWO DIGITS OF THE YEAR (55=1955).
C      9-10 MONTH NO.  (01-12=JANUARY-DECEMBER).
C     11-12 DAY OF THE MONTH (01-31).
C       13  CARD NUMBER (=1).
C     14-16 PRECIPITATION AMOUNT FOR HOUR ENDING 01 LOCAL STANDARD TIME.
C           VALUES ARE PUNCHED AS INTEGERS TO ONE-HUNDREDTHS OF AN INCH
C           OR TO ONE-TENTHS OF A MILLIMETEP (021=0.21 IN. OR 2.1 MM).
C           FOR NO PRECIPITATION (THAT IS LESS THAN 0.01 IN. OR 0.1 MM),
C           LEAVE BLANK OR PUNCH '-BB',WHERE B INDICATES BLANK.THE
C           LATTER IS USA-NCC PRACTICE.
C       .
C       .
C       .
C     47-49 PRECIPITATION AMOUNT FOR HOUR ENDING 12 LST.
C     50-80 BLANK.
C
C     CARD NO. 2
C     COLUMNS
C
C      1-6  SAME AS IN CARD 1.
C      7-8  SAME AS IN CARD 1.
C      9-10 SAME AS IN CARD 1.
C     11-12 SAME AS IN CARD 1.
C      13   CARD NUMBER (=2).
C     14-16 PRECIPITATION AMOUNT FOR HOUR ENDING 13 LST.
C       .
C       .
C       .
C     47-49 PRECIPITATION AMOUNT FOR HOUR ENDING 24 LST.
C     50-53 LEAVE BLANK OR PUNCH DAILY TOTAL.LATTER IS USA-NCC PRACTICE.
C     54-57 LEAVE BLANK OR PUNCH MONTHLY TOTAL ON THE LAST DAY OF THE
C           MONTH WITH PRECIPITATION.LATTER IS USA-NCC PRACTICE.
C     58-78 BLANK.
C     79-80 NEXT DAY WITH PRECIPITATION.FOR THE LAST DAY OF THE MONTH
C           WITH PRECIPITATION,THIS IS '01'.
C
C     (2)   FOR FIRST DAY OF THE MONTH, THE TWO CARDS SHOULD ALWAYS BE
```

```
C          PUNCHED. IF THERE IS NO PRECIPITATION FOR THAT DAY,DAILY
C          TOTAL CAN BE PUNCHED AS '-BBB'.IF THERE IS NO PRECIPITATION
C          FOR THAT MONTH,MONTHLY TOTAL CAN BE PUNCHED AS '-BBB'.
C          THESE TWO ARE USA-NCC PRACTICES.
C     (3)  HOURLY PRECIPITATION DATA FOR RECORDING RAINGAGES IN THE
C          U.S. WEATHER SERVICE SYSTEM ARE AVAILABLE AT COST ON PUNCHED
C          CARDS FROM U.S. DEPARTMENT OF COMMERCE,NATIONAL OCEANIC AND
C          ATMOSPHERIC ADMINISTRATION,ENVIRONMENTAL DATA SERVICE,
C          NATIONAL CLIMATIC CENTER,FEDERAL BUILDING,ASHEVILLE,
C          N.C. 28801.
C          SINCE THEY USE THE ABOVE FORMAT, THE CARD DECK OBTAINED FROM
C          THEM CAN DIRECTLY BE FED INTO THE PROGRAM.HOWEVER,IT SHOULD
C          FIRST BE CHECKED TO CORRECT MISTAKES AND FILL IN INCOMPLETE
C          INFORMATION.
C
C     (4)  THE ANALYSIS IS CARRIED OUT ON SEASONAL BASIS.THE LENGTH OF
C          A SEASON IS AT LEAST 1 MONTH AND AT MOST 12 MONTHS.A SEASON
C          CAN EXTEND OVER TWO CALENDAR YEARS.THE CARD DECK SHOULD
C          CONTAIN ONLY THE DATA FOR THE SEASONS AND THE YEARS TO BE
C          ANALYZED.THE DATA SHOULD BE PLACED IN TIME ORDER.ANY SEASON
C          HAVING MISSING DATA SHOULD BE COMPLETELY REMOVED FROM
C          THE DECK.
C
C-----------------------------------------------------------------------
C
C     NUMBERING OF RAINSTORM PARAMETERS.
C
C
C     RAINSTORM PARAMETERS
C
C     NO.  DESCRIPTION
C
C      1   TIME BETWEEN RAINSTORMS,HR
C      2   RAINSTORM DURATION,HR (TIME BETWEEN THIS RAINSTORM AND THE
C          PREVIOUS ONE IS GIVEN BY RAINSTORM PARAMETER NO. 1)
C      3   TOTAL DEPTH OF RAINSTORM,MM OR IN.
C      4   AVERAGE INTENSITY OF RAINSTORM,MM/HR OR IN./HR
C      5   STANDARD DEVIATION OF RAINSTORM DEPTH,MM OR IN.
C      6   FIRST MOMENT ARM OF THE HYETOGRAPH WITH RESPECT TO THE
C          BEGINNING TIME OF THE RAINSTORM,HR
C      7   SECOND MOMENT ARM OF THE HYETOGRAPH WITH RESPECT TO THE
C          BEGINNING TIME OF THE RAINSTORM,HR.SQ.
C      8   DIMENSION A FOR TRIANGULAR REPRESENTATION OF HYETOGRAPH,HR
C      9   DIMENSION B OF TRIANGLE,HR
C     10   DIMENSION H OF TRIANGLE,MM OR IN.
C     11   NONDIMENSIONAL STANDARD DEVIATION OF RAINSTORM DEPTH
C     12   FIRST MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH
C     13   SECOND MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH
```

```
C       14     DIMENSION A FOR TRIANGULAR REPRESENTATION OF THE
C              NONDIMENSIONAL HYETOGRAPH
C       15     DIMENSION B FOR TRIANGULAR REPRESENTATION OF
C              NONDIMENSIONAL HYETOGRAPH
C       16     DIMENSION H FOR TRIANGULAR REPRESENTATION OF
C              NONDIMENSIONAL HYETOGRAPH
C
C***********************************************************************
C
C       PROGRAM 1 ...
C
        DIMENSION X(2500),D(50),P(50,16),PA(500,16),PA1(500,16),A(500)
        DIMENSION B(500),FREQ(150),PCT(150),DEN(150),CDEN(150),XX(150)
        DIMENSION MONTH(12),KYR(12),KMO(12),KNUMDY(12),IT(24),IP(16)
        DIMENSION UBO(3),STATS(5),NOV(2),UBO2(3,2),STAT1(3,20),STAT2(3,20)
        DIMENSION XINSZ(2),FREQ1(20,20),PCT1(20,20),DEN1(20,20),XX1(21)
        DIMENSION XX2(21)
C
C *** INPUT 1-1
        READ(5,57) IDIMX,IDIMPA
   57 FORMAT(2I4)
C ***
C
C *** INPUT 1-2
        READ(5,1) IUNIT
        READ(5,1) IPRSI
    1 FORMAT(I1)
C ***
C
C *** INPUT 1-3
        READ(5,2) NUMMON
    2 FORMAT(I2)
        READ(5,3) (MONTH(I),I=1,NUMMON)
    3 FORMAT(I2)
C ***
C
C *** INPUT 1-4
        READ(5,4) NUMYR,LASTYR
    4 FORMAT(2I2)
C ***
C
C *** INPUT 1-5
        READ(5,5) IPR1
    5 FORMAT(I1)
C ***
C
        WRITE(6,6)
    6 FORMAT('1')
```

```
      WRITE(6,7) NUMYR
    7 FORMAT(//////////43X,'FREQUENCY ANALYSIS OF HOURLY PRECIPITATION D
     1ATA'//////57X,I2,' YEARS OF RECORD'/////63X,'MONTHS'/)
      WRITE(6,8) (MONTH(I),I=1,NUMMON)
    8 FORMAT(65X,I2)
      WRITE(6,6)
      WRITE(6,60)
   60 FORMAT(/1X,'RAINSTORM PARAMETERS ( ENGLISH UNITS )'/)
      WRITE(6,48)
   48 FORMAT(2X,'1.TIME BETWEEN RAINSTORMS,HR'/2X,'2.RAINSTORM DURATION,
     1HR'/2X,'3.TOTAL DEPTH OF RAINSTORM,IN'/2X,'4.AVERAGE INTENSITY OF
     2RAINSTORM,IN/HR'/2X,'5.STANDARD DEVIATION OF RAINSTORM DEPTH,IN'/2
     3X,'6.FIRST MOMENT ARM OF THE HYETOGRAPH WITH RESPECT TO THE BEGINN
     4ING TIME OF THE RAINSTORM,HR'/2X,'7.SECOND MOMENT ARM OF THE HYETO
     5GRAPH WITH RESPECT TO THE BEGINNING TIME OF THE RAINSTORM,HR.SQ.')
      WRITE(6,49)
   49 FORMAT(2X,'8.DIMENSION A FOR TRIANGULAR REPRESENTATION OF HYETOGRA
     1PH,HR'/2X,'9.DIMENSION B OF TRIANGLE,HR'/1X,'10.DIMENSION H OF TRI
     2ANGLE,IN'/1X,'11.NONDIMENSIONAL STANDARD DEVIATION OF RAINSTORM DE
     3PTH'/1X,'12.FIRST MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH'/1X,
     4'13.SECOND MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH'/1X,'14.DIM
     5ENSION A FOR TRIANGULAR REPRESENTATION OF THE NONDIMENSIONAL HYETO
     6GRAPH'/1X,'15.DIMENSION B FOR TRIANGULAR REPRESENTATION OF NONDIME
     7NSIONAL HYETOGRAPH'/1X,'16.DIMENSION H FOR TRIANGULAR REPRESENTATI
     8ON OF NONDIMENSIONAL HYETOGRAPH'///)
      WRITE(6,61)
   61 FORMAT(////1X,'RAINSTORM PARAMETERS ( SI UNITS )'/)
      WRITE(6,38)
   38 FORMAT(2X,'1.TIME BETWEEN RAINSTORMS,HR'/2X,'2.RAINSTORM DURATION,
     1HR'/2X,'3.TOTAL DEPTH OF RAINSTORM,MM'/2X,'4.AVERAGE INTENSITY OF
     2RAINSTORM,MM/HR'/2X,'5.STANDARD DEVIATION OF RAINSTORM DEPTH,MM'/2
     3X,'6.FIRST MOMENT ARM OF THE HYETOGRAPH WITH RESPECT TO THE BEGINN
     4ING TIME OF THE RAINSTORM,HR'/2X,'7.SECOND MOMENT ARM OF THE HYETO
     5GRAPH WITH RESPECT TO THE BEGINNING TIME OF THE RAINSTORM,HR.SQ.')
      WRITE(6,39)
   39 FORMAT(2X,'8.DIMENSION A FOR TRIANGULAR REPRESENTATION OF HYETOGRA
     1PH,HR'/2X,'9.DIMENSION B OF TRIANGLE,HR'/1X,'10.DIMENSION H OF TRI
     2ANGLE,MM'/1X,'11.NONDIMENSIONAL STANDARD DEVIATION OF RAINSTORM DE
     3PTH'/1X,'12.FIRST MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH'/1X,
     4'13.SECOND MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH'/1X,'14.DIM
     5ENSION A FOR TRIANGULAR REPRESENTATION OF THE NONDIMENSIONAL HYETO
     6GRAPH'/1X,'15.DIMENSION B FOR TRIANGULAR REPRESENTATION OF NONDIME
     7NSIONAL HYETOGRAPH'/1X,'16.DIMENSION H FOR TRIANGULAR REPRESENTATI
     8ON OF NONDIMENSIONAL HYETOGRAPH'///)
C
      MC3=0
C
   55 DO 9 I=1,IDIMX
```

```
    9 X(I)=0.0
C
      I1=0
      I2=0
      I3=0
   10 I2=I1+I2
      IF(I2.EQ.0) GO TO 20
      IF(INXDY.EQ.1.AND.IMO.EQ.MONTH(NUMMON)) GO TO 11
   20 IF(IUNIT.EQ.2) GO TO 12
C
C *** INPUT 1-6
      READ(5,13) IYR,IMO,IDY,(X(I+I2),I=1,24),INXDY
   13 FORMAT(6X,3I2,1X,12F3.2/13X,12F3.2,29X,I2)
C ***
C
      GO TO 14
C
C *** INPUT 1-6
   12 READ(5,15) IYR,IMO,IDY,(X(I+I2),I=1,24),INXDY
   15 FORMAT(6X,3I2,1X,12F3.1/13X,12F3.1,29X,I2)
C ***
C
   14 I1=24*(INXDY-IDY)
      IF(I1.GT.0) GO TO 10
      GO TO(16,18,16,17,16,17,16,16,17,16,17,16),IMO
   16 NUMDY=31
      GO TO 19
   17 NUMDY=30
      GO TO 19
   18 X1=IYR
      X2=IYR/4
      X3=X1/4.
      IF(X2.EQ.X3) NUMDY=29
      IF(X2.NE.X3) NUMDY=28
   19 I1=(NUMDY+1-IDY)*24
      I3=I3+1
      KYR(I3)=IYR
      KMO(I3)=IMO
      KNUMDY(I3)=NUMDY
      GO TO 10
C
   11 IC=0
      LC=0
      MC=1
      NC=1
      DO 59 I=1,I2
      IF(X(I).GT.0.) GO TO 21
      IF(NC.NE.MC) GO TO 22
```

```
      IC=IC+1
      GO TO 59
   21 IF(LC.GT.0) GO TO 23
      P(MC,1)=IC
      NC=MC+1
      IC=1
   23 LC=LC+1
      D(LC)=X(I)
      GO TO 59
   22 P(MC,2)=LC
      SUM1=0.
      DO 24 J=1,LC
   24 SUM1=SUM1+D(J)
      P(MC,3)=SUM1
      P(MC,4)=P(MC,3)/P(MC,2)
      SUM2=0.
      SUM3=0.
      SUM4=0.
      DO 25 J=1,LC
      XJ=J
      SUM2=SUM2+D(J)*(XJ-0.5)
      SUM3=SUM3+D(J)*(XJ-0.5)**2
   25 SUM4=SUM4+(D(J)-P(MC,4))**2
      P(MC,5)=SQRT(SUM4/P(MC,2))
      P(MC,6)=SUM2/P(MC,2)
      P(MC,7)=SUM3/P(MC,3)+1./12.
      P(MC,8)=3.*P(MC,6)-P(MC,2)
      P(MC,9)=P(MC,2)-P(MC,8)
      P(MC,10)=2.*P(MC,4)
      P(MC,11)=P(MC,5)/P(MC,3)
      P(MC,12)=P(MC,6)/P(MC,2)
      P(MC,13)=P(MC,7)/(P(MC,2)**2)
      P(MC,14)=P(MC,8)/P(MC,2)
      P(MC,15)=P(MC,9)/P(MC,2)
      P(MC,16)=P(MC,10)/P(MC,3)
      MC=MC+1
      LC=0
   59 CONTINUE
C
      MC1=MC-1
      MC2=MC-2
      MC3=MC3+MC2
      DO 26 I=1,16
      DO 26 J=2,MC1
   26 PA(MC3-MC2+J-1,I)=P(J,I)
C
      DO 40 I=1,16
   40 IP(I)=I
```

```
C
      IF(IPR1.EQ.0) GO TO 27
C
      I9=0
      WRITE(6,6)
      DO 29 I=1,24
   29 IT(I)=I
C
      IF(IUNIT.EQ.1) GO TO 28
C
   54 WRITE(6,30)
   30 FORMAT(1X,'HOURLY PRECIPITATION DATA.'/)
      WRITE(6,31)
   31 FORMAT(1X,'VALUES ARE IN MILLIMETERS.'///)
      WRITE(6,32)
   32 FORMAT(67X,'HOUR ENDING'/)
      WRITE(6,33) (IT(I),I=1,24)
   33 FORMAT(11X,24(3X,I2)/)
C
   44 CONTINUE
      I7=0
      DO 34 I=1,I3
      I4=KYR(I)
      I5=KMO(I)
      I6=KNUMDY(I)
      IF(I.EQ.1) GO TO 35
      I7=I7+KNUMDY(I-1)
   35 CONTINUE
      DO 34 J=1,I6
      I8=(J-1+I7)*24
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 45
      WRITE(6,36) I4,I5,J,(X(K+I8),K=1,24)
   36 FORMAT (1X,I2,1X,I2,1X,I2,3X,24F5.1)
      GO TO 34
   45 WRITE(6,46) I4,I5,J,(X(K+I8),K=1,24)
   46 FORMAT (1X,I2,1X,I2,1X,I2,3X,24F5.2)
   34 CONTINUE
C
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 47
      WRITE(6,6)
      WRITE(6,62)
   62 FORMAT(1X,'VALUES OF RAINSTORM PARAMETERS IN SI UNITS.'//)
      GO TO 50
   47 WRITE(6,6)
      WRITE(6,63)
   63 FORMAT(1X,'VALUES OF RAINSTORM PARAMETERS IN ENGLISH UNITS.'//)
   50 CONTINUE
      WRITE(6,41) (IP(I),I=1,16)
```

```
   41 FORMAT(4X,16(I2,6X)/)
C
      WRITE(6,42) ((P(I,J),J=1,16),I=2,MC1)
   42 FORMAT(16(1X,F7.2))
      WRITE(6,56) MC2
   56 FORMAT(///1X,'NUMBER OF RAINSTORMS=',I4)
C
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 51
      GO TO 27
C
   28 WRITE(6,30)
      WRITE(6,43)
   43 FORMAT(1X,'VALUES ARE IN INCHES.'///)
      WRITE(6,32)
      WRITE(6,33) (IT(I),I=1,24)
      I9=1
      GO TO 44
   51 IF(IPRSI.EQ.0) GO TO 27
      I9=2
      DO 52 I=1,I2
   52 X(I)=X(I)*25.4
      DO 53 I=2,MC1
      P(I,3)=P(I,3)*25.4
      P(I,4)=P(I,4)*25.4
      P(I,5)=P(I,5)*25.4
   53 P(I,10)=P(I,10)*25.4
      WRITE(6,6)
      GO TO 54
C
   27 IF(IYR.LT.LASTYR) GO TO 55
C
C***************************************************************************
C***************************************************************************
C
C     PROGRAM 2 ...
C
C *** INPUT 2-1
      READ(5,80) IPR2
   80 FORMAT (I1)
C ***
C
      IF(IPR2.EQ.0) GO TO 105
C
      DO 83 J=1,16
      DO 83 I=1,MC3
   83 PA1(I,J)=PA(I,J)
C
C     OPERATION 2-1 ...
```

```
C
C *** INPUT 2-2
      READ(5,81) IPR21
   81 FORMAT(I1)
C ***
C
      IF(IPR21.EQ.0) GO TO 90
C
      I10=1
      I9=0
      K1=MC3
      GO TO 93
C
C     OPERATION 2-2 ...
C
C *** INPUT 2-3
   90 READ(5,82) IPR22
   82 FORMAT(I1)
C ***
C
      IF(IPR22.EQ.0) GO TO 98
C
      I10=2
      I9=0
      K1=MC3
      DO 95 J=1,16
      DO 96 I=1,K1
      A(I)=PA(I,J)
   96 B(I)=PA(I,J)
      CALL SORT(A(1),K1,B(1))
      DO 97 I=1,K1
   97 PA1(I,J)=A(I)
   95 CONTINUE
      GO TO 93
C
C     OPERATION 2-3 ...
C
C *** INPUT 2-4
   98 READ(5,85) IPR23
   85 FORMAT(I1)
C ***
C
      IF(IPR23.EQ.0) GO TO 104
C
      I10=3
      K1=MC3
      I11=0
C
```

```
C *** INPUT 2-5
      READ(5,99) NUMT
   99 FORMAT(I2)
C ***
C
  103 I9=0
      IF(I11.GE.NUMT) GO TO 104
C
C *** INPUT 2-6
      READ(5,94) NOPAR
   94 FORMAT(I2)
C ***
C
      DO 100 J=1,16
      DO 101 I=1,K1
      A(I)=PA(I,NOPAR)
  101 B(I)=PA(I,J)
      CALL SORT(A(1),K1,B(1))
      DO 102 I=1,K1
  102 PA1(I,J)=B(I)
  100 CONTINUE
      I11=I11+1
      GO TO 93
C
C     OPERATION 2-4 ...
C
C *** INPUT 2-7
  104 READ(5,126) IPR24
  126 FORMAT(I1)
C ***
C
      IF(IPR24.EQ.0) GO TO 105
C
      I10=4
      I11=0
C
C *** INPUT 2-8
      READ(5,127) NUMT
  127 FORMAT(I2)
C ***
C
  106 I9=0
      IF(I11.GE.NUMT) GO TO 105
C
C *** INPUT 2-9
      READ(5,107) NOPAR,XLL,XUL
  107 FORMAT(I2,8X,2F10.0)
C ***
```

```
C
      K1=0
      DO 108 I=1,MC3
      COUNT=0.
      IF(PA(I,NOPAR).GE.XLL.AND.PA(I,NOPAR).LT.XUL) COUNT=1.
      IF(COUNT.EQ.0.) GO TO 108
      K1=K1+1
      DO 109 J=1,16
  109 PA1(K1,J)=PA(I,J)
  108 CONTINUE
      DO 121 J=1,16
      IF(J.EQ.NOPAR) GO TO 121
      DO 122 I=1,K1
      A(I)=PA1(I,NOPAR)
  122 B(I)=PA1(I,J)
      CALL SORT(A(1),K1,B(1))
      DO 123 I=1,K1
  123 PA1(I,J)=B(I)
  121 CONTINUE
      DO 129 I=1,K1
  129 PA1(I,NOPAR)=A(I)
      I11=I11+1
C
   93 IF(IUNIT.EQ.1) I9=1
   91 WRITE(6,6)
      GO TO(110,111,112,113),I10
  110 WRITE(6,114)
  114 FORMAT(1X,'VALUES OF RAINSTORM PARAMETERS.'///)
      GO TO 120
  111 WRITE(6,115)
  115 FORMAT(1X,'VALUES OF RAINSTORM PARAMETERS SORTED IN ASCENDING ORDE
     1R.'///)
      GO TO 120
  112 WRITE(6,116) NOPAR
  116 FORMAT(1X,'VALUES OF RAINSTORM PARAMETER ',I2,' SORTED IN ASCENDIN
     1G ORDER AND CORRESPONDING VALUES OF OTHER PARAMETERS REARRANGED SI
     2MULTANEOUSLY.'///)
      GO TO 120
  113 IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 118
      IF(IUNIT.EQ.2) GO TO 125
      IF(NOPAR.NE.3.AND.NOPAR.NE.4.AND.NOPAR.NE.5.AND.NOPAR.NE.10) GO TO
     1125
      XLL=XLL*25.4
      XUL=XUL*25.4
  125 WRITE(6,117) NOPAR,XLL,XUL
  117 FORMAT(1X,'VALUES OF RAINSTORM PARAMETER ',I2,' GREATER THAN OR EQ
     1UAL TO ',F7.2,' AND LESS THAN ',F7.2,' (IN SI UNITS)'/1X,'SORTED I
     2N ASCENDING ORDER AND CORRESPONDING VALUES OF OTHER PARAMETERS REA
```

```
     3RRANGED SIMULTANEOUSLY.'////)
      GO TO 120
  118 WRITE(6,119) NOPAR,XLL,XUL
  119 FORMAT(1X,'VALUES OF RAINSTORM PARAMETER ',I2,' GREATER THAN OR EQ
     1UAL TO ',F7.2,' AND LESS THAN ',F7.2,' (IN EN UNITS)'/1X,'SORTED I
     2N ASCENDING ORDER AND CORRESPONDING VALUES OF OTHER PARAMETERS REA
     3RRANGED SIMULTANEOUSLY.'////)
  120 CONTINUE
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 87
      WRITE(6,130)
  130 FORMAT(1X,'( SI UNITS )'////)
      GO TO 88
   87 WRITE(6,131)
  131 FORMAT(1X,'( ENGLISH UNITS )'////)
   88 CONTINUE
      WRITE(6,41) (IP(I),I=1,16)
      WRITE(6,42) ((PA1(I,J),J=1,16),I=1,K1)
      WRITE(6,56) K1
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 89
   84 GO TO (90,98,103,106),I10
   89 IF(IPRSI.EQ.0) GO TO 84
      I9=2
      DO 92 I=1,K1
      PA1(I,3)=PA1(I,3)*25.4
      PA1(I,4)=PA1(I,4)*25.4
      PA1(I,5)=PA1(I,5)*25.4
   92 PA1(I,10)=PA1(I,10)*25.4
      GO TO 91
C
  105 CONTINUE
C
C***********************************************************************
C***********************************************************************
C
C     PROGRAM 3 ...
C
C *** INPUT 3-1
      READ(5,150) IPR3
  150 FORMAT(I1)
C ***
C
      IF(IPR3.EQ.0) GO TO 151
C
      DO 156 I=1,IDIMPA
  156 A(I)=0.0
C
C     OPERATION 3-1 ...
C
```

```
C *** INPUT 3-2
      READ(5,152) IPR31
  152 FORMAT(I1)
C ***
C
      IF(IPR31.EQ.0) GO TO 153
C
      K1=MC3
      I10=1
      I11=0
C
C *** INPUT 3-3
      READ(5,154) NUMT
  154 FORMAT(I2)
C ***
C
  183 I9=0
      IF(I11.GE.NUMT) GO TO 153
C
C *** INPUT 3-4
      READ(5,155) NOPAR,UBO(1),UBO(3),UBO(2)
  155 FORMAT(I2,8X,3F10.0)
C ***
C
      DO 157 I=1,MC3
  157 A(I)=1.0
      CALL TAB1(PA,A,NOPAR,UBO,FREQ,PCT,STATS,IDIMPA,16)
      I11=I11+1
      GO TO 177
C
C     OPERATION 3-2 ...
C
C *** INPUT 3-5
  153 READ(5,184) IPR32
  184 FORMAT(I1)
C ***
C
      IF(IPR32.EQ.0) GO TO 151
C
      I10=2
      I11=0
C
C *** INPUT 3-6
      READ(5,185) NUMT
  185 FORMAT(I2)
C ***
C
  186 I9=0
```

```
      IF(I11.GE.NUMT) GO TO 151
C
C *** INPUT 3-7
      READ(5,187) NOPARB,NOPAR,XLL,XUL,UBO(1),UBO(3),UBO(2)
  187 FORMAT(2I2,6X,5F10.0)
C ***
C
      K1=0
      DO 188 I=1,MC3
      IF(PA(I,NOPARB).GE.XLL.AND.PA(I,NOPARB).LT.XUL) A(I)=1.0
      IF(A(I).EQ.0.) GO TO 188
      K1=K1+1
  188 CONTINUE
      CALL TAB1(PA,A,NOPAR,UBO,FREQ,PCT,STATS,IDIMPA,16)
      I11=I11+1
C
  177 XINTSZ=(UBO(3)-UBO(1))/(UBO(2)-2.)
      IUBO2=UBO(2)
      SUM=0.
      DO 158 I=1,IUBO2
      PCT(I)=0.01*PCT(I)
      DEN(I)=PCT(I)/XINTSZ
      SUM=SUM+PCT(I)
  158 CDEN(I)=SUM
C
  160 IF(IUNIT.EQ.1) I9=1
  161 WRITE(6,6)
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 169
      WRITE(6,170)
  170 FORMAT(61X,'( SI UNITS )'//)
      GO TO 171
  169 WRITE(6,172)
  172 FORMAT(58X,'( ENGLISH UNITS )'//)
  171 WRITE(6,159) NOPAR
  159 FORMAT(1X,'FREQUENCY ANALYSIS FOR RAINSTORM PARAMETER ',I2/)
      IF(I10.EQ.1) GO TO 162
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 164
      IF(IUNIT.EQ.2) GO TO 164
      IF(NOPARB.NE.3.AND.NOPARB.NE.4.AND.NOPARB.NE.5.AND.NOPARB.NE.10)
     1GO TO 164
      XLL=XLL*25.4
      XUL=XUL*25.4
  164 WRITE(6,165) NOPARB,XLL,XUL
  165 FORMAT(1X,'ONLY VALUES CORRESPONDING TO VALUES OF RAINSTORM PARAME
     1TER ',I2,' GREATER THAN OR EQUAL TO ',F7.2,' AND LESS THAN ',F7.2,
     2' CONSIDERED.'/)
  162 CONTINUE
      WRITE(6,167) UBO(1),UBO(3),UBO(2),XINTSZ
```

```
  167 FORMAT(//1X,'LOWER LIMIT=',F7.2,5X,'UPPER LIMIT=',F7.2,5X,'NUMBER
     1OF CLASS INTERVALS=',F4.0,5X,'INTERVAL SIZE=',F7.2//)
      WRITE(6,168) STATS(4),STATS(5),STATS(2),STATS(3)
  168 FORMAT(1X,'MINIMUM=',F7.2,5X,'MAXIMUM=',F7.2,5X,'MEAN=',F7.2,5X,
     1'STANDARD DEVIATION=',F7.2///)
      DO 173 I=2,IUBO2
  173 XX(I)=UBO(1)+(I-2)*XINTSZ
      XX(1)=100000.
      XX(IUBO2+1)=100000.
      WRITE(6,175)
  175 FORMAT(7X,'INTERVAL',11X,'FREQ',4X,'REL FREQ',4X,'PROB DEN',7X,'NO
     1N-EXCE PROB'/)
      WRITE(6,176) (XX(I),XX(I+1),FREQ(I),PCT(I),DEN(I),CDEN(I),I=1,IUBO
     12)
  176 FORMAT(1X,F7.2,3X,F7.2,5X,F7.1,5X,F6.4,5X,F8.4,10X,F6.4)
      WRITE(6,56) K1
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 178
  179 GO TO 180
  178 IF(IPRSI.EQ.0) GO TO 179
      I9=2
      IF(NOPAR.NE.3.AND.NOPAR.NE.4.AND.NOPAR.NE.5.AND.NOPAR.NE.10) GO TO
     1161
      UBO(1)=UBO(1)*25.4
      UBO(3)=UBO(3)*25.4
      XINTSZ=XINTSZ*25.4
      STATS(2)=STATS(2)*25.4
      STATS(3)=STATS(3)*25.4
      STATS(4)=STATS(4)*25.4
      STATS(5)=STATS(5)*25.4
      DO 181 I=1,IUBO2
  181 DEN(I)=PCT(I)/XINTSZ
      GO TO 161
  180 DO 182 I=1,IDIMPA
  182 A(I)=0.0
      GO TO (183,186),I10
C
  151 CONTINUE
C
C***********************************************************************
C***********************************************************************
C
C     PROGRAM 4 ...
C
C *** INPUT 4-1
      READ(5,200) IPR4
  200 FORMAT(I1)
C ***
C
```

```
      IF(IPR4.EQ.0) GO TO 201
C
      DO 202 I=1,IDIMPA
  202 A(I)=0.0
      I11=0
C
C *** INPUT 4-2
      READ(5,203) NUMT
  203 FORMAT(I2)
C ***
C
  204 I9=0
      IF(I11.GE.NUMT) GO TO 201
C
C *** INPUT 4-3
      READ(5,205) NOV(1),UBO2(1,1),UBO2(3,1)
      READ(5,205) NOV(2),UBO2(1,2),UBO2(3,2)
  205 FORMAT(I2,8X,2F10.0)
C ***
C
      UBO2(2,1)=20.
      UBO2(2,2)=20.
C
      DO 206 I=1,MC3
  206 A(I)=1.0
      CALL TAB2(PA,A,NOV,UBO2,FREQ1,PCT1,STAT1,STAT2,IDIMPA,16)
      I11=I11+1
      XINSZ(1)=(UBO2(3,1)-UBO2(1,1))/(UBO2(2,1)-2.)
      XINSZ(2)=(UBO2(3,2)-UBO2(1,2))/(UBO2(2,2)-2.)
      SUM=0.
      DO 207 I=1,20
      DO 207 J=1,20
      PCT1(I,J)=0.01*PCT1(I,J)
  207 DEN1(I,J)=PCT1(I,J)/(XINSZ(1)*XINSZ(2))
C
  208 IF(IUNIT.EQ.1) I9=1
  209 WRITE(6,6)
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 210
      WRITE(6,236)
  236 FORMAT(61X,'( SI UNITS )'//)
      GO TO 211
  210 WRITE(6,237)
  237 FORMAT(58X,'( ENGLISH UNITS )'//)
  211 WRITE(6,212) NOV(1),NOV(2)
  212 FORMAT(1X,'TWO-WAY FREQUENCY ANALYSIS FOR RAINSTORM PARAMETERS ',
     1I2,' AND ',I2///)
      DO 213 I=1,2
      WRITE(6,214) NOV(I),UBO2(1,I),UBO2(3,I),UBO2(2,I),XINSZ(I)
```

```
214 FORMAT(1X,'RAINSTORM PARAMETER=',I2/1X,'LOWER LIMIT=',F7.2,5X,'UPP
   1ER LIMIT=',F7.2,5X,'NUMBER OF CLASS INTERVALS=',F4.0,5X,'INTERVAL
   2SIZE=',F7.2//)
213 CONTINUE
    DO 216 I=1,21
    IF(I.EQ.1.OR.I.EQ.21) GO TO 217
    XX1(I)=UBO2(1,1)+(I-2)*XINSZ(1)
    GO TO 216
217 XX1(I)=100000.
216 CONTINUE
    DO 218 I=1,21
    IF(I.EQ.1.OR.I.EQ.21) GO TO 219
    XX2(I)=UBO2(1,2)+(I-2)*XINSZ(2)
    GO TO 218
219 XX2(I)=100000.
218 CONTINUE
    DO 215 K=1,4
    L=(K-1)*5
    WRITE(6,220)
220 FORMAT(///64X,'FREQUENCIES'//)
    WRITE(6,221) NOV(2)
221 FORMAT(5X,'RAINSTORM',45X,'RAINSTORM PARAMETER ',I2/)
    WRITE(6,222) NOV(1),(XX2(I+L),XX2(I+1+L),I=1,5)
222 FORMAT(4X,'PARAMETER ',I2,7X,5(F7.2,2X,F7.2,3X)/)
    WRITE(6,223) (XX1(I),XX1(I+1),FREQ1(I,1+L),FREQ1(I,2+L),FREQ1(I,3+
   1L),FREQ1(I,4+L),FREQ1(I,5+L),I=1,20)
223 FORMAT(1X,F7.2,3X,F7.2,9X,F10.4,9X,F10.4,9X,F10.4,9X,F10.4,9X,F10.
   14)
215 CONTINUE
    DO 224 K=1,4
    L=(K-1)*5
    WRITE(6,225)
225 FORMAT(///59X,'RELATIVE FREQUENCIES'//)
    WRITE(6,221) NOV(2)
    WRITE(6,222) NOV(1),(XX2(I+L),XX2(I+1+L),I=1,5)
    WRITE(6,223) (XX1(I),XX1(I+1), PCT1(I,1+L), PCT1(I,2+L), PCT1(I,3+
   1L), PCT1(I,4+L), PCT1(I,5+L),I=1,20)
224 CONTINUE
    DO 226 K=1,4
    L=(K-1)*5
    WRITE(6,227)
227 FORMAT(///59X,'PROBABILITY DENSITIES'//)
    WRITE(6,221) NOV(2)
    WRITE(6,222) NOV(1),(XX2(I+L),XX2(I+1+L),I=1,5)
    WRITE(6,223) (XX1(I),XX1(I+1), DEN1(I,1+L), DEN1(I,2+L), DEN1(I,3+
   1L), DEN1(I,4+L), DEN1(I,5+L),I=1,20)
226 CONTINUE
    WRITE(6,56) MC3
```

```
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 228
  229 GO TO 230
  228 IF(IPRSI.EQ.0) GO TO 229
      I9=2
      IF(NOV(1).NE.3.AND.NOV(1).NE.4.AND.NOV(1).NE.5.AND.NOV(1).NE.10)
     1GO TO 231
      UBO2(1,1)=UBO2(1,1)*25.4
      UBO2(3,1)=UBO2(3,1)*25.4
      XINSZ(1)=XINSZ(1)*25.4
  231 IF(NOV(2).NE.3.AND.NOV(2).NE.4.AND.NOV(2).NE.5.AND.NOV(2).NE.10)
     1GO TO 232
      UBO2(1,2)=UBO2(1,2)*25.4
      UBO2(3,2)=UBO2(3,2)*25.4
      XINSZ(2)=XINSZ(2)*25.4
  232 DO 233 I=1,20
      DO 233 J=1,20
  233 DEN1(I,J)=PCT1(I,J)/(XINSZ(1)*XINSZ(2))
      GO TO 209
  230 DO 234 I=1,IDIMPA
  234 A(I)=0.0
      GO TO 204
C
  201 CONTINUE
C
      STOP
      END
```

```
C***********************************************************************
C
C       SUBROUTINE SORT
C
C       IDENTIFICATION
C          SORTS A REAL ARRAY A AND REARRANGES SIMULTANEOUSLY
C          THE CORRESPONDING ELEMENTS OF AN ASSOCIATED REAL ARRAY B.
C
C       PURPOSE
C          TO SORT JJ ELEMENTS OF A REAL ARRAY A (BEGINNING AT A(I) AS
C          SPECIFIED BY THE USER) IN ASCENDING ORDER.IN ADDITION,
C          THE CORRESPONDING JJ ELEMENTS OF AN ASSOCIATED REAL ARRAY B
C          (BEGINNING AT B(L) AS SPECIFIED BY THE USER) ARE REARRANGED
C          SIMULTANEOUSLY. SORT ALLOWS SORTING UP TO 2**22-1 ELEMENTS.
C
C       USAGE
C          CALL SORT(A(I),JJ,B(L))
C
C       DESCRIPTION OF PARAMETERS
C          A(I) . -ELEMENT OF ARRAY A AT WHICH SORTING IS TO BEGIN.
C                                                          (INPUT-OUTPUT)
C          JJ     -NUMBER OF ELEMENTS OF A,BEGINNING AT A(I),TO BE SORTED.
C                                                                 (INPUT)
C          B(L)   -THE JJ ELEMENTS OF B,BEGINNING AT B(L),ARE REARRANGED
C                  SIMULTANEOUSLY.
C                                                          (INPUT-OUTPUT)
C
C       REMARKS
C          IF B IS AN INTEGER ARRAY THEN DELETE STATEMENT 'REAL NT,NTT'
C          AND ADD A NEW STATEMENT 'INTEGER B'.
C
C***********************************************************************
C
      SUBROUTINE SORT(A,JJ,B)
      REAL NT,NTT
      DIMENSION IU(21),IL(21),A(JJ),B(JJ)
      M=1
      II=1
      I=II
      J=JJ
5     IF(I.GE.J) GO TO 70
10    K=I
      IJ=(J+I)/2
      T=A(IJ)
      IF(A(I).LE.T) GO TO 20
      NT=B(IJ)
      A(IJ)=A(I)
      B(IJ)=B(I)
```

```
      A(I)=T
      B(I)=NT
      T=A(IJ)
20    L=J
      IF(A(J).GE.T)GO TO 40
      NT=B(IJ)
      A(IJ)=A(J)
      B(IJ)=B(J)
      A(J)=T
      B(J)=NT
      T=A(IJ)
      IF(A(I).LE.T) GO TO 40
      NT=B(IJ)
      A(IJ)=A(I)
      B(IJ)=B(I)
      A(I)=T
      B(I)=NT
      T=A(IJ)
      GO TO 40
30    NTT=B(L)
      A(L)=A(K)
      B(L)=B(K)
      A(K)=TT
      B(K)=NTT
40    L=L-1
      IF(A(L).GT.T) GO TO 40
      TT=A(L)
50    K=K+1
      IF(A(K).LT.T) GO TO 50
      IF(K.LE.L) GO TO 30
      IF(L-I.LE.J-K) GO TO 60
      IL(M)=I
      IU(M)=L
      I=K
      M=M+1
      GO TO 80
60    IL(M)=K
      IU(M)=J
      J=L
      M=M+1
      GO TO 80
70    M=M-1
      IF(M.EQ.0) RETURN
      I=IL(M)
      J=IJ(M)
80    IF(J-I.GE.11) GO TO 10
      IF(I.EQ.II) GO TO 5
      I=I-1
```

```
90    I=I+1
      IF(I.EQ.J) GO TO 70
      T=A(I+1)
      IF(A(I).LE.T) GO TO 90
      NT=B(I+1)
      K=I
100   A(K+1)=A(K)
      B(K+1)=B(K)
      K=K-1
      IF(T.LT.A(K)) GO TO 100
      A(K+1)=T
      B(K+1)=NT
      GO TO 90
      END
```

```
C*********************************************************************************
C
C      SUBROUTINE TAB1
C
C      PURPOSE
C         TO TABULATE FOR A GIVEN VARIABLE IN AN OBSERVATION MATRIX,
C         THE FREQUENCIES (NUMBER OF OBSERVATIONS) AND PERCENT
C         FREQUENCIES OVER GIVEN CLASS INTERVALS. IN ADDITION, FOR THE
C         SAME VARIABLE, TOTAL, MEAN, STANDARD DEVIATION, MINIMUM, AND
C         MAXIMUM ARE CALCULATED.
C
C      USAGE
C         CALL TAB1(A,S,NOVAR,UBO,FREQ,PCT,STATS,NO,NV)
C
C      DESCRIPTION OF PARAMETERS
C         A      - INPUT MATRIX OF OBSERVATIONS, NO BY NV
C         S      - INPUT VECTOR SPECIFIYING OBSERVATIONS TO BE CONSIDERED.
C                  ONLY THOSE OBSERVATIONS WITH A CORRESPONDING NON-ZERO
C                  S(I) ARE CONSIDERED. VECTOR LENGTH IS NO
C         NOVAR  - THE VARIABLE TO BE TABULATED.
C         UBO    - INPUT VECTOR OF LENGTH 3
C                     UBO(1)= LOWER LIMIT OF THE FIRST CLASS INTERVAL.
C                     UBO(2)= NUMBER OF CLASS INTERVALS.
C                     UBO(3)= UPPER LIMIT OF THE LAST CLASS INTERVAL.
C                  IF UBO(1)=UBO(3), THE PROGRAM USES THE MINIMUM AND
C                  THE MAXIMUM VALUES OF THE VARIABLE AS UBO(1), AND
C                  UBO(3), RESPECTIVELY.
C                  UBO(2) MUST INCLUDE TWO CLASS INTERVALS FOR THE VALUES
C                  UNDER AND ABOVE LIMITS.
C         FREQ   - OUTPUT VECTOR OF FREQUENCIES. VECTOR LENGTH IS UBO(2)
C         PCT    - OUTPUT VECTOR OF PERCENT FREQUENCIES. VECTOR LENGTH
C                  IS UBO(2)
C         STATS  - OUTPUT VECTOR OF SUMMARY STATISTICS. VECTOR LENGTH IS 5
C                     STATS(1)= TOTAL
C                     STATS(2)= MEAN
C                     STATS(3)= STANDARD DEVIATION
C                     STATS(4)= MINIMUM
C                     STATS(5)= MAXIMUM
C         NO     - NUMBER OF OBSERVATIONS.
C         NV     - NUMBER OF VARIABLES.
C
C      REMARKS
C         INTERVAL SIZE IS COMPUTED AS FOLLOWS
C                           (UBO(3)-UBO(1))/(UBO(2)-2.)
C         A COUNT IS CLASSIFIED INTO A PARTICULAR INTERVAL IF THE VALUE
C         IS .GE. THE LOWER LIMIT OF THAT INTERVAL BUT .LT. THE UPPER
C         LIMIT OF THE SAME INTERVAL.
C         THE DIVISOR FOR STANDARD DEVIATION IS ONE LESS THAN THE NUMBER
```

```
C         OF OBSERVATIONS USED.
C         IF S IS A NULL VECTOR, THEN TOTAL, MEAN, AND STANDARD
C         DEVIATION = 0, MIN=1.E75 AND MAX=-1.E75
C         SUBROUTINE TAB1 IS IN IBM SYSTEM/360 SCIENTIFIC SUBROUTINE
C         PACKAGE VERSION III.
C
C**********************************************************************
C
      SUBROUTINE TAB1(A,S,NOVAR,UBO,FREQ,PCT,STATS,NO,NV)
      DIMENSION A(1),S(1),UBO(1),FREQ(1),PCT(1),STATS(1)
      DIMENSION WBO(3)
      DO 5 I=1,3
    5 WBO(I)=UBO(I)
C
C          CALCULATE MIN AND MAX
C
      VMIN=1.0E75
      VMAX=-1.0E75
      IJ=NO*(NOVAR-1)
      DO 30 J=1,NO
      IJ=IJ+1
      IF(S(J)) 10,30,10
   10 IF(A(IJ)-VMIN) 15,20,20
   15 VMIN=A(IJ)
   20 IF(A(IJ)-VMAX) 30,30,25
   25 VMAX=A(IJ)
   30 CONTINUE
      STATS(4)=VMIN
      STATS(5)=VMAX
C
C          DETERMINE LIMITS
C
      IF(UBO(1)-UBO(3)) 40,35,40
   35 UBO(1)=VMIN
      UBO(3)=VMAX
   40 INN=UBO(2)
C
C          CLEAR OUTPUT AREAS
C
      DO 45 I=1,INN
      FREQ(I)=0.0
   45 PCT(I)=0.0
      DO 50 I=1,3
   50 STATS(I)=0.0
C
C          CALCULATE INTERVAL SIZE
C
      SINT=ABS((UBO(3)-UBO(1))/(UBO(2)-2.0))
```

```
C
C        TEST SUBSET VECTOR
C
      SCNT=0.0
      IJ=NO*(NOVAR-1)
      DO 75 J=1,NO
      IJ=IJ+1
      IF(S(J)) 55,75,55
   55 SCNT=SCNT+1.0
C
C        DEVELOP TOTAL AND FREQUENCIES
C
      STATS(1)=STATS(1)+A(IJ)
      STATS(3)=STATS(3)+A(IJ)*A(IJ)
      TEMP=UBO(1)-SINT
      INTX=INN-1
      DO 60 I=1,INTX
      TEMP=TEMP+SINT
      IF(A(IJ)-TEMP) 70,60,60
   60 CONTINUE
      IF(A(IJ)-TEMP) 75,65,65
   65 FREQ(INN)=FREQ(INN)+1.0
      GO TO 75
   70 FREQ(I)=FREQ(I)+1.0
   75 CONTINUE
      IF (SCNT)79,105,79
C
C        CALCULATE PERCENT FREQUENCIES
C
   79 DO 80 I=1,INN
   80 PCT(I)=FREQ(I)*100.0/SCNT
C
C        CALCULATE MEAN AND STANDARD DEVIATION
C
      IF(SCNT-1.0) 85,85,90
   85 STATS(2)=STATS(1)
      STATS(3)=0.0
      GO TO 95
   90 STATS(2)=STATS(1)/SCNT
      STATS(3)=SQRT(ABS((STATS(3)-STATS(1)*STATS(1)/SCNT)/(SCNT-1.0)))
   95 DO 100 I=1,3
  100 UBO(I)=WBO(I)
  105 RETURN
      END
```

```
C*****************************************************************************
C
C     SUBROUTINE TAB2
C
C     PURPOSE
C        TO PERFORM A TWO-WAY CLASSIFICATION FOR TWO GIVEN VARIABLES IN
C        AN OBSERVATION MATRIX, OF FREQUENCIES (NUMBER OF OBSERVATIONS),
C        PERCENT FREQUENCIES, AND SOME STATISTICS OVER GIVEN CLASS
C        INTERVALS.
C
C     USAGE
C        CALL TAB2(A,S,NOV,UBO,FREQ,PCT,STAT1,STAT2,NO,NV)
C
C     DESCRIPTION OF PARAMETERS
C        A      - INPUT MATRIX OF OBSERVATIONS, NO BY NV
C        S      - INPUT VECTOR SPECIFIYING OBSERVATIONS TO BE CONSIDERED.
C                 ONLY THOSE OBSERVATIONS WITH A CORRESPONDING NON-ZERO
C                 S(I) ARE CONSIDERED. VECTOR LENGTH IS NO
C        NOV    - INPUT VECTOR OF LENGTH 2
C                    NOV(1)= THE NUMBER OF THE FIRST VARIABLE
C                            TO BE CROSS-TABULATED.
C                    NOV(2)= THE NUMBER OF THE SECOND VARIABLE
C                            TO BE CROSS-TABULATED.
C        UBO    - INPUT MATRIX OF LENGTH 3 BY 2
C                    UBO(1,J)= LOWER LIMIT OF THE FIRST CLASS INTERVAL
C                              FOR THE J TH VARIABLE, J=1,2
C                    UBO(2,J)= NUMBER OF CLASS INTERVALS FOR THE
C                              J TH VARIABLE, J=1,2
C                    UBO(3,J)= UPPER LIMIT OF THE LAST CLASS INTERVAL
C                              FOR THE J TH VARIABLE, J=1,2
C                 IF UBO(1,J)=UBO(3,J), THE PROGRAM USES THE MINIMUM AND
C                 THE MAXIMUM VALUES OF THE VARIABLE J AS UBO(1,J) AND
C                 UBO(3,J), RESPECTIVELY.
C                 UBO(2,J) MUST INCLUDE FOR EACH VARIABLE TWO CLASS
C                 INTERVALS FOR THE VALUES UNDER AND ABOVE LIMITS.
C        FREQ   - OUTPUT MATRIX OF TWO-WAY CLASSIFICATION OF FREQUENCIES.
C                 ORDER OF MATRIX IS INT1 BY INT2, WHERE INT1=UBO(2,1)
C                 AND INT2=UBO(2,2)
C        PCT    - OUTPUT MATRIX OF TWO-WAY CLASSIFICATION OF PERCENT
C                 FREQUENCIES. SAME ORDER AS FREQ
C        STAT1  - OUTPUT MATRIX SUMMARIZING TOTALS, MEANS, AND STANDARD
C                 DEVIATIONS FOR EACH CLASS INTERVAL OF VARIABLE 1
C                 ORDER OF MATRIX IS 3 BY INT1
C        STAT2  - SAME AS STAT1 BUT FOR VARIABLE 2
C                 ORDER OF MATRIX IS 3 BY INT2
C        NO     - NUMBER OF OBSERVATIONS.
C        NV     - NUMBER OF VARIABLES.
C     REMARKS
```

```
C         INTERVAL SIZE FOR EACH VARIABLE IS COMPUTED AS FOLLOWS
C                        (UBO(3,J)-UBO(1,J))/(UBO(2,J)-2.)
C         FOR EACH VARIABLE, A COUNT IS CLASSIFIED INTO A PARTICULAR
C         INTERVAL IF THE VALUE IS .GE. THE LOWER LIMIT OF THAT INTERVAL
C         BUT .LT. THE UPPER LIMIT OF THE SAME INTERVAL.
C         THE DIVISOR FOR STANDARD DEVIATION IS ONE LESS THAN THE NUMBER
C         OF OBSERVATIONS USED.
C         IF S IS A NULL VECTOR, OUTPUT AREAS ARE SET TO ZERO.
C         SUBROUTINE TAB2 IS IN IBM SYSTEM/360 SCIENTIFIC SUBROUTINE
C         PACKAGE VERSION III.
C
C***********************************************************************
C
      SUBROUTINE TAB2(A,S,NOV,UBO,FREQ,PCT,STAT1,STAT2,NO,NV)
      DIMENSION A(1),S(1),NOV(2),UBO(3,2),FREQ(1),PCT(1),STAT1(1),
     1STAT2(2),SINT(2)
      DIMENSION WBO(3,2)
      DO 5 I=1,3
      DO 5 J=1,2
    5 WBO(I,J)=UBO(I,J)
C
C         DETERMINE LIMITS
C
      DO 40 I=1,2
      IF(UBO(1,I)-UBO(3,I)) 40, 10, 40
   10 VMIN=1.0E75
      VMAX=-1.0E75
      IJ=NO*(NOV(I)-1)
      DO 35 J=1,NO
      IJ=IJ+1
      IF(S(J)) 15,35,15
   15 IF(A(IJ)-VMIN) 20,25,25
   20 VMIN=A(IJ)
   25 IF(A(IJ)-VMAX) 35,35,30
   30 VMAX=A(IJ)
   35 CONTINUE
      UBO(1,I)=VMIN
      UBO(3,I)=VMAX
   40 CONTINUE
C
C         CALCULATE INTERVAL SIZE
C
   45 DO 50 I=1,2
   50 SINT(I)=ABS((UBO(3,I)-UBO(1,I))/(UBO(2,I)-2.0))
C
C         CLEAR OUTPUT AREAS
C
      INT1=UBO(2,1)
```

```
      INT2=UBO(2,2)
      INTT=INT1*INT2
      DO 55 I=1,INTT
      FREQ(I)=0.0
   55 PCT(I)=0.0
      INTY=3*INT1
      DO 60 I=1,INTY
   60 STAT1(I)=0.0
      INTZ=3*INT2
      DO 65 I=1,INTZ
   65 STAT2(I)=0.0
C
C         TEST SUBSET VECTOR
C
      SCNT=0.0
      INTY=INT1-1
      INTX=INT2-1
      IJ=NO*(NOV(1)-1)
      IJX=NO*(NOV(2)-1)
      DO 95 J=1,NO
      IJ=IJ+1
      IJX=IJX+1
      IF(S(J)) 70,95,70
   70 SCNT=SCNT+1.0
C
C         CALCULATE FREQUENCIES
C
      TEMP1=UBO(1,1)-SINT(1)
      DO 75 IY=1,INTY
      TEMP1=TEMP1+SINT(1)
      IF(A(IJ)-TEMP1) 80,75,75
   75 CONTINUE
      IY=INT1
   80 IYY=3*(IY-1)+1
      STAT1(IYY)=STAT1(IYY)+A(IJ)
      IYY=IYY+1
      STAT1(IYY)=STAT1(IYY)+1.0
      IYY=IYY+1
      STAT1(IYY)=STAT1(IYY)+A(IJ)*A(IJ)
      TEMP2=UBO(1,2)-SINT(2)
      DO 85 IX=1,INTX
      TEMP2=TEMP2+SINT(2)
      IF(A(IJX)-TEMP2) 90,85,85
   85 CONTINUE
      IX=INT2
   90 IJF=INT1*(IX-1)+IY
      FREQ(IJF)=FREQ(IJF)+1.0
      IX=3*(IX-1)+1
```

```
      STAT2(IX)=STAT2(IX)+A(IJX)
      IX=IX+1
      STAT2(IX)=STAT2(IX)+1.0
      IX=IX+1
      STAT2(IX)=STAT2(IX)+A(IJX)*A(IJX)
   95 CONTINUE
      IF (SCNT)98,151,98
C
C        CALCULATE PERCENT FREQUENCIES
C
   98 DO 100 I=1,INTT
  100 PCT(I)=FREQ(I)*100.0/SCNT
C
C        CALCULATE TOTALS, MEANS, STANDARD DEVIATIONS
C
      IXY=-1
      DO 120 I=1,INT1
      IXY=IXY+3
      ISD=IXY+1
      TEMP1=STAT1(IXY)
      SUM=STAT1(IXY-1)
      IF(TEMP1-1.0) 120,105,110
  105 STAT1(ISD)=0.0
      GO TO 115
  110 STAT1(ISD)=SQRT(ABS((STAT1(ISD)-SUM*SUM/TEMP1)/(TEMP1-1.0)))
  115 STAT1(IXY)=SUM/TEMP1
  120 CONTINUE
      IXX=-1
      DO 140 I=1,INT2
      IXX=IXX+3
      ISD=IXX+1
      TEMP2=STAT2(IXX)
      SUM=STAT2(IXX-1)
      IF(TEMP2-1.0) 140,125,130
  125 STAT2(ISD)=0.0
      GO TO 135
  130 STAT2(ISD)=SQRT(ABS((STAT2(ISD)-SUM*SUM/TEMP2)/(TEMP2-1.0)))
  135 STAT2(IXX)=SUM/TEMP2
  140 CONTINUE
      DO 150 I=1,3
      DO 150 J=1,2
  150 UBO(I,J)=WBO(I,J)
  151 RETURN
      END
```

APPENDIX B

LISTING OF COMPUTER PROGRAM FOR THE ILLINOIS SURFACE RUNOFF MODEL

The Illinois Surface Runoff Model is programmed in Fortran IV language for computer solutions. The input to the computer program is the drainage basin characteristics and the rainfall hyetographs. The output is the catchment hydrographs and pollutographs which serve as the input to the sewer system.

The computer program allows the consideration of a maximum number of 100 gutters at a time. Along each gutter, the subcatchments can be approximated by as many as 10 rectangular strips. As many as five different zones of rainfall can be considered for the entire basin. Quality computations for two different pollutants are performed at a time. The computations can be proceeded for as many as 100 time steps. The storage requirement for the computer program in its present form is 400K. It can be modified to consider larger basins by changing the arrays in DIMENSION statements if more storage is available.

A listing of the computer program of the Illinois Surface Runoff Model is given below.

```
C
C     MAIN PROGRAM FOR ILLINOIS SURFACE RUNOFF MODEL
C
C     MAIN PROGRAM PERFORMS THE ROUTING COMPUTATIONS FOR QUANTITY AND
C     QUALITY OF GUTTER FLOW
C
      COMMON/Z1/TEMP,IRYN,RYNST,RYNEND,DAYRYN,TM,RAIN
      COMMON/Z2/FFIN,FINS,CINF,QRAIN,QRSI
      COMMON/Z3/XNU,C1,FK,SO,ULL,NOV,YOIN,DT,QOVERL
      COMMON/Z4/QCACH,T8A,GR8TYP,W,GR8L,GD,T,VP,YP,QP,FLOLAT,OPR
      COMMON/Z5/SG,RAFCUF,YING,COTTH,SINTH,COSTH,TANTH,YUPST,QUPST
      COMMON/Z20/CONIN1,CONIN2,COVER1,COVER2
      COMMON/Z6/K1,K2,K3,K4,K6,OR1,OR2,OR3,OR4,OR5,OR6,K5
      COMMON/Z7/I,GTRTYP,B,TIME,QUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
     *CIC1,CIC2
      COMMON/Z8/G,L1,TYM
      COMMON/Z9/JTOTAL,INLET,LIMIT,TYMEND,DTM,NSW,SWRTYP,BYSFLO
      COMMON/Z10/L2,L3,NDTM,NGTR
      COMMON/Z11/GL
      COMMON/Z12/CJ1,CJ2
      DIMENSION CJ1(100),CJ2(100)
      DIMENSION RYNST(5),RYNEND(5),TM(5,100),RAIN(5,100),DAYRYN(5)
      DIMENSION NOKTA(5)
      DIMENSION QCACH(100,100),T8A(100),GR8TYP(100),W(100),GR8L(100)
      DIMENSION GTRTYP(100),B(100),TIME(100),QUP(100,100),OPR(100),NGTR(
     *100)
      DIMENSION SG(100),RAFCUF(100),YING(100)
      DIMENSION K1(100),K2(100),K3(100),K4(100),K5(100),K6(100)
      DIMENSION OR1(100),OR2(100),OR3(100),OR4(100),OR5(100),OR6(100)
      DIMENSION QO(11),QN(11),YO(11),YN(11)
      DIMENSION NO(100,10),FFINAL(100,10),FINSHL(100,10),CINFLT(100,10)
      DIMENSION DK(100,10),DS(100,10),OL(100,10),YINO(100,10)
      DIMENSION IRNZON(100),N(100),PL(100),FFINP(100),FINSP(100)
      DIMENSION CGTR1(100),CGTR2(100),CO1(100,10),CO2(100,10)
      DIMENSION CCC1(100,100),CCC2(100,100)
      DIMENSION DUMMY(100),GL(100),FFING(100),FINSG(100),CINFG(100)
      DIMENSION INLET(100,10),NSW(100),SWRTYP(100),BYSFLO(100)
      DIMENSION H(100),CINFP(100)
      INTEGER UNIT
C
C     FOLLOWING ARE THE STATEMENT FUNCTIONN DESCRIBING CROSS-SECTIONAL
C     PROPERTIES AND THE FRICTION SLOPE EVALUATION FOR TWO TYPES OF GUTTER
C
      A1(E)=0.5*E**2/COTTH
      T1(E)=E/COTTH
      R1(E)=0.5*E*SINTH/(1.+COSTH)
      A1P(E)=E/COTTH
      T1P(E)=1./COTTH
      R1P(E)=0.5*SINTH/(1.0+COSTH)
      A1PP(E)=1./COTTH
      R1PP(E)=0.
      R1PPP (E)=0.
      A2(E)=U*E-0.5*COTTH*U**2
      T2(E)=U
      R2(E)=(U*E-0.5*COTTH*U**2)/(E+(U/SINTH))
      A2P(E)=U
      R2P(E)=(U**2/SINTH+0.5*COTTH*U**2)/(E+(U/SINTH))**2
      T2P(E)=0.0
      A2PP(E)=0.
      R2PP(E)=(U**2/SINTH+0.5*COTTH*U**2)*(-2./(E+U/SINTH)**3)
      R2PPP(E)=6.*(U**2/SINTH+COTTH*U**2/2.)/(E+(U/SINTH))**4
      A3(E)=U*E
```

```
 T3(E)=U
 R3(E)=U*E/(U+2.*E)
 A3P(E)=U
 T3P(E)=0.0
 R3P(E)=U**2/(U+2.*E)**2
 A3PP(E)=0.
 R3PP(E)=U**2*(-4./(U+2.*E)**3)
 R3PPP(E)=24.*U**2/(U+2.*E)**4
 A4(E)=U*E
 T4(E)=U
 R4(E)=U*E/(U+HHHH+E)
 A4P(E)=U
 R4P(E)=(U**2+U*HHHH)/(U+HHHH+E)**2
 T4P(E)=0.0
 A4PP(E)=0.0
 R4PP(E)=0.0
 R4PPP(E)=6.*(U**2+U*HHHH)/(U+HHHH+E)**4
 G1(E)=COF*(.67*A1(E)*R1P(E)/R1(E)**.33+A1P(E)*R1(E)**.67)
 G2(E)=COF*(.67*A2(E)*R2P(E)/R2(E)**.33+A2P(E)*R2(E)**.67)
 G3(E)=COF*(.67*A3(E)*R3P(E)/R3(E)**.33+A3P(E)*R3(F)**.67)
 G4(E)=COF*(.67*A4(E)*R4P(E)/R4(E)**.33+A4P(E)*R4(F)**.67)
 G1P(E)=COF*(A1PP(E)*R1(E)**.67+1.33*A1P(E)*R1P(E)/R1(E)**.33-.22*A
*1(E)*R1P(E)**2/R1(E)**1.33+.67*A1(E)*R1PP(E)/R1(E)**.33)
 G2P(E)=COF*(A2PP(E)*R2(E)**.67+1.33*A2P(E)*R2P(E)/R2(E)**.33-.22*A
*2(E)*R2P(E)**2/R2(E)**1.33+.67*A2(E)*R2PP(E)/R2(E)**.33)
 G3P(E)=COF*(A3PP(E)*R3(E)**.67+1.33*A3P(E)*R3P(E)/R3(E)**.33-.22*A
*3(E)*R3P(E)**2/R3(E)**1.33+.67*A3(E)*R3PP(E)/R3(E)**.33)
 G4P(E)=COF*(A4PP(E)*R4(E)**.67+1.33*A4P(E)*R4P(E)/R4(E)**.33-.22*A
*4(E)*R4P(E)**2/R4(E)**1.33+.67*A4(F)*R4PP(E)/R4(E)**.33)
 G1PP(E)=COF*(2.*A1PP(E)*R1P(E)/R1(E)**.33-.67*A1P(E)*R1P(E)**2/R1(
*E)**1.33+2.*A1P(E)*R1PP(E)/R1(E)**.33-.67*A1(E)*R1P(E)*R1PP(E)/R1(
*E)**1.33+.296*A1(E)*R1P(E)**3/R1(E)**2.34+.67*A1(F)*R1PPP(E)/R1(E)
***.33)
 G2PP(E)=COF*(2.*R2P(E)*A2PP(E)/R2(E)**.33-.67*A2P(E)*R2P(E)**2/R2(
*E)**1.33+2.*A2P(E)*R2PP(E)/R2(E)**.33-.67*A2(E)*R2P(F)*R2PP(E)/R2(
*E)**1.33+.296*A2(E)*R2P(E)**3/R2(E)**2.34+.67*A2(E)*R2PPP(E)/R2(E)
***.33)
 G3PP(E)=COF*(2.*R3P(E)*A3PP(E)/R3(E)**.33-.67*A3P(E)*R3P(E)**2/R3(
*E)**1.33+2.*A3P(E)*R3PP(E)/R3(E)**.33-.67*A3(E)*R3P(F)*R3PP(E)/R3(
*E)**1.33+.296*A3(E)*R3P(E)**3/R3(E)**2.34+.67*A3(F)*R3PPP(E)/R3(E)
***.33)
 G4PP(E)=COF*(2.*R4P(E)*A4PP(E)/R4(E)**.33-.67*A4P(E)*R4P(E)**2/R4(
*E)**1.33+2.*A4P(E)*R4PP(E)/R4(E)**.33-.67*A4(E)*R4P(F)*R4PP(E)/R4(
*E)**1.33+.296*A4(E)*R4P(E)**3/R4(E)**2.34+.67*A4(F)*R4PPP(E)/R4(F)
***.33)
 Q1(E)=COF*A1(E)*R1(E)**0.67
 Q2(E)=COF*A2(E)*R2(E)**0.67
 Q3(E)=COF*A3(E)*R3(E)**0.67
 Q4(E)=COF*A4(E)*R4(E)**0.67
 F1(E)=F-(BET-.25*GAM*T1(E)/DT+.5*G1(E)*YD/DX)/(.25*T1(E)/DT+.5*G1(
*E)/DX+ALFA)
 F2(E)=F-(BET-.25*GAM*T2(E)/DT+.5*G2(E)*YD/DX)/(.25*T2(E)/DT+.5*G2(
*E)/DX+ALFA)
 F3(E)=F-(BFT-.25*GAM*T3(E)/DT+.5*G3(E)*YD/DX)/(.25*T3(E)/DT+.5*G3(
*E)/DX+ALFA)
 F4(E)=F-(BET-.25*GAM*T4(E)/DT+.5*G4(E)*YD/DX)/(.25*T4(E)/DT+.5*G4(
*E)/DX+ALFA)
 PAY1(E)=.25*GAM*T1P(E)/DT-.50*G1P(E)*YD/DX
 PAY2(E)=.25*GAM*T2P(E)/DT-.50*G2P(E)*YD/DX
 PAY3(E)=.25*GAM*T3P(E)/DT-.50*G3P(E)*YD/DX
 PAY4(E)=.25*GAM*T4P(E)/DT-.50*G4P(E)*YD/DX
 PA1(E)=(.25*T1P(E)/DT+.5*G1P(E)/DX)*(BET-.25*GAM*T1(E)/DT+.5*YD*G1
```

```
     *(E)/DX)
      PA2(E)=(.25*T2P(E)/DT+.5*G2P(E)/DX)*(BET-.25*GAM*T2(E)/DT+.5*YD*G2
     *(E)/DX)
      PA3(E)=(.25*T3P(E)/DT+.5*G3P(E)/DX)*(BET-.25*GAM*T3(E)/DT+.5*YD*G3
     *(E)/DX)
      PA4(E)=(.25*T4P(E)/DT+.5*G4P(E)/DX)*(BET-.25*GAM*T4(E)/DT+.5*YD*G4
     *(E)/DX)
      PAYD1(E)=0.25*T1(E)/DT+0.5*G1(E)/DX+ALFA
      PAYD2(E)=0.25*T2(E)/DT+0.5*G2(E)/DX+ALFA
      PAYD3(E)=0.25*T3(E)/DT+0.5*G3(E)/DX+ALFA
      PAYD4(E)=0.25*T4(E)/DT+0.5*G4(E)/DX+ALFA
      F1P(E)=1.0+PAY1(E)/PAYD1(E)+PA1(E)/PAYD1(E)**2
      F2P(E)=1.0+PAY2(E)/PAYD2(E)+PA2(E)/PAYD2(E)**2
      F3P(E)=1.0+PAY3(E)/PAYD3(E)+PA3(E)/PAYD3(E)**2
      F4P(E)=1.0+PAY4(E)/PAYD4(E)+PA4(E)/PAYD4(E)**2
      PAY1P(E)=0.5*YD*G1PP(E)/DX*(-1.)
      PAY2P(E)=0.5*YD*G2PP(E)/DX*(-1.)
      PAY3P(E)=0.5*YD*G3PP(E)/DX*(-1.)
      PAY4P(E)=0.5*YD*G4PP(E)/DX*(-1.)
      PA1P(E)=(+.5*G1PP(E)/DX)*(BET-.25*GAM*T1(E)/DT+.5*YD*G1(E)/DX)+(.2
     *5*T1P(E)/DT+.5*G1P(E)/DX)*(-.25*GAM*T1P(E)/DT+.5*YD*G1P(E)/DX)
      PA2P(E)=(+.5*G2PP(E)/DX)*(BET-.25*GAM*T2(E)/DT+.5*YD*G2(E)/DX)+(.2
     *5*T2P(E)/DT+.5*G2P(E)/DX)*(-.25*GAM*T2P(E)/DT+.5*YD*G2P(E)/DX)
      PA3P(E)=(+.5*G3PP(E)/DX)*(BET-.25*GAM*T3(E)/DT+.5*YD*G3(E)/DX)+(.2
     *5*T3P(E)/DT+.5*G3P(E)/DX)*(-.25*GAM*T3P(E)/DT+.5*YD*G3P(E)/DX)
      PA4P(E)=(+.5*G4PP(E)/DX)*(BET-.25*GAM*T4(E)/DT+.5*YD*G4(E)/DX)+(.2
     *5*T4P(E)/DT+.5*G4P(E)/DX)*(-.25*GAM*T4P(E)/DT+.5*YD*G4P(E)/DX)
      PAYD1P(E)=.25*T1P(E)/DT+.5*G1P(E)/DX
      PAYD2P(E)=.25*T2P(E)/DT+.5*G2P(E)/DX
      PAYD3P(E)=.25*T3P(E)/DT+.5*G3P(E)/DX
      PAYD4P(E)=.25*T4P(E)/DT+.5*G4P(E)/DX
      F1PP(E)=(PAY1P(E)*PAYD1(E)-PAYD1P(E)*PAY1(E))/PAYD1(E)**2+(PA1P(E)
     **PAYD1(E)**2-2.*PAYD1(E)*PAYD1P(E)*PA1(E))/PAYD1(E)**4
      F2PP(E)=(PAY2P(E)*PAYD2(E)-PAYD2P(E)*PAY2(E))/PAYD2(E)**2+(PA2P(E)
     **PAYD2(E)**2-2.*PAYD2(E)*PAYD2P(E)*PA2(E))/PAYD2(E)**4
      F3PP(E)=(PAY3P(E)*PAYD3(E)-PAYD3P(E)*PAY3(E))/PAYD3(E)**2+(PA3P(E)
     **PAYD3(E)**2-2.*PAYD3(E)*PAYD3P(E)*PA3(E))/PAYD3(E)**4
      F4PP(E)=(PAY4P(E)*PAYD4(E)-PAYD4P(E)*PAY4(E))/PAYD4(E)**2+(PA4P(E)
     **PAYD4(E)**2-2.*PAYD4(E)*PAYD4P(E)*PA4(E))/PAYD4(E)**4
C
C     FOLOWING IS INPUT DATA FOR THE GENERAL DESCRIPTION OF DRAINAGE BASIN
C
C
C     UNIT=1 FOR ENGLISH SYSTEM,AND UNIT=2 FOR METRIC SYSTEM
C     ITOTAL,JTOTAL,AND NOZONE RESPECTIVELY ARE THE NUMBER OF GUTTERS,
C     THE SEWER NODES,AND THE RAINZONES CONSIDERED
C     NDTM IS THE FREQUENCY OF PRINTED OUTPUT
C     DTM IS TIME INTERVAL OF COMPUTATION
C     TYMEND IS THE STOP EXECUTION TIME
C     G,TEMP,AND XNU RESPECTIVELY ARE THE GRAVITATIONAL ACCELERATION,
C     THE AVERAGE DAILY TEMPERATURE,ANDTHE KINEMATIC VISCOSITY OF WATER
C     C1 IS A CONSTANT USED IN THE FRICTION FACTOR COMPUTATION
C
      READ(5,490)UNIT
  490 FORMAT(I1)
      READ(5,101)ITOTAL,JTOTAL,NOZONE,NDTM,DTM,TYMEND,G,TEMP,XNU,C1
      DTM=DTM*60.
      TYMEND=TYMEND*60.
      IF(UNIT.EQ.1)GO TO 8001
      G=G*32.2/9.81
      TEMP=32.+1.8*TEMP
      XNU=XNU/(304.8*304.8)
```

```
 8001 CONTINUE
  101 FORMAT(4I5,6F10.0)
C
C       FOLLOWING IS RAINFALL DATA
C
C
C       RYNST IS TIME AT WHICH RAIN STARTS
C       RYNEND IS TIME AT WHICH RAIN STOPS
C       DAYRYN IS TOTAL DAILY RAINFALL
C       NOKTA IS THE NUMBER OF POINTS TO DESCRIBE A HYETOGRAPH
C       TM AND RAIN RESPECTIVELY IS THE TIME AND RAINFALL INTENSITY
C
        DO 201 IRYN=1,NUZONE
        READ(5,202)RYNST(IRYN),RYNEND(IRYN),DAYRYN(IRYN),NOKTA(IRYN)
        RYNST(IRYN)=RYNST(IRYN)*60.
        RYNEND(IRYN)=RYNEND(IRYN)*60.
        IF(UNIT.EQ.2)DAYRYN(IRYN)=DAYRYN(IRYN)*0.03937
        NOK=NOKTA(IRYN)
        READ(5,203)(TM(IRYN,JJ),RAIN(IRYN,JJ),JJ=1,NOK)
  203 FORMAT(16F5.0)
        DO 491 NNOK=1,NOK
        TM(IRYN,NNOK)=60.*TM(IRYN,NNOK)
        IF(UNIT.EQ.1)GO TO 491
        RAIN(IRYN,NNOK)=RAIN(IRYN,NNOK)*0.03937
  491 CONTINUE
  201 CONTINUE
C
C       FOLLOWING IS GUTTER,INLET AND SUBCATCHMENT DATA
C
        I=1
C
C       NGTR IS THE GUTTER NUMBER
C       N IS THE NUMBER OF COMPUTATIONAL GRID POINTS FOR GUTTER ROUTING
C       IRNZON IS THE RAIN-ZONE NUMBER THE GUTTER BELONGS TO
C       GTRTYP,GL,B,SG,H,AND TBA ARE RESPECTIVELY THE TYPE,LENGTH,WIDTH,
C       SLOPE,DEPTH AND THE ANGLE BETWEEN THE VERTICAL AND PLANE OF GUTTER
C       GRBTYP,W,GRBL,AND OPR ARE RESPECTIVELY THE TYPE,WIDTH LENGTH AND
C       OPENING RATIO OF GRATE INLET
C       PL IS WIDTH OF STREET PAVEMENT
C       YING IS INITIAL WATER DEPTH IN GUTTER
C       RAFCOF IS MANNING'S FRICTION FACTOR FOR GUTTER
C
   36 READ(5,102)NGTR(I),N(I),IRNZON(I),GTRTYP(I),GL(I),B(I),SG(I),
     *H(I),TBA(I),GRBTYP(I),W(I),GRBL(I),OPR(I),PL(I),YING(I),RAFCOF(I)
        IF(UNIT.EQ.1)GO TO 8002
        GL(I)=GL(I)*3.28
        IF(GTRTYP(I).EQ.0.0)GL(I)=GL(I)*3.28
        YING(I)=YING(I)*3.28
        B(I)=B(I)*3.28
        H(I)=H(I)*3.28
        W(I)=W(I)*3.28
        GRBL(I)=GRBL(I)*3.28
        PL(I)=PL(I)*3.28
 8002 CONTINUE
  102 FORMAT(3I5,13F5.0)
C
C       FINSG AND FFING ARE THE INITIAL AND FINAL INFILTRATION CAPACITY
C       OF GUTTER SURFACE
C       CINFG IS CONSTANT OF DECAY OF INFILTRATION FOR GUTTER SURFACE
C       FINSP,FFINP AND CINFP  ARE THOSE FOR STREET PAVEMENT
C       CGTR1 AND CGTR2 ARE INITIAL CONCENTRATIONS OF 1ST AND 2ND POLLUTANTS
C       IN GUTTER
```

```
C
      READ(5,402)FINSG(I),FFING(I),CINFG(I),FINSP(I),FFINP(I),CINFP(I),
     *CGTR1(I),CGTR2(I)
  402 FORMAT(8F5.0)
      IF(UNIT.EQ.1)GO TO 8003
      FINSG(I)=FINSG(I)*0.03937
      FFING(I)=FFING(I)*0.03937
      FINSP(I)=FINSP(I)*0.03937
      FFINP(I)=FFINP(I)*0.03937
 8003 CONTINUE
C
C     K1,K2,K2,K3,K4,K5,K6 ARE IDENTIFICATION NUMBERS OF SIX IMMEDIATELY
C     UPSTREAM INLETS
C     OR1,OR2,OR3,OR4,OR5,OR6 ARE CARRY-OVER DISTRIBUTION FACTORS
C
      READ(5,502)K1(I),K2(I),K3(I),K4(I),K5(I),K6(I),       OR1(I),OR2(I)
     *,OR3(I),OR4(I),OR5(I),OR6(I)
  502 FORMAT(6I5,6F5.0)
      NNN=N(I)
      DO 1 J=2,NNN
C
C     OL,OS,OK  RESPECTIVELY ARE THE LENGTH SLOPE AND SURFACE ROUGHNESS
C     OF SUBCATCHMENT STRIP
C     FINSHL,FFINAL,CINFLT ARE THE INFILTRATION PARAMETERS FOR SUBCATCHMENT
C     YINO IS INITIAL DEPTH OF WATER IN SUBCATCHMENT
C     CO1 AND CO2 ARE INITIAL CONCENTRATIONS OF THE  1ST AND 2ND
C     POLLUTANTS IN SUBCATCHMENT
C     NO IS NUMBER OF COMPUTATIONAL GRID POINTS ALONG SUBCATCHMENT STRIP
C
      READ(5,103)OL(I,J),OS(I,J),OK(I,J),FINSHL(I,J),FFINAL(I,J),CINFLT(
     *I,J),YINO(I,J),CO1(I,J),CO2(I,J),NO(I,J)
  103 FORMAT(9F5.0,I5)
      IF(UNIT.EQ.1)GO TO 8004
      OL(I,J)=OL(I,J)*3.28
      OK(I,J)=OK(I,J)*3.28
      FINSHL(I,J)=FINSHL(I,J)*0.03937
      FFINAL(I,J)=FFINAL(I,J)*0.03937
      YINO(I,J)=YINO(I,J)*3.28
 8004 CONTINUE
    1 CONTINUE
      IF(I.EQ.ITOTAL)GO TO 712
      I=I+1
      GO TO 36
C
C     FOLLOWING IS INPUT DATA FOR SEWER NODES
C
  712 DO 702 JUNC=1,JTOTAL
C
C     NSW AND SWRTYP ARE IDENTIFICATION NUMBER AND TYPE OF SEWER NODE
C     BYSFLO IS RATE OF BASE FLOW INTO SEWER NODE
C     CJ1 AND CJ2 ARE CONCENTRATIONS OF TWO POLLUTANTS IN BASE FLOW
C     INLET'S ARE IDENTIFICATION NUMBERS OF  GUTTER INLETS DISCHARGING
C     INTO SEWER NODE
C
      READ(5,703)NSW(JUNC),SWRTYP(JUNC),BYSFLO(JUNC),CJ1(JUNC),CJ2(JUNC)
     *,(INLET(JUNC,INL),INL=1,10)
      IF(UNIT.EQ.1)GO TO 8005
      BYSFLO(JUNC)=BYSFLO(JUNC)*3.28*3.28*3.28
 8005 CONTINUE
  202 FORMAT(3F10.0,I5)
  703 FORMAT(I5,4F5.0,10I5)
  702 CONTINUE
```

```
      DO 900 JK=1,7
      DO 113 I=1,ITOTAL
      GO TO(901,902,903,904,905,906,907),JK
  901 IF(K1(I).EQ.0)GO TO 908
      GO TO 113
  902 IF(K2(I).EQ.0.AND.K1(I).NE.0)GO TO 908
      GO TO 113
  903 IF(K3(I).EQ.0.AND.K2(I).NE.0)GO TO 908
      GO TO 113
  904 IF(K4(I).EQ.0.AND.K3(I).NE.0)GO TO 908
      GO TO 113
  905 IF(K5(I).EQ.0.AND.K4(I).NE.0)GO TO 908
      GO TO 113
  906 IF(K6(I).EQ.0.AND.K5(I).NE.0)GO TO 908
      GO TO 113
  907 IF(K6(I).NE.0)GO TO 908
      GO TO 113
  908 CONTINUE
    2 TYM=0.0
      L1 = 0
      L2=NDTM
      L3=1
      GTR=GTRTYP(I)
      IF(GTR.EQ.0.)GO TO 500
      IF(GTR.EQ.2.0)GO TO 888
      SINTH=SIN(TBA(I))
      COSTH=COS(TBA(I))
      COTTH=COSTH/SINTH
      TANTH=TAN(TBA(I))
  888 DX=GL(I)/(N(I)-1)
      XN=RAFCOF(I)
      HHHH=H(I)
      NNN=N(I)
      SS=SG(I)
      U=B(I)
      GD=U*COTTH
      DO 3 I1=1,NNN
      QD(I1)=0.0
    3 YD(I1)=YING(I)
      COF=1.49*SQRT(SS)/XN
  500 DT=DTM
      TYM=TYM+DT
      L1=L1+1
      L2=L2-1
      IF(GTR.NE.0.)GO TO 2666
      CALL UPSBO
      QP=QUPST
      CALL STROUT
      GO TO 2678
 2666 AYSE=0.
      FATMA=0.
      GUL=0.
      BOK1=0.
      BOK2=0.
 2875 DO 5 J1=1,NNN
      IF(J1.NE.1)GO TO 77
 1000 CALL UPSBO
      QN(J1)=QUPST
      YN(J1)=YUPST
      IF(GTR.EQ.1.0)AUPST=0.5*COTTH*YUPST*YUPST
      IF(GTR.EQ.2.0)AUPST=YUPST*B(I)
      GO TO 5
```

```
   77 IF(J1.NE.2)GO TO 7
      FFIN=FFINP(I)
      FINS=FINSP(I)
      CINF=CINFP(I)
      IRYN=IRNZON(I)
      CALL RASNIN
      QL1=QRSI*PL(I)
      FFIN=FFING(I)
      FINS=FINSG(I)
      CINF=CINFG(I)
      IRYN=IRNZON(I)
      CALL RASNIN
      QL3=QRSI
      GO TO 8
    7 IF(OK(I,J1).EQ.OK(I,J1-1).AND.OS(I,J1).EQ.OS(I,J1-1).AND.OL(I,J1).
     *EQ.OL(I,J1-1))GO TO 9
      GO TO 8
    9 IF(FFINAL(I,J1).EQ.FFINAL(I,J1-1).AND.FINSHL(I,J1).EQ.FINSHL(I,J1
     *-1).AND.CINFLT(I,J1).EQ.CINFLT(I,J1-1))GO TO 1200
      GO TO 8
 1200 IF(CO1(I,J1).EQ.CO1(I,J1-1).AND.CO2(I,J1).EQ.CO2(I,J1-1))GO TO 12
    8 FK=OK(I,J1)
      SO=OS(I,J1)
      OLL=OL(I,J1)
      FFIN=FFINAL(I,J1)
      FINS=FINSHL(I,J1)
      CINF=CINFLT(I,J1)
       NOV=NO(I,J1)
      YOIN=YINO(I,J1)
      IRYN=IRNZON(I)
      CONIN1=CO1(I,J1)
      CONIN2=CO2(I,J1)
      CALL OVLFLO
      QL2=QOVERL
   12 QLAT=QL1+QL2+QL3*U
      FLOLAT=QLAT
      YA=YO(J1-1)
      YB=YO(J1)
      YD=YN(J1-1)
      GAM=YD-YA-YB
      MT=0
      MTT=0
      YO=YD
      IF(J1.EQ.2)YO=YO+(DX*XN*QLAT/SQRT(100.*SS))**(3./8.)
      DELYO=YO-YING(I)
      IF(GTR.EQ.1.0)GO TO 19
      IF(YD.GT.HHHH)GO TO 70
      BET=QLAT-.25*T3(YD)*GAM/DT+.5*YD*G3(YD)/DX
      ALFA=.25*T3(YD)/DT+.5*G3(YD)/DX
      GO TO 23
   70 ALFA=.25*T4(YD)/DT+.5*G4(YD)/DX
      BET=QLAT-.25*T4(YD)*GAM/DT+.5*YD*G4(YD)/DX
   23 IF(GTR.EQ.1.)GO TO 19
      IF(YO.GT.HHHH)GO TO 21
       CONVER=F3(YO)*F3PP(YO)/(F3P(YO)**2)
      CONVER=ABS(CONVER)
      GO TO 20
   21 CONVER=F4(YO)*F4PP(YO)/(F4P(YO)**2)
      CONVER=ABS(CONVER)
   20 IF(CONVER.LT.1.0)GO TO 22
      GO TO 32
   19 IF(YD.GT.GD)GO TO 80
```

```
      ALFA=.25*T1(YD)/DT+.5*G1(YD)/DX
      BET=QLAT-.25*T1(YD)*GAM/DT+.5*YD*G1(YD)/DX
      GO TO 89
   80 ALFA=.25*T2(YD)/DT+.5*G2(YD)/DX
      BET=QLAT-.25*T2(YD)*GAM/DT+.5*YD*G2(YD)/DX
   89 IF(YO.GT.GD)GO TO 31
      CONVER=F1(YO)*F1PP(YO)/(F1P(YO)**2)
      CONVER=ABS(CONVER)
      GO TO 30
   31 CONVER=F2(YO)*F2PP(YO)/(F2P(YO)**2)
      CONVER=ABS(CONVER)
   30 IF(CONVER.LT.1.0)GO TO 22
   32 IF(MTT.NE.0)GO TO 35
      YO=YO-DELYO/20.
      IF(YO.LT.YING(I))YO=YING(I)
      MT=MT+1
      IF(MT.LT.20)GO TO 23
   35 IF(MTT.EQ.0)YO=YD
      IF(MTT.EQ.0.AND.J1.EQ.2)YO=YD+(DX*XN*QLAT/SQRT(100.*SS))**(3./8.)
      YO=YO+YD/5.0
      MTT=MTT+1
      IF(MTT.LE.99)GO TO 23
   22 DO 37 M1=1,40
      YO=ABS(YO)
      IF(GTR.EQ.1.0)GO TO 38
      IF(YO.GT.HHHH)GO TO 39
      FUNC=F3(YO)
      FUNCP=F3P(YO)
      GO TO 40
   38 IF(YO.GT.GD)GO TO 41
      FUNC=F1(YO)
      FUNCP=F1P(YO)
      GO TO 40
   41 FUNC=F2(YO)
      FUNCP=F2P(YO)
      GO TO 40
   39 FUNC=F4(YO)
      FUNCP=F4P(YO)
   40 Y1=YO-FUNC/FUNCP
      IF(ABS(Y1-YO).LE.0.00001)GO TO 42
   37 YO=Y1
   42 YC=Y1
   45 YN(J1)=YC
      AYSE=AYSE+QL1
      FATMA=FATMA+QL2
      GUL=GUL+QL3
      BOK1=BOK1+COVER1*QL2
      BOK2=BOK2+COVER2*QL2
      IF(J1.NE.NNN)GO TO 5
      YP=YC
      IF(GTR.EQ.2.)GO TO 301
      IF(YP.GT.GD)GO TO 302
      QP=Q1(YC)
      A=A1(YC)
      VP=QP/A
      T=T1(YC)
      GO TO 303
  302 QP=Q2(YC)
      A=A2(YC)
      VP=QP/A
      T=T2(YC)
      GO TO 303
```

```
  301 IF(YP.GT.HHHH)GO TO 304
      QP=Q3(YC)
      A=A3(YC)
      VP=QP/A
      T=T3(YC)
      GO TO 303
  304 QP=Q4(YC)
      A=A4(YC)
      VP=QP/A
      T=T4(YC)
  303 TOTAL=AYSE+FATMA+GUL
      C2LAT=ROK2/TOTAL
      C1LAT=ROK1/TOTAL
      ZIM=TOTAL/(NNN-1)
      IF(L1.EQ.1)CIB1=CGTR1(I)
      IF(L1.EQ.1)CIB2=CGTR2(I)
      CIC1=(C1LAT+CIB1*A/DT+CUPST1*QP/GL(I))/(A/DT+QP/GL(I)+ZIM)
      CIC2=(C2LAT+CIB2*A/DT+CUPST2*QP/GL(I))/(A/DT+QP/GL(I)+ZIM)
      CIB1=CIC1
      CIB2=CIC2
 3303 CALL DOWBOU
    5 CONTINUE
      DO 55 J2=1,NNN
   55 YO(J2)=YN(J2)
 2678 IF(TYM.LT.TYMEND)GO TO 500
      LIMIT=L3
  113 CONTINUE
  900 CONTINUE
      CALL SWRJNT
      STOP
      END
      SUBROUTINE UPSBU
C
C     THIS SUBROUTINE COMPUTES UPSTREAM BOUNDARY CONDITION AT EACH
C     TIME LEVEL
C
      COMMON/Z5/SG,RAFCOF,YING,COTTH,SINTH,COSTH,TANTH,YUPST,QUPST
      COMMON/Z6/K1,K2,K3,K4,K6,OR1,OR2,OR3,OR4,OR5,OR6,K5
      COMMON/Z7/I,GTRTYP,B,TIME,QUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
     *CIC1,CIC2
      COMMON/Z8/G,L1,TYM
      DIMENSION SG(100),RAFCOF(100),YING(100)
      DIMENSION K1(100),K2(100),K3(100),K4(100),K5(100),K6(100)
      DIMENSION DC1(6),DC2(6)
      DIMENSION OR1(100),OR2(100),OR3(100),OR4(100),OR5(100),OR6(100)
      DIMENSION UPINF(6),CGTR1(100),CGTR2(100)
      DIMENSION CCC1(100,100),CCC2(100,100)
      DIMENSION GTRTYP(100),B(100),TIME(100),QUP(100,100),OPR(100),NGTR(
     *100)
      DO 1 MM=1,6
      IF(MM.ZQ.1)MIN=K1(I)
      IF(MM.EQ.2)MIN=K2(I)
      IF(MM.EQ.3)MIN=K3(I)
      IF(MM.EQ.4)MIN=K4(I)
      IF(MM.EQ.5)MIN=K5(I)
      IF(MM.EQ.6)MIN=K6(I)
      IF(MIN.EQ.0)UPINF(MM)=0.0
      IF(MIN.EQ.0)DC1(MM)=CGTR1(I)
      IF(MIN.EQ.0)DC2(MM)=CGTR2(I)
      IF(MIN.EQ.0)GO TO 1
      MAL=2
   69 IF(TYM-TIME(MAL))112,111,110
```

```
110 MAL=MAL+1
    GO TO 69
111 UPINF(MM)=QUP(MIN,MAL)
    DC1(MM)=CCC1(MIN,MAL)
    DC2(MM)=CCC2(MIN,MAL)
    GO TO 1
112 UPINF(MM)=QUP(MIN,MAL-1)+(QUP(MIN,MAL)-QUP(MIN,MAL-1))/(TIME(MAL)-
   *TIME(MAL-1))*(TYM-TIME(MAL-1))
    RATIO=(TYM-TIME(MAL-1))/(TIME(MAL)-TIME(MAL-1))
    DC1(MM)=CCC1(MIN,MAL-1)+(CCC1(MIN,MAL)-CCC1(MIN,MAL-1))*RATIO
    DC2(MM)=CCC2(MIN,MAL-1)+(CCC2(MIN,MAL)-CCC2(MIN,MAL-1))*RATIO
  1 CONTINUE
    DISCH=(1.0)*(OR1(I)*UPINF(1)+OR2(I)*UPINF(2)+OR3(I)*UPINF(3)+OR4(I
   *)*UPINF(4)+OR5(I)*UPINF(5)+OR6(I)*UPINF(6))
    IF(DISCH.EQ.0.)CUPST1=CGTR1(I)
    IF(DISCH.EQ.0.)CUPST2=CGTR2(I)
    IF(DISCH.EQ.0.0)GO TO 200
    DIS1=OR1(I)*UPINF(1)*DC1(1)+OR2(I)*UPINF(2)*DC1(2)+OR3(I)*UPINF(3)
   **DC1(3)+OR4(I)*UPINF(4)*DC1(4)+OR5(I)*UPINF(5)*DC1(5)+OR6(I)*UPINF
   *(6)*DC1(6)
    DIS2=OR1(I)*UPINF(1)*DC2(1)+OR2(I)*UPINF(2)*DC2(2)+OR3(I)*UPINF(3)
   **DC2(3)+OR4(I)*UPINF(4)*DC2(4)+OR5(I)*UPINF(5)*DC2(5)+OR6(I)*UPINF
   *(6)*DC2(6)
    CUPST1=DIS1/DISCH
    CUPST2=DIS2/DISCH
200 IF(GTRTYP(I).EQ.0.0)GO TO 4
    IF(GTRTYP(I).EQ.2.0)GO TO 2
    YUPST=(DISCH*RAFCOF(I)*COTTH/(.745*SQRT(SG(I))))**(.375)*((1.+COST
   1H)/(0.5*SINTH))**(.25)
    GO TO 3
  2 YUPST=(DISCH*RAFCOF(I)/(1.49*B(I)*SQRT(SG(I))))**(3./5.)
  3 IF(YUPST.LT.YING(I))YUPST=YING(I)
  4 QUPST=DISCH
    RETURN
    END
    SUBROUTINE DOWBOU
C
C   THIS SUBROUTINE COMPUTES FLOW INTO GUTTER INLET AND CARY-OVER
C   AT EACH TIME LEVEL
C
    COMMON/Z4/QCACH,TBA,GR8TYP,W,GR8L,GD,T,VP,YP,QP,FLOLAT,OPR
    COMMON/Z7/I,GTRTYP,B,TIME,QUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
   *CIC1,CIC2
    COMMON/Z8/G,L1,TYM
    COMMON/Z10/L2,L3,NDTM,NGTR
    DIMENSION CGTR1(100),CGTR2(100)
    DIMENSION CCC1(100,100),CCC2(100,100)
    DIMENSION GTRTYP(100),B(100),TIME(100),QUP(100,100),OPR(100),NGTR(
   *100)
    DIMENSION QCACH(100,100),TBA(100),GR8TYP(100),W(100),GR8L(100)
    TIME(1)=0.
    QUP(I,1)=0.0
    IF(GR8TYP(I).EQ.0.)GO TO 20
    IF(GR8TYP(I).EQ.1.0.OR.GR8TYP(I).EQ.2.)COF=1.0
    IF(GR8TYP(I).EQ.3.0)COF=0.4
    IF(GR8TYP(I).EQ.4.0)COF=0.8
    IF(GR8TYP(I).EQ.5.)COF=0.9
    IF(GTRTYP(I).EQ.2.)GO TO 14
    DEP=W(I)/TAN(TBA(I))
    IF(YP.GT.DEP)GO TO 13
    WL=T*OPR(I)/SIN(TBA(I))
    YM=YP/2.
```

```
      GO TO 12
   13 WL=W(I)+DPR(I)/SIN(TBA(I))
      YM=YP-DEP/2.
      GO TO 12
   14 WL=W(I)+DPR(I)
      YM=YP
   12 WFL=3.*COF*WL*(YM+VP*VP/(2.*G))**1.5
      IF(WFL.LE.QP)GO TO 11
      WFL=0.6*WL*GRBL(I)*SQRT(YM+VP*VP/(2.*G))
      IF(WFL.GT.QP)WFL=QP
   11 FLOWIN=WFL+GRBL(I)*FLOLAT
      CARYOV=QP-WFL
      GO TO 21
   20 FLOWIN=0.
      CARYOV=QP
   21 IF(L2.NE.0)GO TO 31
      L3=L3+1
      L2=NDTM
      OUP(NGTR(I),L3)=CARYOV
      QCACH(NGTR(I),L3)=FLOWIN
      CCC1(NGTR(I),L3)=CIC1
      CCC2(NGTR(I),L3)=CIC2
      TIME(L3)=TYM
   10 CONTINUE
   31 RETURN
      END
      SUBROUTINE RASNIN
C
C     THIS SUBROUTINE COMPUTES DIRECT LATERAL INFLOW FROM RAIN,SNOW,
C     AND INFILTRATION
C
      COMMON/Z1/TEMP,IRYN,RYNST,RYNEND,DAYRYN,TM,RAIN
      COMMON/Z2/FFIN,FINS,CINF,QRAIN,QRSI
      COMMON/Z8/G,L1,TYM
      DIMENSION RYNST(5),RYNEND(5),TM(5,100),RAIN(5,100),DAYRYN(5)
      IF(TYM.LT.RYNST(IRYN).OR.TYM.GT.RYNEND(IRYN))GO TO 1
      J2=1
   69 IF(TYM-TM(IRYN,J2))112,111,110
  110 J2=J2+1
      GO TO 69
  111 QRAIN=RAIN(IRYN,J2)/(12.0*3600.)
      GO TO 2
  112 QRAIN=RAIN(IRYN,J2-1)+(TYM-TM(IRYN,J2-1))*(RAIN(IRYN,J2)-RAIN(IRYN
     *,J2-1))/(TM(IRYN,J2)-TM(IRYN,J2-1))
      QRAIN=QRAIN/(3600.*12.0)
      GO TO 2
    1 QRAIN=0.0
      IF(TYM.LT.RYNST(IRYN))QINFLT=0.0
      IF(TYM.LT.RYNST(IRYN))GO TO 5
    2 FAFA=FINS/(12.0*3600.0)
      IF(QRAIN.LE.FAFA)GO TO 4
      ARG=-CINF*(TYM-RYNST(IRYN))/(3600.)
      QINFLT=FFIN+(FINS-FFIN)*EXP(ARG)
      QINFLT=QINFLT/(3600.*12.0)
      GO TO 5
    4 QINFLT=QRAIN
    5 QSNOW=0.007*DAYRYN(IRYN)*(TEMP-32.)/(24.*3600.*12.0)
      IF(TEMP.LT.32.0)QSNOW=0.
      QRSI=QRAIN-QINFLT+QSNOW
      RETURN
      END
      SUBROUTINE OVLFLO
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C
C     THIS SUBROUTINE COMPUTES THE QUANTITY AND QUALITY OF FLOW IN
C     SUBCATCHMENT STRIPS
C
      COMMON/Z1/TEMP,IRYN,RYNST,RYNEND,DAYRYN,TM,RAIN
      COMMON/Z2/FFIN,FINS,CINF,QRAIN,QRSI
      COMMON/Z20/CONIN1,CONIN2,COVER1,COVER2
      COMMON/Z3/XNU,C1,FK,SO,OLL,NOV,YOIN,DT,QOVERL
      COMMON/Z8/G,L1,TYM
      DIMENSION YOLD(11),YNEW(11),QOLD(11),QNEW(11)
      DIMENSION RYNST(5),RYNEND(5),TM(5,100),RAIN(5,100),DAYRYN(5)
      GL(E)=3.*ALF*E**2
      GLP(E)=6.*ALF*E
      GLPP(E)=6.*ALF
      FL(E)=E-(BET+RAT*YD*GL(E))/(GAM+RAT*GL(E))
      FUNC1L(E)=YD*GLP(E)/(GL(E)+GAM/RAT)*(-1.)
      FUNCPL(E)=(YD*GLPP(E)*(GL(E)+GAM/RAT)-YD*GLP(E)**2)/(GL(E)+GAM/RAT
     1)**2*(-1.)
      PAY(E)=RAT*GLP(E)*(BET+YD*RAT*GL(E))
      PAYP(E)=RAT*(BET+RAT*YD*GL(E))*GLPP(E)+YD*(RAT*GLP(E))**2
      PAYD(E)=(GAM+RAT*GL(E))**2
      PAYDP(E)=2.*RAT*GLP(E)*(GAM+RAT*GL(E))
      FLP(E)=1.0+FUNC1L(E)+PAY(E)/PAYD(E)
      FLPP(E)=FUNCPL(E)+(PAYP(E)*PAYD(E)-PAYDP(E)*PAY(E))/(PAYD(E))**2
      QL(E)=ALF*E**3
      GT(E)=SQRT(6.*G*SO*E)*(3.*ALOG10(2.*E/FK)+3.47)
      GTP(E)=SQRT(8.*G*SO/E)*(1.5*ALOG10(2.*E/FK)+3.04)
      GTPP(E)=SQRT(8.*G*SO/E**3)*(-0.75*ALOG10(2.*E/FK)-0.85)
      FT(E)=E-(BET+RAT*YD*GT(E))/(GAM+RAT*GT(E))
      FUNC1T(E)=YD*GTP(E)/(GT(E)+GAM/RAT)*(-1.)
      FUNCPT(E)=(YD*GTPP(E)*(GT(E)+GAM/RAT)-YD*GTP(E)**2)/(GT(E)+GAM/RAT
     1)**2*(-1.)
      PAT(E)=RAT*GTP(E)*(BET+YD*RAT*GT(E))
      PATP(E)=RAT*(BET+RAT*YD*GT(E))*GTPP(E)+YD*(RAT*GTP(E))**2
      PATD(E)=(GAM+RAT*GT(E))**2
      PATDP(E)=2.*RAT*GTP(E)*(GAM+RAT*GT(E))
      FTP(E)=1.0+FUNC1T(E)+PAT(E)/PATD(E)
      FTPP(E)=FUNCPT(E)+(PATP(E)*PATD(E)-PATDP(E)*PAT(E))/(PATD(E))**2
      QT(E)=SQRT(8.*G*SO*E**3)*(2.*ALOG10(2.*E/FK)+1.74)
      REYCR(E)=C1*(2.*ALOG10(2.*E/FK)+1.74)**2
      IF(OLL.EQ.0.0)QOVERL=0.0
      IF(OLL.EQ.0.0)GO TO 38
      CALL RASNIN
      QOL=QRSI
      DOX=OLL/(NOV-1)
      ALF=8.*G*SO/(C1*XNU)
      RAT=DT/DOX
      IF(L1.NE.1)GO TO 2
      FLOW=1.
      DO 1 IOV=1,NOV
    1 YOLD(IOV)=YOIN
    2 DO 3 KOV=1,NOV
       IF(KOV.EQ.1)GO TO 4
      YOA=YOLD(KOV-1)
      YOD=YNEW(KOV-1)
      YD=YOD
      YOB=YOLD(KOV)
      IF(FLOW.EQ.2.0.AND.YOB.GT.FK.AND.YOD.GT.FK)GO TO 31
      BET=2.*QOL*DT-(YOD-YOA-YOB)+RAT*YOD*GL(YOD)
      GAM=1.+RAT*GL(YOD)
      MOT=0
      MOTT=0
```

```
      YO=((QNEW(KOV-1)+QOL*DOX)/ALF)**0.33
      D1YO=(YO-YOIN)/20.
      D2YO=YO/10.
      YYO=YO
    7 CONVER=FL(YO)*FLPP(YO)/(FLP(YO)**2)
      CONVER=ABS(CONVER)
      IF(CONVER.LT.1.0)GO TO 5
      IF(MOT.NE.0)GO TO 6
      YO=YO-D1YO
      MOTT=MOTT+1
      IF(MOTT.LE.20)GO TO 7
    6 IF(MOT.EQ.0)YO=YYO
      YO=YO+D2YO
      MOT=MOT+1
      IF(MOT.LE.75)GO TO 7
      GO TO 31
    5 DO 9 M1=1,50
      YO=ABS(YO)
      Y1=YO-FL(YO)/FLP(YO)
      IF(ABS(YO-Y1).LE.0.00001)GO TO 10
    9 YO=Y1
   10 YOC=Y1
      IF(YOC.LT.YOIN)YOC=YOIN
      YNEW(KOV)=YOC
      QNEW(KOV)=QL(YOC)
      REYNO=QNEW(KOV)/XNU
      IF(REYNO.LT.REYCR(YOC).OR.YOD.LT.FK)GO TO 3
      IF(REYNO.LT.REYCR(YOC).OR.YOB.LT.FK)GO TO 3
      FLOW=2.
   31 BET=2.0*QOL*DT-(YOD-YOA-YOB)+RAT*YOD*GT(YOD)
      GAM=1.+RAT*GT(YOD)
      MAT=0
      MATT=0
      YO=YOB+QOL*DT
      D1YO=(YO-YOIN)/20.
      D2YO=YO/10.
      YYO=YO
   37 CONVER=FT(YO)*FTPP(YO)/(FTP(YO)**2)
      CONVER=ABS(CONVER)
      IF(CONVER.LT.1.)GO TO 35
      IF(MAT.NE.0)GO TO 36
      YO=YO-D1YO
      IF(YO.LT.FK)YO=FK
      MATT=MATT+1
      IF(MATT.LE.20)GO TO 37
   36 IF(MAT.EQ.0)YO=YYO
      YO=YO+D2YO
      MAT=MAT+1
      IF(MAT.LE.75)GO TO 37
      WRITE(6,8)
    8 FORMAT(2X,'OVERLAND ITERATION FAILS')
      STOP
   35 DO 39 M2=1,50
      YO=ABS(YO)
      Y1=YO-FT(YO)/FTP(YO)
      IF(ABS(YO-Y1).LE.0.00001)GO TO 40
   39 YO=Y1
   40 YOC=Y1
      IF(YOC.LT.YOIN)YOC=YOIN
      YNEW(KOV)=YOC
      QNEW(KOV)=QT(YOC)
      GO TO 3
```

```
    4 YNEW(1)=YOIN
      QNEW(1)=0.0
    3 CONTINUE
      IF(L1.EQ.1)CONB1=CONIN1
      IF(L1.EQ.1)CONB2=CONIN2
      QOVERL=QNEW(NOV)
      COVER1=(CONB1*YNEW(NOV)/DT+CONIN1*QOVERL/OLL)/(YNEW(NOV)/DT+QOVERL
     */OLL+QOL)
      COVER2=(CONB2*YNEW(NOV)/DT+CONIN2*QOVERL/OLL)/(YNEW(NOV)/DT+QOVERL
     */OLL+QOL)
      DO 47 JOV=1,NOV
      CONB1=COVER1
      CONB2=COVER2
   47 YOLD(JOV)=YNEW(JOV)
   38 RETURN
      END
      SUBROUTINE STROUT
C
C     THIS SUBROUTINE COMPUTES FLOW INTO INLETS WHERE THERE IS NO
C     DOWNSTREAM GUTTERS
C
      COMMON/Z4/QCACH,TBA,GRBTYP,W,GRBL,GD,T,VP,YP,QP,FLOLAT,OPR
      COMMON/Z3/XNU,C1,FK,SO,OLL,NOV,YOIN,DT,QOVERL
      COMMON/Z8/G,L1,TYM
      COMMON/Z10/L2,L3,NDTM,NGTR
      COMMON/Z7/I,GTRTYP,B,TIME,QUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
     *CIC1,CIC2
      COMMON/Z11/GL
      DIMENSION QCACH(100,100),GRBTYP(100),TBA(100)
      DIMENSION GTRTYP(100),B(100),TIME(100),QUP(100,100)
      DIMENSION CGTR1(100),CGTR2(100)
      DIMENSION CCC1(100,100),CCC2(100,100)
      DIMENSION GL(100),W(100),NGTR(100),GRBL(100),OPR(100)
      IF(L1.NE.1)GO TO 1
      AREA=W(I)*GRBL(I)*OPR(I)
      XK=-(0.80*AREA*SQRT(2.*G))/2.
      XC1=GL(I)/DT
      XI1=0.
      XH1=0.
      XQ1=0.
    1 XI2=QP
      XC2=.5*(XI1+XI2+XQ1)+XC1*XH1
      DISC=XK**2+4.0*XC1*XC2
      XH0=(+XK+SQRT(DISC))/(2.*XC1)
      IF(XH0.LT.0.0)XH0=0.
      XH2=XH0**2
      XQ2=-2.*XK*XH0
      IF(L2.NE.0)GO TO 2
      L2=NDTM
      L3=L3+1
      QCACH(NGTR(I),L3)=XQ2
      IF(XQ2.EQ.0.)CCC1(NGTR(I),L3)=0.
      IF(XQ2.EQ.0.)CCC2(NGTR(I),L3)=0.
      IF(XQ2.EQ.0.)GO TO 2
      CCC1(NGTR(I),L3)=CUPST1*XI2/XQ2
      CCC2(NGTR(I),L3)=CUPST2*XI2/XQ2
    2 XQ1=XQ2
      XH1=XH2
      XI1=XI2
      RETURN
      END
      SUBROUTINE SWRJNT
```

```
C
C     THIS SUBROUTINE COMPUTES THE QUANTITY AND QUALITY OF DIRECT INFLOW
C     TO SEWER NODES
C
      COMMON/Z4/QCACH,T8A,GR8TYP,W,GR8L,GO,T,VP,YP,QP,FLOLAT,OPR
      COMMON/Z7/I,GTRTYP,B,TIME,QUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
     *CIC1,CIC2
      COMMON/Z8/G,L1,TYM
      COMMON/Z9/JTOTAL,INLET,LIMIT,TYMEND,DTM,NSW,SWRTYP,BYSFLO
      COMMON/Z12/CJ1,CJ2
      DIMENSION CJ1(100),CJ2(100)
      DIMENSION ITYM(100),QDIS(100)
      DIMENSION INLET(100,10),NSW(100),SWRTYP(100),BYSFLO(100)
      DIMENSION QCACH(100,100),T8A(100),GR8TYP(100),W(100),GR8L(100)
      DIMENSION COB1(100),COB2(100),CGTR1(100),CGTR2(100)
      DIMENSION CCC1(100,100),CCC2(100,100)
      DIMENSION GTRTYP(100),B(100),TIME(100),QUP(100,100),OPR(100),NGTR(
     *100)
      IHD=TYMEND
      IDTM=DTM
      ISTP=IHD+IDTM
      DO 1 JJ=1,JTOTAL
      WRITE(6,200)
  200 FORMAT(2X,'*******************************************************')
      WRITE(6,80)NSW(JJ)
   80 FORMAT(1X,'FLOW INTO SEWER JUNCTION=',I5,'**********************')
      WRITE(6,200)
      WRITE(6,81)
   81 FORMAT(///2X,'TIME',3X,'DISCHARGE',3X,'1ST POLLUTANT',3X,'2ND POLL
     *UTANT')
      WRITE(6,86)
   86 FORMAT(1X,'(SEC)',6X,'(CFS)',6X,'CONCENTRATION',3X,'CONCENTRATION'
     *)
      TIME(1)=0.
      ITYM(1)=0
      QDIS(1)=BYSFLO(JJ)
      COB1(1)=CJ1(JJ)
      COB2(1)=CJ2(JJ)
      DO 2 NN=2,LIMIT
      ITYM(NN)=TIME(NN)
    6 DIS=0.
      CIS1=0.
      CIS2=0.
      DO 3 MM=1,10
      K=INLET(JJ,MM)
      IF(K.EQ.0)GO TO 15
      CIS1=CIS1+QCACH(K,NN)*CCC1(K,NN)
      CIS2=CIS2+QCACH(K,NN)*CCC2(K,NN)
    3 DIS=DIS+QCACH(K,NN)
   15 QDIS(NN)=DIS+BYSFLO(JJ)
      COB1(NN)=(CIS1+BYSFLO(JJ)*CJ1(JJ))/QDIS(NN)
      COB2(NN)=(CIS2+BYSFLO(JJ)*CJ2(JJ))/QDIS(NN)
    2 CONTINUE
   14 NL=LIMIT+1
      ITYM(NL)=ISTP
      QDIS(NL)=BYSFLO(JJ)
      IF(SWRTYP(JJ).EQ.2.0)GO TO 20
      WRITE(7,21)NSW(JJ),IHD,BYSFLO(JJ)
   21 FORMAT('JFBD',1X,I5,1X,'0',I10,1X,F10.2)
      GO TO 22
   20 WRITE(7,23)NSW(JJ),IHD,BYSFLO(JJ)
   23 FORMAT('FBD',1X,I5,1X,'0',I10,1X,F10.2)
```

```
 22 WRITE(7,25)(ITYM(MM),QDIS(MM),MM=1,NL)
 25 FORMAT(I5,1X,F5.2,1X,I5,1X,F5.2,1X,I5,1X,F5.2,1X,I5,1X,F5.2,1X,I5,
   *1X,F5.2,1X,I5,1X,F5.2)
    IF(SWRTYP(JJ).EQ.2.0)GO TO 30
    WRITE(7,31)
 31 FORMAT('JEND')
    GO TO 100
 30 WRITE(7,32)
 32 FORMAT('FEND')
100 DO 82 MAMA=1,LIMIT
    WRITE(6,83)TIME(MAMA),QDIS(MAMA),COB1(MAMA),COB2(MAMA)
 83 FORMAT(F7.2,2X,F8.2,9X,F6.2,9X,F6.2)
 82 CONTINUE
  1 CONTINUE
    RETURN
    END
```

APPENDIX C

LISTING OF COMPUTER PROGRAM FOR THE ILLINOIS SEWER SYSTEM WATER QUALITY MODEL

The Illinois Sewer System Water Quality Model is programmed in Fortran IV language. The input to the computer program includes the depth and discharge hydrographs at the entrance and exit of each sewer and the volume of water at each storage junction at given times as provided by the output of the ISS model. In addition, the direct inflow hydrographs and pollutographs to the sewer junctions as obtained from surface runoff computations and the sewer system layout are also input to the program for runoff quality routing in the sewer system. The output from the computer program are the pollutographs at the sewer system outlet, at the storage junctions and at the entrance and exit of all sewers.

The computer program allows the consideration of as many as 100 sewers at a time. The runoff quality routing is performed for two different pollutants. The computations can be proceeded for as many as 100 time steps. The storage requirement for the computer program in its present form is 300K. It can be modified to consider larger sewer systems by changing the arrays in DIMENSION statements if more storage is available.

A listing of the computer program for the Illinois Sewer System Water Quality Model is given below.

```
C
C     SEWER SYSTEM WATER QUALITY MODEL
C
      DIMENSION QD1(100),QD2(100),QI1(100),QI2(100)
      DIMENSION QOUT1(100,100),QOUT2(100,100),QIN1(100,100),QIN2(100,100
     *)
      DIMENSION TYM(100),QU(100),HU(100),QD(100),HD(100)
      DIMENSION QQU(100),QQD(100),HHU(100),HHD(100),AAU(100),AAD(100)
      DIMENSION QN(100),HN(100),NODE(100)
      DIMENSION AREA(100),C1IN(100),C2IN(100)
      DIMENSION NODEUP(100),NODOWN(100),PLENGT(100),D(100),C1U(100)
      DIMENSION C1D(100),CN1(100),CN2(100)
      DIMENSION C2U(100),C2D(100),Y(100),TM(100)
C
C     FOLLOWING ARE THE GENERAL INPUT PARAMETERS
C
C     NPIPE,TOTAL NUMBER OF SEWERS IN THE SYSTEM
C     NSTJUN,TOTAL NUMBER OF RESERVOIR-TYPE JUNCTIONS IN THE SYSTEM
C     NPYPD8,NUMBER OF POINTS USED TO DESCRIBE EACH OF THE DISCHARGE AND
C            STAGE HYDROGRAPHS AT THE ENTRANCE AND EXIT OF EACH SEWER
C     NJNCD8,NUMBER OF POINTS USED TO DESCRIBE DIRECT INFLOW HYDROGRAPHS
C            AT EACH OF THE SEWER NODES
C     NROOT,IDENTIFICATION NUMBER OF THE SEWER NODE AT THE SYSTEM OUTLET
C     DT,THE TIME INTERVAL AT WHICH ORDINATES OF THE DIRECT INFLOW
C            HYDROGRAPHS AT THE SEWER NODES ARE PROVIDED
C
      READ(5,1)NPIPE,NSTJUN,NPYPD8,NJNCD8,NROOT,DT
    1 FORMAT(5I5,F5.0)
C
C     FOLLOWING ARE THE INPUT PARAMETERS FOR EACH SEWER
C
C     NODEUP(I),IDENTIFICATION NO OF JUNCTION NODE UPSTREAM OF SEWER I
C     NODOWN(I),IDENTIFICATION NO OF JUNCTION NODE DOWNSTREAM OF SEWER I
C     PLENGT(I),LENGTH OF SEWER I
C     D(I),DIAMETER OF SEWER I
C     C1U(I),INITIAL CONCENTRATION OF THE FIRST POLLUTANT AT ENTRANCE OF I
C     C1D(I),INITIAL CONCENTRATION OF THE FIRST POLLUTANT AT EXIT OF I
C     C2U(I),INITIAL CONCENTRATION OF THE SECOND POLLUTANT AT ENTRANCE OF I
C     C2D(I),INITIAL CONCENTRATION OF THE SECOND POLLUTANT AT EXIT OF I
C     TYM(J1),TIME AT J1'ST TIME LEVEL
C     QU(J1),DISCHARGE AT ENTRANCE AT J1'ST TIME LEVEL
C     HU(J1),FLOW DEPTH AT ENTRANCE AT J1'ST TIME LEVEL
C     QD(J1),DISCHARGE AT EXIT AT J1'ST TIME LEVEL
C     HD(J1),FLOW DEPTH AT EXIT AT J1'ST TIME LEVEL
C
      DO 2 I=1,NPIPE
      READ(5,3)NODEUP(I),NODOWN(I),PLENGT(I),D(I),C1U(I),C1D(I),C2U(I),C
     *2D(I)
      IF(C1D(I).EQ.C1U(I))C1D(I)=1.05*C1U(I)
      IF(C2D(I).EQ.C2U(I))C2D(I)=1.05*C2U(I)
    3 FORMAT(2I5,6F5.0)
      READ(5,4)(TYM(J1),QU(J1),HU(J1),QD(J1),HD(J1),J1=1,NPYPD8)
    4 FORMAT(F6.0,4F5.0,F6.0,4F5.0,F6.0,4F5.0)
      IF(NODOWN(I).NE.NROOT)GO TO 230
      WRITE(6,74)
      WRITE(6,73)
   73 FORMAT(1X,'WATER QUALITY CONDITIONS AT SEWER SYSTEM OUTLET******')
      WRITE(6,74)
   74 FORMAT(1X,'*******************************************************')
      WRITE(6,75)
   75 FORMAT(////4X,'TIME',5X,'DISCHARGE',3X,'1ST POLLUTANT',3X,'2ND POL
     *LUTANT')
```

```
      WRITE(6,76)
  76  FORMAT(3X,'(SEC)',6X,'(CFS)',6X,'CONCENTRATION',3X,'CONCENTRATION'
     *)
      GO TO 900
  66  FORMAT(F7.2,5X,F8.2,9X,F6.2,9X,F6.2)
 230  WRITE(6,74)
      WRITE(6,233)NODEUP(I),NODOWN(I)
 233  FORMAT(1X,'CONDITIONS IN SEWER FROM NODE',I5,'TO NODE',I5,'*****')
      WRITE(6,74)
      WRITE(6,234)
 234  FORMAT(//6X,'AT UPSTREAM SECTION',45X,'AT DOWNSTREAM SECTION')
      WRITE(6,235)
 235  FORMAT(2X,'TIME',5X,'DISCHARGE',3X,'1ST POLLUTANT',3X,'2ND POLLUTA
     *NT',18X,'DISCHARGE',3X,'1ST POLLUTANT',3X,'2ND POLLUTANT')
      WRITE(6,236)
 236  FORMAT(1X,'(SEC)',6X,'(CFS)',6X,'CONCENTRATION',3X,'CONCENTRATION'
     *,7X,'(CFS)',18X,'CONCENTRATION',3X,'CONCENTRATION')
 900  DO 6 LIN=1,4
      GO TO(7,8,9,10),LIN
   7  DO 11 K1=1,NPYPD8
  11  Y(K1)=QU(K1)
      GO TO 15
   8  DO 12 K1=1,NPYPD8
  12  Y(K1)=HU(K1)
      GO TO 15
   9  DO 13 K1=1,NPYPD8
  13  Y(K1)=QD(K1)
      GO TO 15
  10  DO 14 K1=1,NPYPD8
  14  Y(K1)=HD(K1)
  15  K=1
      TE=DT
  17  K=K+1
      J2=2
  69  IF(TE-TYM(J2))112,111,110
 110  J2=J2+1
      GO TO 69
 111  YE=Y(J2)
      GO TO 200
 112  YE=Y(J2-1)+(Y(J2)-Y(J2-1))/(TYM(J2)-TYM(J2-1))*(TE-TYM(J2-1))
 200  TM(K)=TE
      GO TO(21,22,23,24),LIN
  21  QQU(K)=YE
  22  HHU(K)=YE
  23  QQD(K)=YE
  24  HHD(K)=YE
      TE=TE+DT
      IF(TE.GT.TYM(NPYPD8))KKK=K
      IF(TE.GT.TYM(NPYPD8))GO TO 6
      GO TO 17
   6  CONTINUE
      TM(1)=TYM(1)
      QQU(1)=QU(1)
      QQD(1)=QD(1)
      HHU(1)=HU(1)
      HHD(1)=HD(1)
      DO 80 KK=1,KKK
      ARGU=SQRT(D(I)*HHU(KK)-HHU(KK)*HHU(KK))/(.5*D(I)-HHU(KK))
      A1U=.25*D(I)*D(I)*ATAN(ARGU)
      A2U=(.5*D(I)-HHU(KK))*SQRT(D(I)*HHU(KK)-HHU(KK)*HHU(KK))
      AAU(KK)=A1U-A2U
      ARGD=SQRT(D(I)*HHD(KK)-HHD(KK)*HHD(KK))/(.5*D(I)-HHD(KK))
```

```
      A1D=.25*D(I)*D(I)*ATAN(ARGD)
      A2D=(.5*D(I)-HHD(KK))*SQRT(D(I)*HHD(KK)-HHD(KK)*HHD(KK))
      AAD(KK)=A1D-A2D
   80 CONTINUE
      DO 30 L=2,KKK
      QA=QQU(L-1)
      AA=AAU(L-1)
      QB=QQD(L-1)
      AB=AAD(L-1)
      QDD=QQU(L)
      AD=AAU(L)
      QC=QQD(L)
      AC=AAD(L)
      DO 31 LL=1,2
      GO TO(41,42),LL
   41 IF(L.NE.2)GO TO 43
      CA=C1U(I)
      CB=C1D(I)
      GO TO 45
   43 CA=C1A
      CB=C1B
      GO TO 45
   42 IF(L.NE.2)GO TO 44
      CA=C2U(I)
      CB=C2D(I)
      GO TO 45
   44 CA=C2A
      CB=C2B
   45 X1=AC/DT+QC/PLENGT(I)
      X2=QDD/PLENGT(I)
      B1=-QC/PLENGT(I)
      B2=AD/DT-QDD/PLENGT(I)
      C=AC*CB/DT
      E=AD*CA/DT
      CC=(C*B2-E*B1)/(X1*B2-X2*B1)
      CD=(X1*E-C*X2)/(X1*B2-X2*B1)
      IF(CC.LT.0.)CC=0.
      IF(CD.LT.0.)CD=0.
      GO TO(51,52),LL
   51 QOUT1(I,L)=CD*QDD
      DIS1=CC*QC
      TIS1=CD*QDD
      C1A=CD
      C1B=CC
      QIN1(I,L)=CC*QC
      GO TO 31
   52 QOUT2(I,L)=CD*QDD
      DIS2=CC*QC
      TIS2=CD*QDD
      C2A=CD
      C2B=CC
   63 QIN2(I,L)=CC*QC
   31 CONTINUE
      P1=DIS1/QC
      P2=DIS2/QC
      T1=TIS1/QDD
      T2=TIS2/QDD
      IF(NODOWN(I).NE.NROOT)GO TO 888
      WRITE(6,66)TM(L),QC,P1,P2
      GO TO 30
  888 CONTINUE
  237 WRITE(6,533)TM(L),QC,P1,P2,QDD,T1,T2
```

```
533 FORMAT(2F10.2,2X,F10.2,9X,F10.2,15X,F10.2,2X,F10.2,9X,F10.2)
 30 CONTINUE
  2 CONTINUE
    DO 50 JJ=1,NSTJUN
C
C   FOLLOWING ARE THE INPUT PARAMETERS FOR EACH RESERVOIR-TYPE JUNCTION
C
C   NODE(JJ),IDENTIFICATION NUMBER OF THE JUNCTION JJ
C   AREA(JJ),CROSS SECTIONAL AREA OF THE JUNCTION JJ
C   C1IN(JJ),INITIAL CONCENTRATION OF THE FIRST POLLUTANT IN JUNCTION JJ
C   C2IN(JJ),INITIAL CONCENTRATION OF THE SECOND POLLUTANT
C   TYM(J3),TIME AT J3'RD TIME LEVEL
C   QN(J3),RATE OF DIRECT INFLOW AT J3'RD TIME LEVEL
C   CN1(J3),CONCENTRATION OF THE FIRST POLLUTANT OF DIRECT INFLOW
C   CN2(J3),CONCENTRATION OF THE SECOND POLLUTANT OF DIRECT INFLOW
C   HN(J3),WATER STAGE IN JUNCTION AT J3'RD TIME LEVEL
C
    READ(5,151)NODE(JJ),AREA(JJ),C1IN(JJ),C2IN(JJ)
151 FORMAT(I5,3F5.0)
     READ(5,152)(TYM(J3),QN(J3),CN1(J3),CN2(J3),HN(J3),J3=1,NJNCD8)
152 FORMAT(F6.0,4F5.0,F6.0,4F5.0,F6.0,4F5.0)
    WRITE(6,74)
    WRITE(6,54)NODE(JJ)
 54 FORMAT(1X,'CONDITIONS AT JUNCTION NO=',I5,'**********************')
    WRITE(6,74)
    WRITE(6,55)
 55 FORMAT(///2X,'TIME',6X,'STORAGE',4X,'1ST POLLUTANT',3X,'2ND POLLUT
   *ANT')
    WRITE(6,56)
 56 FORMAT(1X,'(SEC)',6X,'(FT3)',6X,'CONCENTRATION',3X,'CONCENTRATION'
   *)
    IF(KKK.GT.NJNCD8)KKK=NJNCD8
    DO 90 II=1,KKK
    QO1(II)=0.
    QO2(II)=0.
    QI1(II)=0.
 90 QI2(II)=0.
    DO 91 MM=1,NPIPE
    IF(NODE(JJ).NE.NODEUP(MM))GO TO 92
    DO 93 L1=2,KKK
    QO1(L1)=QO1(L1)+QOUT1(MM,L1)
 93 QO2(L1)=QO2(L1)+QOUT2(MM,L1)
 92 IF(NODE(JJ).NE.NODOWN(MM))GO TO 91
    DO 94 L2=2,KKK
    QI1(L2)=QI1(L2)+QIN1(MM,L2)
 94 QI2(L2)=QI2(L2)+QIN2(MM,L2)
 91 CONTINUE
    DO 57 K=2,KKK
    IF(K.EQ.2)C1OLD=C1IN(JJ)
    IF(K.EQ.2)C2OLD=C2IN(JJ)
    BET1=QI1(K)+QN(K)*CN1(K)-QO1(K)
    BET2=QI2(K)+QN(K)*CN2(K)-QO2(K)
    HNEW=HN(K)
    HOLD=HN(K-1)
    CNEW1=(C1OLD*HOLD+BET1*DT/AREA(JJ))/HNEW
    CNEW2=(C2OLD*HOLD+BET2*DT/AREA(JJ))/HNEW
    IF(CNEW1.LT.0.)CNEW1=0.
    IF(CNEW2.LT.0.)CNEW2=0.
    S=HNEW*AREA(JJ)
    WRITE(6,66)TYM(K),S,CNEW1,CNEW2
    C1OLD=CNEW1
    C2OLD=CNEW2
```

```
57 CONTINUE
50 CONTINUE
   STOP
   END
```