Urban Stormwater Hydrology

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David F. Kibler,
Editor

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PREFACE

This monograph has been written over the last 2 years by eight members of the AGU Urban Hydrology Committee as a means of conveying state-of-the-art practices in the expanding field of urban hydrology and stormwater management. Although numerous references to on-going research are cited, the monograph is intended to serve primarily as a practical guide to methods and models currently in use to analyze different types of stormwater management problems. With this objective in mind, the authors have made a special effort to include examples which help illustrate the steps in a particular procedure or analysis. The monograph presumes a basic background in hydrology and makes no real effort to present conventional procedures which can be found readily in available hydrology textbooks. However, the detailed example calculations in Chapters 2, 3, and 4 should assist the unfamiliar reader in understanding basic hydrologic computations related to urban rainfall-runoff analysis. Given this coverage, the monograph should be useful as a reference to practicing engineers and urban planners as well as to graduate students in engineering-environmental disciplines with career interests in the growing field of urban hydrology and stormwater management.

On behalf of the individual chapter authors and the AGU Urban Hydrology Committee, I wish to express a note of appreciation to those who have served in a review capacity and helped to bring this monograph to fruition. In this regard, a special note of thanks goes to Ben Urbonas of the Denver Urban Drainage and Flood Control District, David Lystrom and Ernest Cobb of the U.S. Geological Survey, and David Dawdy of Northern Technical Services, Inc.

David F. Kibler, Monograph Editor

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1 INTRODUCTION TO URBAN HYDROLOGY AND STORMWATER MANAGEMENT

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Overview of Monograph

The primary purpose of this monograph is to present in a coherent fashion the state of the art in urban hydrology and stormwater management. A secondary objective is to communicate recent research findings as they apply to improved methods of analysis of urban runoff. This monograph attempts to bridge the gap between current practice and research.

The first chapter is an introduction to urban hydrology and stormwater management. The effects of urbanization on the quantity and quality of the runoff and the associated problems are presented in general terms. A brief history of urban hydrology highlights the progress made during the last decade. The interaction of the land use and urban runoff is presented in quantitative terms followed by a brief discussion of urban air quality since it affects stormwater quality. The chapter closes with a section on stormwater planning in the urban metroplex which includes a brief discussion on urban water balance, an introduction to stormwater and land use models and their use as elements of urban planning.

Chapter 2 discusses the concept of the 'design storm' which provides a means of estimating rainfall depth or intensity for a specified duration and frequency, which in turn can be used in estimating runoff peaks and volumes. In the early applications the rainfall intensity-duration-frequency relationships were used to obtain a rainfall intensity of uniform duration for use with the rational formula. More recent applications include the development of a synthetic hyetograph or storm profile. Recent research has demonstrated the limitations of the design storm concept. For example, it usually does not provide all the probability information desired for risk evaluation in planning and design, particularly in those cases involving runoff storage and treatment which may be needed for nonpoint source pollution abatement. Because of the importance of this design tool, new methods have evolved from which rationally developed and statistically acceptable design storms may be obtained.

Chapter 3 deals with rainfall losses in the form of interception, depression storage, and infiltration. The methods of estimating these losses are basic to both the desk top and the computer-oriented methods of analysis. It is one of the topics which has received little attention by researchers during the last decade. Nevertheless, it is a very important part of the rainfall-runoff process in urban areas. Frequently, the most important parameter in determining the abstractions from urban areas is the exact determination of the impervious areas directly connected to the drainage systems since these areas contribute, almost instantaneously, a runoff volume very close to the amount of incident precipitation, while most of the rainfall on the pervious areas and on the areas not directly connected may infiltrate and does not produce immediate runoff.

Chapter 4 is concerned with simplified methods for urban stormwater calculations. Traditionally, the rational formula has been used for estimating the peak runoff from small urban catchments. During the last 15 years an increased awareness of the limitations of the rational formula has evolved. An important limitation of the rational formula is that it is concerned only with a peak flow and is thus of little help to quantify those effects which depend upon the availability of a sewer outlet hydrograph. A modification of the rational method for estimating detention storage volumes is presented in chapter 4 along with several other methods capable of producing an outlet hydrograph. Simplified methods are also given to estimate the annual pollutant loads from urban watersheds. The pollutants considered are the biological oxygen demand (BOD), total organic carbon (TOC), suspended solids, volatile solids, phosphates, total nitrogen, and coliforms.

The detailed formulation of stormwater runoff processes and their inclusion in large-scale simulation models are discussed in chapter 5. The surface runoff and transport subsystems are discussed first, leading to the receiving water subsystem. The combined sewer systems encountered in many older cities are described, followed by the conceptualization of the physical drainage as used in simulation models. The basic hydraulic transport equations for the channel and sewer transport system are stated.

Chapter 6 describes the quality aspects of urban runoff starting with the water quality criteria for urban stormwater and combined sewer overflows. The entry of pollutants from dry fall and wet fall is quantified, leading to the determination of the pollutants washed off.

Chapter 7 discusses data collection and instrumentation. It covers in some detail the techniques and procedures used by the U.S. Geological Survey (USGS) for the measurement of precipitation (dry fall and wet fall) quantity and chemical constituents and runoff quantity and chemical and bacteriological constituents. Urban basins differ from rural basins in that they are usually small and have short response time. Therefore measurements are often made at 5-min or at 1-min intervals. The primary flow measuring devices are compared, and the water quality constituents analyzed in the current EPA/USGS urban runoff program are listed, along with a discussion of automatic sampling techniques. The typical installation of the USGS urban hydrology monitoring system is described in detail.

Chapter 8 gives an overview of the principal large-scale planning and design urban runoff models. These include such programs as STORM, SEMSTORM, ILLUDAS, SWMM, RUNQUAL, HSPF, and others. The basic objectives and generalized flowcharts are given for these several models.

The application of the models, with examples, are discussed in chapter 9. Described are the combined sewer overflow project in the Dorchester Bay area of Metropolitan Boston which involved primarily the use of SWMM, the Four Mile Run (near Washington, D. C.) study which made use of both STORM and SWMM, and the application of SWMM to the city of Bucyrus, Ohio. The concept of the minor drainage system which carries limited flow and the major system which becomes active in extreme occurrences is one of the new stormwater management techniques used for Edmonton, Alberta, Canada. The several flow management alternatives are discussed. An example is given of the application of ILLUDAS and QUAL-ILLUDAS to Bloomington-Normal, Illinois. Finally, the chapter closes with examples of the application of the transport block of SWMM to analyze complex sewer systems subject to extensive surcharging.

Urban Hyrology and the Stormwater Problem

The potential climatological and hydrologic effects of urbanization are summarized in Tables 1 and 2. From the point of view

TABLE 1. Climatic Effects of Urbanization

Climatic Variable	Ratio of City to Environs
Solar radiation (insolation) in horizontal surfaces	0.85
Ultraviolet radiation, summer	0.95
Ultraviolet radiation, winter	0.70
Mean annual temperature greater in the city by 1^0 to 1.3^0 F	-
Annual mean relative humidity	0.94
Annual mean wind speed	0.75
Speed of extreme wind gusts	0.85
Frequency of calms	1.15
Frequency and amount of cloudiness	1.10
Frequency of fog, summer	1.30
Frequency of fog, winter	2.00
Total annual precipitation	1.10
Days with less than 2/10 in. of precipitation	1.10

From Lowry [1967].

TABLE 2. Potential Hydrologic Effects of Urbanization

Urbanizing Influence	Potential Hydrologic Response
Removal of trees and vegetation	Decrease in evapotranspiration and interception; increase in stream sedimentation
Initial construction of houses, streets, and culverts	Decrease infiltration and lowered groundwater table; increased storm flows and decreased base flows during dry periods
Complete development of residential, commercial, and industrial areas	Increased imperviousness reduces time of runoff concentration thereby increasing peak discharges and compressing the time distribution of flow; volume of runoff and flood damage potential greatly increased
Construction of storm drains and channel improvements	Local relief from flooding; concentration of floodwaters may aggravate flood problems downstream

From American Society of Civil Engineers Tech. Memo 24 [1974].

of surface hydrology, the major changes in the runoff process in urbanizing areas are due to two principal factors. The first factor is the covering of parts of the catchment with impervious surfaces: roofs, streets, sidewalks, parking lots. The infiltration capacity of impervious areas is essentially zero. The depression storage capacity is greatly reduced. Dust, dirt, sediments, and pollutants of various kinds, settled from the atmosphere and generated by the urban activities, accumulate on these impervious areas between storm events and are eventually washed off by the runoff during rains. The urban areas not covered by impervious material are usually relandscaped, covered with grass and vegetation, treated with fertilizers and insecticides. Frequently, the landscape modifications increase the overland flow which in turn increases the pollutant washoff.

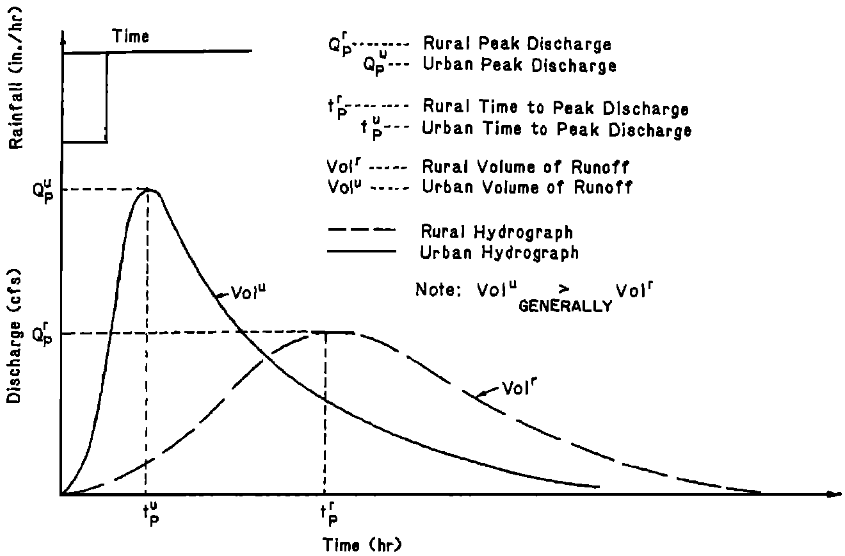

Fig. 1. Urbanization impacts on basin response without increased detention storage [after Riodan et al., 1978].

The second factor in the urban runoff process is the increased hydraulic conveyance of the flow channels. Natural channels are often straightened, deepened, and lined. Gutters, storm sewers, and drains are installed. These major changes in surface impervious fraction and hydraulic conveyance efficiency result in an increase in the runoff volume and peak flow rate. At the same time this peak runoff occurs sooner because of the increased flow velocities in channels (see Figure 1). Therefore the stormwater accumulates downstream in a greater amount and more quickly than with the natural stream channels.

The increased flow velocities enhance the transport of suspended solids and of pollutants and aggravate the scouring of channels. The pollution loading at the downstream end of the urban runoff conveyances is thus increased. This may be expressed, for example, by an increase both in the concentration and the mass emission rate of suspended solids and of the biological oxygen demand (BOD). When the storm and sanitary sewers are combined, during many storm events the capacity of the sewage treatment facilities, which are generally sized to treat the dry weather flow, may be greatly exceeded thus causing scour and raw sewage from combined sewers to overflow in the receiving stream, lake, or estuary.

The increase of urban storm runoff may cause flooding in low lying downstream areas causing disruption of traffic, flooding of underpasses, damage to houses and properties, and costly interruptions of urban activities in general. Existing drainage ditches, culverts, and bridge openings may have inadequate flow capacities. Newly developed residential areas may suffer erosion with a consequent reduction in property value. The decrease in infiltration also tends to decrease the amount of water available for recharge of the aquifers and thus tends to decrease the dry weather base flow in urban streams. At the same time, when aquifer recharge does takes place, it may be contaminated by road salt applied in winter time.

Urban storm drainage design, simply stated, aimed until recently at devising measures to protect urban development from stormwater. It consisted, for instance, of evaluating the peak rate of runoff and designing a network of pipes and ditches to collect safely and convey the stormwater downstream, away from the urbanized areas.

Increasing urbanization brought about the concept of stormwater management for environmental protection, in terms of both the control of the quality of the receiving waters and of the flooding of the downstream portions of watersheds. Thus the urban hydrologic picture was broadened to encompass a watershed in its entirety, wherein the occurrence of a rainstorm results in a string of interrelated events, shown schematically in Figure 2. A portion of the rainfall over the urbanized basin is captured by the depression storage from where it either infiltrates to groundwater or evaporates. Depending on the degree of development in the basin and the existence of a storm drainage system,

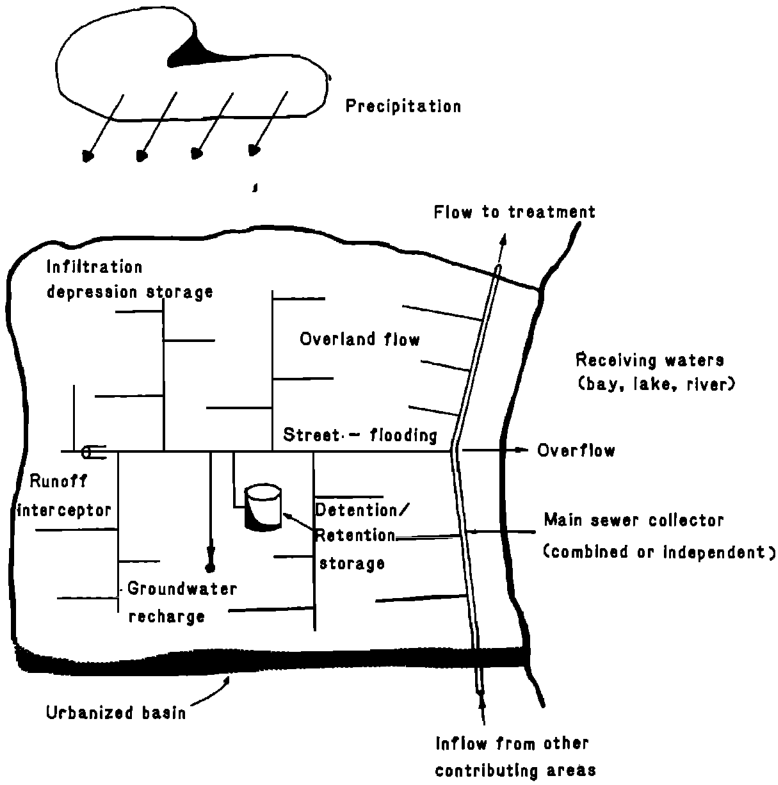

Fig. 2. Schematic description of an urban storm-drainage system.

all or a portion of the ensuing runoff is intercepted by storm drains or combined sewers and conveyed to treatment facilities, detention or retention storage facilities, or spilled at overflow points.

Historical Perspective of Urban Hydrology

Some of the important technical developments are listed in chronological order in Table 3. Also listed are the recent acts of the U.S. Congress which had an important impact on the development and orientation of urban hydrology research and on stormwater management. The table also includes some of the recent reviews of literature.

Interaction of Land Use and Urban Stormwater Runoff

Urban Geography

The major change in the runoff process in urbanizing areas is due to the covering of parts of the catchment with impervious materials: concrete, asphalt, roofs, etc. The sensitivity of the runoff volume and peak to the amount of impervious areas requires that they be carefully delineated. USGS quadrangle maps may be used for this purpose and may be supplemented by aerial photographs. USGS land use maps are becoming available and are expected to cover the entire country, except Alaska, by 1982. These are two-colored maps which depict residential, agricultural, commercial, and industrial use, and land cover such as forest and wetlands at a scale 1:250,000. The maps are prepared from photographs taken by high-flying (U-2) aircraft. Infrared photography is particularly useful because of the good contrast between the vegetation associated with pervious areas and the impervious areas [NASA, 1970]. Nevertheless, the proper identification of the impervious areas contributing directly to the drainage system and those draining on adjacent pervious areas usually requires a detailed field inspection.

LANDSAT satellite imagery can be used to classify a watershed according to the following categories: (1) highly impervious, (2) recent residential, (3) old residential, (4) highway, (5) bare land, (6) grass land, and (7) forest areas. For planning purposes the LANDSAT data would be a cost-effective data collection method for a wide variety of conditions [Ragan and Jackson, 1975]. The LANDSAT data have been used to calibrate the urban runoff model STORM and also in conjunction with the Soil Conservation Service (SCS) curve numbers for soil groups and land covers [Jackson and Ragan, 1977; Ragan and Jackson, 1980]. However, the complexity of urban areas introduces uncertainties in the interpretation of the satellite imagery requiring the use of supplemental information such as city and park boundaries, seasonal reflectance variations, and hierarchical classification schemes [Link and Aron, 1977].

TABLE 3. Historical Perspective of Urban Hydrology

Year	Author or Agency	Development or Specific Application to Urban Areas
1850	Mulvany	Rational formula
1880	Kuichling	First recorded application of rational formula in United States
1906	Lloyd-Davies	U.K. equivalent of rational formula
1930	Metcalf and Eddy	Zone principle
1932	Gregory and Arnold	General rational formula
1944	Hicks	Los Angeles hydrograph
1958	Bock and Viessman	Inlet hydrograph method
1960	Tholin and Keifer	Chicago hydrograph
1964	Jens and McPherson	State of the art
1964	Public Law 88-379 (Water Resources Research Act)	Office of Water Resources Research, Department of the Interior
1965	Urban Hydrology Research Council	Engineering Foundation Conference
1965	Eagleson and March	Unit hydrograph application to urban hydrology
1966 1968	Viessman	Unit hydrograph application to urban hydrology
1967	American Society of Civil Engineers	Urban Water Resources Research Program
1971	Environmental Protection Agency (Metcalf and Eddy, Inc. et al.)	Storm water management model (SWMM) and later versions
1972	Public Law 92-500 (Water Pollution Control Act)	Area-wide abatement and management of water pollution ('208' projects)

TABLE 3. Continued

Year	Author or Agency	Development or Specific Application to Urban Areas
1972	Rao et al.	Instantaneous unit hydrograph applied to urban hydrology
1974 1977	U.S. Army Corps of Engineers, Hydrologic Engineering Center	Storage-treatment-runoff model (STORM) and later improvements
1974	Illinois State Water Survey (Terstriep and Stall)	Illinois urban drainage simulator (ILLUDAS) and later improvements
1975	U.S. Department of Agriculture, Soil Conservation Service	Technical Release No. 55
1975	McPherson and Mangan	Summary of 28 technical memoranda ASCE program
1977	McPherson and Mangan	Closing discussion to above, reference to 20 additional reports issued by ASCE
1977	McPherson and Zuidema	Summary of international research in urban hydrologic modeling
1977 1978	UNESCO	Urban hydrology progress in 12 countries
1978	McPherson	Review of literature (309 references)
1979	Lystrom and Alley	USGS-EPA urban hydrology studies program, establishment of urban watershed data base and evaluation of stormwater management alternatives
1979	Public Law 95-217 (Clean Water Act)	Nonpoint source pollution studies
1979	McPherson	Review of literature (220 references)

TABLE 3. Continued

Year	Author or Agency	Development or Specific Application to Urban Areas
1979	Steele and Stefan	Review of literature (water quality)
1980	Alley, et al.	Parametric-deterministic model
1980	Delleur and Dendrou	Review of literature (183 references)
1980	Field	EPA research in urban stormwater pollution control (60 references)
1981	Delleur	Review of literature (77 references)
1974-1981	University of Kentucky	Proceedings yearly conferences

Quantitative Relationships Between Land Use and Urban Runoff

Brater and Sherill [1975] state that as the population density changes from 100 to 13,000 persons per square mile the peak rate of surface runoff for a given total surface runoff becomes about 10 times greater while the time parameters decrease to about one tenth of the values for rural areas. For basins in Michigan they find that the hydrologically significant impermeable areas in percent of the total area, HSIA, are related to the population density in thousands of persons per square mile, P_d, by HSIA = 1.38 P_d. Rantz [1971] has given a simple graphical relationship between the percent imperviousness and lot size and Heany and Nix [1977] have reported graphical relationships between percentage imperviousness and population density for several locations in the United States and Canada.

Graham et al. [1974], in a study of the metropolitan Washington D. C., region, found that the percent imperviousness was slightly better correlated to the number of households per acre, H, than to the population per acre, P, whereas the specific curb length

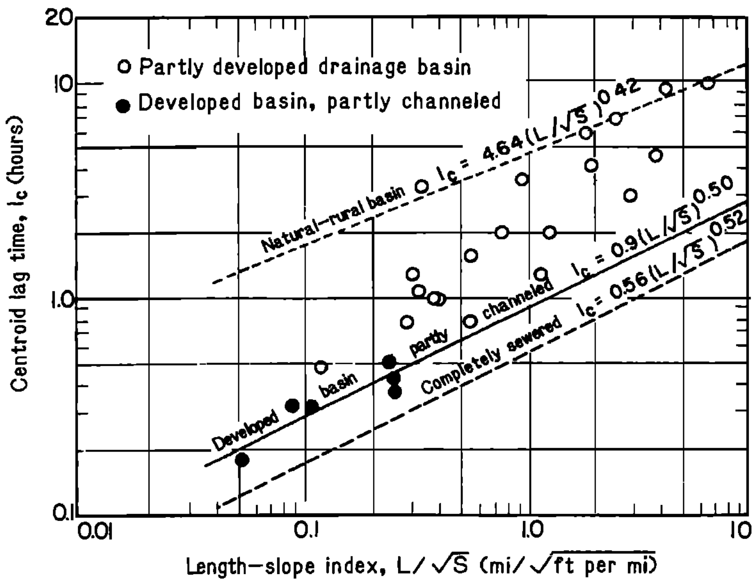

Fig. 3. Relation between lag time and slope [Anderson, 1970].

in feet per acre, often used in the estimation of pollutant buildup, is slightly better correlated to P than to H. Gluck and McCuen [1975] have related the principal types of urban land uses to demographic characteristics that can be obtained from census summaries or projections usually compiled by planning agencies. Their equations are based on data for the Anacostia River basin near Washington, D. C.

The second major change that occurs as a result of urbanization is the improvement of the hydraulic efficiency of the drainage network through the straightening and lining of channels, construction of sewers, culverts, etc. This results in a reduction of the lag time (time elapsed between the centroid of the rainfall hyetograph to the centroid of the runoff hydrograph) to 10-25% of its natural basin value, as may be seen from the relationship obtained by Anderson [1970] for basins near Washington, D. C. (see Figure 3). In a study of the stream network characteristics of a developing watershed in Iowa City, Iowa, Graf [1977] has shown that with suburbanization the cumulative length of links increases, causing a substantial increase in drainage density. Internal links become more significant than external links, and the basins tend to acquire a more rectangular shape. These changes tend to decrease the lag and increase the hydrograph peak. Graf suggests that design and planning of suburban drainage networks must take into account the significant role of internal links in the network, where corrective measures designed to counter flood problems can have the greatest effect. Likewise, Bannister [1979] in a study of the southeastern shore of Lake Michigan found that road networks modify natural stream systems by capture and rechannelization along road drainage ditches. This results in an increase in drainage density and erosion losses.

Leopold [1968] combined the effects of increase in percentage of impervious area and of area served by sewers on the mean annual flood on a 1-mi^2 basin in the vicinity of Washington, D. C., as shown in Figure 4. Rantz [1971] prepared similar curves for recurrence intervals of 2, 5, 10, 25, 50, and 100 years based on data for the San Francisco Bay Area in California (Figure 5).

In constrast to the quantity of urban runoff the methodologies for relating urban stormwater quality to basin and storm characteristics and to environmental practices are not well established. Lager et al. [1977] and Harremoës [1981] have characterized typical urban runoff pollutant loads and these are reported in chapter 6. The U.S. Geological Survey has been taking detailed measurements of rainfall and runoff (quantity and quality) at several sites in southern Florida. In an analysis of these data, Miller et al. [1978] developed regression equations for the runoff and seven major chemical constituents for a residential and a transportation site in Broward County, Florida. The original data are contained in two reports by Hardee et al. [1978] and by Mattraw et al. [1978]. The detailed basin descriptions have been given by Miller [1979]. The measurement techniques used at these sites are described in chapter 7. The regression equations for the two sites show that

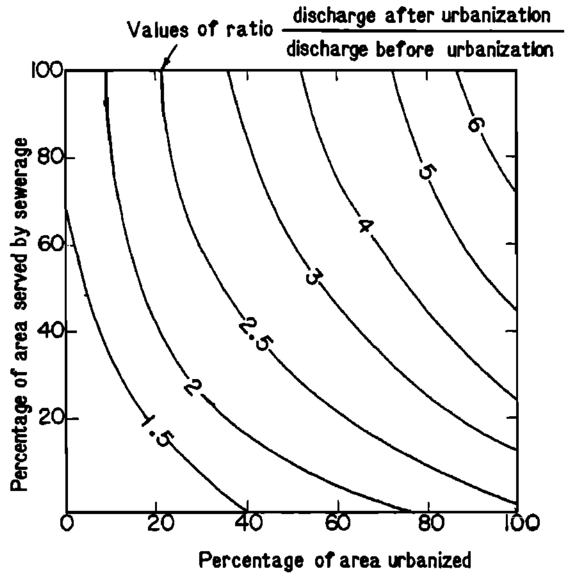

Fig. 4. The effect of urbanization and storm sewerage on mean annual flood for a 1-mi^2 basin [Leopold, 1968]. Note that 100% ubanization is approximately equivalent to 50% imperviousness.

the peak discharge dominated the water quality regression equations for the residential area (which is relatively pervious) and the depth of rainfall dominated the equation for the sewered transportation area. The Federal Highway Administration [1981] has documented the constituents of highway runoff and has monitored rainfall and related runoff at six highway sites for a period of 12 to 16 months. From these data a predictive model was developed for the runoff quantity and quality from three types of highway sites. The model is based on total rain, rainfall duration, dry days, and daily traffic values.

Urban Air Quality as Related to Storm Runoff Quality

The atmosphere, in general, provides an important source of consituents found in river waters draining both urban and rural basins. Usually, dry and wet depositions are considered separately. For analytical purposes it is convenient to distinguish three fractions of precipitation according to Lewis and Grant

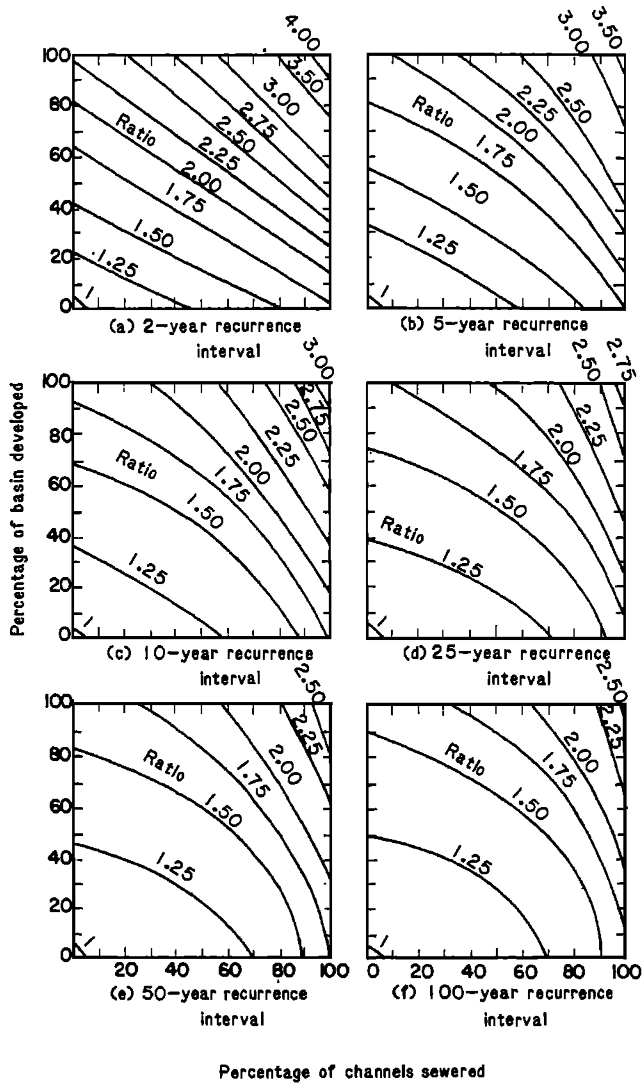

Fig. 5. Ratios of flood peak magnitude for urbanized basins to that for urbanized basins for floods of various occurrence intervals [Rantz, 1971]. Note the 100% urbanization is approximately equivalent to 50% imperviousness.

[1978]. There are (1) the dissolved materials in liquid precipitation; (2) the water soluble portion of dry precipitation, and (3) the water insoluble components of dry and wet precipitation. Fraction 3 is often neglected, and the separation of fraction 1 and 2 is recent.

Table 4 taken from Betson [1978] gives average bulk precipitation (dissolved material in wet precipitation plus water soluble materials that have leached out from dry fallout as the sample awaits processing). This bulk precipitation is given for urban and rural areas in the United States and Germany. The higher values for several constituents in the Knoxville sample are attributed to the fact that the sample areas were located less than 50 m from a relatively heavily traveled roadway.

Urban traffic generates two kinds of particulate matter: (1) that emanating from the vehicle exhaust which is measured as lead, chlorides, nitrates, and chemical oxygen demand (COD) [Shaheen, 1975], and (2) that resulting from the wear of tires and the roadway. In addition, the movement of the vehicles propels some of these particles a considerable distance away from the roadway. The sampling of traffic-generated particles requires that the deposition sampler be located very close to the ground surface, whereas in open areas the collector is usually mounted at 3 m above ground. A wet fall/dry fall atmospheric sampler used by the USGS is shown in chapter 7. Lewis and Grant [1978] give a review of the literature on the measurement and analysis of bulk precipitation constituents and give a design for a bulk precipitation collector for use in open areas. ASTM [1970] also gives the specifications for dustfall bucket installation and analysis.

Precipitation and runoff quality are measured simultaneously by the USGS at several locations as part of the EPA/USGS Nationwide Urban Runoff Program. As an example, the data for the highway area near Pompano Beach, Florida, have been published by Hardee et al. [1978]. From these data it is apparent that precipitation as rain, snow, and dry fall supplies large fluxes of many chem-

TABLE 4. Average Bulk Precipitation or Rainfall Concentrations[a]

	Urban			Rural			
Constituent	Knoxville Tennessee	St. Louis Missouri	Gottingen Germany	Gatlinburg Tennessee	Oak Ridge Tennessee	Blairsville Georgia	Canton-Clyde N. Carolina
Calcium, mg/l	3.8		3.0	0.2	1.0	0.1	
Magnesium, mg/l	0.74	0.24	0.7	0.03	0.19	0	
Sodium, mg/l	1.5		0.8	0.05	0.25	0.7	
Chloride, mg/l	4.0		0.9	0.15			
Sulfate, mg/l	7.1		8.8	3.2		2.9	19
pH	5.1	4.9		4.2		5.5	
Organic N, mg/l	2.5						1.2
NH_3-N, mg/l	0.41			0.18	0.13	0.3	0.8
NO_{2-3}-N, mg/l	0.47			0.28	0.25		0.62
Total PO_4, mg/l	1.1		1.1		0.12		1.5
Potassium, mg/l	2.6		1.0	0.1	0.2	0.8	1.2
Total iron, mg/l	0.18	0.67	1.4			0.7	
Manganese, mg/l	0.05	0.01	0.04		0.01[b]	0.09	
Lead, mg/l	0.05	0.03	0.04		0.02[b]	0.01	
Mercury, mg/l	0.0009		0.0015		0.0002[b]		
Suspended solids, mg/l	16						
COD, mg/l	65						

[a]From Betson [1978].

[b]Rainfall only.

ical constituents which are of importance in urban runoff studies. Ideally, wet fall samples should be collected for each observed storm, and dry fall samples should be collected at least once every 2 months.

Stormwater Planning in the Urban Metroplex

Urban Water Balance

Although there are some global and national water balances, there are few water inventories for cities, in spite of the fact that a water budget would seem essential in comprehensive planning in urban areas. McPherson [1973] defines water balance inventory as the determination of the quantity and quality of water since its appearance as precipitation through its departure from a metropolis as runoff and evapotranspiration. McPherson [1975] has given a schematic water balance based on a national average of 32 in/y of rainfall for a hypothetical 150 mi^2 urban area with one million inhabitants. The average urban runoff to receiving water bodies is 500,000 tons/d. It is not possible, however, to give precise figures on the breakdown of this figure in flows in storm and combined sewers, flow over unsewered land, and treatment plants releases because there are too few data on total flow quantities in storm and combined sewers. However, there is approximately twice as much storm sewer flow as there is runoff in combined sewers or flow over unsewered land [McPherson, 1978].

Stormwater Management

Urban runoff control measures can be applied at the source, along the line or at the end of the line. The controls can be either structural or nonstructural. Table 5 lists a number of measures for reducing and delaying urban storm runoff taken from U.S. Department of Agriculture, Soil Conservation Service (SCS) [1975], and Table 6 is a list of stream quality management procedures considered by the Illinois Environmental Protection Agency [Bachman, 1979]. Controls at the source prevent flow

TABLE 5. Measures for Reducing and Delaying Urban Storm Runoff

Area	Reducing Runoff	Delaying Runoff
Large flat roof	Cistern storage Rooftop gardens Pool storage or fountain storage Sod roof cover	Ponding on roof by constricted downspouts Increasing roof roughness Rippled roof Graveled roof
Parking lots	Porous pavement Gravel parking lots Porous or punctured asphalt Concrete vaults and cisterns beneath parking lots in high value areas Vegetated ponding areas around parking lots Gravel trenches	Grassy strips on parking lots Grassed waterways draining parking lot Ponding and detention measures for impervious areas Rippled pavement Depressions Basins
Residential	Cisterns for individual homes or groups of homes Gravel driveways (porous) Contoured landscape Groundwater recharge Perforated pipe Gravel (sand) Trench Porous pipe Dry wells Vegetated depressions	Reservoir or detention basin Planting a high delaying grass (high roughness) Gravel driveways Grassy gutters or channels Increased length of travel of runoff by means of gutters, diversions, etc.
General	Gravel alleys Porous sidewalks Mulched planters	Gravel alleys

From U.S. Department of Agriculture, Soil Conservation Service [1975].

TABLE 6. Urban Runoff Control Measures

Nonstructural	Structural
At the Source	
Air pollution controls and regulations	Air pollution control devices
Animal control	Covered parking lot structures
Auto inspections	Environmental design alternatives
Fertilizer and irrigation controls	Rain gutter runoff control
Land use control alternatives	
Leaf collection	
Litter ordinances	
Onsite detention and retention ordinances	
Refuse collection	
Road salting and sanding	
Sanitary code enforcement	
Stockpile protection	
Studded snow tire restriction	
Unleaded gas	
Waste oil recycling	
Construction site erosion controls	
Along the Line	
Catch basin maintenance	Computerized monitoring and control
Parking lot maintenance	Inline detention
Road maintenance	Parking lot storage basins
Streetsweeping alternatives	Porous pavements
End of the Line	
Discharge permits	Storage treatment alternatives
Water quality monitoring	Sedimentation basin
	Dissolved air flotation
	Oil skimming
	Storage effluent treatment alternatives
	Disinfection
	Screening
	Biological treatment
	Physical-chemical treatment
	Land treatment
	Swirl concentrator

and/or pollution from reaching the sewer system, while controls along the line prevent the flow and/or pollutants from being carried into receiving waters. The controls at the end of the line usually consist of a flow-regulating storage facility and some type of biological or physical-chemical treatment. Some alternatives must be carefully analyzed. For example, when several detention basins are used, their interaction must be considered, since a combination of the timing of their releases could aggravate downstream flooding rather than alleviating it.

Further listing of management alternatives and some costs are given by Wanielista [1978, Chap. 7]. The efficency and costs of many of these procedures vary from one location to another. Many of the alternatives, such as on site storage basins, erosion control, and flow reduction alternatives, may be feasible only for areas of new development. The several alternatives may be compared making use of production theory and marginal cost analysis [Heany and Nix, 1977]. On the basis of technical feasibility and cost the list of alternatives of Table 6 was reduced by the Illinois EPA for application in eight northeastern Illinois locations to the following alternatives: (1) storage-treatment, (2) street sweeping, (3) road salting control, (4) planning, (5) erosion control, (6) runoff control, (7) waste oil recycling, and (8) unleaded gasoline. Street sweeping can be improved by using better equipment and controlling the sweeper speed and the frequency of passes. Reduction of road salting may be a viable control measure. Waste oil recycling has been encouraged by the Illinois Institute of Natural Resources. Land use plans and ordinances should recognize environmental concerns. Construction erosion control has a high potential for reduction of pollutant loading, for improved esthetics, and for reduction of street and sewer cleanups. Runoff controls reduce the ability of storm runoff to transport pollutants and sediments and may reduce the need for capital-intensive storm sewer systems.

Stormwater Models

Need, definition, and components. The analysis of the complex urban systems in Figure 2 under stochastic rainstorm conditions far exceeds today's analytical capabilities for simple closed-form solutions. Consequently, engineers and planners have scale mathematical simulation models developed to address various levels of storm-drainage related problems. While all models revolve around the same basic process, the hydrologic cycle, they differ in the purpose of their development and, consequently, in their treatment of different parts of the hydrologic cycle.

A model, in the context of this monograph, is a mathematical description of physical, chemical, and bacteriological processes or phenomena. The features and components encountered most commonly in storm drainage models grouped by hydrologic and hydraulic processes are presented in Table 7. Stormwater runoff is the focal point of the urban hydrologic component, and the evaluation of the performance of the man-made systems is the central point of the hydraulic component.

Modeling approaches. Two basic approaches have been used in modeling the urban hydrologic processes, based on the scale and

TABLE 7. Major Components of Urban Stormwater Models

Hydrologic Processes	Hydraulic Processes
Rainfall	Routing through drainage pipes
Initial abstractions	Routing through sewer trunks
Depression storage	Routing through storage facilities
Evaporation	Routing though treatment plants
Infiltration	Control devices
Runoff	Natural channels
Quantity	
Quality	
Diffusion/dispersion of pollutants in receiving waters	
Erosion/sedimentation	

level of investigation; the microapproach and the macroapproach. In the macroapproach, only the relevant characteristics of a system are retained in a cause-effect or input-output pattern. The transformation of the rainfall pattern, or input, into the time distribution of runoff, or output, is detected from actual data and is described by a limited number of parameters. Although these parameters do represent, in a lumped manner, some of the properties of the system, they seldom have a direct physical interpretation. This approach, also known as lumped parameter model approach, has been used extensively in urban hydrology in the class of models known as conceptual linear and nonlinear models. These models are deterministic in nature. They seek to establish relationships between average values of physical quantities. Other models consider that certain of the variables describing the system or certain of the coefficients in the dynamic equations may include a random component. For example, the rainfall is known to be highly variable in space and time and could be considered as a random input. These models are stochastic.

The microapproach consists of modeling all the physical processes involved in the system to a degree of minute detail. This approach is also known as distributed parameter model. However, some of the insight gained by the physical interpretation of the large number of parameters involved is counterbalanced by the difficulty of their evaluation. Small portions of the urban hydrologic process are best modeled in this manner. Many physically based models have been used in large, extensive simulation models. A major distinction must be drawn, however, between the large, physically based models and models of man-made systems that simulate the performance of storm-drainage systems including network of collectors, detention and/or retention storages, and treatment plants. In fact, simulation models of such man-made systems often use both lumped parameter (conceptual) and distributed parameter physical process models. Chapter 8 deals exclusively with large-scale simulation models of man-made systems and draws on models of physical processes presented in chapters 5 and 6.

Models for Forecasting Land Use

Land use models are needed for projecting the future drainage systems and sewer services. Some of the simpler models give the future population distribution decreasing exponentially as a function of distance. Adjustments can be made to allow for the fact that the population density at the center may be less than that predicted by the exponential formula and to reflect the rate of growth as a function of time and distance [Newling, 1966]. Residential population density profiles through time may be represented by a partial differential equation, similar to that for recharge in an unconfined aquifer, which describes the population density, the birth rate, the death rate, the out migration rate, and indicators which vary with topography, transportation network, and other land use considerations [Meier, 1976].

Computer-oriented models are required for more detailed description of the land use and population distribution. The dynamic land use allocation model (DYLAM), originally developed by Walsh and Grava [1969], has the advantage of simplicity and suitability to hydrologic studies. Dendrou et al. [1978a] extended the DYLAM model to form the model LANDUSE, which was used in conjunction with the drainage planning model STORM (see chapter 8) to form the computer package LANDSTORM. The model LANDUSE transforms the aggregated land use demand for a region into actual allocations at the end of the planning horizon. Proprietary extended versions DYLAM II and III and their applications to Fairfax County, Virginia, are described by Seader [1975]. Further discussion on this application, a flow chart, and application to Cleveland, Ohio, and Lakewood, Colorado, are given by McPherson [1975]. Some models such as ADAP (areal design and planning tool, [Males et al., 1980]) use a triangular grid network to form the spatial data bases describing land information and natural and man-made drainage.

Among other land use models that may be useful in the evaluation of the land/water interface the following six models may be considered: PLUM (project land use model [Rosenthal et al., 1972]), EMPIRIC [Dickey, 1975], CRP (community renewal program [Little, Arthur D., Inc., 1966]), REHSM (regional housing simulation model [Sinha, 1977]), TOPAZ (technique for optimum placement of activities in zones [Dickey et al., 1974]), and LUPDUM (land use plan design model [Sinha et al., 1973]).

Coordination of Urban Subbasins

Urban storm drainage basins usually behave independently. Where these basins are large enough to justify local and individual storage facilities and a treatment plant, they can be studied individually. Usually, the topographic configuration is such that the flow from the uphill basins will be added to those of the downhill ones before being released to the receiving body of water. Further, the size of urban subbasins usually excludes the alternative of individual treatment plants. The subbasins in an urbanized watershed usually form a treelike branching graph. On the basis of this branching configuration, two levels of aggregation emerge (Figure 6): a first level where a land use component provides urban growth information to subbasin models (the subbasins are optimized individually) and a second level where the subbasins are coordinated to satisfy the objectives of the city-wide storm drainage system [Dendrou et al., 1978b].

New Directions in Urban Hydrology and Stormwater Management

The decade of the 1970's has been dominated by the effects of the Federal Water Pollution Control Act (Public Law 92-500) which mandated that regional planning for water pollution abatement management be undertaken in metropolitan areas. As a result, the attention focussed on urban runoff quantity and quality and stormwater management. The Clean Water Act of 1977 (Public Law 95-217)

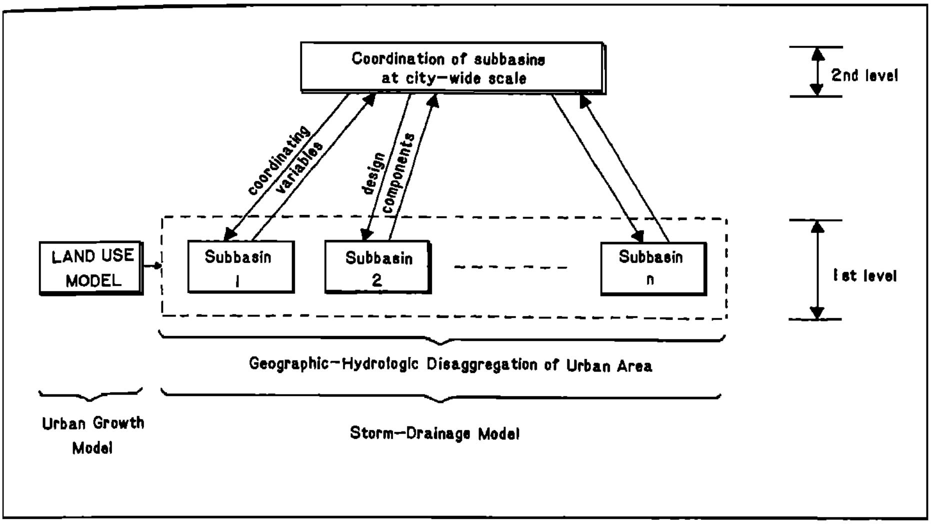

Fig. 6. Multilevel representation of urban storm drainage models [Dendrou et al., 1978a].

recognized the existing limitations in analyzing pollution from non-point sources.

The decade of the 1980's will most likely be dominated by the results of the USGS/EPA urban hydrology program initiated in 1979. At present, 31 locations have been selected in the United States. These studies will provide an important urban hydrology data base which will result in improved methods of analysis and more suitable decisions for the urban stormwater management.

In the analysis and development of models it is expected that there will be less empiricism and more reliance on the detailed movement of urban runoff and its associated constituents. Such tendencies are already present in the model ARM (agricultural runoff model [Donigian et al., 1977]) and the HSPF model (see chapter 8). Such model refinements are expected in the simulation of specific processes such as sediment transport, adsorption, volatilization, and photosynthesis. The trend is not toward the development of new models but to the improvement of subsystem simulation, at least in the short term [Torno, 1979; Sonnen, 1980].

Another important advance in the state of the art is the realization that atmospheric transport of gaseous and particulate emission from industries, automobiles, and other sources of atmospheric pollution has a major impact on precipitation chemistry and on regional water quality. [Steele and Stefan, 1979]. This indicates that the control of urban runoff pollution may not be independent of the control of air pollution.

The application of urban runoff models will continue to be affected by the development of computer technology. Two trends are discernible: one is the increase in speed and memory capacity of large computers and the other is the development and improvement of mini computers. The first development will make it possible to accommodate improvements in the simulation of the several processes involved in urban drainage models. The second development will make it possible to obtain more realistic tabletop techniques and to adapt some models to mini computers. For example, an adaptation of ILLUDAS to a micro computer has been done by Patry et al. [1979]. The use of computer-based mathematical models in the planning and design of urban drainage will be fully established.

Finally, substantial improvement of the operation of storm sewer systems appears to be possible with real-time forecasting of the rainfall time and space distributions. Such efforts now appear possible through the recent improvements in measurements of rainfall by radar, in large cities such as Chicago [Vogel and Changnon, 1981] and Montreal [Austin, 1980], and for flash flood forecasting [Johnson and Harris, 1981].

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2 RAINFALL FOR URBAN STORMWATER DESIGN

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Introduction

A design rainfall can be defined as a rainfall event, either historical or artificial, which is used as a basis for determining the design for a proposed drainage or water-related system. The design rainfall is chosen under the general assumption that if the system is designed with the capability of accomodating the event at full capacity, the operation of the system will meet the design objectives.

Two basic types of design rainfall can be identified. The first and most common type is an artificial event, commonly termed a design storm, which is based on depth-duration-frequency analysis of historical data. This is used in connection with drainage design or other problems where peak flow is of primary consideration. The second type involves the direct use of actual historical rainfall events. This type might be used in problems where runoff volume is of interest such as in the design of detention storage reservoirs or where water quality considerations are important. The latter type may not, in fact, be a single event but a series of events, and in that sense the term design rainfall event is misleading. Therefore hereinafter in this chapter the term 'design rainfall' refers to the first type. Most of the discussion will be devoted to various aspects of this type because of its extensive case.

A design rainfall generally has the following components or characteristics which serve to uniquely describe it: (1) frequency or return period, (2) total depth, (3) duration, and (4) time distribution of depth or intensity (hyetograph). The areal distribution of rainfall is usually assumed to be uniform with the justification that the drainage area is small enough to make the areal effects negligible.

In a typical design situation the design frequency or return period is first chosen. This choice presumably reflects an acceptable trade-off between construction costs and the damage costs associated with flooding, delays, and inconvenience. Inherent in this choice is the assumption that the return period of the design rainfall is equal to the return period associated with exceeding the design capacity of the system. The choice of the other components of the design rainfall varies, depending on the particular design procedure being used.

Perhaps the most common use of the design rainfall is in connection with the rational method as used to size storm and combined sewers [American Society of Civil Engineers (ASCE), 1969]. In this application the design rainfall hyetograph is assumed to be uniform, and its duration is taken as the time of concentration for the catchment at the point of interest or outlet. The time of concentration can be defined as the longest travel time to the outlet for surface flow. Although its calculation is not precise, it is a useful concept since it implies that all of the upstream area is contributing to the flow. Methods of estimation are given in chapter 4.

In this chapter, the development of the components of a design rainfall are discussed, some examples are presented some limitations and problems associated with its use are discussed, and a brief summary of current research on the subject is presented.

Rainfall Data

The development of design rainfall requires detailed rainfall data, sometimes in time increments as short as 5 min. The largest data source is the U.S. Weather Service. Basic or raw data in various formats can be obtained from the National Climatic Center, NOAA Environmental Data Service in Asheville, North Carolina.

However, small time increment data for some first-order stations has been published in basically three formats since 1896 [U.S. Weather Bureau, 1958]. These data are termed 'excessive precipitation' by the Weather Service. Because these data have been used for design rainfall development, a short summary of sources and format is given in the following paragraphs.

For the period 1896-1934, excessive precipitation data were published in the annual reports of the chief of the Weather Bureau. Also, these same data for 1897-1920 were published in the Monthly Weather Review. From 1935-1949 the data appeared in the U.S. Meterological Yearbook, and since 1950 they have been included in the annual publication, Climatological Data, National Summary.

From 1904 through 1972, the Weather Service has set down a criterion for classifying a storm event as 'excessive.' For most of the states this criterion has been constant since that time, namely, that the cumulative precipitation p_1 (in inches) must exceed

$$P = 0.01t + 0.20 \quad (1)$$

where t is the time in minutes, at some point during the event. In 1934, (1) was changed to

$$P = 0.02t + 0.30 \quad (2)$$

for the states of Alabama, Arkansas, Florida, Georgia, Louisiana, Mississippi, North Carolina, Oklahoma, South Carolina, Tennessee, and Texas. In 1949, (2) was dropped as a criterion, and (1) has been used for all states since then.

It is important to be aware of exactly how the data were recorded since there were fundamental changes made in the procedure in 1936 and 1972. Prior to 1936 the cumulative precipitation data were examined to identify when (1) was exceeded. The time t in (1) and (2) was from the beginning of the excessive period, not necessarily from the beginning of the event. Accumulated depths were then listed at 5-min increments until the excessive criterion was no longer satisfied for a period of at least 30 min. It should be noted that this procedure resulted in descriptions of only the most intense portions of the events and that the accumulative values were presented in the time order in which they occurred. Starting in 1936 to 1972, the data were no longer necessarily listed in their historical time order. The excessive criterion was still used to identify events to be tabulated. However, the maximum precipitation for each event was tabulated for each of the durations 5, 10, 15, 20, 30, 45, 60, 80, 100, 120, 150, and 180 min regardless of the time order in which they occurred. The criterion for separation of events was a period of 180 min in which (1) is not satisfied. Thus the tabulation no longer represents an historical event but simply the largest precipitation during various nonconsecutive time periods within the event. From 1973 to date, the excessive criterion has been dropped, and the monthly maximum amounts for the above durations are tabulated for the first-order stations along with an annual summary. It is important to recognize that this latter change represents a different time series when performing statistical analyses with these data. Prior to 1973 the tabulated data were appropriate for the formation of partial duration series, whereas after 1973 the data can be used to form monthly maximum series. These series may not differ significantly, but the user should be aware of the fundamental change which occurred in 1973.

In addition to the Weather Service data, the U.S. Department of Agriculture, Science and Education Administration (formerly Agricultural Research Service) maintains a hydrologic data bank containing continuous rainfall data from 390 precipitation stations on 242 experimental watersheds [Hershfield, 1971]. These data are in the form of mass curve breakpoints and require reduction before they can be analyzed.

The U.S. Geological Survey also maintains files of selected precipitation data in their WATSTOR system.

Depth-Duration-Frequency Analysis

A useful method of presenting rainfall data for design purposes is in the form of relationships between the average depth or intensity of rainfall for various durations and return periods or frequencies. The return period is defined as the average period of time (usually expressed in years) in which the rainfall of a specified depth or intensity for a specified duration is equaled or exceeded, i.e., the inverse of the exceedance probability of the rainfall. Data in this form serve as a basis for the development of design rainfall and thus can be more useful than in the basic format described earlier.

The source of depth-duration-frequency data on a national basis is the reports published by the National Weather Service (NWS). These reports present the data in the form of a series of isopluvial maps containing lines of constant rainfall depth for specified durations and return periods. The paper by Hershfield [1961] (hereinafter referred to as TP 40) contains maps for the entire United States, covering durations of 30 min to 24 hours and return periods of 1 to 100 years together with procedures for interpolation between the values given. In 1973 the data in TP 40 for the 11 western states was updated and issued as separate volumes of NOAA Atlas 2 [Miller et al. 1973]. These volumes provide maps for 6- and 24-hour durations and return periods of 2 to 100 years and procedures for extrapolating to duration as short as 5 min. Of particular utility in urban areas is the report by Frederick et al. [1977] which provides maps for the central and eastern United States for durations of 5 to 60 min. Data from these reports can be abstracted and depth values converted to intensity and presented graphically, as shown in Figure 1. An example of this procedure is presented by Jens [1979], and Figure 1 shows the results of this procedure for Santa Fe, New Mexico. The intensity variable in this figure could be transformed into depth by multiplying by the duration, producing another common graphical display format.

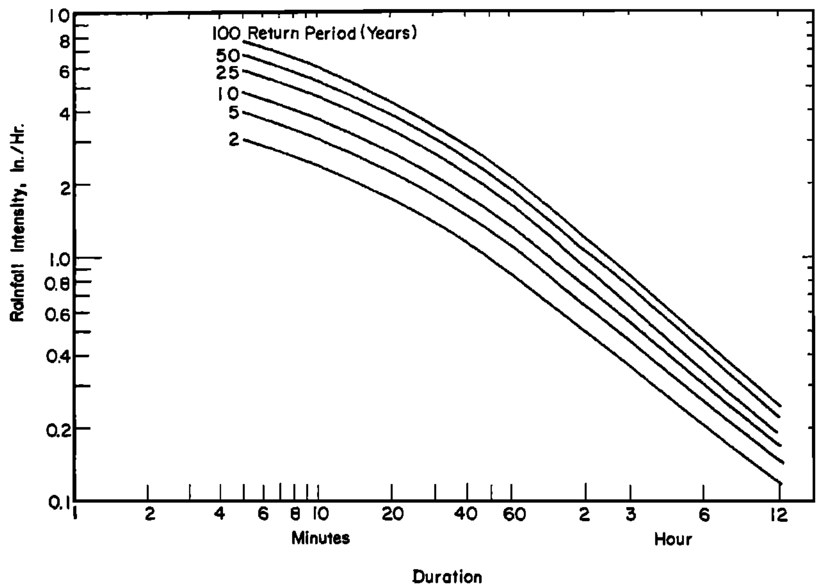

Fig. 1. Intensity-duration-frequency curves for Santa Fe, New Mexico.

Although the NWS reports are very useful, some interpolation and smoothing techniques were used in their development. Therefore if local rainfall data for a specific site are available and the effort can be economically justified, an independent depth-duration-frequency analysis may be carried out.

General Procedure

The procedure for performing a depth-duration-frequency analysis involves the following steps:

1. Starting with essentially continuous rainfall data, established a criterion for identifying independent events. This criterion could be a minimum time interval during which average rainfall is zero or very low. For example, the National Weather Service uses a period of 180 min with less than 1.80 in. of rainfall or an average intensity of 0.6 in./h. Another approach is to perform an auto correlation analysis for various durations to establish the time lag between rainfall periods such that there is no significant statistical correlation between them.

2. Identify a series of durations to be analyzed. For urban design, durations of less than 60 min and sometimes as small as 5 min are desirable.

3. For each analysis duration, scan the events which have durations equal to or greater than that value, identifying the largest rainfall which occurred during any time period equal to the analysis duration within each event.

4. Identify a partial duration series for each analysis duration by ranking the depths and choosing the N largest values from a record length of N years, i.e., an annual exceedance series. In some cases, it may be impractical to formulate a partial duration series as would be the case if NWS excessive precipitation tabulations were used after 1973. In this case, an annual series can be used and the result converted to a corresponding partial series result utilizing either a theoretical relationship between them [Chow, 1964] or an empirical relationship, as was done by the NWS in the development of TP 40, as given in Table 1.

5. For each series, assign exceedance probability or return period estimates using a plotting position formula. A number of such formulas have been proposed. For the annual exceedance series the following have been used:

$$T = \frac{m}{N} \tag{3a}$$

$$T = \frac{2m - 1}{2N} \tag{3b}$$

and for the annual maximum series

$$T = \frac{m}{N + 1} \tag{3c}$$

In all of these formulas, T is the return period in years and m is rank for each depth in decreasing order of magnitude. Other formulas have been proposed as well [Chow, 1964], and they differ

TABLE 1. Empirical Factors for Converting Partial Duration Series to Annual Series

Return Period (years)	Conversion Factor
2	0.88
5	0.96
10	0.99

Table adapted from Hershfield [1961].

primarily in the return period estimates for the largest events in the series.

6. Plot the data on same type of graph paper, and fit a curve to the data for each analysis duration either by eye or some analytical procedure. There is no uniformly established graph paper or probability distribution to be used for rainfall data. Hershfield [1961] utilized an empirical distribution in TP 40 for return periods from 1 to 10 years and the Gumbel or extremal type I distribution for return periods above 20 years. Other distributions that have been used include the log normal, log Pearson, gamma, and exponential.

An example of this procedure is shown in Table 2 and Figure 2. The basic data was supplied by the Science and Education Administration (formerly Agricultural Research Service) of the U.S. Department of Agriculture. The data are for a single station at Coshocton, Ohio, for a period of 25 years. Table 2 shows the magnitude and event date for five selected durations and Figure 2 shows the data plotted for 15- and 240- min durations along with best fit lines for intermediate durations. These lines were determined using a least squares fit procedure with the largest value (rank = 1) for each duration excluded since it was judged to have been assigned an inaccurate plotting position from (3a) which was used in this example. It should be noted that rainfall intensity could have been plotted instead of depth by simply dividing by the associated analysis duration.

One alternative step in the above procedure is to use the

TABLE 2. Maximum Depth Time Series

Rank	Return Period (years)	Maximum Depth (in.) and Date for Duration Shown				
		15 min	30 min	60 min	120 min	240 min
1	25.00	1.423 6/12/57	2.625 6/12/57	3.220 6/12/57	3.421 6/12/57	3.040 7/4/69
2	12.50	0.940 7/11/51	1.326 7/24/68	1.830 6/27/75	1.900 7/27/69	2.085 6/28/57
3	8.33	0.920 6/12/59	1.238 5/13/64	1.756 7/27/69	1.883 8/21/60	2.062 8/21/60
4	6.25	0.910 5/13/64	1.177 6/23/52	1.510 8/21/60	1.792 7/4/69	1.900 7/27/69
5	5.00	0.890 6/27/75	1.170 7/22/58	1.431 7/24/68	1.733 7/24/68	1.900 7/24/68
6	4.17	0.884 6/23/52	1.167 6/27/75	1.375 7/22/58	1.703 8/4/59	1.771 8/4/59
7	3.57	0.860 8/14/73	1.149 6/17/70	1.313 6/17/70	1.623 6/12/59	1.714 4/25/61
8	3.13	0.810 7/27/69	1.087 6/15/75	1.306 5/13/64	1.609 6/28/57	1.595 8/14/57
9	2.78	0.805 6/22/51	1.063 8/22/51	1.290 6/23/52	1.604 6/13/72	1.524 11/15/55
10	2.50	0.783 6/24/56	1.060 7/11/51	1.269 4/25/61	1.600 7/28/61	1.480 6/23/69
11	2.27	0.770 8/15/75	1.040 6/12/59	1.225 6/12/59	1.570 4/25/61	1.470 9/24/70
12	2.08	0.770 7/22/58	1.037 7/19/67	1.213 7/4/69	1.482 7/22/58	1.470 8/11/64
13	1.92	0.750 7/10/73	1.027 9/5/75	1.204 6/13/72	1.393 8/11/64	1.460 6/24/57
14	1.79	0.750 6/17/70	1.023 7/10/73	1.203 8/11/64	1.353 5/13/64	1.367 5/13/64
15	1.67	0.733 7/19/67	1.000 7/10/55	1.200 8/3/63	1.351 9/24/70	1.343 5/5/71
16	1.56	0.732 7/30/58	0.975 7/27/69	1.194 8/2/64	1.335 6/23/69	1.330 9/12/57
17	1.47	0.710 7/3/52	0.972 7/30/58	1.192 9/12/57	1.310 8/14/57	1.317 6/23/52
18	1.39	0.707 8/3/63	0.934 8/27/74	1.174 7/28/61	1.305 6/24/57	1.300 4/23/70
19	1.32	0.700 7/24/68	0.919 7/28/61	1.143 6/22/51	1.300 6/11/60	1.300 9/13/51
20	1.25	0.700 6/4/63	0.907 9/12/57	1.130 9/24/70	1.300 6/23/52	1.280 8/31/65
21	1.19	0.700 6/22/60	0.890 8/14/73	1.130 7/19/67	1.290 8/2/64	1.270 5/10/73
22	1.14	0.692 4/3/74	0.880 6/24/56	1.109 9/5/75	1.274 9/12/57	1.255 3/14/73
23	1.09	0.688 8/27/74	0.873 6/11/60	1.095 7/6/58	1.230 7/3/52	1.220 7/22/57
24	1.04	0.687 9/12/57	0.869 7/4/69	1.094 6/28/57	1.220 7/6/58	1.200 6/22/51
25	1.00	0.670 4/13/55	0.850 8/11/64	1.063 8/27/74	1.200 9/5/75	1.180 9/13/62

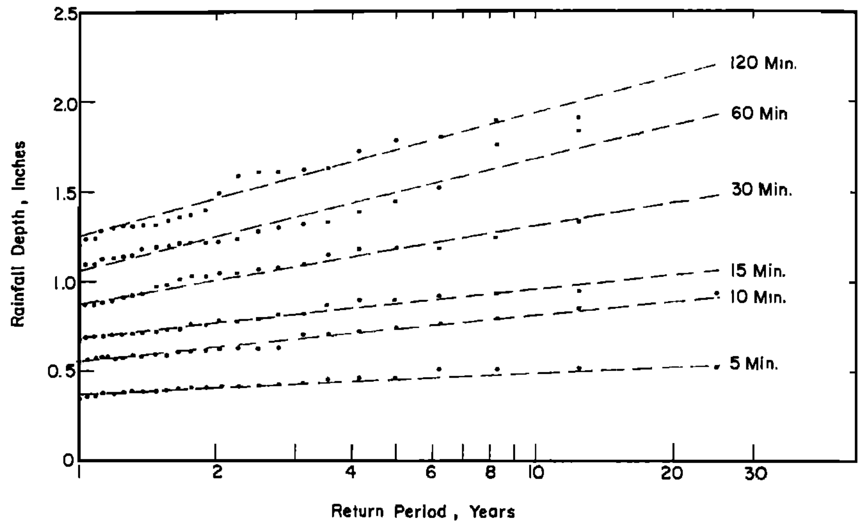

Fig. 2. Depth-duration-frequency data for Coshocton, Ohio.

extended duration approach in which the duration of all events is extended to the largest analysis duration. This means that all events are scanned for each analysis duration rather than eliminating those with shorter durations. The result is to increase the maximum depths in the resulting time series, with the effect becoming more severe with increasing analysis duration. In this example, essentially no increase occurred until the duration exceeded 60 min. For 120-min duration the depths increased about 2% above those shown in Table 2, with increases of approximately 14% for 240 min. Note in Table 2 that if the extended duration procedure were used, the storm of June 12, 1957, would be included and ranked first in the 240-min series.

The event dates associated with the depths in Table 2 are shown to illustrate the synthetic nature of the procedure. Each depth is a portion of an actual event. The arrows show the movement of several events through the rankings for the various analysis durations. It should be clear that any design rainfall developed from this procedure no longer represents a single historical event but is made up of maximum portions of a number of historical events.

Mathematical Depth-Duration-Frequency Relationships

There are situations were it is convenient to express graphical information such as Figure 1 in mathematical form. Several equations have been used for this purpose, all of which are similar in format:

$$i = \frac{a}{(t_d + c)^b} \tag{4a}$$

$$i = \frac{a}{t_d^b + c} \tag{4b}$$

in which a is a constant for a given return period, b and c are constants independent of return period, i is the rainfall intensity, and t_d is the duration. There is no theoretical basis for these equations; they have simply been found to fit the graphical analysis results rather well.

The value of the constants can be determined by a curve-fitting procedure such as least squares or by choosing three points in the range of interest along the intensity-duration curve for a specific return period, substituting into the desired equation and solving the resulting three simultaneous equations. Chen [1976] developed a procedure for evaluating the constants in (4a) which is based on the ratio of the 1- to 24-hour rainfall depth for a specific return period.

Caution should be used in applying (4) outside of the range of intensity and duration for which the constants have been determined. Furthermore, the constants may vary considerably with location and thus should be evaluated locally. Table 3 shows a comparison of the constants in (4a) and (4b) for various locations for a 10-year return period. The constants for (4a) for the first seven cities listed in Table 3 were developed by Chen

[1976]. The constants for the remaining cities were developed from data summarized by Chen [1976] as taken from TP 40 [Hershfield, 1961] for durations from 5 min to 12 hours, using an optmization technique to fit the data to (4a). The constants for (4b) were developed by fitting intensities for 5-, 30-, and 120-min durations to the equation, and the data were taken from Chen [1976]. The purpose of Table 3 is to indicate typical values for various locations across the United States and to show the variability of the constants. The lack of transferability of the values is apparent.

TABLE 3. Constants for Equations (4) for 10-Year Return Period at Various Locations

	$i = \frac{a}{(t_d+c)^b}$			$i = \frac{a}{t_d^b+c}$		
Location	a	b	c	a	b	c
Chicago	60.9	0.81	9.56	94.9	0.88	9.04
Denver	50.8	0.84	10.50	96.6	0.97	13.90
Houston	98.3	0.80	9.30	97.4	0.77	4.80
Los Angeles	10.9	0.51	1.15	20.3	0.63	2.06
Miami	79.9	0.73	7.24	124.2	0.81	6.19
New York	51.4	0.75	7.85	78.1	0.82	6.57
Olympia	6.3	0.40	0.60	13.2	0.64	2.22
Atlanta	64.1	0.76	8.16	97.5	0.83	6.88
Helena	30.8	0.81	9.56	36.8	0.83	6.46
St. Louis	61.0	0.78	8.96	104.7	0.89	9.44
Cleveland	47.6	0.79	8.86	73.7	0.86	8.25
Santa Fe	32.2	0.76	8.54	62.5	0.89	9.10

Constants correspond to i in in./h and t_d in min.

Areal Effect on Point Rainfall

Studies of the effect of area on average areal rainfall indicate that point rainfall values should be reduced as the area under consideration increases. Figure 3 shows the reductions recommended by the National Weather Service [Miller et al., 1973]. This figure was developed for any location and for return periods from 2 to 100 years. Current efforts are under way by the National Weather Service to investigate the geographical variation of depth-area ratios, utilizing new methodology and data from dense rain gage networks [Myers and Zehr, 1980]. Many urban drainage catchments are under 10 mi^2 (25.9 km^2) in area, and it is generally recommended, as summarized by Jens [1979], that corrections for areas below this size are not necessary.

Design Rainfall Duration

The duration of the design rainfall is chosen with the objective that the maximum peak runoff rate is achieved within the context of the chosen design return period. This implies that the duration should be long enough to allow runoff from the entire catch-

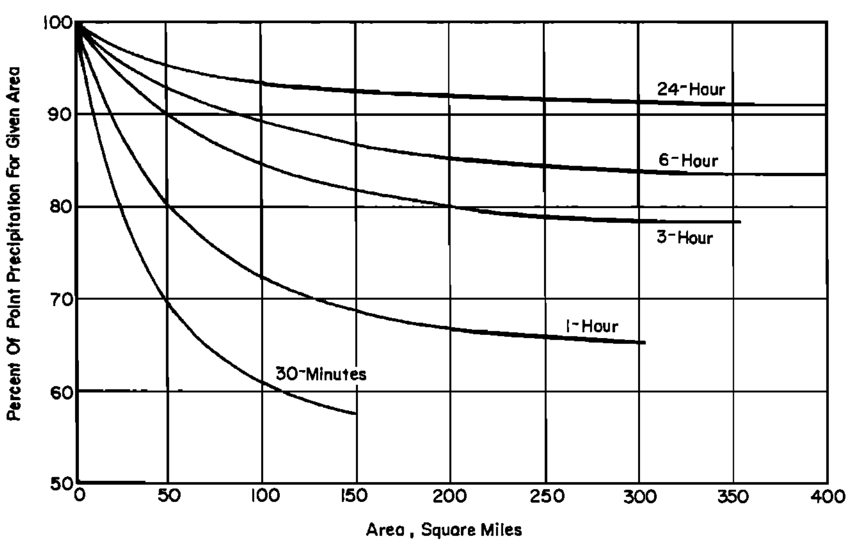

Fig. 3. Reductions in point rainfall with area as recommended by the National Weather Service.

ment to contribute to flow at the design point. This criterion is generally associated with the concept of the 'time of concentration' of the catchment, i.e., the longest travel time to the design point. The use of the time of concentration as the design rainfall duration is a basic part of rational method. In other design procedures the duration is not necessarily equal to the time of concentration but can be varied so as to achieve a maximum peak runoff.

Time Distribution of Design Rainfall

The time distribution or hyetograph of the total rainfall corresponding to the design return period and duration can have a significant effect on the peak runoff. The rational method implicitly utilizes a uniform distribution, but nonuniform hyetographs are associated with other runoff computation procedures.

The U.S. Department of Agriculture, Soil Conservation Service [1973] has developed 24-hour rainfall distributions and a 6-hour distribution for use in developing runoff hydrographs. Table 4 shows these distributions. Type I is for the coastal side of the Sierra Nevada mountains and the interior regions of Alaska, and type II is for the remaining United States, Puerto Rico, and the Virgin Islands.

Hershfield [1962] developed an average time distribution using rainfall data from 50 widely separated situations for durations of 6, 12, 18, and 24 hours. The average distribution for these durations is presented in Table 5. Wide variations in distributions were found, and Hershfield states that a rearrangement of the average distribution for a particular event is not unreasonable.

Huff [1967] presented a rather thorough analysis of 11 years of data from a 49 gage, 400 mi^2 recording rain gage network in east central Illinois. Storms were defined as rainfall periods preceded and followed by at least 6 hours of no rainfall. Time distributions were classified into four groups depending on whether the maximum intensity occurred in the first, second,

TABLE 4. SCS Rainfall Distributions

Hour	t/24	P_t/P_{24} Type I	P_t/P_{24} Type II	Hour	t/6	P_t/P_6
0	0	0	0	0	0	0
2.0	0.083	0.035	0.022	0.6	0.10	0.04
4.0	0.167	0.076	0.048	1.2	0.20	0.10
6.0	0.250	0.125	0.080	1.5	0.25	0.14
7.0	0.292	0.156	-	1.8	0.30	0.19
8.0	0.333	0.194	0.120	2.1	0.35	0.31
8.5	0.354	0.219	-	2.28	0.38	0.44
9.0	0.375	0.254	0.147	2.40	0.40	0.53
9.5	0.396	0.303	0.163	2.52	0.42	0.60
9.75	0.406	0.362	-	2.64	0.44	0.63
10.0	0.417	0.515	0.181	2.76	0.46	0.66
10.5	0.438	0.583	0.204	3.00	0.50	0.70
11.0	0.459	0.624	0.235	3.30	0.55	0.75
11.5	0.479	0.654	0.283	3.60	0.60	0.79
11.75	0.489	-	0.357	3.90	0.65	0.83
12.0	0.500	0.682	0.663	4.20	0.70	0.86
12.5	0.521	-	0.735	4.50	0.75	0.89
13.0	0.542	0.727	0.772	4.80	0.80	0.91
13.5	0.563	-	0.799	5.40	0.90	0.96
14.0	0.583	0.767	0.820	6.00	1.0	1.00
16.0	0.667	0.830	0.880			
20.0	0.833	0.926	0.952			
24.0	1.00	1.000	1.000			

third, or fourth quarter of the duration. For each quarter, dimensionless time distributions were presented for various probability levels. It was found that short-duration storms dominated the first and second quartile groups. The first-quartile point rainfall medium distribution was thus selected by Terstriep and Stall [1974] for optional use in the

TABLE 5. Hershfield Average Rainfall Distribution

t/t_d	P/P_{max}
0	0
0.10	0.06
0.20	0.12
0.30	0.20
0.40	0.29
0.45	0.34
0.50	0.45
0.55	0.63
0.60	0.73
0.65	0.81
0.70	0.86
0.80	0.94
0.90	0.99
1.00	1.00

Illinois urban drainage area simulator (ILLUDAS) and urban runoff and design model. Table 6 shows this distribution. It must be emphasized that this distribution is not universal and should not be applied in areas of different climate and topography from east-central Illinois. Distributions developed from local data should be used if possible.

Figure 4 shows a dimensionless plot of the above distributions in hyetograph form, which provides a graphical comparison. The average intensity $\bar{i}$ is the total depth divided by the total duration t_d. The variations in peak relative intensity and relative peak time are clearly indicated.

TABLE 6. Huff First-Quartile Point Rainfall Distribution for East Central Illinois

t/t_d	P/P_{max}
8.3	0.21
16.7	0.44
25.0	0.59
33.3	0.68
41.7	0.75
50.0	0.80
58.3	0.84
66.7	0.87
75.0	0.90
83.3	0.94
91.7	0.97
100.0	1.00

Keifer and Chu [1957] introduced a hyetograph for use in sewer design, sometimes called the Chicago method. It is based on an intensity-duration curve for a specific return period given by (4b). The resulting design hyetograph is expressed mathematically, using the time of peak intensity as the origin or the time scale:

Before peak

$$i = \frac{a[(1 - b)(t_b/r)^b + c]}{[(t_b/r)^b + c]^2} \tag{5a}$$

After peak

$$i = \frac{a[(1 - b)(t_a/(1 - r))^b + c]}{[(t_a/(1 - r))^b + c]^2} \tag{5b}$$

where i is the value of the rainfall intensity at time t_b before the peak and at time t_a after the peak and r represents the ratio of the portion of the duration before to that after the time of peak intensity.

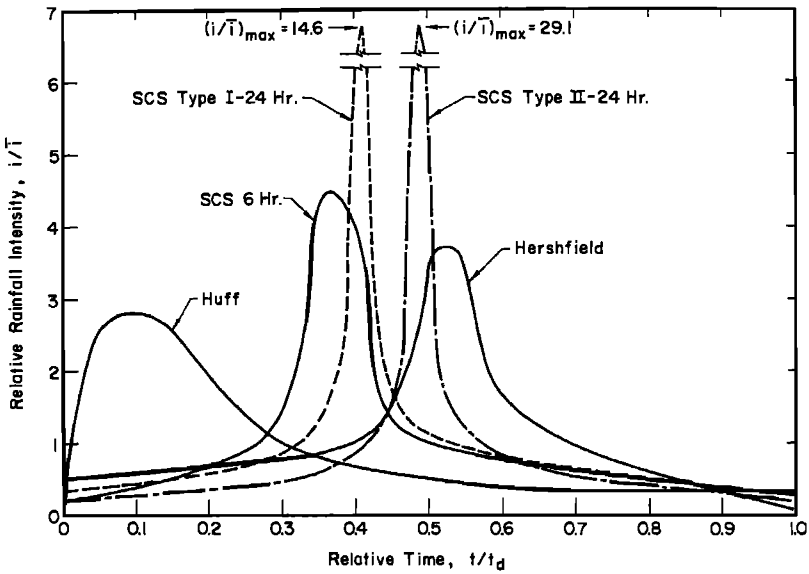

Fig. 4. Comparison of dimensionless design rainfall hyetographs.

Two methods of evaluating r are proposed. The first and more reasonable method is to compute the ratio of the peak intensity time to the storm duration for a series of events for various durations. The mean value of this ratio, weighted according to the duration of the events then is taken as r. The second method involves the estimation of antecedent rainfall for events of various durations less than the largest duration in the analysis. Equation (4b) is used as the basis of this procedure. This latter method has been criticized [McPherson, 1958] and is not detailed here. Some values of r reported in the literature are shown in Table 7. Although the range of r values in Table 7 is not great, it is recommended that if (5) is used to develop a design event, local data be used to evaluate all coefficients. In addition, it should be pointed out that the peak intensity can be high, and caution should be taken when utilizing short time increments which may produce high peak runoff.

Pilgrim and Cordery [1975] have developed a method for developing design hyetographs based on analysis which retains the iden-

TABLE 7. Values of r (Equation (5)) for Various Locations

Location	r	Reference
Baltimore	0.399	McPherson [1958]
Chicago	0.375	Keifer and Chu [1957]
Chicago	0.294	McPherson [1958]
Cincinnati	0.325	Preul and Papadakis [1973]
Cleveland	0.375	Havens and Emerson [1968]
Gauhati, India	0.416	Bandyopadnyay [1972]
Ontario	0.480	Marsalek [1978]
Philadelphia	0.414	McPherson [1958]

tity of the events. The procedure for a specified duration involves the following steps:

1. Identify a sample of events with large rainfall depths for the specified duration. Pilgrim and Cordery suggest that the largest 50 events of record be used so that the results will have statistical significance.

2. Divide the duration into a number of time periods. The length of the period will depend on the interval desired in the resulting design hyetograph and possibly the smallest interval of the rainfall data.

3. Rank the periods for each event according to the depth of rainfall in each period. The average ranking for each period is computed using all events. The average rankings are used to assign a rank to each period indicating the most likely order of period of the largest depth, second largest, etc.

4. Determine the percentage of total rainfall for each event in each of the ranked periods for that event. Average percentages for the periods of rank 1, 2, 3,... for all events are computed.

5. Form the hyetograph by arranging the periods in the most likely order as determined in step 3 with relative magnitudes for each period as determined in step 4.

A more recent approach by Yen and Chow [1977, 1980] applied the method of statistical moments to describing hyetographs. Data from over 9000 rainstorms at four locations were used (Boston, Massachusetts; Elizabeth City, North Carolina; San Luis Obispo, California; and Urbana, Illinois), with the analysis focused primarily on the first two moments with respect to the beginning of the event. They found that a general nondimensional triangular hyetograph could be established, utilizing only the first moment. Assuming that the distribution is represented by a series of incremental depths d_j equal time intervals Δt, the dimensionless first moment is given by

$$\tilde{\tilde{t}} = \frac{\Delta t}{t_d D} \sum_{j=1}^{n} (j - 0.5) d_j \quad (6)$$

where t_d is the duration and D is the total depth. The triangular hyetograph can be described by

$$\tilde{t}_{max} = t_{max}/t_d = 3\tilde{\tilde{t}} - 1 \quad (7a)$$

$$\tilde{\tilde{t}} = t_d/t_d = 1 \quad (7b)$$

$$\tilde{i}_{max} = i_{max}/(D/t_d) = 2 \quad (7c)$$

where $\tilde{t}_{max}$ is the dimensionless time from the beginning of the hyetograph to the time of maximum dimensionsless intensity $\tilde{i}_{max}$, and $\tilde{t}_d$ is the dimensionless duration of unity. Therefore if $\tilde{\tilde{t}}$ is known together with the total depth D and duration t_d, the dimensional triangular hyetograph is defined. They found that mean value of $\tilde{t}_{max}$ ranged from 0.32 to 0.51, indicating an advanced general pattern. This work represents an initial study and the Federal Highway Administration is currently extending this approach.

Another procedure is the development of a composite design hyetograph utilizing depth (or intensity)-duration data directly. For a specified return period, maximum depth values

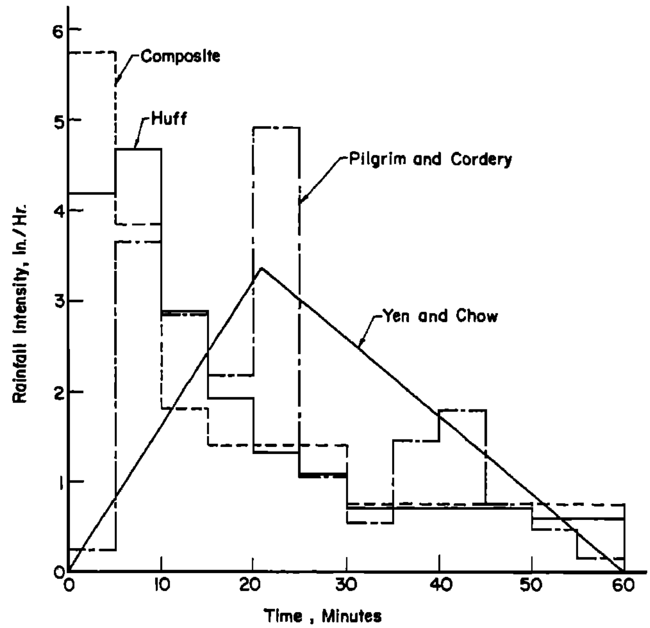

Fig. 5. Comparison of example 60-min design rainfall hyetographs.

for successively larger durations are obtained. Incremental depth and corresponding incremental durations are computed and average incremental intensities are calculated for each of the incremental durations. The resulting intensities are then rearranged in an arbitrary sequence to form the design hyetograph.

Examples of Design Rainfall Development

The following examples are all based on the depth-duration-frequency data in Figure 2. The objective is to construct a design rainfall with a 60-min duration and a return period of 10 years. Five methods are shown with the results compared graphically in Figure 5.

Huff Distribution

From Figure 2 the 10-year 60-min depth is 1.67 in. (42.4 mm). Table 8 is determined directly from the percentage values for time and depth given in Table 6.

TABLE 8. Huff Design Rainfall

Time (min)	Cumulative Rainfall (in.)	Incremental Rainfall (in.)	Average Incremental Intensity (in./h)
5.0	0.35	0.35	4.20
10.0	0.74	0.39	4.68
15.0	0.98	0.24	2.88
20.0	1.14	0.16	1.92
25.0	1.25	0.11	1.32
30.0	1.34	0.09	1.08
35.0	1.40	0.06	0.72
40.0	1.46	0.06	0.72
45.0	1.51	0.06	0.72
50.0	1.57	0.06	0.72
55.0	1.62	0.05	0.60
60.0	1.67	0.05	0.60

Depth in millimeters = 25.4 x depth in inches.

Keifer and Chu Distribution

This method requires that the depth-duration data for a 10-year return period from Figure 2 be converted to intensity-duration data and then fitted to (4b). Data points from the least squares fit lines for 2-, 10-, and 25-year return periods were divided by their respective intensities and are shown in Figure 6. Values for 5, 30, and 60 min were used to determine the constant in (4b). The result is a = 75.0, b = 0.88, and c = 8.9 with the resulting curve shown in Figure 6. The value of r for this example is the average for the 60-min events listed in Table 2. The result of this calculation is r = 0.44. Substitution of these values in (5) yields the results shown in Table 9 of hyetograph ordinates at 5-min increments.

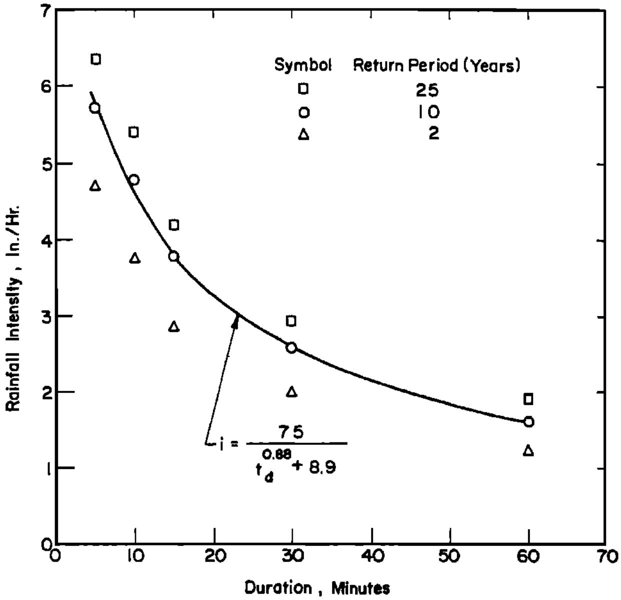

Fig. 6. Intensity-duration-frequency data for Coshocton, Ohio.

TABLE 9. Keifer and Chu Design Rainfall

Time From Peak (min)	Time From Start (min)	Rainfall Intensity (in./h)
20	2.5	0.59
15	7.5	0.81
10	12.5	1.25
5	17.5	2.32
0	22.5	7.83
5	27.5	3.31
10	32.5	2.00
15	37.5	1.38
20	42.5	1.03
30	52.5	0.66
40	62.5	0.47

Pilgrim and Cordery Method

The data base for this example is the 25 60-min events shown in Table 2. The 60-min period of maximum rainfall for each event was divided into 5-min increments. The first step in the procedure is to compute the average ranking of each of the 12 successive periods according to rainfall depth in the period. Then the average percent or fraction of total rainfall for periods of rank 1, 2, ... 12 are computed. The highest average fractional rainfall is then assigned to the period with the lowest average ranking from the first step, the second highest rainfall as assigned to the next highest ranking period, etc. This results in a design rainfall hyetograph with relative ordinates which can be transformed into an actual hyetograph by multiplying each ordinate by the total design depth. For this case the 10-year 60-min depth is 1.67 in. The calculations are summarized in Table 10.

The calculations supporting the values shown in the second and fourth rows in Table 10 are not presented here. It should be noted that the variation in average rank for the highest five periods is not great and thus may not be statistically significant because of the relatively small sample size. However, the order is retained for illustrative purposes.

Yen and Chow Method

The triangular distribution requires the specification of the dimensionless first moment. The value of $\approx{t}$ computed using the entire 25-year Coshocton record was 0.448. The value of $\approx{t}$ computed using the 25 60-min events in Table 2 was 0.446. Therefore the value of t is taken as 0.45 with the depth and duration of 1.67 in. and 60 min, respectively.

$$\tilde{t}_{max} = 3\overset{\approx}{t} = 0.35$$

$$t_{max} = t_d\tilde{t}_{max} = 21.0 \text{ min}$$

$$i_{max} = 2(D/t_d) = 3.34 \text{ in./h}$$

Thus the triangular hyetograph has a maximum intensity of 3.44 in./h which occurs 21 min from the start of rainfall.

TABLE 10. Pilgrim and Cordery Design Rainfall

Period	Average Rank According to Depth	Assigned Order of Periods	Average Fractional Rainfall for Ranked Periods	Rainfall Depth (in.)	Rainfall Intensity (in./h)
1	8.12	11	0.013	0.02	0.26
2	5.24	2	0.182	0.30	3.65
3	5.24	3	0.142	0.24	2.85
4	5.68	4	0.109	0.18	2.18
5	5.04	1	0.245	0.41	4.91
6	6.52	7	0.052	0.09	1.04
7	7.00	9	0.028	0.05	0.56
8	6.36	6	0.073	0.12	1.46
9	5.84	5	0.090	0.15	1.80
10	6.68	8	0.037	0.06	0.74
11	8.12	10	0.020	0.04	0.48
12	8.16	12	0.008	0.01	0.16

TABLE 11. Composite Design Rainfall

Duration (min)	Depth (in.)	Increment (in.)	Intensity (in./h)	Time Period (min)
5	0.48	0.48	5.76	0-5
10	0.80	0.32	3.84	5-10
15	0.95	0.15	1.80	10-15
30	1.30	0.35	1.40	15-30
60	1.67	0.37	0.74	30-60

Composite Method

The depth and corresponding duration values for the 10-year return period are taken from Figure 2. The calculation of the incremental intensities are shown in Table 11. These can be arranged in any order but are plotted in an advanced pattern in Figure 5.

Advantages and Disadvantages of Design Rainfalls

The design rainfall approach has several advantages. The chief advantage is simplicity and speed, at least for some of the methods. Also, since maximum historical rainfall data are used, it is likely to yield a conservative design, although this is not necessarily true. It is a widely used approach, and continued use could be argued from the standpoint of consistency and comparison of design alternatives.

However, there are some very definite and serious disadvantages and weaknesses. First, the basic assumption of the transfer of the design return period directly from the design rainfall to the drainage system has not been verified, and the implications, if the assumption is inaccurate, have not been fully explored. Recent works by Marsalek [1978] and Wenzel and Voorhees [1978, 1979] indicate that the choice of rainfall duration, time distribution, and antecedent soil moisture can have a significant effect on the peak runoff-frequency relationship. Furthermore, these parameters interact in a manner which has not been generally established. It is safe to say that if the design rainfall approach does produce a peak runoff with the rainfall return period, it would be simply by chance and not an inherent result of the procedure.

Another disadvantage is amount of local data analysis (in addition to the usual intensity-frequency-duration analysis) required for proper use of some of the methods such as Keifer and Chu or Pilgrim and Cordery. There may be a strong temptation to transfer published coefficients or constants from other locations to save time but without justification.

The approach does not directly consider spatial variability of rainfall, which could be significant, particularly for larger catchments. Furthermore, it does not consider multipeaked time distributions or multiple events which could be important in designs involving detention storage.

There is the basic concern that design rainfall methods based on intensity-duration-frequency analysis do not represent all aspects of historical events. This is demonstrated in this chapter and earlier by McPherson [1958, 1977]. For example, events which involve high depth and long duration may be important in sizing detention storage facilities, but these events are typically excluded by the nature of depth-duration-frequency analysis.

Finally, the design rainfall which may be used for peak flow considerations is unlikely to be suitable for water quality considerations since the rainfall characteristics which produce high pollutograph concentrations may not be those which produce high runoff hydrograph peaks.

Current Design Rainfall Research

Currently, there is considerable research interest on an international level in various aspects of design storms. A seminar was held on this topic at Ecole Polytechnique, University of Montreal [Patry and McPherson, 1979] at which ideas and research efforts were discussed. The following is a brief summary of the topics of major interest. The names after each topic refer to individuals involved as discussed in the seminar report.

1. Use of a series of historic storms with an event model

followed by subsequent discharge-probability or volume-probability analyses of the output (H.G. Wenzel and M.L. Voorhees, Illinois; J. Marsalek, Canada; B. Urbonas, Denver; and S.G. Walesh and D.H. Lau, Wisconsin).

2. Reducing the cost of continuous simulation (W.F. Geiger, Germany; W.C. Huber, Florida; W.M. Alley, USGS; and S.G. Walesh and D.F. Snyder, Wisconsin).

3. Probabilistic development of a design storm (Hydroscience for U.S. EPA and E.M. Laurenson, Australia).

4. Use of modeling to identify appropriate combinations of hyetographs, shape or peakedness, antecedent moisture conditions, duration, and areal variability (B. Urbonas, Denver; M.J. Lowing, United Kingdon; J. Falk, Sweden; and W.F. Geiger, Germany).

5. Development of computer programs intended to efficiently and effectively screen long-term historic or precipitation data to select subsets of storms suitable for simulation, with event models, or discharges, volumes, or water quality (M.L. Terstriep, Illinois State Water Survey; H.G. Wenzel and M.L. Voorhees, Illinois; and M.J. Lowing, United Kingdom).

Alternatives to Design Rainfall Approach

There are a number of alternatives to the use of the traditional design storm. They are relatively new, and there is no single alternative which is presently suitable for all design situations. They are summrized as follows.

Continuous Simulation

This involves the use of long-term historical precipitation data together with a simulation model to evaluate the simulated historical response of a proposed system. The design is modified so as to produce an acceptable statistical performance of the system.

This approach has the advantage of eliminating the design rainfall approach but may be costly because of the required computer time. Furthermore, it is subject to the validity of the simulation model which may vary depending on the physical situation and the objectives of the design. The required precipitation data for the study area may not be available in sufficient quantity or detail for design purposes.

Modified Continuous Simulation

This general approach is similar to the above except that simplifications and/or screening procedures are used to reduce the costs without sacrificing the utility of the results.

Historic Storms

A series of large historic storm events are used in connection with an event model to evaluate system performance. Probability analyses of the model output results can be performed.

Another approach involves the use of one or a few readily identifiable and remembered historic rainfall events without explicit assignment of a return period. This may be effective in gaining public understanding of the design approach.

Improved Design Storm

One approach is to combine a simulation model with historic storm sequences to determine by trial and error the combinations of hyetograph shape, duration, and antecedent soil moisture that yield accurate discharges for specified return period at a given location.

Another approach is to view a hyetograph as being a function of duration, mean intensity, time distribution, areal distribution, and antecedent soil moisture. These variables can be described in terms of joint probability distributions and ultimately combine to form design hyetographs.

Design Rainfall for Detention Storage

It was stated at the beginning of this chapter that the design rainfall approach is not generally suitable for problems in which runoff volume is of concern. This is because of the lack of relationship between peak discharge and runoff volume. Therefore a synthetic rainfall event chosen for peak discharge design will not be suitable for peak volume design.

Detention or retention storage design has not been standardized. The performance criteria for the structure is normally in terms of a maximum allowable downstream discharge, adequate storage volume, and aesthetic considerations. Thus the shape and volume of the direct runoff hydrograph into the structure are important. This means that the duration of the rainfall is important as well as its intensity pattern. Furthermore, a succession of rainfall events may serve as a basis for some aspects of the design. It seems reasonable therefore to base storage facility design on historical rainfall data rather than a synthetic event, as described earlier in this chapter.

Rainfall for Water Quality Studies

With the advent of section 208 studies, urban water quality has become of increased concern. With regard to nonpoint source pollution, the concept of design takes on a different connotation than with drainage or detention problems. The objective, perhaps, is to meet in-stream water quality standards which are, in some cases, still to be established. The 'design' process involves the determination of ways to meet these standards. Thus the process is more one of evaluation than design in the normal sense, although the problem of combined sewer overflows certainly may involve the design and construction of control structures.

It is clear therefore that design rainfall of the type discussed in detail in this chapter has no utility in urban water quality analysis. Two identical rainfall events could produce significantly different pollutographs depending on the amount of pollutant build-up occurring prior to the events. The many factors which can effect the quality of runoff dictate that historical rainfall be utilized for any viable evaluation procedure. The historical record incorporates the various characteristics of rainfall, including the time between events, which are important. The quality of urban runoff is so much more complex than quantity that it would be misleading indeed to attempt to use a single rainfall event of any type for design or analysis. It is the performance of a system in a statistical or probabilistic sense which is significant. Such an evaluation requires the use of some effective form of continuous rainfall data. The profession is still in the process of experimenting with various models and approaches.

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3 RAINFALL ABSTRACTIONS

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Introduction

Rainfall abstractions or losses in the form of interception, depression storage, and infiltration are among the most important factors in the estimation of runoff rates from rainfall; yet they are the ones subject to the largest amount of uncertainty and thus are the weakest link. Modern developments have provided the hydrologist with refinements in unit hydrographs, routing procedures, statistical flood frequency estimates, and watershed simulation as well as more sophisticated parametric infiltration equations, but when it comes to the actual estimate or choice of a loss rate parameter, we usually have to go back to reports from the 1940's or earlier for experimental test data. If rainfall-runoff data are available for the site under design consideration, the unknown parameters can be evaluated by calibration, but even in this fortunate case the parameter estimates usually are subject to major uncertainties.

Interception

Interception is that portion of the rain which is retained by leaves and stems of vegetation or other forms of cover. Some of the interception flows down to the ground in the form of stemflow and thus becomes water available for infiltration, depression storage or runoff, with a time delay which could be significant on small watersheds.

Usually interception is considered as taking water from the earliest portions of a storm. The Soil Conservation Service, for

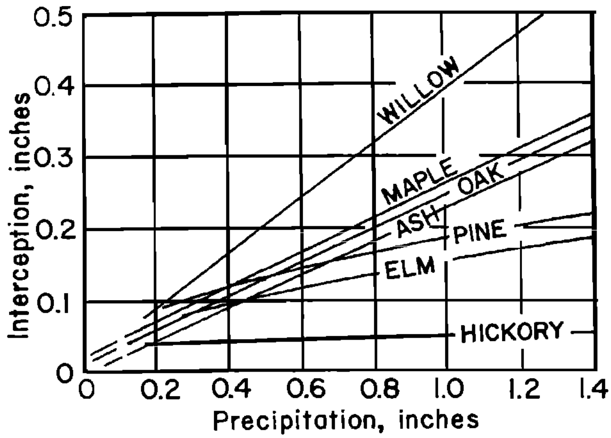

Fig. 1. Interception of rainfall by various tree species [Horton, 1919].

example, goes to the extreme of designating an initial abstraction, which is proportional to soil storage capacity and must be satisfied before any water is available for runoff.

Horton [1919] measured and plotted interception as a function of rainfall amount for single storms and various types of trees. His results are presented in Figure 1. In more general terms, he also expressed interception by the equation

$$I_{ri} = a + bP^n \tag{1}$$

where I_{ri} and P are interception and rainfall in inches, respectively, and a, b, and n are parameters listed in Table 1. The projection factor is used as an adjustment for partial area coverage of a given type of vegetation. Where no projection factor is given, it should be estimated as being equal to that portion of the area covered by the vegetation.

Depression Storage

Depression storge accounts for that amount which gets trapped in small puddles without either infiltrating or running off. This type of rainfall abstraction has been even less measured than interception.

TABLE 1. Evaluation of Constants a, b, and n in an Interception Equation

Vegetal Cover	Interception = $a + bP^n$			Projection Factor
	a	b	n	
Orchards	0.04	0.18	1.00	
Ash, in woods	0.02	0.18	1.00	
Beech, in woods	0.04	0.18	1.00	
Oak, in woods	0.05	0.18	1.00	
Maple, in woods	0.04	0.18	1.00	
Willow, shrubs	0.02	0.40	1.00	
Hemlock and pine woods	0.05	0.20	0.50	
Beans, potatoes, cabbage and other small hilled crops	0.02h	0.15h	1.00	0.25h
Clover and meadow grass	0.005h	0.08h	1.00	1.00
Forage, alfalfa, vetch millet, etc.	0.01h	0.10h	1.00	1.00
Small grains, rye, wheat, barley	0.005h	0.05h	1.00	1.00
Corn	0.005h	0.005h	1.00	0.10h

From Horton [1919]
h is the vegetation height in feet.

For runoff modeling purposes, Linsley et al. [1975] suggest a gradual accumulation of volume V_d trapped in depressions which can be written in incremental form by the equation

$$\Delta V_d = e^{-(P_e/S_d)} \Delta P_e \tag{2}$$

where P_e is the rainfall excess, or rainfall minus evaporation,interception, and infiltration and S_d is the total available depression storage. Linsley et al.'s suggested default values in absence of locally obtained data are 0.25 in. for pervious areas and 0.0625 in. on impervious surfaces.

Hicks [1944] suggests using maximum depression storage depths of 0.02 in. for sand, 0.15 in. for loam, and 0.10 in. for clay

TABLE 2. Typical Depression and Detention for Various Land Covers

Land Cover	Depression and Detention, inches	Recommended, inches
Impervious		
Large paved areas	0.05 - 0.15	0.1
Roofs, flat	0.1 - 0.3	0.1
Roofs, sloped	0.05 - 0.1	0.05
Pervious		
Lawn grass	0.2 - 0.5	0.3
Wooded areas and open fields	0.2 - 0.6	0.4

From Denver Regional Council of Governments [1969].

soils. The Denver Regional Council of Governments [Wright-McLaughlin Engineers, 1969] has compiled Table 2 of suggested depression and detention depths which are similar to those by Hicks. While the values of surface depression and detention are reported only for use in the Colorado unit hydrograph procedure, they are in general agreement with accepted ASCE [1970] values of 1/16 in. for impervious areas and 1/4 in. for pervious areas. Mitchell and Jones [1978] have developed a relationship of the form $S = aD^b$ to express the surface depression storage available in the microrelief of an overland flow surface. They have shown further how this function can be combined with an infiltration equation to predict the time distribution of rainfall excess for watershed simulation purposes.

Infiltration

The topic of infiltration has been the subject of numerous publications, offering a variety of equations expressing infiltration as functions of time, soil permeability, capillary suction, or soil storage capacity. However, estimates of any of the proposed infiltration parameters in most cases remain largely guesswork unless site-specific infiltrometer results are available.

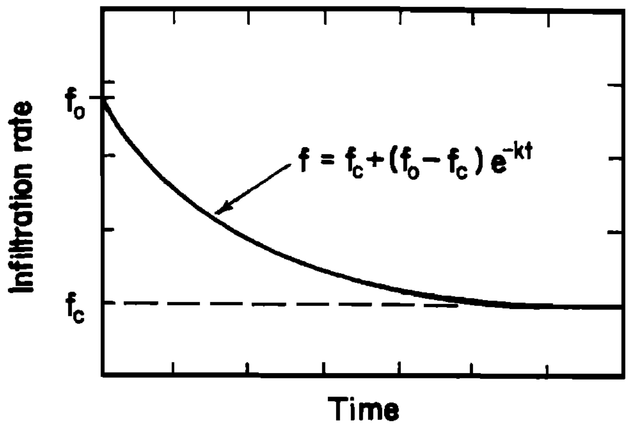

Fig. 2. Characteristic Horton infiltration rate.

Even at a site at which infiltration tests have been performed, a change in ion concentration due to major rainfall or runoff events or due to some form of surface pollution may alter the soil permeability drastically. Rose [1966] provides one of the most thorough descriptions of soil physical and chemical factors which may affect infiltration rates; however, no specific or typical infiltration rates can be found in his otherwise highly instructive book.

Horton Equation

The best-known and most widely used infiltration equation is the one developed by Horton [1940] (illustrated in Figure 2):

$$f = f_c + (f_0 - f_c)e^{-kt} \tag{3}$$

where f_o and f_c are the initial and final infiltration rates in inches per hour, or centimeters, respectively, and k is an exponential decay coefficient to be evaluated by field experiments, in units of 1/h.

The curves shown in Figure 3 have been proposed by American Society of Civil Engineers and Water Pollution Control Federation [1970] for use on sandy soils, residential areas, and industrial-commercial areas. These values should, however, be used with great caution since no reference is made to soil type

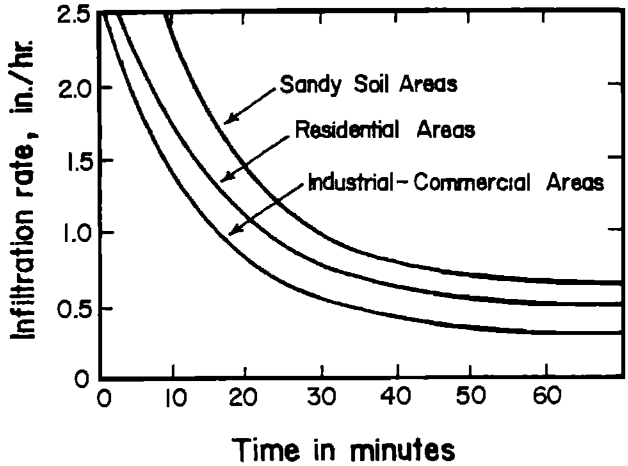

Fig. 3. Recommended typical infiltration rates [ASCE, 1970].

or moisture state. A truly hazardous situation has been created in hydrologic design through the availability of so-called default values which are used indiscriminately in lack of even reasonably reliable field data.

A further problem arises with the Horton equation because it considers infiltration entirely as a function of time rather than of soil water storage available after varying amounts of infiltration have taken place. This problem is illustrated in Figure 4 in which a storm starts at an intensity which is less than the infiltration capacity. According to Horton's equation the infiltration capacity should decrease with time regardless of the storm intensity, yet it is intuitively obvious that the infiltration capacity should decrease at a slower rate than it would under conditions of a storm intensity larger than infiltration capacity. In such a situation cumulative rainfall and infiltration should be used to adjust the Horton equation by shifting the curve to the right, as shown in Figure 4.

Up to time t_1, more or less all of the storm should be absorbed by the soil. The location is found at which the slopes of the cumulative Horton infiltration and rainfall are equal. At this point in time the cumulative infiltration curve is shifted to the right to become tangent to the cumulative storm line which

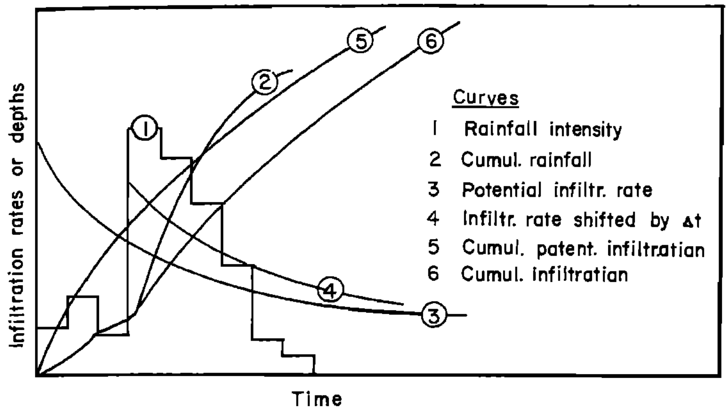

Fig. 4. Horton infiltration curve shift for light rainfall.

is also actual infiltration. The Horton line for infiltration rate is then shifted by the same time interval.

Green and Ampt Equation

Green and Ampt [1911] developed the infiltration equation

$$f = K(H_0 + H_c + L_f)/L_f \tag{4}$$

where

- K hydraulic conductivity of the soil behind the wetting front, in the same units of [L/T] used for f;
- H_o depth of ponded water on the soil surface, often negligible;
- H_c capillary suction head;
- L_f depth from soil surface to wetting front.

The quantities H_o, H_c, and L_f should also have the same units (usually feet, meters, or centimeters).

The infiltration concept envisioned by Green and Ampt is described in Figure 5. It is based physically on the Darcy law of flow through porous media, plus the assumptions that there is

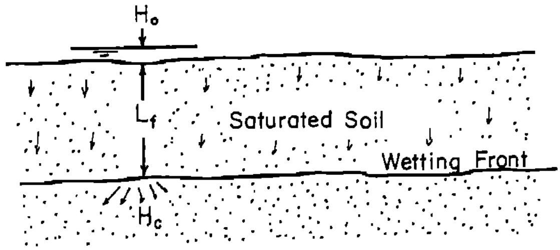

Fig. 5. Green and Ampt infiltration concept.

indeed a well-defined wetting front and capillary suction at depth L_f, and that the degree of saturation as well as the hydraulic conductivity behind the wetting front are constant. The equation has been used successfully at experiment stations, but without detailed field data it is again virtually impossible to estimate the value of K, H_c and L_f for any given soils and location. The test data reported by Mein and Larson [1971] are probably the most useful data for general use of the Green and Ampt equation.

Assuming that all available pores will be filled by the advancing wetting front, the infiltration rate f equals the rate of advance dL_f/dt multiplied by the wettable porosity θ_f which is essentially the difference between the total soil porosity θ_o and the volumetric water content of the soil prior to infiltration and may vary between 5% in tight clayey silts and 30% in coarse sands. Under these conditions (4) can be rewritten as

$$f = \theta_f \frac{dL_f}{dt} = K(H_o + H_c + L_f)/L_f \tag{5}$$

If values of H_o, H_c, K, and θ_f were available, the variables f and L can be computed as functions of time by numerical approximations in appropriately small time steps, and an infiltration curve can be drawn which is similar in shape to the Horton equation.

As an example, let H_o, H_c, K, and θ_f be 2 cm, 18 cm, 3 cm/h, and 0.20, respectively, and find f and L_f as a function

TABLE 3. Infiltration Rates and Wetting Front Depth Computed by Green and Ampt's Equation

t, min	f, cm/h	ΔL_f, cm	L_f, cm
0	63.0		1.0
0.5	19.6	2.63	3.63
1	16.5	0.81	4.44
2	13.3	1.38	5.82
3	11.7	1.11	6.93
4	10.6	0.97	7.90
6	9.2	1.77	9.66
8	8.4	1.53	11.20
10	7.8	1.39	12.59
15	6.8	3.24	15.83
20	6.2	2.83	18.66
30	5.5	5.18	23.84
40	5.1	4.60	28.43
60	4.6	8.52	36.95
90	4.2	11.56	48.51
120	4.0	10.59	59.10
180	3.8	20.08	79.18
240	3.6	18.79	97.97
300	3.5	18.06	116.03

of time. In Table 3, a starting wetting front depth $L_f = 1$ cm is further assumed because if $L_f = 0$, f would be infinite for a very small time interval. In each time step, f is computed as a function of the value L_f at the beginning of the time step. This requires that the time step be made very short at the start, then gradually lengthened as the infiltration rate becomes less time-varying. The wetting front depth L_f at the end of each step is then used to compute f for the next time interval.

The procedure of computing the infiltration rate at the beginning of each time interval and using it as average f for the interval results in a slight overprediction of total infiltration depths. Fortunately, these errors are self-balancing, since overestimates of wetting front depths L_f result in underestimates of infiltration rates f.

To avoid the need for very short initial time increments, the graphical procedure shown in Figure 6 may be used because it

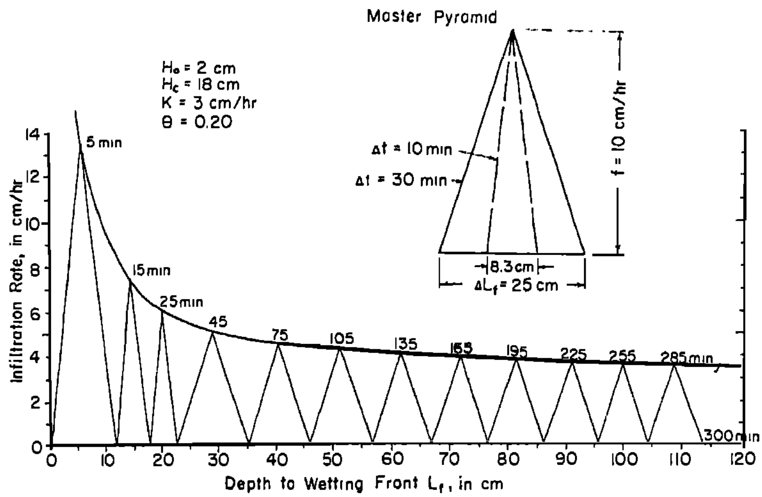

Fig. 6. Graphical solution of Green and Ampt infiltration problem.

automatically uses a mean rather than initial f for each time interval. In this procedure a curve describing f as a function of L_f according to (4) is plotted. Next, a set of 'master pyramids' are traced describing the wetting front advance ΔL_f over a time interval of Δt for an arbitrarily chosen infiltration rate. In the figure, f = 10 cm/h was chosen, and the corresponding $\Delta L_f = f\Delta t/\theta$ values of 8.33 and 25 cm were computed for time intervals for 10 and 30 min. The master pyramids are simply isosceles triangles with height f and base width ΔL_f. Starting at t = 0 and L_f = 0 (no initial wetting depth is necessary) and using the short time interval of 10 min, a triangle is drawn from point 0,0 up to the f versus L_f curve and back down to the base line at the slope prescribed by the 10-min 'master pyramid.' The end point of this triangle marks the depth to the wetting front at 10 min, whereas the triangle vertex describes the average infiltration rate during this time interval. Two more triangles for Δt = 10 min are drawn in the same manner, followed by five triangles with sides parallel to the 30-min master pyramid. The final point marks about 114 cm as the L_f value after 300 min, compared to the 116 cm computed in Table 3.

Other infiltration equations have been proposed by Holtan and Kirkpatrick [1950], Philip [1954], and other soil scientists and are all subject to the same requirement of site-specific data, which are always difficult to obtain with consistency.

The SCS method, which will be described below, uses parameters which can be evaluated from published soil classification and land use tables. Thus it is one of the very few generally applicable models; the reliability of the results, however, should not be accepted with unquestioning faith, as pointed out by Hawkins' [1978] discussion of a paper by Aron et al. [1977].

SCS Method

The U.S. Department of Agriculture, Soil Conservation Service [1972, 1973, 1975] has published an extensive National Engineering Handbook as well as several smaller specific bulletins describing their method for estimating rainfall-runoff relationships.

The keystone of the SCS equations is the soil cover complex number CN, for which values are listed in various tables as a functions of soil classification and land use or cover. From this CN value the soil water storage capacity is computed in inches as

$$S = \frac{1000}{CN} - 10 \tag{6}$$

The next step is the identification of initial abstraction, IA, as a fixed percentage of S, which must be satisfied before any infiltration or runoff can begin to take place. This initial abstraction constitutes all losses except infiltration and thus principally interception and depression storage. It seems questionable that IA would be that directly related to S, and even more so that it should account for 20% of S as suggested by SCS. With a CN value of 75 for example, typical for a medium to fair draining soil and a dense residential development, S would be 3.33 in. and an amount of 0.67 in. of rain could fall without causing a single drop of runoff.

The SCS method further uses the definitions of

- P cumulative rainfall since the beginning of a storm, inches;
- P_e cumulative excess rainfall (in inches), equal to P - IA;
- F cumulative infiltration since the beginning of a storm, inches;
- Q cumulative runoff (in inches), equal to P_e - F.

The runoff concept is based on the assumption that

$$F/S = Q/P_e \tag{7}$$

which upon substitution of the identities listed above results in the runoff equation

$$Q = \frac{P_e^2}{P_e + S} = \frac{(P - IA)^2}{P - IA + S} \tag{8}$$

Figure 7 contains a graphical description of the SCS runoff process, in which after subtraction of IA, most of the excess rainfall begins to go into infiltration, while Q builds up more slowly, then increases as the soil storage gets filled. As P approaches infinity, F approaches S, and Q increases at the same rate as P. Figure 7 describes the precipitation as a straight line and thus of constant intensity, but this was merely done for the sake of simplicity of description, and contrary to the belief of some critics of the SCS method, was not intended or implied to be one of the constraints of the method.

It has been proposed by Chen [1975] and later by Aron et al. [1977] that the SCS equation be used in runoff modeling to yield the expression

$$\Delta F = \frac{S^2}{(P_e + S)^2} \Delta P \tag{9}$$

for incremental infiltration due to an incremental amount of rainfall excess. This equation has the attractive property in that infiltration is strongly influenced by soil water storage

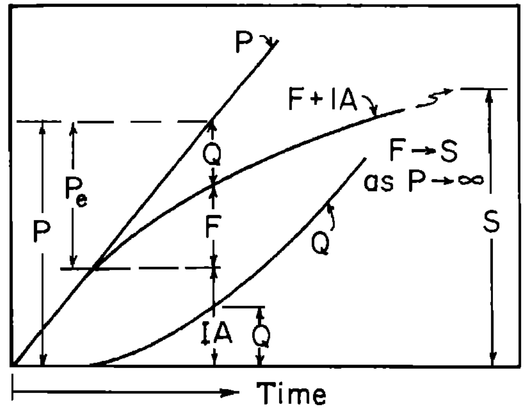

Fig. 7. SCS rainfall-runoff concept.

capacity and cumulative precipitation; however,it does not place any limit on the time rate of infiltration other than being less than rainfall intensity, which can indeed be very large for short periods of time. The term ΔF in (9) should be interpreted as the infiltration increment due to a given rainfall increment but not necessarily occurring during the time interval in which ΔP_e fell. If some upper limit on infiltration rates can be established from percolation measurements taken nearby, any large simulated infiltration amounts resulting from large amounts of rainfall occurring over a few time increments could be reduced to such limiting rate, and the total infiltration depth for the storm could thus be spread out over a longer time without changes in magnitude. Likewise, (9) implies that all infiltration will stop as soon as rainfall stops. This is not realistic either, and some minimum rates could be specified. U.S. Bureau of Reclamation [1973] suggests the lower limits on infiltration rates given in Table 4.

TABLE 4. Minimum 15-Minute Retention Loss Rates

Hydrologic Soil Group	Minimum Loss Rate, in./15 min
A	0.10
B	0.06
C	0.03
D	0.02

TABLE 5. Infiltration Rates According to SCS-Based Method

Time, min.	Incremental Rainfall ΔP, in.	Cumulative Rainfall P^a, in.	Incremental Infiltration ΔF, in.	Infiltration Rate f, in./h
0				
	0.20	0.10	0.10	0.20
30				
	0.50	0.45	0.27	0.54
60				
	0.30	0.85	0.10	0.20
90				
	0.70	1.35	0.14	0.28
120				
	0.30	1.85	0.04	0.08
150				
	0.10	2.05	0.01	0.02
180				

[a]Note that cumulative rainfall is computed for the midpoint of each time interval.

One advantage of using (9) over Horton's equation is that the designer is usually searching completely in the dark when selecting a set of Horton coefficients (except where infiltration data are available), whereas in the use of the SCS equation at least some measure of reliance on the selection of CN from soil classifications and cover type can be accepted. Aron et al. [1977] have suggested, however, that the initial abstraction be reduced from the SCS recommended 20% to somewhere between 5 and 10% of soil storage capacity.

A sample application of (9) is presented in Table 5. A soil with curve number 91 and thus S = 1.0 in. was chosen. Initial abstraction was taken to be 10% of S and thus equal to 0.10 in. Cumulative rainfall depth at any one time was considered to equal the total rain up to the midpoint of any one time interval, and the initial abstraction had to be satisfied before any runoff or infiltration could take place. For example, during the second time step, $\Delta P = 0.5$ in., average cumulative $P = 0.20 + (0.50/2) = 0.45$ in., average excess rainfall $P_e = P - IA = 0.45 - 0.10 = 0.35$ in., and incremental infiltration $\Delta F = [1.0/(0.35 + 1.0)]^2 \times 0.50 = 0.27$ in.

The ϕ Index

The ϕ index is used as a bookkeeping method to estimate a uniform loss rate when rainfall and runoff records are available

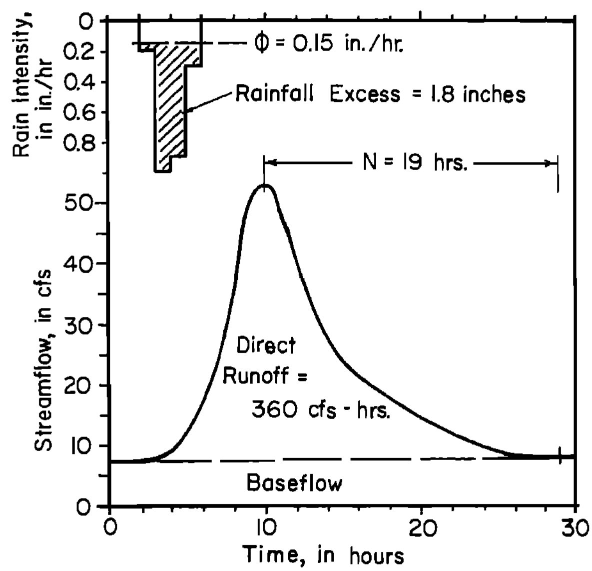

Fig. 8. Rainfall loss estimation by the ϕ index method.

for a storm event. Assume that the rainfall and runoff rates shown in Figure 8 have been measured. Base flow should be extracted from the hydrograph by one of the methods suggested in hydrology textbooks. In this example, the time between the flood peak and the end of storm runoff is computed by the empirical equation

$$N = A^{0.2} \tag{10}$$

where N is the direct runoff recession time in days and A is the watershed area in square miles. The area above the base flow line is the storm runoff volume in cfs which is for all practical purposes equal to a volume in acre inches. Thus in the example presented in Figure 8, the runoff can be determined as

$$\frac{360 \text{ cfs h}}{200 \text{ ac}} \times \frac{0.992 \text{ ac in.}}{\text{cfs h}} \simeq 1.8 \text{ in.}$$

In comparison, the total rainfall shown for the event is 2.4 in. The difference between rainfall and runoff is 0.6 in., which should be taken off the rainfall base as losses. Since the total rainfall duration in the example was 4 hours, this amounts to a

0.15 in./h loss rate, which is called the ϕ index. This index is then assumed to be constant for that particular watershed and similar antecedent moisture conditions.

The ϕ index method is a rather coarse procedure, particularly because of the assumption of a constant rather than decreasing loss rate, but it may often be the best available, particularly for the larger watershed.

Importance of Losses in Urbanized Basins: Effective Runoff Areas

In modeling urban basins it is usually found that losses from the impervious areas are so small that they do not have any appreciable effect on runoff peak or volume. The loss coefficients applied to the pervious areas may affect the total runoff volume appreciably. The flood peak, however, will be determined almost exclusively by the impervious areas, and whatever runoff is produced by the pervious areas tends to be delayed well beyond the time of the peak flow. Thus it was found by Kibler and Aron [1978] that infiltration losses or roughness coefficients on pervious areas in watersheds more than 50% developed had very little effect on the magnitude of flood peaks from moderate storms. Certain runoff methods recognize the effective or contributing area explicitly.

In the British Road Research Laboratory method described by Watkins [1962], for example, only those impervious areas directly connected to the main runoff conveyance paths are counted on to contribute to the flood peak. An exception may be encountered in the case of pervious areas near the watershed outlet, with a storm in which the intense portion was delayed long enough to allow the pervious area to contribute a substantial runoff at the time at which the flood peak from the upstream impervious areas arrived. When runoff volumes are of importance, however, as in the case of the design of detention basins, the runoff from pervious areas can have an appreciable effect and should not be treated lightly.

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4 DESK-TOP METHODS FOR URBAN STORMWATER CALCULATION

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Role of Desk-Top Methods in Urban Stormwater Analysis

Despite the versatility of large present-day stormwater models, there remains a need to apply desk-top procedures for computing runoff from the simple urban drainage system. Desk-top methods are readily implemented on a programmable calculator and therefore do not in general require large storage and iterative solution techniques. The treatment of simplified runoff methods in this chapter is not intended to diminish the importance of full-blown computer models but rather to emphasize that the design analysis of simplified urban stormwater systems often can be carried out satisfactorily with the array of desk-top methods available today [Croley, 1979]. Obviously, the distinction between desk-top procedure and full-blown computer model will diminish with time as microprocessor technology provides greater and greater computing capability.

Typically, the simplified urban drainage system is characterized by (1) basin area less than 1 mi^2, (2) branching sewer system without looped network, and (3) absence of weir diversions and complicated outfall structures. In short, basin size will be small, the storm sewer branching or tree-shaped, and backwater will be insignificant in the urban drainage system for which desk-top methods apply. Storage detention basins may be present above the analysis point provided that a storage routing technique is used. Under these conditions, the application of a stormwater model even for repetitive calculation may be unwarranted in view of the low cost of the simpler runoff method. Very often the simple urban drainage system lies in the jurisdiction of a municipal engineering authority where the level of financial and technical resources are more closely aligned with the simplified methods presented here.

The purpose of this chapter is to present the philosophy and the essential steps in applying three desk-top runoff methods to a simple urban basin. These methods are (1) rational method, (2) Soil Conservation Service (SCS) method, and (3) synthetic unit hydrograph (UH) methods, specifically the Espey 10-min UH procedure. Chapter 4 concludes with a brief discussion of desk-top methods for calculating nonpoint source pollutant loadings. Before turning to the detailed example calculations, it is appropriate to describe briefly the Calder Alley drainage system to which each of the aforementioned runoff methods will be applied.

Calder Alley Drainage System

The Calder Alley watershed is an urbanized basin with a separate storm drain system for which desk-top runoff methods are well-suited. It occupies 227 acres of commercial-residential land within the State College Borough located in central Pennsylvania. The configuration of the principal storm drains is indicated in Figure 1. Table 1 summarizes the physical data for each subarea in the Calder Alley drainage system.

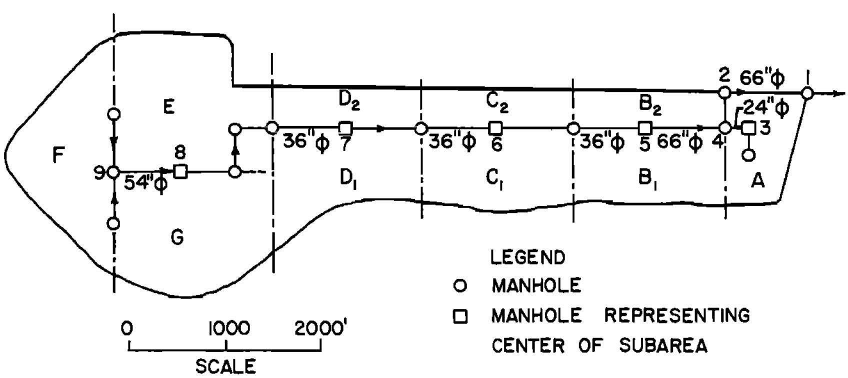

Fig. 1. Schematic of Calder Alley storm drain system.

TABLE 1. Physical Subarea Data for Calder Alley Basin

Subarea	Principal Land Use	Area, Acres	Imperv. Percent	Imperv. Acres	Avg. Surface Slope, %	Runoff Coefficient C[a]	Inlet No.
A	Detached multifamily residential	12.1	4	0.5	3.3	0.33	3
B_1	Small business	14.0	45	6.3	4.5	0.66	5
B_2	Multifamily residential	8.0	77	6.2	1.2	0.70	5
C_1	Commercial	17.9	77	13.4	3.4	0.82	6
C_2	Commercial	6.1	100	6.1	1.5	0.95	6
D_1	Commercial	19.0	95	18.1	3.4	0.90	7
D_2	Commercial	4.9	100	4.9	1.5	0.95	7
E	Residential and small business	47.6	40	19.0	3.5	0.60	8
F	Residential and small business	52.6	20	10.5	3.0	0.42	9
G	Residential and small business	45.0	32	14.4	3.5	0.52	8
Total		227.2		99.4		0.59	

[a]Values of runoff coefficient C were obtained from Table 2 and checked against values computed from the Schaake et al. [1967] equation for the Baltimore area: $C = 0.14 + 0.65\ IMP\ (\text{decimal}) + 0.05\ S$ (slope of main drainage path, percent).

Rational Method for Storm Drain Design

The rational method has served as the basis for American storm drain design practice since the turn of the century. It is essentially a peak discharge method based on the following formula:

$$Q_T = C\, i_T\, A \tag{1}$$

where

Q_T peak flow rate in cfs for return interval T years;

C runoff coefficient dependent on land use;

i_T design rainfall intensity in inches per hour for return period of T-years and duration equal to the time of concentration for the basin;

A drainage area in acres.

The units of discharge for practical purposes are taken as cubic feet per second, since 1 acre inch of runoff per hour equals 1.008 ft^3/s.

The underlying principle of the rational method is that under steady rainfall intensity, maximum discharge will occur at a basin outlet at a time when the entire area above the outlet is contributing runoff. This is a time commonly known as the time of concentration T_c and is defined as the time required for runoff to travel the distance from the most distant point in the basin (in time sense) to the outlet. Other key assumptions are that (1) the frequency or return period of the computed peak flow is the same as that for the design storm, and (2) rainfall intensity is constant over the duration and spatially uniform for the area under analysis. It is further assumed that necessary basin characteristics can be identified and that the runoff coefficient does not vary during a storm. The limits of applicability for the rational method traditionally have been kept at small urban basins less than 1 mi^2 in area. In larger basins the sewer or channel system is more complex and usually requires a full-hydrograph method involving the analysis of flow routing and channel storage effects.

TABLE 2. Typical C Coefficients for 5- to 10-Year Frequency Design

Description of Area	Runoff Coefficients
Business	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential	
Single-family areas	0.30–0.50
Multiunits, detached	0.40–0.60
Multiunits, attached	0.60–0.75
Residential (suburban)	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30
Streets	
Asphaltic	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Drives and walks	0.75–0.85
Roofs	0.75–0.95
Lawns: Sandy soil	
Flat 2%	0.05–0.10
Average 2–7%	0.10–0.15
Steep 7%	0.15–0.20
Lawns: Heavy soil	
Flat 2%	0.13–0.17
Average 2–7%	0.18–0.22
Steep 7%	0.25–0.35

From ASCE [1972] and Viessman et al. [1977].

The individual steps in applying the rational method can be identified briefly as follows:

1. Measure drainage area tributary to a given design point from field surveys, air photos, or available topographic maps.

Delineate subareas, their land use characteristics and inlet points to the storm sewer system.

2. Determine runoff coefficients for each subarea from a known reference, such as Table 2. (Note that the values in Table 2 are sometimes adjusted for storm return period and antecedent rainfall as reported by the Federal Highway Administration [1979, Chap. 3] (also see Rossmiller [1980] and Kibler et al. [1981]).

3. Estimate overland flow time t_o as a partial measure of time of concentration for each design point. Note that T_c = $t_o + t_{pipe}$. In some cases the inlet time from a downstream subarea may be sufficiently large so as to control the duration of design rainfall. The largest value of T_c should be used in general.

4. Select design rainfall intensity from available rainfall frequency-duration-intensity data, given T_c as the duration and assumed return period of from 2 to 25 years for most storm drain designs.

5. Compute Q_T from equation (1) and proceed to design of storm drain, as indicated by the following example problem.

Rational Method Applied to Calder Alley Drainage System

The existing Calder Alley storm drain system is to be redesigned to convey the 25-year peak discharge. Pipe diameters and invert elevations are to be set such that a minimum of 2.5 ft/s velocity

TABLE 3. Design Data for Calder Alley Drainage System

Inlet Number	Tributary Area to Inlet, acres	Subtotal Area, acres	Runoff Coefficient C	Distance to Inlet, ft	Overland Flow time, min
9	52.6	52.6	0.42	1800	36.0
8	92.6	145.2	0.56	1600	25.6
7	23.9	169.1	0.91	1300	8.5
6	24.0	193.1	0.85	900	9.4
5	22.0	215.1	0.68	850	14.8
3	12.1	12.1	0.33	800	26.3
Total	227.2				

will be maintained for scouring purposes. Pipe slopes should conform to surface slopes to avoid unnecessary excavation. Assume concrete pipe to have Manning's n of 0.014. Tributary areas and composite runoff coefficients have been listed in Table 3. The composite C values in Table 3 were obtained by area weighting of the subarea C values listed in Table 1. In addition to C values, the overland distances and flow times are computed by the Federal Aviation Agency (FAA) empirical formula [FAA, 1970]:

$$t_0 = \frac{1.8\,(1.1 - C)D^{1/2}}{S^{1/3}} \tag{2}$$

where

t_o overland flow time, min;
C runoff coefficient;
D travel distance, ft;
S overland slope, %.

(Note that the FAA formula is only one of many for computing time of concentration. Seven of the more common T_c methods are presented in Table 11 with example calculations in Table 12.)

The selection of design rainfall is made from the design storm manual for Pennsylvania [Kerr et al., 1970] which provides rainfall-frequency-duration data. Any of the design rainfall procedures of Chapter 2 could be invoked depending on regional applicability. The 1-hour 25-year rainfall depth is estimated at 2.06 in. from the Pennsylvania storm manual. This value is now adjusted for time of concentration at each inlet by means of Figure 2, which contains standardized rainfall duration-intensity curves developed by the U.S. Army Corps of Engineers. At inlet 9, for example, the T_c is equal to t_o of 36 min. Entering Figure 2 with a duration of 36 min, one arrives at the 2.0-in. curve and moves slightly above to 2.06 in. Transferring horizontally to the ordinate scale from this point, one estimates the 36-min 25-year rainfall intensity at 2.9 in/h. Table 4 contains the selected design rainfall intensities together with computed peak discharges and design information for the Calder Alley drain-

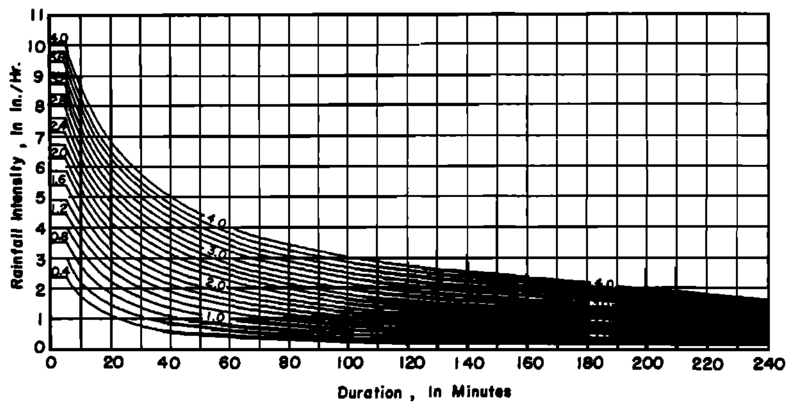

Fig. 2. Standard rainfall intensity-duration curves or standard supply curves. (Curve numbers correspond to 1-hour values of rainfall or supply indicated by respective curves; all points on the same curve are assumed to have the same average frequency of occurrence [Wisler and Brater, 1963].)

age system. A graphical solution of the Manning equation for full pipe flow was used to develop the design data in Table 4 and this is shown in Figure 3.

Modification to Basic Rational Method

More complex applications of the basic rational method involving hydrograph lag and full hydrograph construction have been reported by Yen [1978] and by the American Society of Civil Engineers (ASCE) [1972] and are not described here. An interesting modification of the rational method has been reported by Poertner [1974] which has particular relevance for the design of detention storage facilities for areas of 20-30 acres or less. The trapezoidal hydrographs shown in Figure 4 were obtained by setting rising and recession limbs equal to T_c and computing the peak discharge by the rational method. The problem of estimating required storage for small detention reservoirs when the maximum release rate is known can be analyzed simply by the modified rational method.

As an example, an allowable release rate of 2.5 cfs has been set for a detention storage facility which receives runoff from a 2.0-acre parking lot. The full hydrographs for various rainfall

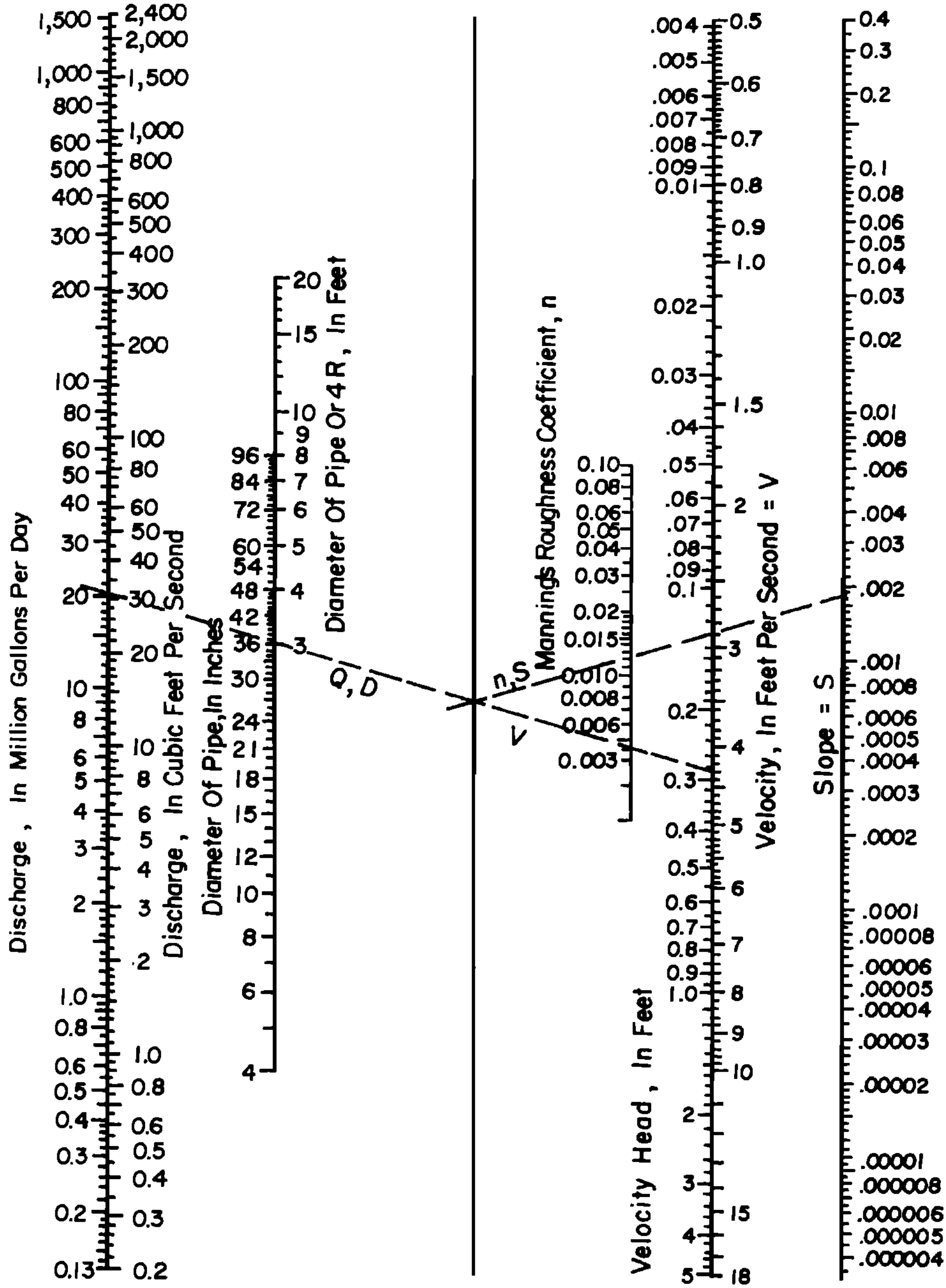

Fig. 3. Solution of Manning equation for circular full pipes [after ASCE 1972].

TABLE 4. Calder Alley Watershed Drainage Design by Rational Formula

Inlet	Line	Length (ft)	Overland Time to Inlet (min)	Pipe Flow Time (min)	T_c (min)	Runoff Coefficient C_{ave}	Rain Intensity i (in./h)	Tributary Area A (acres)	Σ C A
9	9–8	850	36.0	1.1	36.0	0.42	2.9	52.6	22.09
8	8–7	2000	25.6	3.6	37.1	0.56	2.8	92.6	73.95
7	7–6	1650	8.5	2.0	40.7	0.91	2.7	23.9	95.70
6	6–5	1550	9.4	1.7	42.7	0.85	2.6	24.0	116.10
5	5–4	650	14.8	0.6	44.4	0.68	2.5	22.0	131.06
3	3–4	350	26.3	0.8	26.3	0.33	3.4	12.1	3.99
4	4–2	300	0	0.3	45.0	–	2.4	–	–
2	2–1	900	0	0.9	45.3	–	2.4	–	–
1	Outfall								

Inlet	Design Q[a] cfs	Pipe Diameter[c] in.	Cap. Q cfs	Pipe Velocity[d] Full Cap. ft/s	Pipe Slope[e] ft/ft	Fall in Sewer ft	Invert Elev. Entering Pipe	Invert Elev. Exiting Pipe	Crown Elev. at Inlet[f]	Ground Elev. at Inlet
9	64.1	36	92	13.2	0.025	21.2	-	1174.0	1177.0	1183.0
8	207.1	66	210	9.4	0.005	10.0	1152.8	1150.3	1155.8	1162.0
7	258.4	60	260	14.0	0.013	21.4	1140.3	1140.3	1145.8	1152.0
6	301.9	60	305	15.5	0.015	23.2	1118.9	1118.9	1123.9	1130.0
5	327.6	60	350	18.0	0.028	18.2	1095.7	1095.7	1100.7	1107.0
3	13.6	24	24	7.2	0.011	3.8	-	1084.3	1086.3	1093.0
4	335.6[b]	72	370	15.0	0.010	3.0	1077.5	1076.5	1082.5	1089.0
2	335.6	72	410	17.0	0.015	13.5	1073.5	1073.5	1079.5	1086.0
1							1060.0	-	-	1060.0

[a] Design Q obtained by multiplying ΣCA by rain intensity.

[b] Subarea A peaks at 26.3 min. At 45.0 min. outflow peak is estimated at 8 cfs by interpolation of triangular hydrograph.

[c] Pipe diameters obtained by aligning slope and n = 0.014 on right-hand side of Figure 3, finding pivot and aligning design Q on left-hand side of Figure 3.

[d] Pipe velocity obtained from Figure 3, under full-flow capacity.

[e] Slope set equal to ground slope in direction of pipe run, except at pipe 2-1.

[f] Crown and invert elevations set so that crowns are matched when entering pipe is smaller or equal to exiting pipe. When entering pipe is larger than exiting pipe, inverts are aligned. (See Fair and Geyer [1961, pp. 223-226] for further details of manhole drop calculations).

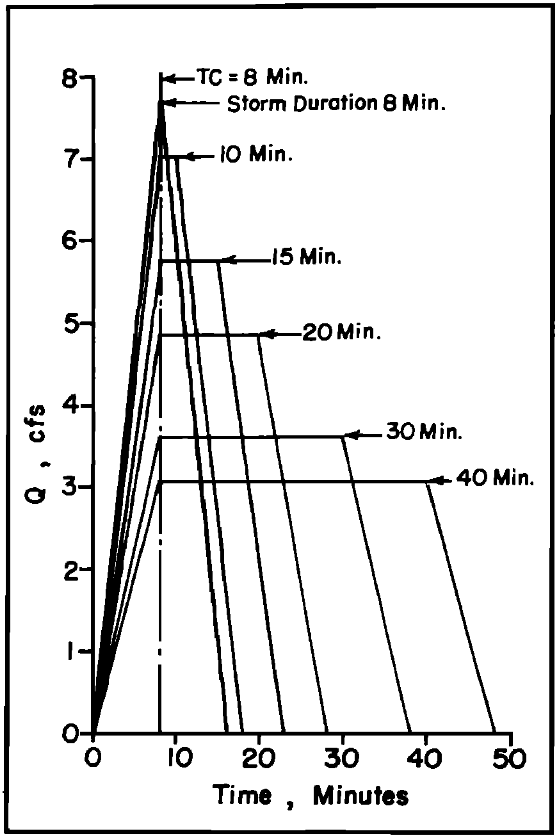

Fig. 4. Use of modified rational method hydrographs to estimate detention storage [after Poertner, 1974].

durations are shown in Figure 4. The corresponding storage volumes required to maintain the maximum release rate of 2.5 cfs are shown in Table 5. The critical storage requirement occurs for a rainfall of 15 min, even though the highest peak occurs for rainfall duration equal to T_c. Clearly, for larger more complex basins, improved runoff and flood-routing methods should be used. However, for small homogeneous areas the modified rational method is quite useful for estimating preliminary detention reservoir sizes.

TABLE 5. Determination of Critical Storage Requirement

Storm Duration (min)	Storm Runoff Volume (ft^3)	Release Flow Volume (ft^3)	Required Storage Volume (ft^3)
8	3710	1200	2510
10	4206	1500	2706
15	5184	2250	2934
20	5820	3000	2820
30	6480	4500	1980
40	7344	6000	1344

After Poertner [1974].

Soil Conservation Service Composite Hydrograph Method for Small Urban Drainage Systems (TR 55)

The SCS composite hydrograph method for small urban drainage basins utilizes a subarea approach to represent nonuniform runoff contributions and flood-routing effects. Like the rational method, the U.S. Department of Agriculture SCS [1975] (hereinafter referred to as TR 55) method is a design storm procedure and cannot be used to reconstruct actual runoff events. The SCS curve number (CN) and rainfall-runoff equation came into use in the mid-1950's primarily as a means of estimating runoff potential over 24-hour periods on ungauged agricultural basins. The runoff curve numbers were developed empirically from daily rainfall-runoff records--a fact which is often overlooked in attempting to analyze incremental runoff amounts during the course of a storm. Although intended originally as a simple tool for evaluating the effects of land treatment on runoff from ungauged rural watersheds, the SCS method has been adapted recently to flood peak and hydrograph analysis on urbanizing basins. The interested reader is referred to an excellent review article on the origin of the SCS rainfall-runoff equation by Rallison [1980]. A comparative summary of rational and SCS methods based on the author's experiences is presented in Table 6.

The SCS method presented here is a tabular solution of the SCS runoff and routing equations taken from SCS [1975, Chapter 5].

TABLE 6. Comparison of Rational Method and SCS TR 55

Item	Rational Method	SCS TR 55
Origin	Late 1800's in Great Britain; known as Lloyd-Davies method; designated as rational method in United States because Q=ciA has cfs units	Published 1975; based on computer runs with TR-20; dimensionless UH developed 1949; CN and rainfall runoff equation published 1955
Intended original use	Flood peaks from small uniform areas for culvert and sewer design	CN technique developed as index of runoff potential over a 24-hour period on ungaged basins; wanted simple design method for effects of land treatment
Development	Design storm procedure; RO frequency = RF frequency; basin delivers maximum peak when full area contributes at time T_c	Design storm procedure; RO frequency = RF frequency; CN developed from analysis of daily rainfall-runoff data from rural basins
Parameters	T_c and runoff coefficient C	T_c and CN
Area limits	Approximately 1 mi^2 for basic method; modified rational ~ 10-20 acres	Approximately 20 mi^2 for full hydrograph method; for peak discharge charts, 2000 acres is upper limit
Strengths	Rapid method for design peaks; very useful for simple urban storm drainage systems; modified rational method good for detention basins below small parking lots	Average CN available for more than 4000 soils in TR 55; easy to estimate CN for given soil and surface cover conditions; can give full hydrograph

TABLE 6. Comparison of Rational Method and SCS TR 55 (cont.)

Item	Rational Method	SCS TR 55
Weaknesses	Selection of C value is subjective; very little calibration against actual rain-fall-runoff data	CN developed from 24-hour data; use to estimate incremental runoff during storm is questionable
	Depends on T_c and IDF curves	Depends on T_c and design rainfall
	Very questionable for full hydrograph in areas greater than 20 acres	CN and UH developed empirically from rural watersheds; only just beginning to test SCS on urban basins
	Assumes that rainfall is uniform over basin	Tends to underestimate smaller flood peaks

The underlying equations for estimating rainfall excess have already been presented in chapter 3 as follows:

$$Q = \frac{(P - IA)^2}{P - IA + S} \tag{3}$$

$$S = \frac{1000}{CN} - 10 \tag{4}$$

where

Q accumulated runoff since beginning of storm period, in.;

P accumulated rainfall since beginning of storm period, in.;

IA initial abstraction loss or sum of interception, depression storage and infiltration required prior to runoff initiation, in.;

S maximum potential soil retention, in.;

CN runoff curve number determined from soil type, land cover, and antecedent moisture conditions.

Although IA has been estimated at 0.2S from experimental rainfall-runoff data, there is some evidence indicating a lower percentage of the soil retention parameter. (See chapter 3 discussion and also SCS [1969].) A graphical solution of equations (3) and (4) is shown in Figure 5.

Individual steps in the SCS composite hydrograph procedure are noted as follows:

1. Delineate watershed boundary and individual subareas including their respective land uses and sewer inlet points.

2. Obtain principal soil type for total basin and determine corresponding hydrologic soil group parameter from Appendix B of TR 55. Enter Table 7 with land use and hydrologic soil group for each subarea to obtain the runoff curve number CN.

3. Estimate local travel time across each subarea, T_c, and also the total travel time T_t from each subarea to basin outlet. Three approaches are available for overland flow times. The local subarea flow times may be estimated either from the SCS lag equation (Figure 3-3 or equation 3-2 of TR 55) or from the FAA overland flow method identified in equation (2). In the former alternative, one must obtain T_c as $T_c = 1.6 \times \text{lag}$. A third alternative is to use average overland velocities for typical surfaces, as shown in Figure 6. (Note that for very short overland distances, Figure 6 will tend to underestimate flow times since it is based on normal flow assumptions.) Total travel time includes in-channel or in-pipe flow time to basin outlet from each subarea inlet point.

4. Establish 24-hour rainfall depth from TP 40 [Hershfield, 1961] or other local source. Note that the maximum 24-hour storm will usually contain maximum depths for shorter durations, such as 1-hour, and thus it is possible to compare TR 55 with other procedures using durations as low as 30 min, even though TR 55 was developed for a 24-hour storm. This is due primarily to the SCS dimensionless hyetograph (type II distribution) which places approximately 54% of total storm rainfall in the central 2 hours of the 24-hour period.

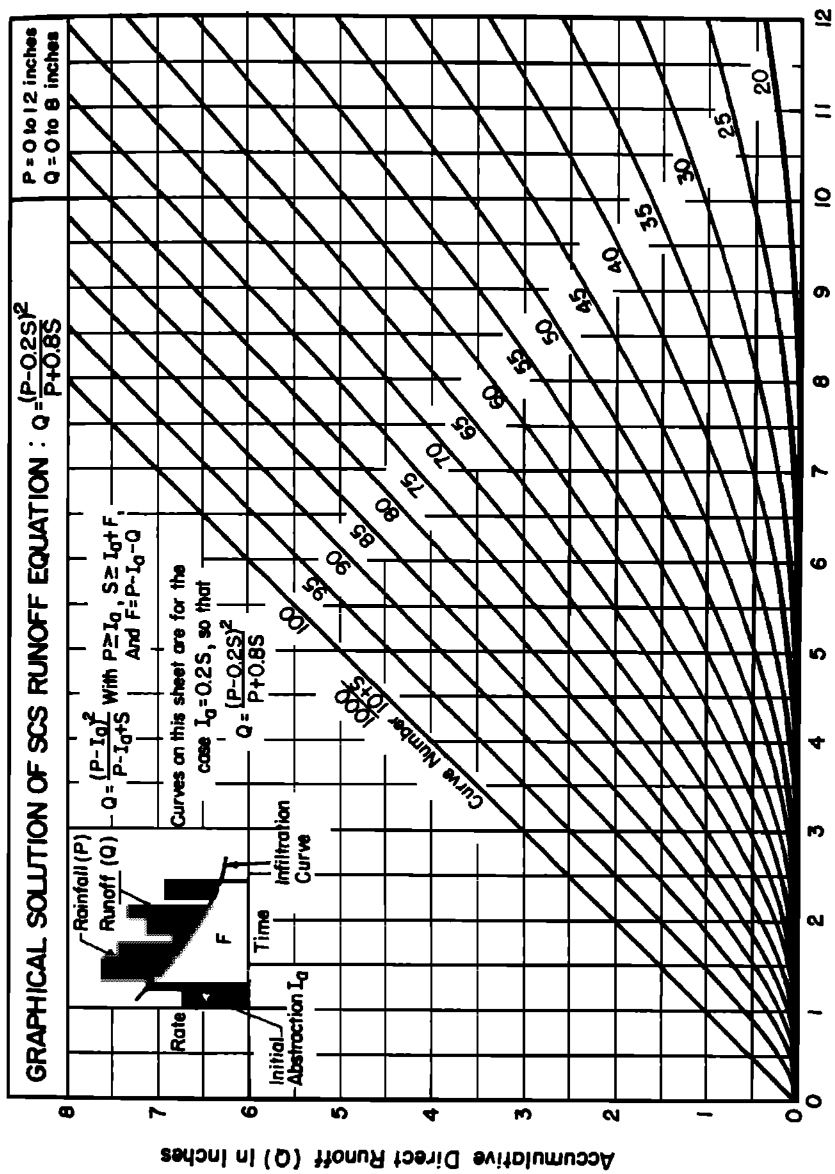

Fig. 5. Solution of the SCS runoff equations.

TABLE 7. Runoff Curve Numbers for Selected Agricultural, Suburban, and Urban Land Use

Land Use Description	Hydrologic Soil Group			
	A	B	C	D
Cultivated land				
Without conservation treatment	72	81	88	91
With conservation treatment	62	71	78	81
Pasture or range land				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow, good condition	30	58	71	78
Wood or forest land, thin stand, poor cover, no mulch	45	66	77	83
Good Cover	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.				
Good condition (grass cover on 75% or more of the area)	39	61	74	80
Fair condition (grass cover on 50% to 75% of the area)	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential				
Average lot size and average % imperv.				
1/8 acre or less, 65%	77	85	90	92
1/4 acre, 38%	61	75	83	87
1/3 acre, 30%	57	72	81	86
1/2 acre, 25%	54	70	80	85
1 acre, 20%	51	68	79	84
Paved parking lots, roofs, driveways,etc.	98	98	98	98
Streets and roads				
Paved with curbs and storm sewers	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89

After SCS [1975].

5. Compute total runoff depth in inches using equations (3) and (4) or Figure 5.

6. Using the total travel time T_t and T_c for each subarea, enter Table 5-3 of TR 55 containing unit discharges at various

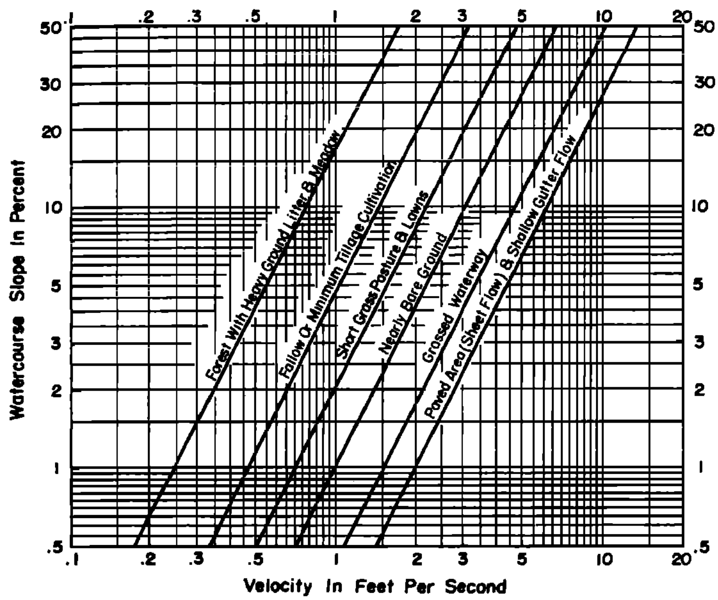

Fig. 6. Average velocities for estimating travel time for overland flow [after SCS 1975].

points on the total hydrograph at the basin outlet. The entire procedure is illustrated by an application to the Calder Alley drainage system.

Application of SCS TR 55 Method to Calder Alley Drainage System

The outflow hydrograph for the 24-hour 25-year storm is to be computed for the existing Calder Alley system by the SCS composite hydrograph method. Land use and related runoff data are taken from Table 1. The principal soil is a Hagerstown silt loam which is classified as a C hydrologic soil type in Appendix B of TR 55. Runoff curve numbers CN are assigned to each subarea using Table 7.

Because of various uncertainties in the SCS lag equation, the local travel times have been estimated using average overland velocities in Figure 6 and measured overland distances to each inlet point. In-pipe velocities have been computed for a single average pipe representing the entire system. An average sewer velocity of 10.8 ft/s was computed assuming a 36-in.-diameter circular section flowing full with slope of 1.5% and Manning n = 0.014. A summary of subarea runoff data and travel times is presented in Table 8.

The 24-hour 25-year rainfall depth for Calder Alley location is estimated as 4.10 in. from the Pennsylvania rainfall manual [Kerr et al., 1970]. This total rainfall depth is now converted to total runoff depth for each subarea by solving (3) and (4) graphically in Figure 5.

The final step is to obtain unit discharges in csm (cubic feet per second per square mile) per inch of runoff from Table 9. This table is taken directly from Chapter 5 of TR 55 [SCS, 1975]. It was developed with the SCS computer program TR-20, which was used to generate subarea hydrographs (subarea of 1.0 mi^2, CN = 75, runoff = 3 in.) for a range of T_c values and to route them through channel reaches having a range of travel time T_t. Note that the unit discharges of Table 9 should not be used when large changes in CN occur among subareas within a basin and when runoff volumes are less than 1.5 in. for CN less than 60. The tabular method used here is considered valid for most urbanizing watersheds for subareas up to approximately 20 mi^2 in area.

A summary of the SCS composite hydrograph method is presented in Table 10 for the Calder Alley system. It is noted that the peak outflow is estimated at 429 cfs for the 24-hour 25-year storm. This peak should be comparable to the 1-hour 25-year runoff peak under the premise that the maximum 1-hour rainfall is contained in the 24-hour storm. Since the T_c for the Calder Alley system is roughly 40 min, the watershed will reach its maximum outflow at this time, and rain falling beyond this point will not contribute to the outflow peak. Thus the SCS 24-hour 25-year peak of 429 cfs should be comparable to the 1-hour 25-year peak estimated by the rational method - all input being equal. The rational method peak is only 336 cfs - low by roughly 25%. However, while the subarea input data has remained constant, the methods used to estimate overland travel times are different.

Under the SCS procedure, a total T_c for the Calder Alley system was 0.58 hours or 35 min (from Table 8). For the rational method, using FAA travel time charts, the total T_c was 45 min. (from Table 4). If one were to apply the rational method to the entire Calder Alley basin with a reduced T_c of 35 min, we would increase the rain intensity to 3.0 in./h using Figure 2. With a basin-wide C value of 0.59, the estimate of Q_{25} becomes 0.59 x 3.0 x 227.2 = 402 cfs. Alternatively, if we use a basin average CN value of 84 in the SCS procedure together with T_t = 0.0 hours and T_c = 0.75 hours, we get a peak discharge of 388 csm/in. or 388 csm/in. x (227/640 acres/mi^2) x 2.5 in. = 344 cfs. And so it seems the choice of overland flow travel time will largely dictate the accuracy of the discharge estimate by either method. This is an area of extreme importance to the urban drainage engineer which is discussed briefly below.

Time of Concentration in the Urban Basin

The preceding discussion of Q_{25} results by the rational and SCS TR 55 methods illustrates the sensitivity of most desk-top methods to T_c estimation. This parameter shows up in one form or another in almost every method now in use. At the present time there are perhaps half a dozen different T_c formulas having applicability to the developing watershed. A brief summary of the more important ones is presented in Table 11. Each was developed under special laboratory or field conditions and should only be used in those portions of the urban basin where it is applicable. Only limited testing of T_c methods in a composite urban watershed has been reported [Kibler et al., 1981].

The impact of using alternative T_c methods in subarea F of the Calder Alley drainage system is shown by example calculations in Table 12. The variation in T_c is dramatic-ranging from 9.6 to 36.0 min, for a composite drainage path, with a corresponding range in Q_{25} from 117 cfs down to 62 cfs. The reasonableness of these estimates is clearly dependent on the range of applicability for the different T_c equations. This problem has never

TABLE 8. Calder Alley Subarea Data for SCS Composite Hydrograph Construction

Subarea	Area Acres	Land Use	CN[a]	Overland Surface	Overland Slope, %	Overland[c] Velocity ft/s	Overland Distance ft
A	12.1	Residential (1/2 acre)	74	Lawn	3.3	1.3	800
B_1	14.0	Small business (2/3)		Paved	4.5	4.0	850
B_2	8.0	Residential (1/3)	88	Lawn	1.2	1.0	800
C_1	17.9	Commercial		Paved		3.0	900
C_2	6.1	Commercial	94	Paved	2.9	3.0	800
D_1	19.0	Commercial		Paved		3.0	1300
D_2	4.9	Commercial	97	Paved	3.0	3.0	800
		Residential (2/3 dense)		Lawn			
E	47.6	Small Business (1/3)	83	Paved	3.5	2.5	1600
F	52.6	Residential (1/4 acre)	79	Lawn	3.0	1.3	1800
G	45.0	Residential (1/4 acre)	81	Lawn	3.5	1.5	1300
Total	227.2	-	84[b]	-	-	-	-

Subarea	Overland Flow Time hours	T_c, hours[d]	Sewer Distance to Outlet, ft	Outlet Flow Time[e] hours	Travel Time[f] T_t, hours
A	0.17	0.20	1550	0.04	0.00
B_1 B_2	0.06 0.22	0.20	1850	0.05	0.07
C_1 C_2	0.08 0.07	0.10	3400	0.09	0.07
D_1 D_2	0.12 0.07	0.10	5050	0.13	0.15
E	0.18	0.20	7050	0.18	0.16
F	0.38	0.40	7900	0.20	0.18
G	0.24	0.20	7050	0.18	0.22
Total	-	-	-	-	-

[a] CN values from Table 7.

[b] Weighted average.

[c] Overland velocities estimated from Figure 6.

[d] Time of concentration, T_c, has been rounded to coincide with values in Table 9.

[e] Sewer flow times based on average pipe velocity of 10.8 ft/s.

[f] Travel time, T_t, has been adjusted to reflect rounding of T_c values.

TABLE 9. Tabular Discharges for Type II Storm Distribution

T_t	Hydrograph Time, hours										
	11.5	12.0	12.2	12.4	12.6	12.8	13.0	13.2	13.5	14.0	14.5
	Time of Concentration = 0.1 hour										
0	51	477	152	121	85	70	65	52	48	39	33
0.25	38	626	546	236	137	97	75	66	52	41	35
0.50	27	133	482	543	310	168	110	81	63	47	38
0.75	20	42	125	392	515	360	206	127	80	53	42
1.00	15	28	41	115	328	470	389	245	121	64	47
1.50	10	16	19	25	38	92	236	410	360	133	66
2.00	6	10	12	14	18	23	34	74	244	371	142
2.50	4	6	7	9	11	13	16	21	41	243	343
3.00	2	4	4	5	7	8	10	12	17	50	239
3.50	1	2	2	3	4	5	6	7	10	17	59
4.00	0	1	1	2	2	3	4	5	6	10	18
	Time of Concentration = 0.2 hours										
0	47	641	245	138	104	75	68	56	49	40	34
0.25	34	419	627	341	173	114	83	70	55	43	36
0.50	24	87	341	545	397	219	133	92	67	49	39
0.75	18	36	84	284	491	422	263	157	89	56	43

1.00	14	25	35	79	240	426	427	299	147	69	49
1.50	9	14	18	23	32	67	176	330	399	159	72
2.00	6	9	11	13	16	21	29	56	192	363	168
2.50	3	6	7	8	10	12	15	19	33	200	337
3.00	2	3	4	5	6	8	9	11	15	40	203
3.50	0	2	2	3	4	5	6	7	9	16	46
4.00	0	1	1	1	2	3	3	4	6	9	16
Time of Concentration = 0.4 hours											
0	39	558	451	247	155	105	80	66	53	42	35
0.25	28	196	467	464	295	180	119	87	64	47	38
0.50	20	53	172	395	453	332	211	137	84	54	42
0.75	16	29	51	150	338	429	356	241	128	65	47
1.00	12	21	28	49	132	292	403	368	220	88	55
1.50	8	12	15	19	25	43	102	220	365	224	93
2.00	5	8	9	11	14	17	23	37	119	338	225
2.50	3	5	6	7	9	11	13	16	25	132	317
3.00	1	3	3	4	5	7	8	10	13	28	140
3.50	0	1	2	2	3	4	5	6	8	13	32
4.00	0	0	1	1	1	2	3	3	5	8	14

After SCS [1975]. In cubic feet per second per square mile per inch of runoff.

TABLE 10. SCS Composite Hydrograph Construction for Calder Alley System

Subarea	T_c hours	T_t hours	CN	Runoff[b] in.	Hydrograph Outflows at Given Clock Hours[c], cfs										
					11.5	12.0	12.2	12.4	12.6	12.8	13.0	13.2	13.5	14.0	14.5
A	0.20	0.00	74	1.68	1	20	8	4	3	2	2	2	2	1	1
B_1 B_2	0.20	0.07	88	2.83	4	56	34	19	12	8	7	6	5	4	3
C_1 C_2	0.10	0.07	94	3.43	6	67	34	23	13	10	9	7	6	5	4
D_1 D_2	0.10	0.15	97	3.75	6	79	54	27	16	12	10	8	7	6	5
E	0.20	0.16	83	2.38	7	88	86	47	26	18	14	12	9	7	6
F	0.40	0.18	79	2.05	5	50	78	68	43	27	18	14	10	8	6
G	0.20	0.22	81	2.21	6	69	90	49	26	17	13	11	8	7	6
Total hydrograph[a]					35	429	384	237	139	94	73	60	47	38	31

[a]Total hydrograph obtained by adding subarea hydrographs.

[b]Rainfall excess or runoff obtained from Figure 5.

[c]Hydrograph discharges obtained from unit discharges in Table 9 and multiplying by [area (acre)/640] x runoff (in.).

been fully addressed and deserves full attention in future urban stormwater investigations.

Synthetic Unit Hydrograph Methods for Urban Watersheds

Since the early work of Snyder [1938] on synthetic unit hydrographs for ungaged areas, several investigators have developed empirical relationships between UH parameters and urban indicators such as impervious fraction and the extent of sewered channels [Rantz, 1971; Van Sickle, 1969; Jones, 1970; Nelson, 1970; Brater and Sherrill, 1976]. The Colorado urban hydrograph procedure [Wright-McLaughlin Engineers, 1969] is a well-known example of this effort based upon revised C_p and C_t coefficients for the Denver region.

With the advent of programmable calculators and the availability of software, the discrete convolution can now be readily applied [see Croley, 1979]. The instantaneous unit hydrograph (IUH) is therefore a competitive procedure offering some of the simplicity in the empirical unit hydrograph methods. A conceptual IUH for the urban basin has been presented by Rao et al. [1972], while a regional dimensionless IUH for urban watersheds has been investigated by Hossain et al. [1978]. Delleur et al. [1975] have described the applications of lumped non-linear and quasi-linear IUH models to several watersheds in Indiana and Illinois.

Because of its simplicity and geographical coverage, the Espey 10-min unit hydrograph has been selected to illustrate synthetic UH application to urban basins. It was developed from 41 basins located in Texas (16), Tennessee (1), Mississippi (2), Pennsylvania (1), North Carolina (9), Colorado (2), Kentucky (6), and Indiana (4). These basins are in the size range 0.014 to 15.0 mi^2 and have impervious fractions ranging from 2 to 100%. The Epsey UH method has been documented in a recent U.S. Environmental Protection Agency (EPA) report [1978] which provides the basis of the description here.

The synthetic 10-min UH developed by Espey and Altman [1978] is

TABLE 11. Summary of Time of Concentration (T_c) Methods

Method and Date	Formula for T_c (min.)	Remarks
Kirpich [1940]	$T_c = 0.0078\ L^{0.77}\ S^{-0.385}$ L = length of channel/ditch from headwater to outlet, ft S = average watershed slope, ft/ft	Developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%); for overland flow on concrete or asphalt surfaces multiply T_c by 0.4; for concrete channels multiply by 0.2; no adjustment for overland flow on bare soil or flow in roadside ditches
California Culverts Practice [1942]	$T_c = 60\ [11.9\ L^3/H]^{0.385}$ L = length of longest watercourse, mi H = elevation difference between divide and outlet, ft	Formula is essentially the Kirpich equation; developed from small mountainous basins in California; [U.S. Bureau of Reclamation, 1973, pp. 67-71]
Izzard [1946]	$T_c = [41.025\ (0.0007\ i + c)\ L^{0.33}]/[S^{0.333}\ i^{0.667}]$ i = rainfall intensity, in./h c = retardance coefficient L = length of flow path, ft S = slope of flow path, ft/ft	Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement, c = 0.012 for concrete pavement, and c = 0.06 for dense turf; solution is extremely tedious and requires iteration; product i times L should be ≤ 500

Federal Aviation Agency [1970]	$T_c = 1.8(1.1 - C)L^{0.50}/S^{0.333}$ C = rational method runoff coefficient L = length of overland flow, ft S = surface slope, %	Developed from air field drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems but has been used frequently for overland flow in urban basins
Kinematic wave formulas Morgali and Linsley [1965] Aron and Egborge [1973]	$T_c = 0.94\ L^{0.6}\ n^{0.6}/[i^{0.4}\ S^{0.3}]$ L = length of overland flow, ft n = Manning roughness coefficient i = rainfall intensity in./h S = average overland slope, ft/ft	Overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces; method requires iteration since both i (rainfall intensity) and T_c are unknown; superposition of intensity-duration-frequency curve gives direct graphical solution for T_c
SCS [1975] lag equation	$T_c = \{100\ L^{0.8}[(1000/CN)-9]^{0.7}\}/\{1900\ S^{0.5}]$ L = hydraulic length of watershed (longest flow path), ft CN = SCS runoff curve number S = average watershed slope, %	Equation developed by SCS from agricultural watershed data; it has been adapted to small urban basins under 2000 acres; found generally good where area is completely paved; for mixed areas it tends to overestimate; adjustment factors are applied to correct for channel improvement and impervious area; the equation assumes that T_c = 1.67 x basin lag

TABLE 11. Summary of Time of Concentration (T_c) Methods (cont.)

Method and Date	Formula for T_c (min.)	Remarks
SCS [1975] average velocity charts	$T_c = 1/60 \Sigma L/V$ L = length of flow path, ft V = average velocity in feet per second from Fig. 3-1 of TR 55 for various surfaces	Overland flow charts in Fig. 3-1 of TR 55 show average velocity as function of watercourse slope and surface cover

TABLE 12. Example T_c Calculations for Subarea F in the Calder Alley Drainage System

Method	T(grass), min	T(gutter), min	T(pipe), min	T_c(total), min	T_c(comp)[d], min	Design i[b], in./h	Q_{25}[e], cfs
Kirpich	4.0	1.0	3.2	8.2	9.6	5.3	117
SCS lag	----	---	---	----	12.4	4.9	108
SCS velocity charts	2.8	1.4	2.9[c]	7.1	12.0	5.0	110
Federal Aviation Agency	18.0	3.2	2.9[c]	24.1	36.0	2.8	62
Izzard[a,b]	----	---	----	----	11.1	5.1	113
Kinematic[b] wave	18.0	3.2	2.9[c]	24.1	18.0	4.0	88

Given (1) Subarea F in Calder Alley drainage system, (2) single family residential; area = 52.6 acres, 20% impervious, (3) longest flow distance to nearest major inlet = 1800 ft (200 ft grass, 300 ft gutter, 1300 ft 12-in. storm drain, (4) overland slope and pipe slope = 3%, (5) average runoff coefficient = 0.42; SCS CN = 80, (6) Manning n : grass = 0.20; gutter = pipe = 0.014, and (7) 1-hour, 25-year rainfall ($_1P_{25}$) = 2.06 in. Find T_c to major inlet under 25-year storm conditions.

[a]Izzard [1946] calculations are iterative and too tedious to be used in design analysis.

[b]Penna. intensity-duration-frequency curves are those developed by NWS; these are applied to $_1P_{25}$ to get design i as well as T_c in Izzard and kinematic wave methods; design i is for T_c (comp).

[c]Manning equation under pipe-full flow.

[d]Composite T_c computed for length = 1800 ft using weighted average retardance-roughness coefficients.

[e]Q_{25} = c i A = (0.42) (52.6) i.

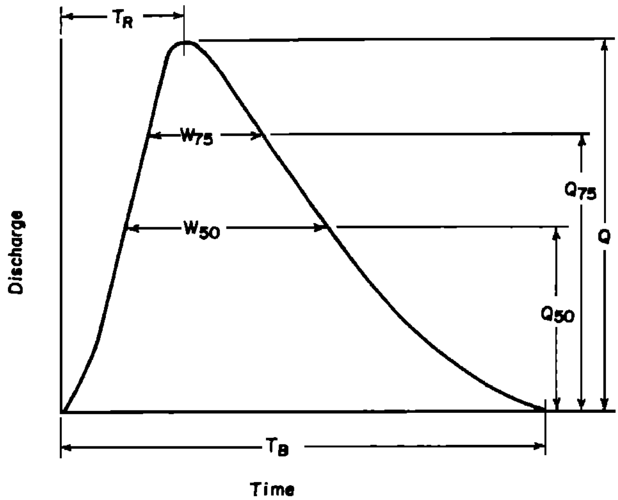

Fig. 7. Definition of Espey 10-min UH parameters [after Espey and Altman, 1978].

described by five parameters shown in Figure 7. These parameters have been related by statistical regression to basin characteristics as indicated in Table 13. The widths W_{50} and W_{75} in Table 13 are normally positioned such that one third lies on the rising side and two thirds on the recession limb of the UH.

The relationships in Table 13 indicate functional dependence between the five UH parameters and basin area, channel length, slope, impervious fraction, and conveyance index. Each basin property is readily obtained, with the possible exception of watershed conveyance. The engineer should be aware that the relationships in Table 13 provide seven points (including origin) which will permit the construction of several possible 10-min UH. The analyst should always compute the area beneath the hydrograph to insure that it represents 1.0 in. of direct runoff. Reshaping the 10-min. UH between computed points may be required to meet this volume requirement.

Perhaps the biggest unknown in the Espey UH method is the watershed conveyance factor, Φ. It appears directly in the rise-time equation of Table 13. It was developed as a means of accounting for reduction in T_R caused by channel improvements

TABLE 13. Espey 10-Minute UH Equations

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

After EPA [1978]. Where L is the total distance (in feet) along the main channel from the point being considered to the upstream watershed boundary; S is the main channel slope (in feet per foot) as defined by H/(0.8L), where L is the main channel length as described above and H is the difference in elevation between two points, A and B (A is a point on the channel bottom at a distance of 0.2L downstream from the upstream watershed boundary; B is a point on the channel bottom at the downstream point being considered); I is the impervious area within the watershed (in percent); ϕ is the dimensionless watershed conveyance factor as described elsewhere in the text; A is the watershed drainage area (in square miles); T_R is the time of rise of the unit hydrograph (in minutes); Q is the peak flow of the unit hydrograph (in cubic feet per second); T_B is the time base of the unit hydrograph (in minutes); W_{50} is the width of the hydrograph at 50% of the Q (in minutes); and W_{75} is the width of the unit hydrograph at 75% of Q (in minutes).

and storm sewers which could not be explained by increases in impervious fraction alone. The most recent investigation of the Φ factor was carried out using the urban watershed data underlying the equations in Table 13. Figure 8 was developed as a rough guide for selection of Φ based on the analysis of these 41 watersheds. Clearly, this is a limited data base for determination of Φ -- a fairly sensitive UH parameter. Thus there is a need for judgment in selecting Φ, and efforts to refine this parameter of UH rise time should be continued.

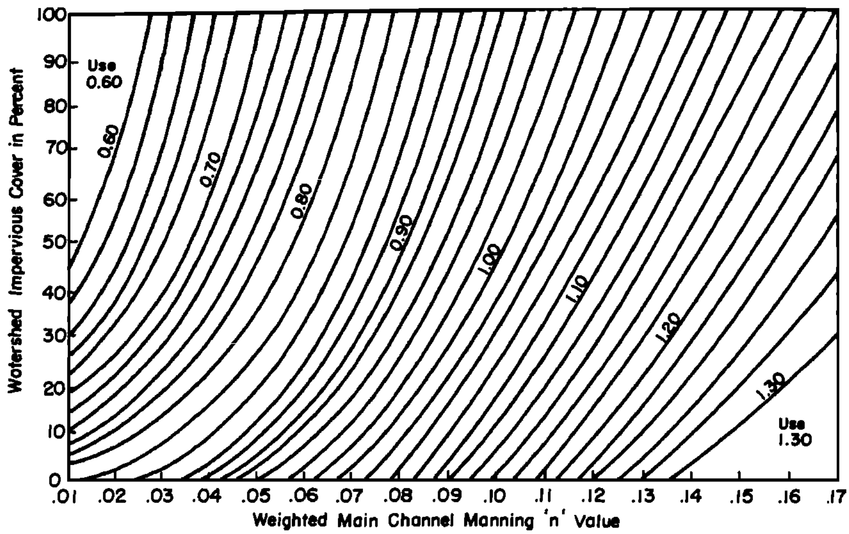

Fig. 8. Watershed conveyance factor Φ, in Espey 10-min UH procedure [after Epsey and Altman, 1978].

Application of Espey-Winslow 10-Minute UH to Calder Alley Basin

In this example, the problem is similar to that for the SCS method, namely, to develop a full outflow hydrograph for the 25-year storm. To be fully comparable with SCS TR 55 estimates of Q_{25}, a 24-hour rainfall should be used. However, this represents a tedious process since in the Espey unit hydrograph procedure a design rainfall hyetograph must first be developed from which storm losses can be deducted. To simplify the application to the Calder Alley basin, a 1-hour 25-year storm has been employed. As indicated in the outflow summary of Table 16, the 24-hour storm produces a peak outflow of approximately 370 cfs by the Espey unit hydrograph method. The basic 10-min UH has been lagged to 30 min, and precipitation excess has been computed in 30-min intervals by the SCS rainfall-runoff equations (3) and (4) using a weighted CN of 84. Space limitations do not permit detailed computations for the 24-hour storm analysis. Recalling that the 1-hour 25-year rainfall is 2.06 in. from the rational method example, we can develop the time distribution for this storm total as shown in Table 14.

TABLE 14. Design Storm Computation for Calder Alley Basin

Duration (min.)	Intensity[a] (in./h)	Precip P (in.)	ΔP (in.)	Δt (min.)	i (in./h)	Time Distribution[b] (min.)
0	0	0				
5	6.4	0.53	0.53	5	6.4	25→30
10	5.4	0.90	0.37	5	4.4	30→35
20	4.0	1.33	0.43	10	2.6	20→25; 35→40
30	3.2	1.60	0.27	10	1.6	15→20; 40→45
40	2.7	1.80	0.20	10	1.2	10→15; 45→50
50	2.35	1.96	0.16	10	1.0	5→10; 50→55
60	2.06	2.06	0.10	10	0.6	0→ 5; 55→60
			$\Sigma=2.06$			

One-hour, 25-year event.

[a]Intensities obtained from standard intensity-duration curves in Figure 2.

[b]Storm distribution is assumed to be symmetrical around peak intensity.

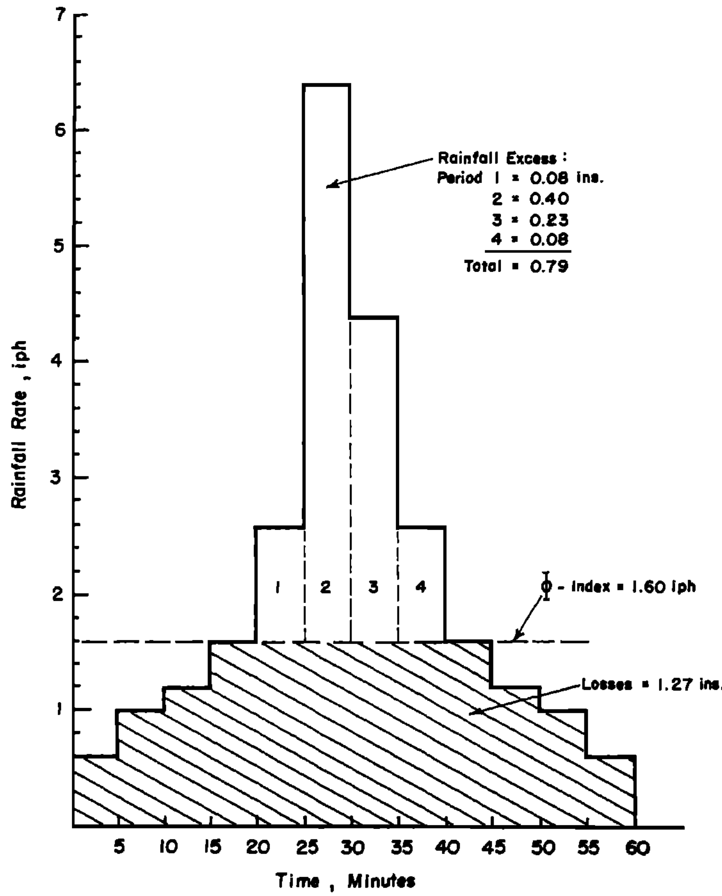

Fig. 9. Time distribution of rainfall excess for 25-year storm by Φ index method on entire Calder Alley Basin.

Losses may now be subtracted from the design rainfall of Table 14 in order to obtain the time distribution of rainfall excess. Any of the infiltration loss models of chapter 3 could be selected. For this example, the SCS rainfall excess model in (3) and (4) was applied to the entire Calder Alley basin with an area-weighted CN of 84. The total rainfall excess for a 1-hour 25-year rainfall of 2.06 in. is 0.79 in., with 1.27 in. going to losses. The time distribution of direct runoff is then computed by Φ index method as shown in Figure 9. This procedure represents a highly simplified loss analysis. A more detailed approach would break the total basin into subareas, delineate pervious and impervious contributing areas, develop separate 10-min UH and include simple flow routing to develop an outflow hydrograph at the basin outlet. The analysis here is intended to illustrate the basic steps in the Espey 10-min UH method.

The 10-min synthetic UH is computed next using the Espey equations in Table 13. From watershed data already presented, the following basin characteristics have been estimated:

L length of main channel up to watershed boundary; equal to 9700 ft;

$Elev_{outlet}$ = 1066 ft;

$Elev_{0.8L}$ = 1175 ft;

S main channel slope, equal to $(Elev_{0.8L} - Elev_{outlet})/0.8L = 0.0148$ ft/ft;

I impervious percent, equal to 43.8%;

Φ watershed conveyance factor from Figure 8 with Manning n = 0.015 and I = 44%, equal to 0.62;

A = 227.2 acres = 0.355 mi^2.

Solving for the UH parameters in Table 13:

$$T_R = 3.1(9700)^{0.23}(0.0148)^{-0.25}(43.8)^{-0.18}(0.62)^{1.57} = 17.6 \text{ min}$$

$$Q = 31620\ (0.355)^{0.96}(17.6)^{-1.07} = 545 \text{ cfs}$$

$$T_B = 125{,}890(0.355)(545)^{-0.95} = 112.3 \text{ min}$$

$$W_{50} = 16{,}220(0.355)^{0.93}(545)^{-0.92} = 18.8 \text{ min}$$

$$W_{75} = 3240(0.355)^{0.79}(545)^{-0.78} = 10.5 \text{ min}$$

The 10-min UH can now be sketched and tabulated such that the volume represented equals 1.0 in. of runoff over 227.2 acres. For convenience the 10-min UH is assumed to peak at 20 min rather than 17.6 min, as computed.

Applying the 10-min UH to the 10-min rainfall excess from Figure 9 one obtains the outflow hydrograph on the left in Table 15 for

TABLE 15. Computation of Outflow Hydrograph for 25-Year Storm on Calder Alley by Espey 10-Minute UH Method

	Ten-Minute UH Analysis			Five-Minute UH Analysis		
Time min	10-min UH cfs	Rain Excess in.	Outflow Hyd. cfs	5-min UH cfs	Rain Excess in.	Outflow Hyd. cfs
0	0	0.00		0	0.00	
5	-	-		160	0.00	
10	225	0.00		390	0.00	
15	-	-		430	0.00	
20	545	0.00	0	600	0.00	0
25	-	-	-	340	0.08	13
30	275	0.48	108	210	0.40	95
35	-	-	-	140	0.23	227
40	125	0.31	331	110	0.08	322
45	-	-	-	75	0.00	397
50	65	0.00	301	60	0.00	325
55	-	-	-	50	0.00	221
60	40	0.00	145	40	0.00	140
65	-		-	30		99
70	30		70	25		71
75	-		-	20		54
80	20		39	15		43
85	-		-	12		35
90	12		27	10		27
95	-		-	9		22
100	10		19	8		17
105	-		-	6		14
110	5		12	4		11
115	-		-	0		9
120	0		9			8
125			-			7
130			6			5
135			-			4
140			2			1

the entire Calder Alley basin. The outflow peak is 331 cfs. Alternatively, one could obtain the 5-min UH by the usual S curve procedure and apply this to the 5-min rainfall excess amounts in Figure 9. Following this method with a more refined rainfall excess distribution, we obtain an outflow peak of 397 cfs. The outflow peak of 370 cfs for the 24-hour rainfall is not substantially different, although it was calculated by a 30-min version

TABLE 16. Summary of Outflow Peaks for 25-Year Storm on Calder Alley Watershed

Method	Case	Q_{25}, cfs
Rational	T_c = 35 min[a]	402
	T_c = 45 min[b]	335
SCS TR 55	T_c = 35 min[a]	429
	T_c = 45 min[b]	344
Espey UH	10-min UH (1-hour storm)	331
	5-min UH (1-hour storm)	397
	30-min UH (24-hour storm)	370
Road Research Laboratory	Virtual hydrograph routing only; no pipe transport	289
Penn State runoff model	Kinematic routing; travel time lagging	420
WRE-SWMM	Kinematic routing overland; dynamic routing in transport system	405

[a]T_c is the time of concentration estimated by SCS average velocity charts.

[b]T_c estimated by Federal Aviation Agency overland flow charts and Manning equation.

of the Espey 10-min UH and probably underpredicts the 24-hydrograph peak (total rain = 4.10 in.; excess rain = 2.46 in.).

Summary of Desk-Top Model Results

A summary of the outflow peaks computed by the three methods in this chapter is presented in Table 16. In addition, outflow peak results generated by the British Road Research Laboratory method and by two urban runoff models are shown for comparison purposes. The Penn State runoff model and the WRE stormwater management model both were applied to the 25-year design storm in

Figure 9, given the subarea data for the Calder Alley system. The outflow results shown in Table 16 range from 289 to 429 cfs for Q_{25} - a variation which is not unreasonable given the different assumptions underlying each method [Kibler et al., 1981].

The deficiencies of the rational method are well known and have been described by McPherson [1969]. It is intended primarily for small urban basins where rainfall intensities can be assumed spatially uniform throughout the time of concentration. It is highly sensitive to T_c, the time of concentration, and also to the runoff coefficient. Calibration of the rational method has been limited to the runoff coefficient C for small parking lots instrumented at the Johns Hopkins University [Schaake et al., 1967]. Finally, the rational method is a design storm method with all of its inherent limitations. Nonetheless, the rational method, properly applied, remains an extremely useful tool for the drainage engineer faced with design of simplified storm drain systems.

The Soil Conservation Service method is also a design storm method as it is presented here and in TR 55 (1975). The principal advantage of the SCS TR 55 method is the ease of assigning runoff curve numbers depending on soils, surface cover, and land use. Runoff peaks and volumes may then be readily computed under a range of alternative land use conditions in a developing watershed. Because the runoff curve numbers were developed from daily rainfall-runoff records, their use in estimating incremental rainfall excess during a storm is questionable. Reference is made once again to the paper by Rallison [1980] on the development of SCS runoff equations and CN.

Another important consideration is that the SCS rainfall-runoff equation and the convex routing method incorporated in TR 55 have never been fully calibrated for urban watersheds. Further, there is growing evidence that the initial abstraction IA and the adjustment to CN for antecedent moisture condition are in need of refinement. In general, this author has found that the SCS method tends to underpredict the low-to-moderate runoff events, while reproducing the major events reasonably well. This is certainly consistent with the use of the SCS TR 55 method as a design storm tool. However, the credibility of the underlying SCS TR 20 model could be improved with calibration of parameters such as the initial abstraction and antecedent moisture adjustment for gaged urban watersheds.

The Espey synthetic UH method offers a full-hydrograph alternative to the rational and SCS TR 55 methods. It is simple to apply, but requires careful judgment in the selection of the watershed conveyance factor Φ. It can be applied to both design and actual storm events. Like the SCS method, the Espey 10-min UH procedure suffers from inadequate calibration on urban watersheds outside the original set of 41 used to develop the empirical UH relationships. To the author's knowledge it has received subsequent testing only on a limited basis against urban runoff data in the Denver area and in the Fourth Creek studies by TVA. Clearly the Φ conveyance factor is a sensitive parameter controlling UH peak and shape. More study is needed of watershed conveyance resulting from urbanization. Kibler et al. [1981] have reported further on the desk-top methods in Table 16.

Desk-Top Methods for Urban Runoff Quality

In closing, we turn to a brief discussion of desk-top methods for assessing annual nonpoint source (NPS) runoff and pollutant loadings. Excellent discussions of the use of simplified desk-top procedures for NPS assessment are presented by Lakatos and Johnson [1979], Zison [1977], and by the EPA [1976]. Typical functions for estimating annual wet weather and dry weather flows are shown below. Annual average pollutant loading rates also can be computed by the method shown below. The relationships shown below [Heaney et al., 1977] were developed for EPA by the University of Florida on the basis of data collected in 248 standard metropolitan statistical areas as part of a nationwide evaluation of combined sewer overflows and stormwater discharges.

Annual Stormwater and Dry Weather Quantity Prediction

The following equations for total annual storm runoff were developed by Heaney et al. [1977]:

$$AR = (0.15 + 0.75\ I/100)P - 5.234(DS)^{0.5957} \tag{5}$$

where AR is the annual runoff, in inches per year.

$$I = 9.6PD_d^{0.573 - 0.089 \log_{10} PD_d} \tag{6}$$

where I is the imperviousness, in percent, PD_d is the population density in developed portion of the urbanized area, in persons/acre, P is the annual precipitation, in inches per year, and

$$DS = 0.25 - 0.1875\ (I/100) \quad 0 \leq I \leq 100 \tag{7}$$

where DS is the depression storage, in inches ($0.005 \leq DS \leq 0.30$).

For annual dry weather flow, the following equation applies [Heaney et al., 1977]:

$$DWF = 1.34\ PD_d \tag{8}$$

where DWF is the annual dry-weather flow, in inches per year and PD_d is the developed population density, persons per acre.

Annual Pollutant Loading Prediction

The following equations may be used to predict annual average loading rates as a function of land use, precipitation and population density [see Heaney et al., 1977]:

Separate areas

$$M_s = \alpha(i, j) \cdot P \cdot f_2(PD_d) \cdot \gamma \frac{lb}{ac\ yr} \tag{9}$$

Combined areas

$$M_c = \beta(i, j) \cdot P \cdot f_2(PD_d) \cdot \gamma \frac{lb}{ac\ yr} \tag{10}$$

where

M pounds of pollutant j generated per acre of land use i per year;

TABLE 17. Pollutant Loading Factors α and β for Separate and Combined Sewer Areas

	Land Use i	Pollutant				
		BOD_5	SS	VS	PO_4	N
	Residential	0.799	16.3	9.4	0.0336	0.131
Separate Areas, α	Commercial	3.200	22.2	14.0	0.0757	0.296
	Industrial	1.210	29.1	14.3	0.0705	0.277
	Other	0.113	2.7	2.6	0.0099	0.060
	Residential	3.290	67.2	38.9	0.1390	0.540
Combined Areas, β	Commercial	13.200	91.8	57.9	0.3120	1.220
	Industrial	5.000	120.0	59.2	0.2910	1.140
	Other	0.467	11.1	10.8	0.0411	0.250

From EPA [1976] and Heany et al. [1977].

Loading factors for each pollutant have units of lb/acre-inch.

P annual precipitation, in./yr;

PD developed population density, persons/acre;

α,β factors given in Table 17;

γ street sweeping effectiveness factor;

$f_2(PD_d)$ population density function.

Land uses

i = 1 residential

i = 2 commercial

i = 3 industrial

i = 4 other developed, e.g., parks, cemeteries, schools (assume PD_d = 0)

Pollutants

j = 1 BOD_5, total

j = 2 suspended solids (SS)

j = 3 volatile solids, total (VS)

j = 4 total PO_4 (as PO_4)

j = 5 total N

Population function

$$\begin{aligned} i &= 1 & f_s(PD_d) &= 0.142 + 0.218 \cdot PD_d^{0.54} \\ i &= 2,3 & f_s(PD_d) &= 1.0 \\ i &= 4 & f_s(PD_d) &= 0.142 \end{aligned} \tag{11}$$

Factors α and β for equations: Separate factors, π, and combined factors, β, have units lb/acre-in. To convert to kg/ha cm, multiply by 0.422. See Table 17.

Street sweeping: Factor γ is a function of street sweeping interval N_s (days):

$$\begin{aligned} \gamma &= N_s/20 & \text{if } 0 \leq N_s \leq 20 \text{ days} \\ \gamma &= 1.0 & \text{if } N_s > 20 \text{ days} \end{aligned} \tag{12}$$

As an example of the above methodology, annual stormwater runoff and pollutant loadings have been estimated for the Calder Alley basin treated as a separate storm drain system. The estimated average annual runoff quantity has been computed by equation (5), as shown in Table 18. The computation of average annual pollutant loads and concentrations is shown in Table 19 based upon equation (9).

TABLE 18. Estimated Annual Stormwater Runoff from Calder Alley System

Land Use Type	Area Acres[a]	Impervious percent[a]	Depression Storage, in.[b]	Annual Runoff, in.[c]
Residential	93	30	.19	12.30
Commercial	134	53	.15	19.11
Industrial	-	-	-	-
Other	-	-	-	-

[a]From Table 1 of this Chapter.

[b]Computed from equation (7).

[c]Computed from equation (5).

TABLE 19. Estimated Average Annual Pollutant Loads and Concentrations for Calder Alley Storm Drain System

Land Use	Area, Acres	Pop. Factor	Annual Runoff, acre-in.[a]	BOD_5 lbs.[b]	BOD_5 mg/l[c]	Suspended Solids lbs.[b]	Suspended Solids mg/l[c]	Phosphates lbs.[b]	Phosphates mg/l[c]	Nitrogen lb.[b]	Nitrogen mg/l[c]
Residential	93	1.24	1144	3501	14	71,429	276	147	1	574	2
Commercial	134	1.00	2561	16294	28	113,042	195	385	1	1507	3
Industrial	-	-	-	-	-	-	-	-	-	-	-
Other	-	-	-	-	-	-	-	-	-	-	-

[a]Volume of annual runoff obtained by multiplying runoff depth in Table 18 by contributing area for respective land use.

[b]Pollutant load in pounds computed from loading factors in Table 17 and equation (9) with $\gamma=1$ for street sweeping interval > 20 days and average annual precipitation = 38 in.

[c]Pollutant concentrations computed by dividing respective pollutant loads by annual runoff volume and multiplying by 4.42. This constant is the number of mg/l per lb/acre-in.

The reader is cautioned that the use of nationally averaged loading factors in Table 17 can lead to highly erroneous results and that local data should be used whenever possible. Likewise, equations (5) through (12) have been developed from widely scattered data and can be expected to produce results which differ from actual measurements of observed runoff quantity and quality. Despite these limitations, the annual pollutant loading relationships presented here provide a simple planning index of potential runoff changes in the urban drainage system. Space does not permit further discussion and the interested reader is referred to EPA [1976] and Heaney et al. [1977] for full details of the simplified methodology presented here. Expanded coverage of the runoff quantity and quality computations contained in selected stormwater models is presented in Chapters 5, 6, and 8.

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5 URBAN RUNOFF PROCESSES

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Introduction

A predominant characteristic of urban drainage systems is the man-made impervious pathways for guiding the flow of water over the land surface (e.g., curbs, gutters, lined channels, paved parking areas, and streets) and underground (e.g., storm, sanitary, and combined sewers). The system includes all appurtenances that guide, control, or otherwise modify either the quantity, rate of flow, or quality of the runoff from the urban area such as catch basins, storage basins, inlets, manholes, sediment traps, weirs, and outfall structures. Figure 1, which shows a typical drainage system, exhibits an array of subsystems which interact to convey rainfall from its point of impact to the receiving waters. This assemblage of subsystems can be characterized by three basic subsystems: (1) surface runoff, (2) transport through sewers and major drainage facilites, and (3) receiving water. Each of these subsystems is described briefly below.

Surface Runoff Subsystem

The surface runoff subsystem is illustrated for our example in Figure 2, which depicts the drainage area tributary to a sewer inlet as a system of surface elements (rectangles and triangles), gutters (dotted lines), and drainage ditches (dashed lines). Each subarea of the drainage system is characterized by its area, imperviousness, hydraulic roughness, slope, and certain coefficients that relate to its production of quality constituents that may be transported to the inlet by overland flow.

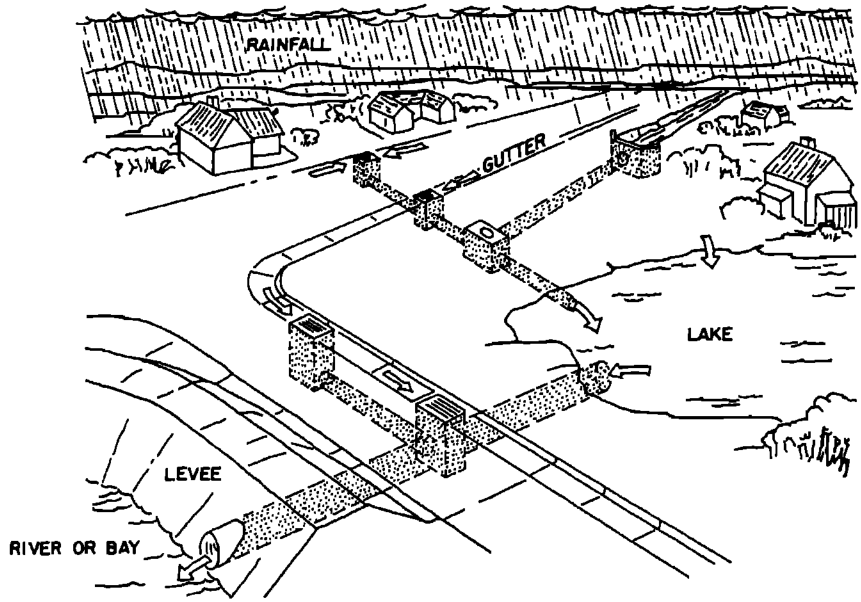


Fig. 1. Typical urban drainage system.

Hydrologic input to the subsystem is composed of a rainfall hyetograph (i.e., a rainfall intensity versus time graph) derived from direct measurements in the watershed. The upper part of Figure 2 illustrates a typical rainfall hyetograph. Additional input includes loss rate parameters and pollutant buildup/washoff coefficients which describe the rate at which quality constituents will be delivered, depending on storm and surface cover conditions.

The overland flow process transforms the rainfall-excess hyetograph (following infiltration and surface retention losses) so that at the inlet one observes an 'inlet hydrograph' or time distribution of inlet flows. In addition, the combined flow and quality processes produce an 'inlet pollutograph,' a time-concentration graph of a particular pollutant as it leaves the surface runoff subsystem and enters the wastewater conveyance system. These two graphs, one of flow and the other of quality, compose the output of the surface runoff subsystem and are input to the transport subsystem.

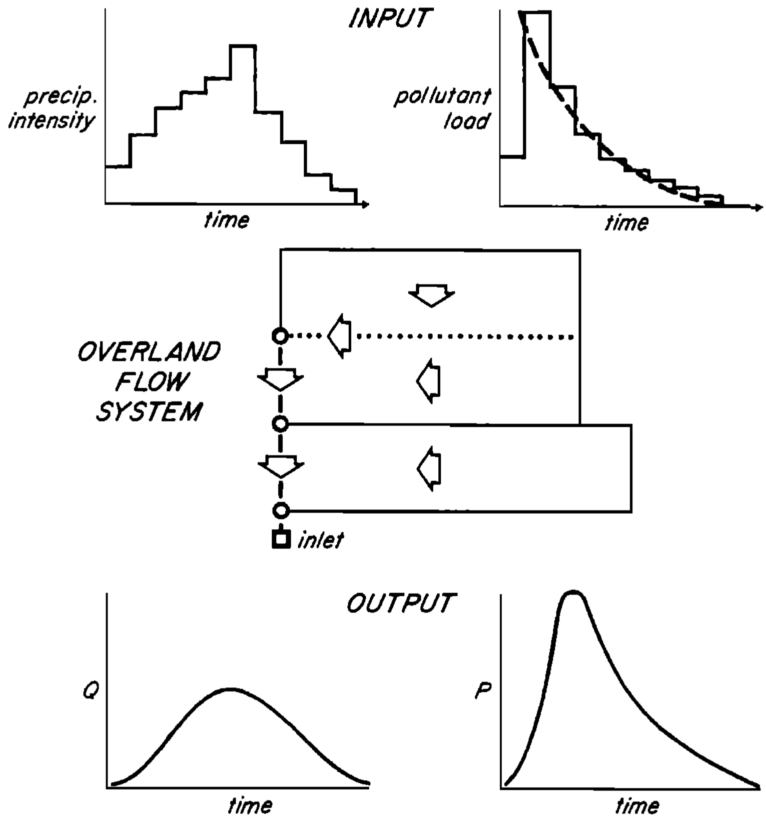


Fig. 2. Surface runoff subsystem.

Transport Subsystem

The transport subsystem is composed of the physical works for conveying storm water and their associated pollutant loads from all of the inlets in the system through a network of storm channels and/or underground conduits to a point (or points) of disposal. Enroute from inlet to discharge, the flow and quality are modified by accretions to the system from other tributary areas and/or point sources of pollution. In addition, flows and pollutant concentrations are attenuated by routing through the system, the degree of modification depending on such factors as system storage, 'off channel' storage, phase relationship of inflow hydrographs and pollutographs, and certain hydraulic properties of the system. Figure 3 illustrates a typical set of outputs

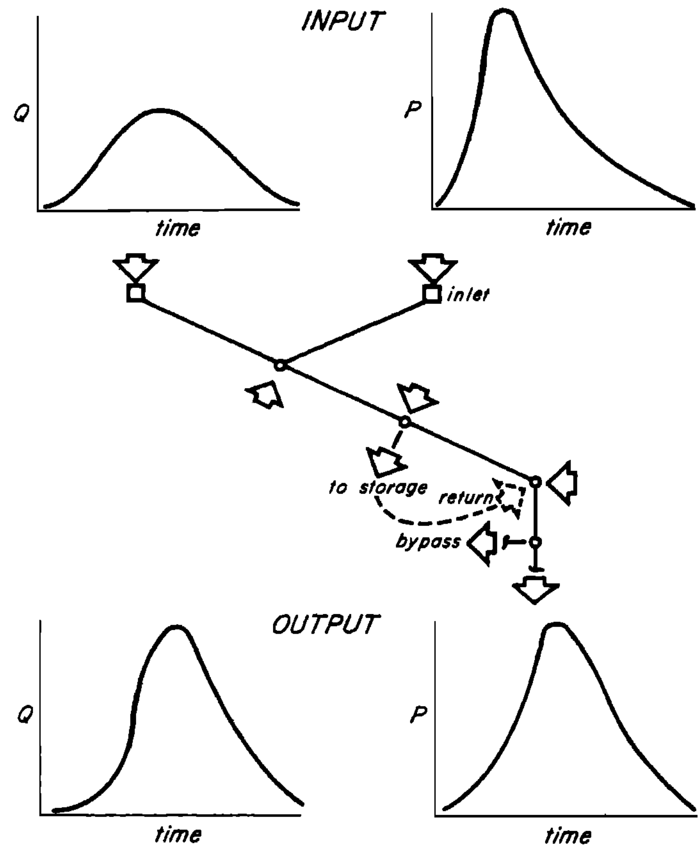


Fig. 3. Transport subsystem.

from the transport subsystem, a hydrograph, and a pollutograph that in turn become inputs to the receiving water.

Receiving Water Subsystem

The receiving water subsystem may be of several forms: stream, lake estuary, or coastal. To illustrate this case, we have assumed that discharge occurs to an estuary.

The impact of the discharge on the estuary will probably be assessed in terms of the concentration of a particular quality constituent; its distribution in space, its persistence in time, and its frequency of exceedance of a certain critical level. For a given hydrologic event the system may be observed synoptically (at the same instant in time) or temporally (at the same point in

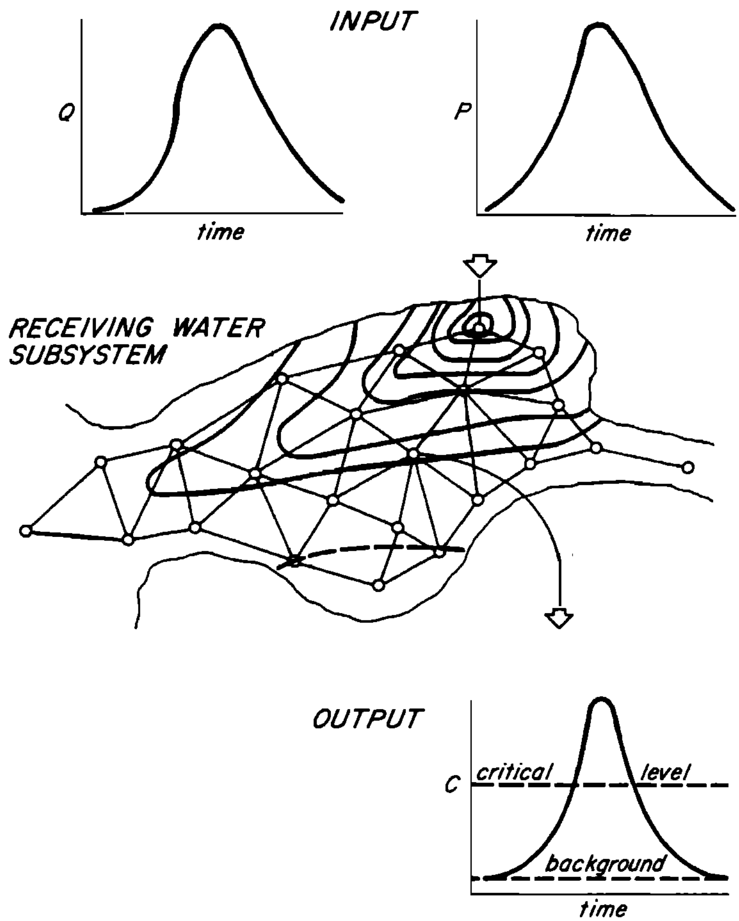


Fig. 4. Receiving water subsystem

space). One gives the distribution in space, the other the persistence in time. From the standpoint of quality management, both viewpoints are usually required for each hydrologic event. To obtain frequency of exceedance of a critical level, the impact on the receiving waters must be observed a 'statistically significant' number of times. Figure 4 illustrates a typical set of responses ('impacts') for our example case.

Combined Sewer Systems

In many older cities in the United States the storm drainage system and the sanitary wastewater system flow in the same pipes.

These sewer systems are called 'combined sewer' systems because the storm sewer and the sanitary sewer are combined into a single sewer. A typical urban drainage system, having combined dry-weather and wet-weather sewers, is shown schematically in Figure 5. The urban system presented here exhibits an impressive array of subsystems which contribute significantly to the complexity of the combined sewer overflow problem. As seen in Figure 5, surface runoff occurs as overland and gutter flow, while subsurface or sewer pipe flow comes from both storm and dry-weather sources. Dry-weather flows (DWF) in turn derive from industrial, municipal, and domestic sources. During dry periods this flow will be intercepted by treatment facilities and discharged as treated effluent to the nearest receiving waters. In periods of wet-weather, however, a significant portion of the DWF must be diverted to a combined sewage overflow where it is discharged untreated to a receiving water. In order to reduce the impact of these discharges on local receiving waters, the combined sewer system must be designed to operate in a manner which will either retain potential overflows in upstream reaches or treat them prior to discharge. The task of optimal design and operation of combined sewer systems is at once the essential problem of reducing system overflows and the challenge of urban stormwater modeling.

The remainder of this chapter describes briefly the conceptual approach and basic mathematical formulations that are used in the U.S. Environmental Protection Agency (EPA) [1977] stormwater management model (SWMM) to simulate urban runoff processes. More detailed descriptions of these processes are also presented by Roesner et al. [1977]. Other models are available, most of which differ substantially in detail but only slightly in the basic conceptual representation. The representation is, of course, subdivided into the major subsystems described above. Only the surface runoff and transport systems will be discussed here in detail. A description of the quantitative representation of the

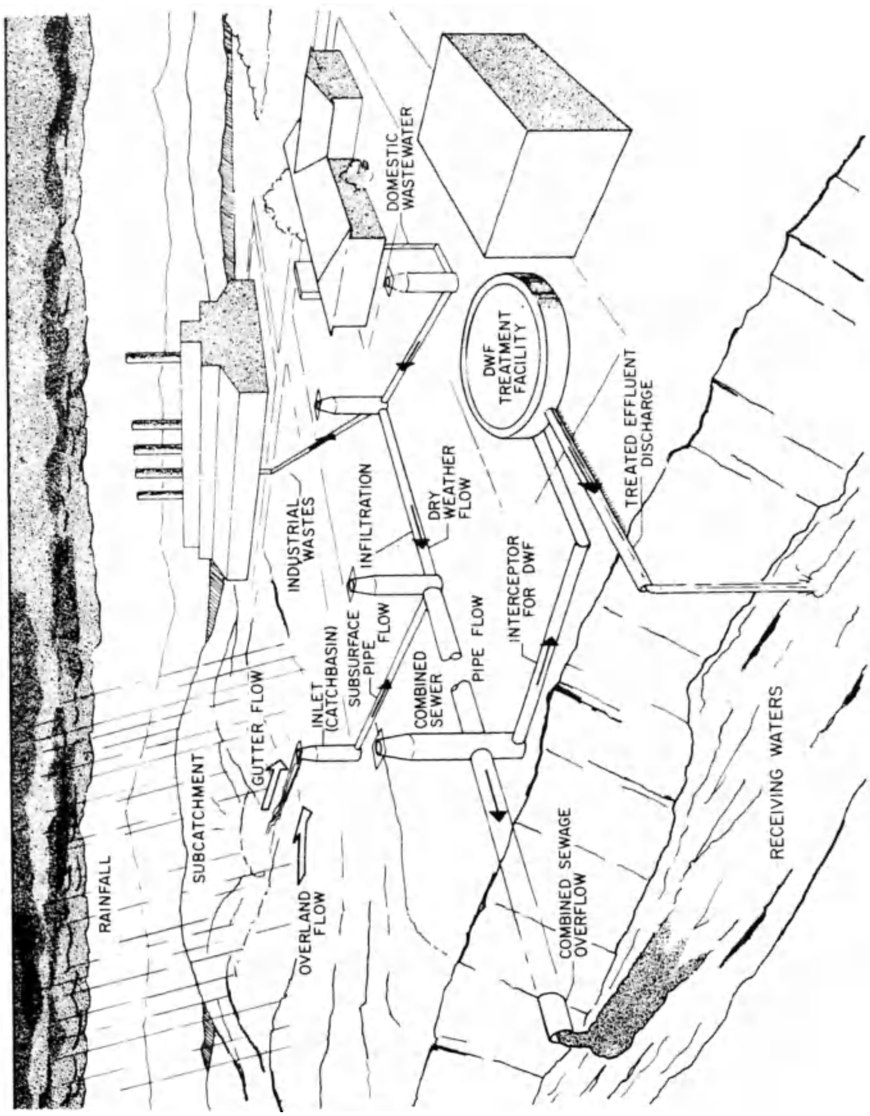


Fig. 5. Typical urban drainage system [Kibler et al., 1975].

receiving water subsystem is beyond the scope of this monograph. The quality aspects of urban runoff are discussed in Chapter 6.

The Surface Runoff Subsystem

Although Figure 2 implies that the surface runoff subsystem is the above ground drainage system, quite often for computational purposes minor sewers are included also in this system. The reason for this is that the computational procedure used to calculate flow through the surface channels can often be used to compute flows through the minor sewers also. This procedure is much faster than the method used for the transport subsystem; thus a significant amount of computer time can be saved by including these appurtenances in the surface runoff subsystem.

Figure 6 shows a typical urban watershed in the City of San Francisco. The transport subsystem is shown as heavy black lines. The surface runoff subsystem is composed of four subareas, each of which consists of a network of yards and streets, gutters, and minor sewers. For runoff computation purposes, however, this whole system can be conceptualized as a single planar surface that discharges to a single surface channel or minor sewer. This conceptual representation is illustrated in Figure 7.

The subareas shown in Figure 7 may contain a complex mixture of land uses, each having a characteristic percentage of its area being impervious. The planar surface is thus subdivided into three planes. The first plane aggregates all the impervious surfaces (having depression storage) regardless of their individual composition, to form a single plane which discharges laterally to a gutter or minor sewer. A second plane is defined as that fraction of the impervious area which has no detention storage at all and thus produces immediate runoff at the start of a storm. Likewise, all the impervious area in the subarea is aggregated to form a third plane, having the same width (width = total subarea area/length). The flow off the subarea is the sum of the flow off the three planes. This aggregated flow is supplied in turn to the gutter or minor sewer element shown in Figure 7.

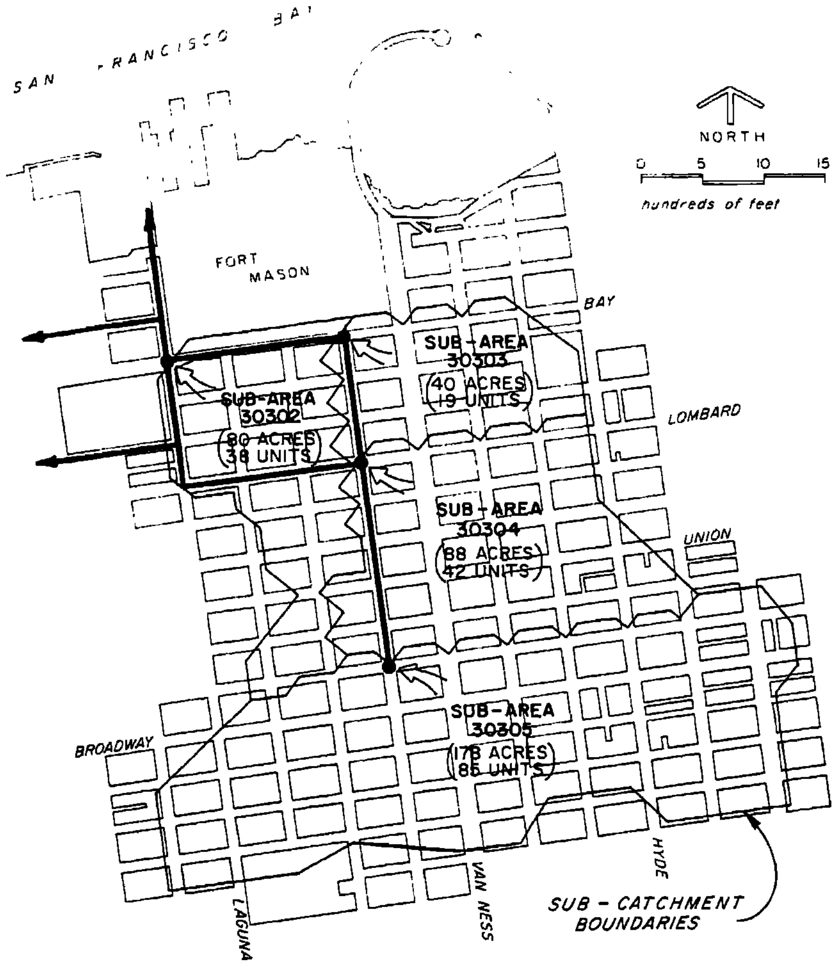


Fig. 6. Geometric representation of the Laguna Street runoff system.

Overland Flow Computation

The basic overland flow routing algorithm in the surface runoff model is the kinematic wave approximation which assumes that the friction slope is equal to the slope of the plane. For this condition, the equations of continuity and uniform flow must be solved simultaneously to define at each time step the depth of flow and the outflow for each of the three flow planes in the surface runoff model. The flow routing algorithm is applied

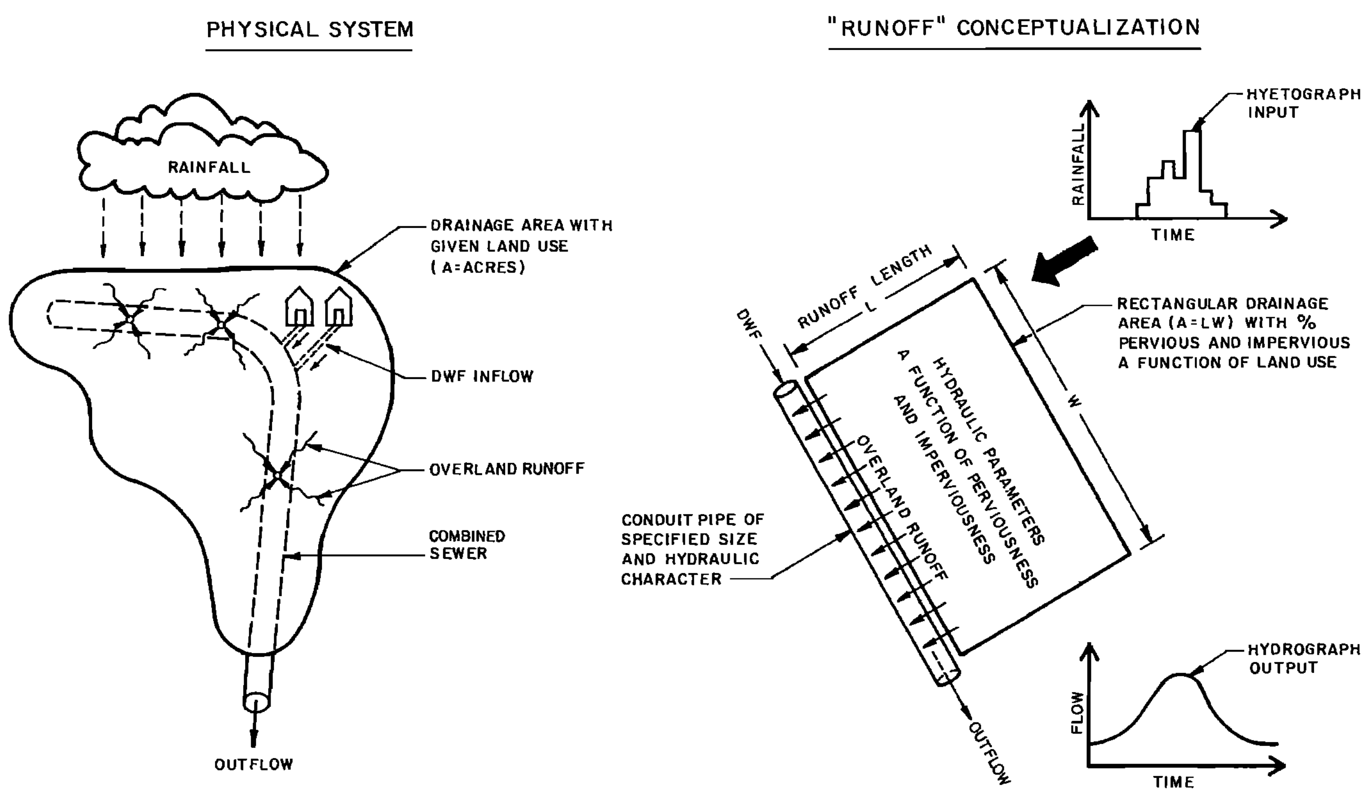
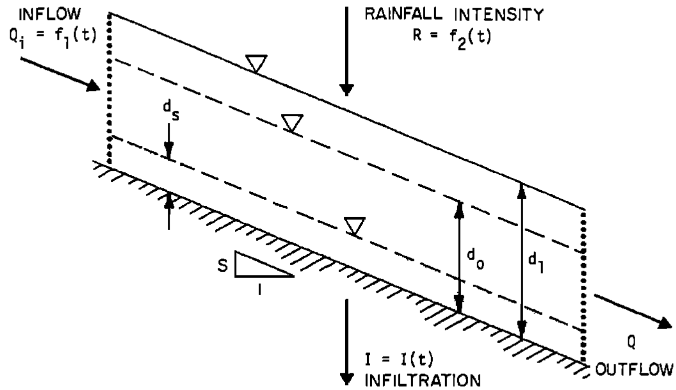


Fig. 7. The runoff model's mathematical conceptualization of a physical drainage area.



INFILTRATION: $I = k_1 + (k_2 - k_1)e^{-k_3 t}$

FLOW: $Q = \frac{1.49}{n} S^{1/2} \left(\frac{d_o + d_1}{2} - d_s\right)^{5/3}$

STORAGE: $\frac{\Delta d}{\Delta t} = \left(R - I + \frac{Q_i - Q}{L}\right)$

Fig. 8. Basic flow calculations for typical watershed subarea.

sequentially to the impervious (with detention), impervious (without detention), and pervious planes, in that order.

The three-plane runoff computation sequence can be generalized for the pervious flow plane shown in profile in Figure 8. At the end of each time step, Δt, we have two unknowns, Q and d_1, and two equations, as indicated in Figure 8. Three flow depths are shown in the figure:

d_o depth at time t;

d_1 depth at time $t+\Delta t$;

d_s maximum depth of detention storage.

The objective of the calculations which pertain to this element is to find the new depth d_1, determining in the process the outflow, Q, and maintaining mass continuity at all times. To accomplish this, two equations must be solved simultaneously. The first is the continuity or storage equation:

$$\frac{\Delta d}{\Delta t} = R - I - \frac{Q}{A_s} \tag{1}$$

where

Δd = $d_1 - d_o$;

R rainfall during Δt;

I infiltration to groundwater during Δt;

Q outflow from subarea during Δt;

A_s surface area of the plane.

The second is the Manning equation for overland flow with the hydraulic radius set equal to average depth (wide channel assumption):

$$Q = \frac{1.49}{n} s^{1/2} w \left[\left(\frac{d_0 + d_1}{2}\right) - d_s\right]^{5/3} \tag{2}$$

where

s slope of ground surface;

n Manning coefficient;

w width of the plane.

Here we have two equations in two unknowns, Q and d_1. Note that the flow computation is based on the average depth during Δt and that surface detention is not included in the effective depth of flow. Rainfall intensity is an input quantity, variable in time but considered constant during each time interval Δt. Infiltration is computed by Horton's (see equation (3), Chapter 3) formula written as

$$I = f_c + (f_0 - f_c) e^{-kt} \tag{3}$$

where

I infiltration loss rate, in./h;

f_c, f_o minimum and maximum infiltration rates, respectively;

k exponential rate of loss in infiltration capacity;

t time, in hours.

The equations (1) and (2) are nonlinear, and their simultaneous solution is performed by a Newton-Raphson iterative technique for locating the zero crossing of the first derivative. First, the equations are combined and rearranged in the form

$$F = \Delta d - \Delta t(k\tilde{d}^{5/3} + R_{net}) \tag{4}$$

where F is Newton's function whose zero crossing is to be located:

$$k = -\left(\frac{1.49}{n} s^{1/2} w\right) \Big/ A_s$$

$$\tilde{d} = \frac{d_0 + d_1}{2} - d_s = d_0 - d_s + \frac{\Delta d}{2}$$

$$R_{net} = (R - I)$$

Then, differentiating yields

$$\frac{dF}{d(\Delta d)} = 1 - \Delta t \frac{5}{6} k\tilde{d}^{2/3} \tag{5}$$

The Newton-Raphson method is a recursive process for finding the value of Δd,

$$(\Delta d)_{n+1} = (\Delta d)_n - \frac{F_n}{(dF_n/d(\Delta d))} \tag{6}$$

where the subscripts refer to the nth and (n+1)th iterations. Repeated application of this expression converges upon F = 0.

The solutions for the impervious with detention and impervious without detention flow planes are similar, the only changes being that in the former case infiltration I is set to zero and in the latter case infiltration I and detention depth d_s are both set to zero. Of course, each of the three planes has its unique surface area A_s, the sum of the three areas being equal to the total area of the watershed subarea as input.

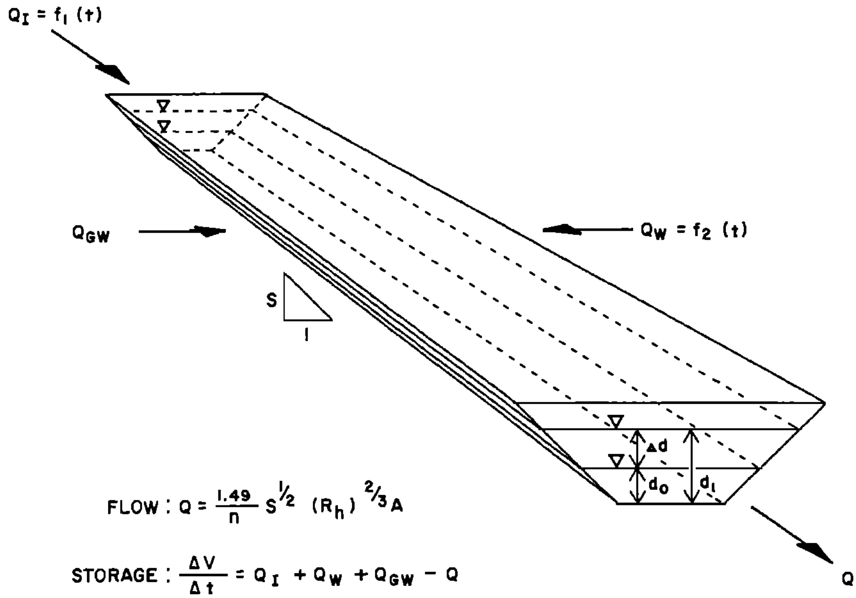


Fig. 9. Basic flow calculations for typical channel.

Channel Routing Quality

Runoff from the three overland flow planes of each subarea is aggregated into an inflow rate Q_W to the drainage channel or minor sewer that drains that subarea (see Figure 7). A typical storm drainage channel is shown in Figure 9. For each time step, outflow Q from the channel is determined.

As with watershed subareas, the two unknowns at the end of each time step are Q and d_1. The known quantities are inflows Q_I, Q_W, and Q_{GW} and depth d_o.

d_o depth at time t;
d_1 depth at time t+Δt;
Q_I inflow from upstream channel(s);
Q_W inflow from adjacent watershed subareas;
Q_{GW} groundwater inflow;
Q outflow from channel.

Q_W is the sum of the outflows from the three planes in adjacent subareas as discussed above. Q_{GW} for each channel is constant with time and is computed by establishing a water balance on the system based on input values of baseflows in all channels.

The solution for d_1 and Q is similar to that used to compute flow off watershed subareas. As in the overland flow representation, the kinematic wave approximation is made, and the equations of continuity and uniform flow are solved simultaneously at each time step. The continuity equation is

$$\frac{\Delta V}{\Delta t} = Q_I + Q_W + Q_{GW} - Q \tag{7}$$

where ΔV is the volume change associated with Δd. The outflow Q is determined from Manning's equation:

$$Q^* = \frac{1.49}{n} s^{1/2} R_H^{2/3} A \tag{8}$$

where

s slope of channel bottom;

n Manning's coefficient;

R_h hydraulic radius (=A/wetted perimeter);

A cross-sectional area of flow.

Q* is computed for both d_o and d_1, and the average taken as Q. The Newton-Raphson iterative technique is employed to solve equations (7) and (8). Newton's function is written

$$F = \Delta V + \Delta t\,(Q - Q_I - Q_W - Q_{GW}) \tag{9}$$

in which ΔV and Q are expressed in terms of d_o and d_1; $dF/d(\Delta d)$ is found and the recursive process for finding Δd,

$$(\Delta d)_{n+1} = (\Delta d)_n - \frac{F^n}{[d\,F_n/d\,(\Delta d)]} \tag{10}$$

where the subscripts refer to the nth and (n+1)th iterations, is employed to reduce the value of F, approaching 0. After a solution for d_1 and Q is reached, the procedure is repeated for the next channel downstream, Q becoming Q_I for that channel.

The Transport Subsystem

The specific function of the transport subsystem is to route surface runoff hydrographs and constituent pollutographs through the network of channels and/or pipes, junctions, and flow diversion structures of the main drainage system to the treatment plant interceptors and/or receiving water outfall points. It has been noted in the introduction to this chapter that the boundary between the runoff and transport subsystems is dependent on the objectives of the simulation. The computational procedure used in the transport subsystem must be used whenever it is important to represent significant backwater conditions, looped channel or sewer system sewer surcharge (pressure flow) and special flow devices such as weirs, orifices, pumps, storage basins, and tide gates. Normally, these conditions occur in the lower reaches of the drainage system where pipe diameters exceed roughly 36 in. (100 cm). The runoff model, on the other hand, is well suited for the simulation of overland and small pipe or channel flow in the upper regions of the system where the assumptions of uniform flow hold.

In the EPA SWMM program there are actually two computational procedures for representing the transport subsystem. One model is called TRANSPORT, the other EXTRAN, for extended transport. TRANSPORT is computationally faster than EXTRAN, but it cannot directly solve backwater, looped sewer, or surcharge problems, while EXTRAN can. Thus the computational procedure used in EXTRAN will be used to illustrate storm routing in the main drainage system.

EXTRAN [see Kibler et al., 1975] uses a link-node description of the sewer system which facilitates the discrete representation of the physical prototype and the mathematical solution of the gradually varied unsteady flow equations which form the mathematical basis of the model.

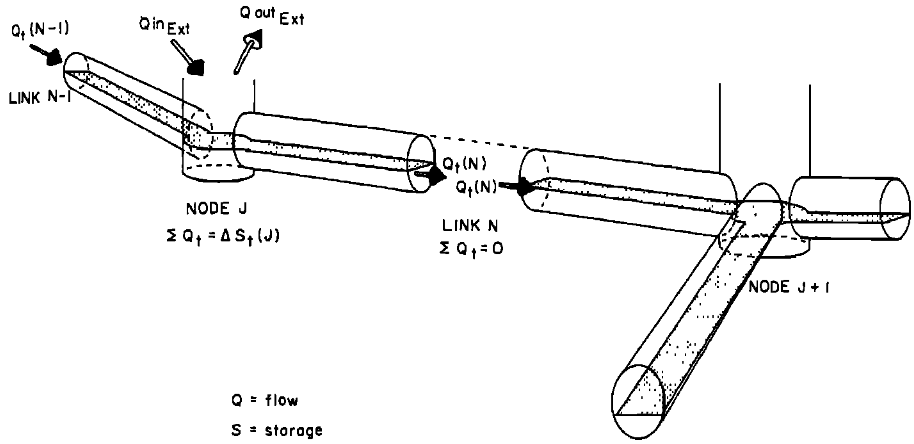


Fig. 10 Conceptual representation of the transport model.

As shown in Figure 10, the conduit system is idealized as a series of links (or pipes) which are connected at nodes (or junctions). Links and nodes have well-defined properties which, taken together, permit representation of the entire pipe network. Moreover, the link-node concept is very useful in representing flow control devices. The specific properties of links and nodes have been summarized in Table 1.

Links transmit flow from node to node. Properties associated with the links are roughness, length, cross-sectional area, hydraulic radius, and surface width. The last three properties are functions of the instantaneous depth of flow. The primary dependent variable in the links is the discharge Q. It is assumed that Q is constant in the link, while velocity and the cross-sectional area of flow, or depth, are variable in the link.

Nodes are the storage elements of the system and correspond to manholes or pipe junctions in the physical system. The variables associated with a node are volume, head and surface area. The primary dependent variable is the head H, which is assumed to be changing in time but constant throughout any one node. Inflows, such as inlet hydrographs, and outflows, such as weir diversions, take place at the nodes of the idealized sewer system. The volume of the node at any time is equivalent to the water volume in the

TABLE 1. Properties of Nodes and Links in the Transport Model

	Properties and Constraints
Nodes	
Constraint	ΣQ = change in storage
Properties computed at each time step	Volume Surface area Head
Constant properties	Invert, crown, and ground elevations
Links	
Constraint	$Q_{in} = Q_{out}$
Properties computed at each time step	Cross-sectional area Hydraulic radius Surface width Discharge Velocity of flow
Constant properties	Head loss coefficients Pipe shape, length, slope, roughness, invert, and crown elevations

half-pipe lengths connected to any one node. The change in nodal volume during a given time step Δt, forms the basis of head and discharge calculations as discussed below

Basic Flow Equations

The basic differential equations for the sewer flow problem come from the gradually varied, unsteady flow equations for open channels. The equation for unsteady spatially varied discharge can be written

$$\frac{\partial Q}{\partial t} = -gAS_f + 2V\frac{\partial A}{\partial t} + V^2\frac{\partial A}{\partial x} - gA\frac{\partial H}{\partial x} \qquad (11)$$

where

Q discharge through the conduit;

V velocity in the conduit;

A cross-sectional area of the flow;

H hydraulic head;

S_f friction slope.

The friction slope is defined by Manning's equation, i.e.,

$$S_f = \frac{k}{gAR^{4/3}} Q \mid V \mid \tag{12}$$

where k = $g(n/1.49)^2$. Use of the absolute value sign on the velocity terms makes S_f a directional quantity and ensures that the frictional force always opposes the flow. Substituting in (11) and expressing the finite difference form give

$$Q_{t+\Delta t} = Q_t - \frac{k}{R^{4/3}} \mid V \mid Q_{t+\Delta t} + 2V \frac{\Delta A}{\Delta t} \Delta t + V^2 \frac{A_2 - A_1}{L} - gA \frac{H_2 - H_1}{L} \Delta t \tag{13}$$

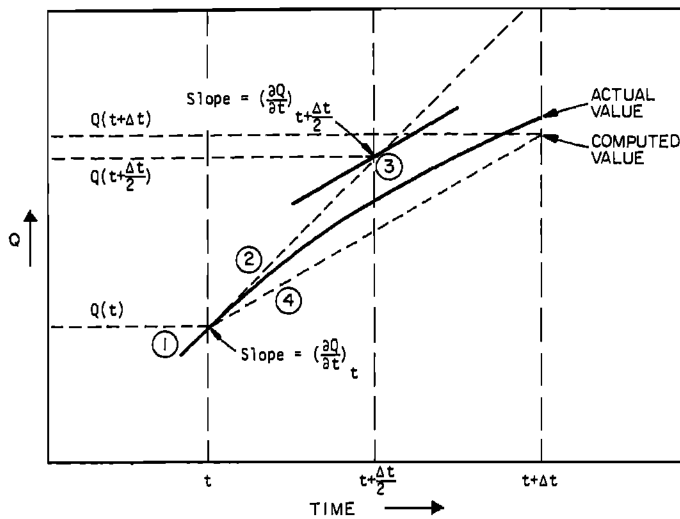
where the subscripts 1 and 2 refer to the properties at the upstream and downstream ends of the conduit, respectively, and L is the length of the conduit. Solving (13) for $Q_{t+\Delta t}$ gives the final finite difference form of the dynamic flow equation as

$$Q_{t+\Delta t} = \left[\frac{1}{1 + (k \cdot \Delta t/\bar{R}^{4/3}) \mid \bar{V} \mid}\right] \left[Q_t + 2\bar{V} \Delta A + \bar{V}^2 \frac{A_2 - A_1}{L} \Delta t - g\bar{A} \frac{H_2 - H_1}{L} \Delta t\right] \tag{14}$$

In (14) the values $\bar{V}$, $\bar{R}$, and $\bar{A}$ are weighted averages of the conduit end values at time t. In addition, head losses may be subtracted from H_2 and H_1 to account for exit and entrance losses.

The basic unknowns in (14) are $Q_{t+\Delta t}$, H_2, and H_1. The variables $\bar{V}$, $\bar{R}$, and $\bar{A}$ can all be related to Q and H. We therefore require another equation relating Q and H. This can be obtained by writing the continuity equation at a node:

$$(\partial H/\partial t)_t = \Sigma Q_t/A_{s_t} \tag{15}$$



1. Compute $(\frac{\partial Q}{\partial t})_t$ from properties of system at time t

2. Project $Q(t+\frac{\Delta t}{2})$ as $Q(t+\frac{\Delta t}{2}) = Q(t) + (\frac{\partial Q}{\partial t})_t \frac{\Delta t}{2}$

3. a. Compute system properties at $t+\frac{\Delta t}{2}$
   b. Form $(\frac{\partial Q}{\partial t})_{t+\frac{\Delta t}{2}}$ from properties of system at time $t+\frac{\Delta t}{2}$

4. Project $Q(t+\Delta t)$ as $Q(t+\Delta t) = Q(t) + (\frac{\partial Q}{\partial t})_{t+\frac{\Delta t}{2}} \Delta t$

Fig. 11 Modified Euler solution for discharge based on half-step, full-step projection.

or in finite difference form

$$H_{t+\Delta t} = H_t + \frac{\Sigma Q_t \Delta t}{A_{s_t}} \tag{16}$$

Equations (14) and (16) can now be solved sequentially to determine discharge in each link and head at each node over a time step Δt. The numerical integration of (14) and (16) is accomplished by a modified Euler method. The results are accurate and stable when certain constraints are met. Figure 11 shows how the process would work if only the discharge equation were involved. The first three operations determine the slope ∂Q/∂t at the 'half-step.' This is used in operation (4) to project the full-step value of discharge. In other words, it is assumed that the slope $\partial Q/\partial t$, at time $t + \Delta t/2$ is the mean slope during the interval. The interested reader can find details of the solution as documented by Kibler et al. [1975].

Head Computation During Surcharge and Flooding

A hydraulic situation which requires special treatment is the occurrence of surcharge (pressure flow) and flooding. Surcharge occurs when all pipes entering a node are full, so that the water surface at the node lies between the crown of the highest entering pipe and the ground surface.

Flooding is a special case of surcharge which takes place when the hydraulic grade line breaks the ground surface and water is lost from the sewer node to the overlying surface system.

During surcharge, the head calculation in (16) is no longer possible because the surface area of the surcharged node is zero. Thus, the continuity equation for node j at time t is

$$\Sigma Q\,(t) = 0 \tag{17}$$

where $\Sigma Q(t)$ is all inflows to and outflows from the node from surface runoff, conduits, diversion structures, pumps, and outfalls.

Since the flow and continuity are not solved simultaneously in TRANSPORT, the flows computed in the links connected to node j will not satisfy (17). However, computing $\partial Q/\partial H_j$ for each link connected to node j, a head adustment can be computed such that the continuity equation is satisfied. Rewriting (17) in terms of the adjusted head gives

$$\Sigma(Q(t) + \frac{\partial Q(t)}{\partial H_j}\,\Delta H_j(t)) = 0 \tag{18}$$

which can be solved for H_j as

$$\Delta H_j(t) = -\Sigma Q(t)/\Sigma \frac{\partial Q(t)}{\partial H_j} \tag{19}$$

Thus, during surcharge, the full-step head is computed as

$$H_j(t + \Delta t) = H_j\left(t + \frac{\Delta t}{2}\right) + k\,\Delta H_j\,(t) \quad (20) \tag{20}$$

where $\Delta H_j(t)$ is described by (19). The value of the constant k theoretically should be 1.0. However, it has been found [see Roesner et al., 1980] that (20) tends to overcorrect the head; therefore, a value of 0.5 is used for k, which gives much better results.

For the conduits connected to a node, $\partial Q/\partial H$ is computed as follows:

$$\frac{\partial Q(t)}{\partial H_j} = \frac{32.2}{1 - K(t)}\,\Delta t\left(\frac{A(t)}{L}\right) \tag{21}$$

where

$$K(t) = -\Delta t\,\frac{32.2\,n^2}{2.208\,R^{4/3}}\,|V(t)|$$

Δt time interval;

$A(t)$ flow cross-sectional area in the conduit;

L conduit length;

n Manning n;

R hydraulic radius for the full conduit;

$v(t)$ velocity in the conduit.

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6 QUALITY OF URBAN RUNOFF

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Background

It has been less than two decades that sanitary engineers have begun to realize the significance of urban runoff as a source of pollution in receiving waters. Historically (see Field and Struzeski, 1972), the earliest sewers were built for the collection and disposal of storm runoff. For convenience, these sewers discharged to the nearest watercourse. In later years, domestic and industrial wastewaters were discharged into these sewers, thereby converting them to the 'combined sewers.' As the significance of the pollutional effects of discharging raw sewage to the watercourses became recognized, the major cities embarked upon programs of 'interceptor' sewers to divert some multiple (generally, 1.5-5) of the 'average dry weather' flow to a central location for treatment prior to disposal.

Even with interceptors, however, stormwater overflows from these combined systems were still observed to discharge significant pollution loads to receiving waters. This fact caused water pollution control agencies to begin thinking that separation of storm runoff and sanitary wastewater was the answer to the pollution problem, and for several years (about the mid-1960's) there were many studies on alternative methods of separating sanitary wastewater and stormwater in existing combined systems. Sewer separation was found to be very expensive, however, as reported by the American Public Works Association (APWA) [1967], and so while a few cities undertook separation programs, most cities began to look for alternative methods of dealing with the problem. Perhaps the most significant result to come out of this push for separation is that combined sewer systems are no longer designed for new developments; separate systems are installed.

At the same time that the feasibility of sewer separation was being studied, the federal government was sponsoring research on the quality characteristics of urban runoff per se. Data compiled from these studies by Field and Struzeski [1972] are presented in Tables 1 and 2 and show the comparison between the characteristics of combined sewer overflow and urban stormwater. These data indicate that at the high range of values, urban runoff can be more polluted than combined sewer overflows. However, as a general rule, the pollution load resulting from overflows of a combined sewer system is larger than the load carried to the receiving waters by a separate storm drainage system. These findings were a significant factor in Congress's decision to fund Section 208 studies under Public Law 92-500 to study the problem of nonpoint source (i.e., stormwater runoff) pollution from urban areas on our receiving waters.

Pollution Potential of Stormwater

Nearly every receiving waterbody has a set of water quality standards specified for it. These standards have generally been set on the basis of the natural quality of the water plus the beneficial uses identified for it. Table 3 lists quality standards for three beneficial uses: drinking water supply, recreational use, and propagation of aquatic life. Comparison of these criteria show that the water standards vary significantly for different uses.

Some idea of the pollutional potential of stormwater runoff can be gained by examining Table 4, which shows measured concentrations of stormwater overflows in San Francisco, California. Note that the overflow qualities are for combined sewer overflows. Where applicable, water quality standards from Table 3 are also shown. It is evident from these data that a large pollution potential exists for untreated stormwater

TABLE 1. Characteristics of Combined Sewer Overflows

Characteristic	Range of Values
BOD_5 (mg/l)	30-600
TSS (mg/l)	20-1,700
TS (mg/l)	150-2,300
Volatile TS (mg/l)	15-820
pH	4.9-8.7
Settleable solids (ml/l)	2-1,550
Organic N (mg/l)	1.5-33.1
NH_3N (mg/l)	0.1-12.5
Soluble PO_4 (mg/l)	0.1-6.2
Total coliforms (number/100 ml)	20,000-90 x 10^6
Fecal coliforms (number/100 ml)	20,000-17 x 10^6
Fecal streptococci (number/100 ml)	20,000-2 x 10^6

Selected data.
From Field and Struzeski [1972].

TABLE 2. Characteristics of Urban Stormwater

Characteristic	Range of Values
BOD_5 (mg/l)	1-700
COD (mg/l)	5-3,100
TSS (mg/l)	2-11,300
TS (mg/l)	450-14,600
Volatile TS (mg/l)	12-1,600
Settleable solids (ml/l)	0.5-5,400
Organic N (mg/l)	0.1-16
NH_3N (mg/l)	0.1-2.5
Soluble PO_4 (mg/l)	0.1-10
Total PO_4 (mg/l)	0.1-125
Chlorides (mg/l)	2-25,000[a]
Oils (mg/l)	0-110
Phenols (mg/l)	0-0.2
Lead (mg/l)	0-1.9
Total coliforms (number/100 ml)	200-146 x 10^6
Fecal coliforms (number/100 ml)	55-112 x 10^6
Fecal streptococci (number/100 ml)	200-1.2 x 10^6

From Field and Struzeski [1972].
[a]With highway deicing.

TABLE 3a. Chemical Standards for Drinking Water

Quality Factor	Recommended Maximum Limits,[a] mg/l	Maximum Permissible Concentrations,[b] mg/l
ABS (detergent)	0.5	
Arsenic	0.01	0.05
Barium		1.0
Cadmium		0.01
Carbon chloroform extract (exotic organic chemicals)	0.2	
Chloride	250.	
Chromium		0.05
Copper	1.0	
Cyanide	0.01	0.02
Fluoride	1.7	2.2
Iron plus manganese	0.3	
Iron	0.3	
Lead		0.05
Manganese	0.05	
Nitrate	45.	
Phenols	0.001	
Selenium		0.01
Silver		0.05
Sulfate	250.	
Total dissolved solids (TDS)	500.	
Zinc	5.	

From U.S. Public Health Service [1961].

[a]Concentrations in water should not be in excess of these limits when more suitable supplies can be made available.

[b]Maximum permissible implies that which constitutes grounds for rejection of supply.

overflows with respect to such pollutants as suspended solids, COD, BOD, nitrogen, phosphorus; i.e., the standard pollutants. What is not shown, however, are the metals which are incorporated in the runoff and which pose a potential for toxicity effects to aquatic life in receiving waters. Table 5 shows stormwater quality data collected in Seattle, Washington. Here it can be seen that most of the metals concentrations approach the limit of the standards or exceed them. Iron and lead in particular both exceed the limit of the standards by nearly an order of magnitude.

TABLE 3b. Water Qualities for Recreational Use

Determination	Water Contact		Boating and Aesthetics	
	Noticeable Threshold	Limiting Threshold	Noticeable Threshold	Limiting Threshold
Coliforms, MPN per 100 ml	1000[a]	[b]		
Visible solids of sewage origin	None	None	None	None
ABS (detergent), mg/l	1[a]	2	1[a]	5
Suspended solids, mg/l	20[a]	100	20[a]	100
Floatable oil and grease, mg/l	0	5	0	10
Emulsified oil and grease, mg/l	10[a]	20	20[a]	50
Turbidity, silica scale units	10[a]	50	20[a]	[c]
Color, standard cobalt scale units	15[a]	100	15[a]	100
Threshold odor number	32[a]	256	32[a]	256
Range of pH	6.5-9.0	6.0-10.0	6.5-9.0	6.0-10.0
Temperature, maximum ^0C	30	50	30	50
Transparency, Secchi disk, ft	-	-	20[a]	[c]

From McGauhey [1968] and McKee and Wolf [1963].

[a]Value not to be exceeded in more than 20% of 20 consecutive samples nor in any 3 consecutive samples.

[b]No limiting concentration can be specified on the basis of epidemiological evidence, provided no fecal pollution is evident. (Note: Noticeable threshold represents the level at which people begin to notice and perhaps complain. Limiting threshold is the level at which recreational use of water is prohibited or seriously impaired.)

[c]No concentration likely to be found in surface waters would impede use.

Sources of Pollutants

Basically, pollutant loads are introduced into urban runoff from three sources: (1) the land surface itself, (2) catch basins, and (3) the sewers in combined systems.

TABLE 3c. Water Quality for Aquatic Life

Determination	Threshold concentration[a]	
	Freshwater	Saltwater
Total dissolved solids (TDS), mg/l	2000[b]	
Electrical conductivity, μmhos/cm 25°C	3000[b]	
Temperature, maximum °C	34	34
Maximum for salmonoid fish	23	23
Range of pH	6.5–8.5	6.5–9.0
Dissolved oxygen (DO), minimum mg/l	5.04[c]	5.04[c]
Floatable oil and grease, mg/l	0	0
Emulsified oil and grease, mg/l	10	10
Detergent, ABS, mg/l	2.0	2.0
Amonia (free), mg/l	0.5[b]	
Arsenic, mg/l	1.0[b]	1.0[b]
Barium, mg/l	5.0[b]	
Cadmium, mg/l	0.01[b]	
Carbon dioxide (free), mg/l	1.0	
Chlorine (free), mg/l	0.02	
Chromium, hexavalent, mg/l	0.05[b]	0.05[b]
Copper, mg/l	0.02[b]	0.02[b]
Cyanide, mg/l	0.02[b]	0.02[b]
Fluoride, mg/l	1.5[b]	1.5[b]
Lead, mg/l	0.1[b]	0.1[b]
Mercury, mg/l	0.01	0.01
Nickel, mg/l	0.05[b]	
Phenolic compounds, as phenol, mg/l	1.0	
Silver, mg/l	0.01	0.01
Sulfide, dissolved, mg/l	0.5[b]	0.5[b]
Zinc, mg/l	0.1	

From McGauhey [1968] and McKee and Wolf [1963].

[a]Threshold concentration is the value that normally might not be deleterious to fish life. Waters that do not exceed these values should be suitable habitats for mixed fauna and flora.

[b]Values not to be exceeded more than 20% of any 20 consecutive samples, nor in any 3 consecutive samples. Other values should never be exceeded. Frequency of sampling should be specified.

[c]Dissolved oxygen concentrations should not fall below 5.0 mg/l more than 20% of the time and never below 2.0 mg/l.

Catch basins basins can be a source of first-flush or shock pollution. The APWA [1969] found in Chicago that

> ...the liquid remaining in a basin between runoff events tends to become septic and that the solids trapped in the basin take on the general characteristics of septic or

> anaerobic sludge. The liquid in catch basins is displaced by fresh runoff water in the ratio of one-half the volume for every equal volume of added liquid. During even minor rainfall or thaw this displacement factor can release the major amount of the retained liquid and some solids. The catch basin liquid was found to have a BOD content of 60 ppm in a residential area. For even minor storms, the BOD of the catch basin liquid would be seven-and-one-half (7 1/2) times that of the runoff which had been in contact with street litter. Improved design of catch basins, and better operational and maintenance practices, could reduce this first-flush pollutional effect.

In combined sewer systems, wastewater is incoporated into the storm runoff. In addition, the storm runoff, as it passes through large sewers, scours sediment deposited by wastewater flows during proceeding dry-weather periods. Figures 1 and 2 illustrate the effects of wastewater sewage and of catch basins and storm sewer scour on the quality of stormwater overflows as reported by Roesner et al. [1972].

The most important contributor of pollutants to urban runoff is the land surface itself, primarily the streets and gutters and other impervious areas directly connected to streets or storm sewers. Pollutants accumulate on these surfaces in a variety of ways. There are, for example, debris dropped or scattered by individuals, sidewalk sweepings; debris and pollutants deposited on or washed into streets from yards and other indigenous open areas; wastes and dirt from building and demolition; fecal dropping from dogs, birds, and other animals; remnants of household refuse dropped during collection or scattered by animals or wind; dirt, oil, tire, and exhaust residue contributed by automobiles; and fallout of air pollution particles. The list could go on and on.

There is still much to learn about the sources and magnitude of pollutants in urban runoff. Studies at the University of Florida by Huber et al. [1979] and the Nationwide Urban Runoff Program (NURP) being conducted by the U.S. Environmental Protection Agency (EPA) [1978] are directed at increasing our knowledge in this area.

Entry of Pollutants Into Urban Runoff

The first raindrops that fall on an urban watershed simply wet the land surface. As additional rain falls, the impervious surface will become wet enough that some of the water begins to form puddles, filling the depression storage. This initial rain begins to dissolve the pollutants in the gutters, streets, and on other impervious surfaces, and eventually, as this water actually begins to flow off the watershed, it carries the dissolved material in it.

As rainfall intensity increases, overland flow velocities become sufficient to pick up solids. Suspended solids are, of course, picked up at smaller velocities than settleable solids. The settleable solids are carried off the watershed in two ways. If the velocity is sufficiently high, the settleable solids may be suspended in the overland flow. At lower velocities, particles may simply be rolled along the bottom surface toward the stormwater inlet.

The rain that initially falls on pervious surfaces infiltrates into the ground. If the rainfall is sufficiently intense, the infiltration capacity may be exceeded and the excess rainfall begins to fill the depression storage on the pervious surfaces. Finally, if the rainfall is of sufficient intensity and duration, runoff will begin to flow off the pervious areas, onto the impervious areas, and thence into the stormwater inlets. Present experience, however, indicates that the amount of runoff, and hence the pollution loads contributed from pervious surfaces in urban areas, is smaller than that coming from the impervious areas. This is especially true of surfaces covered with vegetation such as lawns and gardens. Figure 3 illustrates the differences in simulated runoff and pollution load from a watershed that would occur if it were converted from a park (90% pervious) into a multiple residential area (20% pervious).

TABLE 4. Quality of Combined Sewer Overflows, San Francisco Wet Weather Monitoring Results, Laguna Street Outfall March 15, 1967

Sample Number Clock Time	1 2020	2 2030	4 2100	6 2130	8 2230	Standard from Tables 3a-3c
Flow, cfs	7.0	12.5	7.2	2.0	108	
Coliform MPN - Conf./ Fecal, 10^4/ml	70/13	70/2.3	2.3/2.3	6.2/0.46	70/0.6	0.01 (Tab.3a)
Conductivity, μmho/cm	338	220	178	219.8	76.8	3000. (Table 3b)
Alkalinity, mg/l as $CaCO_3$	82	43.4	35.5	51.3	16.8	
Suspended solids, mg/l	304	442	130	73	237	20. (Table 3b)
Volatile suspended solids, mg/l	234	264	86	56	119	
Grease, mg/l	63.4	18.7	16.2	18.3	17.9	0.2 (Table 3b)
COD, mg/l	458	425	198	251	165	
BOD, mg/l	252	169	104	108	41	
Floatable particulates, mg/l	2.3	1.5	1.0	0.4	1.1	0. (Table 3b,c)
Settleable solids at 30 min, ml/l	40	35	7	10	15	
Total Kjeldahl nitrogen, mg/l -N	19.95	15.75	8.05	10.85	3.85	
Amonia Nitrogen, mg/l N	10.50	4.20	3.50	4.55	2.45	
Total phosphate, mg/l PO_4	3.20	2.12	1.41	1.40	1.13	
Sulfate, mg/l	29	18	18	19	10	250. (Table 3a)
Chloride, mg/l	32	21	14	18	5.5	
Sodium, mg/l	29.5	18.5	14.5	16.0	6.0	
Potassium, mg/l	6.70	3.65	3.00	3.85	1.15	
Calcium, mg/l	11.2	10.4	8.4	8.8	4.8	
Magnesium, mg/l	4.9	2.9	3.2	3.2	2.4	

From Engineering Science, Inc. [1967].
Land use is multiple family residential.

TABLE 5. Urban Runoff Characteristics
Viewridge Two Area, Seattle, Washington

Parameter	Mean Concentration Feb. 14	March 10	March 16	June 6
Temperature, C^0	8.0	9.0	9.4	16.4
pH	7.2	7.5	7.3	6.7
Conductivity, µmho/cm	201	71	160	165
Turbidity, JTU	55	25	17.4	28.4
DO, mg/l	10.3	11.5	10.5	6.6
BOD, mg/l	9.3	16	5.8	39
COD, mg/l	48	78	45	229
Hexane ext., mg/l	17.6	32	25.2	16.5
Chloride, mg/l	22	2.0	4.8	17
Sulfate, mg/l	25	5	24	28
Organic N, mg/l	-	-	0.23	0.66
Ammonia N, mg/l	-	-	0.16	0.75
Nitrite N, mg/l	0.19	0.05	0.09	0.22
Nitrate N, mg/l	0.73	0.51	0.51	1.38
Hydrolyzable P, mg/l	0.20	0.33	0.14	0.44
Ortho P, mg/l	0.03	0.04	0.06	0.22
Copper, mg/l	0.034	0.034	0.031	0.046
Lead, mg/l	0.43	0.094	0.38	0.37
Iron, mg/l	3.4	0.44	3.46	1.02
Mercury, mg/l	-	-	0.00020	-
Chromium, mg/l	0.0050	0.0010	0.0010	0.010
Cadmium, mg/l	0.0044	-	0.057	0.0045
Zinc, mg/l	0.055	0.021	0.030	0.25
Setteable solids, mg/l	136	0.33	42	83
Suspended solids, mg/l	235	132	36	79
TDS, mg/l	154	98	151	199
Total coliform[d] org./100 mls	7,200	11,000	6,800	40,000
F. coliform[d] org./100 mls	490	2,000	480	360

From Farris et al. [1974]. Land use is single family residential.

[a]Chemical standards for drinking water.

[b]Water quality for recreational use.

[c]Water quality for aquatic life.

[d]Medians.

TABLE 5. Urban Runoff Characteristics Viewridge Two Area, Seattle, Washington (cont.)

Parameter	Mean Concentration Aug. 16	Sept. 19	Mean	Standard From Tables 3a-3c
Temperature C°	17.7	17.1	12.9	
pH	7.0	6.5	7.0	6.5–9.0[b]
Conductivity, μmho/cm	194	96	148	3,000.[c]
Turbidity, JTU	30.3	28.7	30.8	10.[b]
DO, mg/l	6.8	8.2	9.0	5.[c]
BOD, mg/l	100	12.4	30.4	
COD, mg/l	125	71	99	
Hexane ext., mg/l	12.3	7.4	18.5	0.2[a]
Chloride, mg/l	21	3	11.6	
Sulfate, mg/l	28	9	20	250.[a]
Organic N, mg/l	6.41	1.41	1.71	
Ammonia N, mg/l	0.27	0.23	0.35	
Nitrite N, mg/l	0.15	0.07	0.13	
Nitrate N, mg/l	0.83	0.49	0.74	
Hydrolyzable P, mg/l	0.81	0.25	0.36	
Ortho P, mg/l	0.20	0.12	0.11	
Copper, mg/l	0.001	0.21	0.059	0.02[c]
Lead, mg/l	-	0.51	0.36	0.05[a]
Iron, mg/l	-	1.62	1.99	0.3[a]
Mercury, mg/l	-	0.00014	0.00017	0.01[a]
Chromium, mg/l	0.0074	0.010	0.0072	0.05[a,c]
Cadmium, mg/l	0.0031	0.004	0.015	0.01[a,c]
Zinc, mg/l	-	0.24	0.12	0.1[c]
Setteable solids, mg/l	371	62	121	
Suspended solids, mg/l	390	80	160	20.[c]
TDS, mg/l	181	78	144	500.[a]
Total coliform[d] org./100 mls	42,000	620,000	26,000	1,000.[b]
F. coliform[d] org./100 mls	6,000	13,000	1,200	

From Farris et al. [1974]. Land use is single family residential.

[a]Chemical standards for drinking water.

[b]Water quality for recreational use.

[c]Water quality for aquatic life.

[d]Medians.

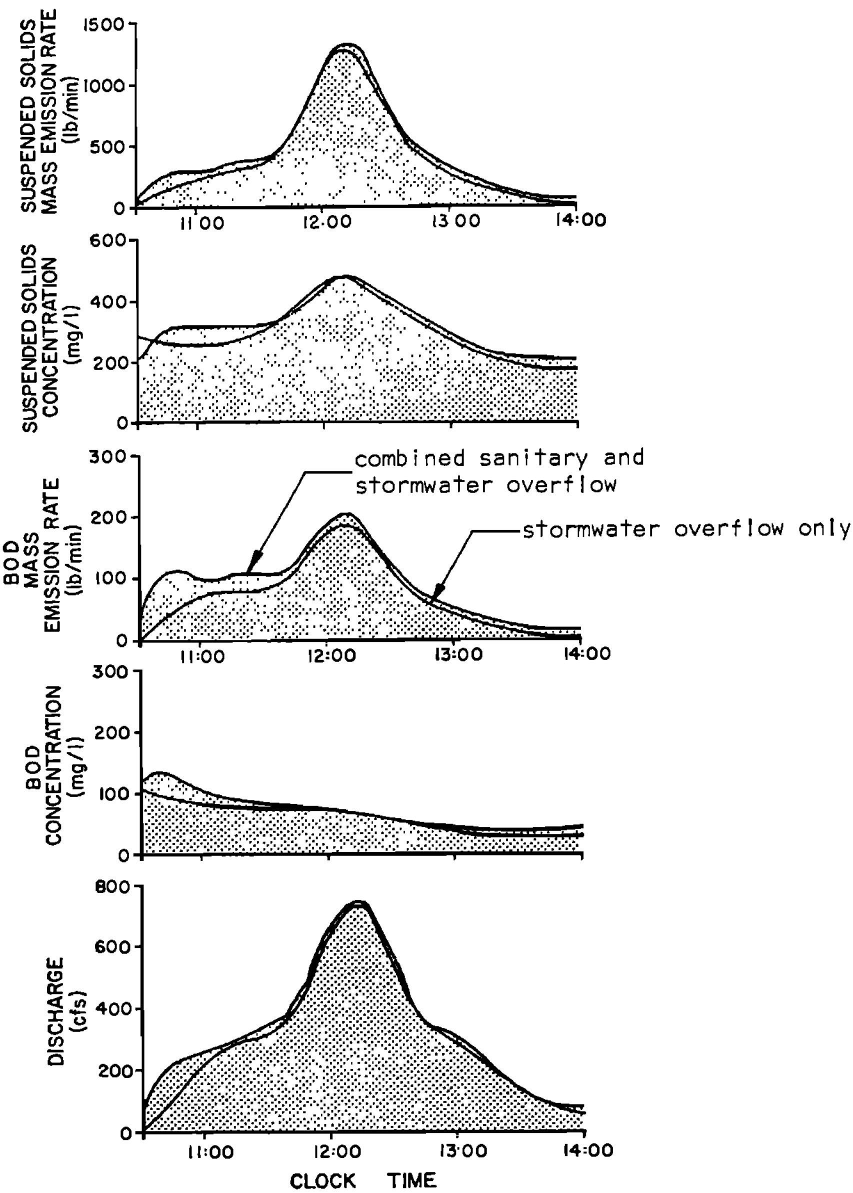

Fig. 1. Comparison of stormwater and combined overflows (Selby Street, San Francisco).

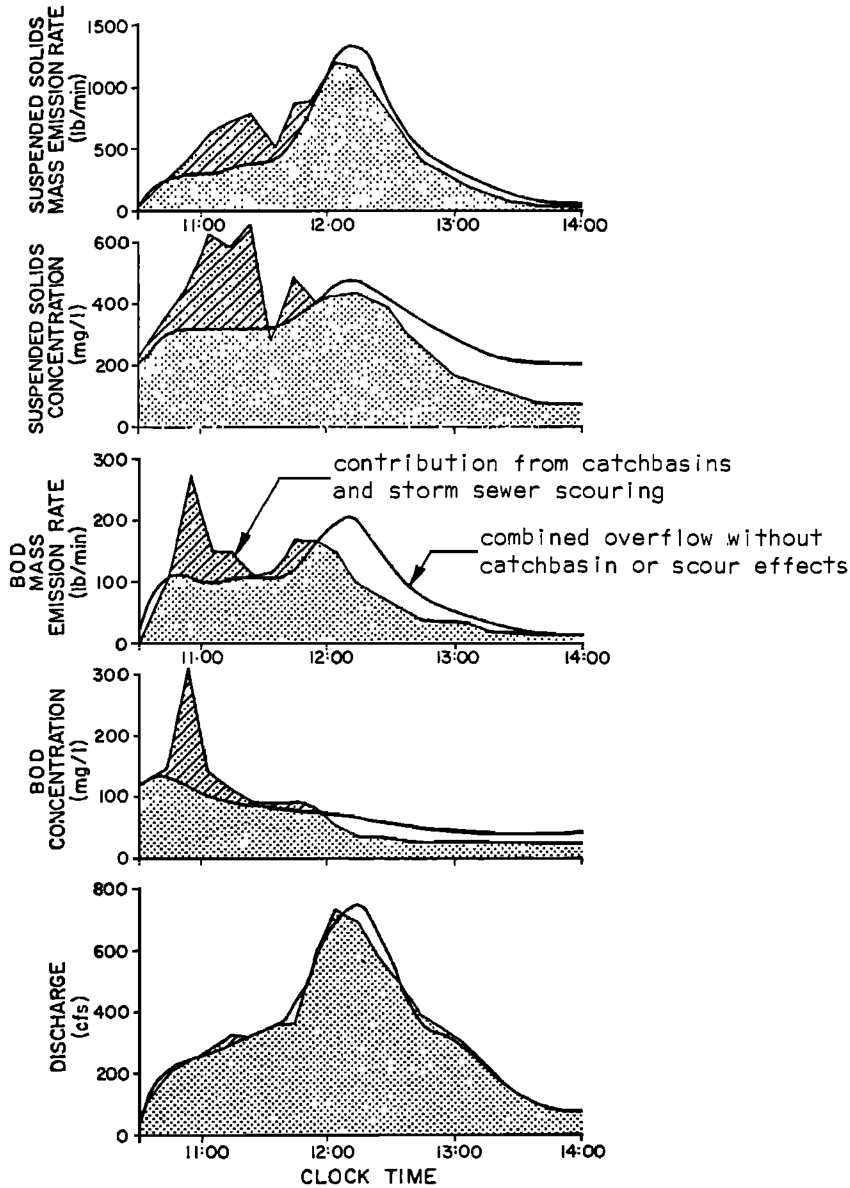

Fig. 2 First-flush pollutional effects of catch basins and sewer scour on combined sewer overflow (Selby Street, San Francisco).

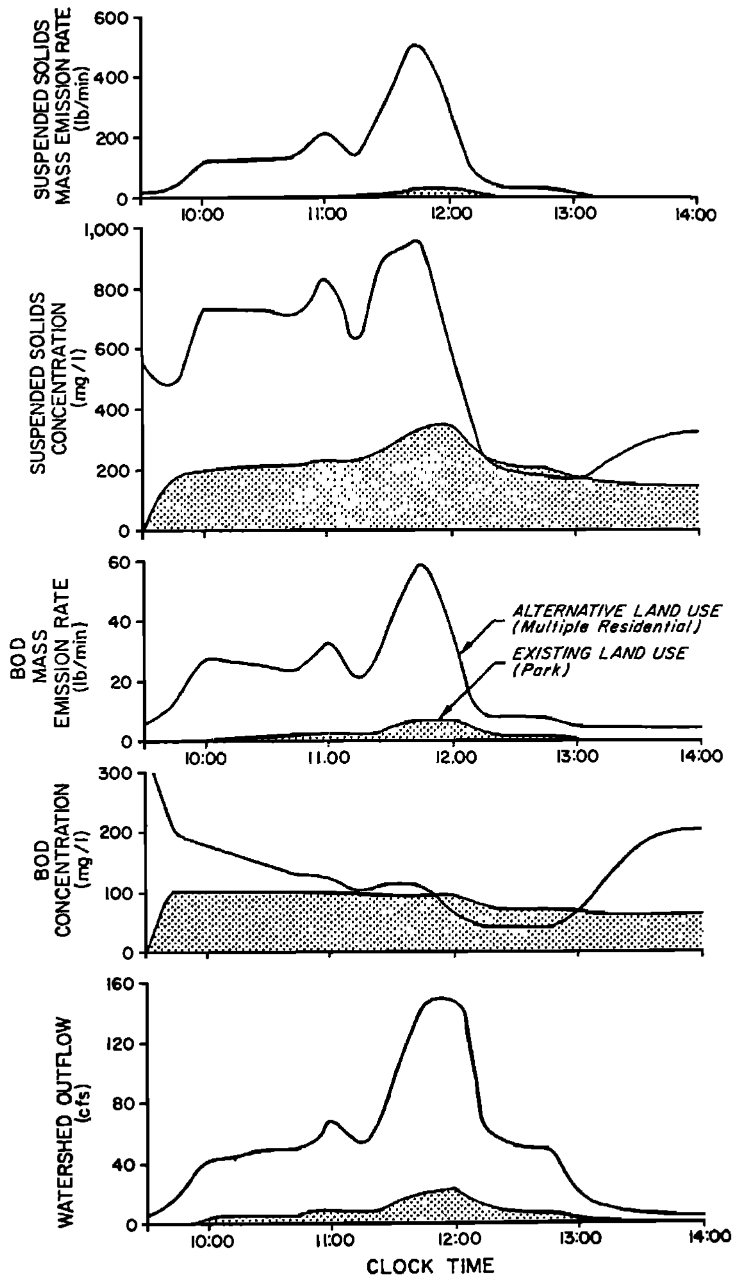

Fig. 3 Effect of changed land use on characteristics of subcatchment runoff, Selby Street [Roesner et al., 1972].

Estimation of the Rate of Pollutant Buildup on Urban Watersheds

It is fairly well accepted that pollutants build up on an urban watershed between rainstorms. The way in which the buildup occurs and the rate of buildup is, however, a much debated subject. The author's experience agrees best with the findings of Sartor and Boyd [1972] who reported that the rate of accumulation is most rapid during the first 2 or 3 days after a significant rainstorm. The rate of accumulation decreases subsequent to that time. This phenomena is presented graphically in Figure 4.

Table 6 shows urban runoff constituent loading rates for various cities in the United States. Those data in brackets have values reported in lb/acre/yr. The values shown in Table 6 should be considered as order of magnitude estimates of annual loads, and comparisons among the various study areas should be made at this level also. Estimates shown in the table were developed in some cases as the product of the mean concentration of urban runoff times the annual volume of runoff. In other cases, the mass washed off from several successive storms was extrapolated to an annual washoff.

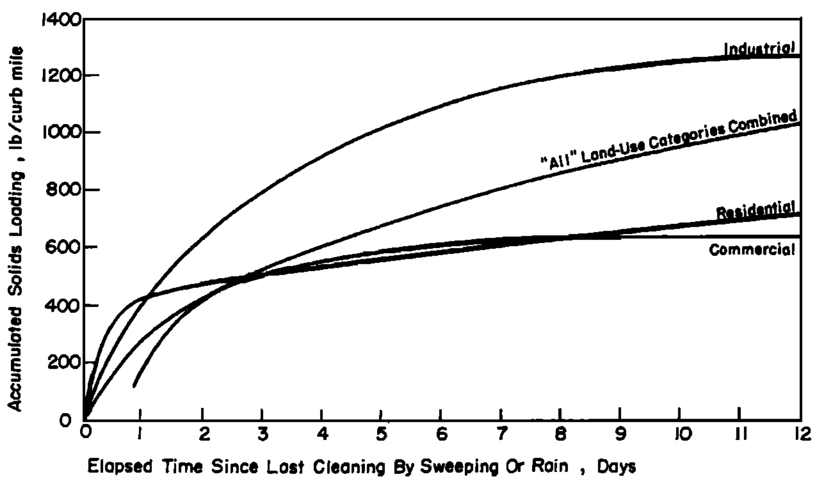

Fig. 4 Solids loading on urban streets versus time [Sartor and Boyd, 1972].

TABLE 6. Urban Runoff Constituent Loading Rates

Description	BOD_5	COD	Nitrogen	Phosphorus	Solids	Lead
Tulsa, Okla., 15 watersheds, 5586 ac, lb/ac/yr	[27.1]	200.7	[Kj 1.93]	[OPO_4 2.54]	T 1240	
lb/day/mi	2.66	18.39	Kj 0.195	OPO_4 0.28	T 127.8	
Eight cities, Mean (10-30% error), lb/curb-mi/day	3.1	21.84	NO_3 0.13 NH_4 0.41 Org 0.46	PO_4 0.46 OPO_4 0.2	T 156	0.28
Lincoln, Neb., 79 ac. Res. 30% imp. (7 samples avg)	3.7	11.4	NO_3 0.014 Org 0.18	PO_4 0.043	T 41.3 SS 25.7	
375 ac. Res. 25% imp. (4 samples avg), lb/ac/storm	0.16	1.25	NO_3 0.005 Org 0.058	PO_4 0.011	T 21.2 SS 23.6	
Durham, N.C., 1067 ac. 20% Paved, 59% Res. 19% LC, Ind, lb/ac/yr		938	[Kj 6.1]	[TP 4.7]	T 7700 [SS 6691]	[71]
Ten cities, 27600 curb-mi, lb/curb-mi/day	13.5	95 (32)	Kj 2.2 NO_3 0.94	PO_4 1.1	T 1400	0.57

Seven cities, street surface contamination, lb/curb-mi/day	4.5	26	Kj NO_3	0.66 0.029	PO_4	0.37	T	730	
Cincinnati, Ohio, lb/ac/yr	[38]	317	|TN	11.5|	[PO_4	3.4]	[SS	864]	
New York, 1210 ac. 60% imp., lb/ac/in. runoff	6.71	10.48	Kj NH_4 NO_3	0.73 0.19 0.09	PO4 OPO_4	0.145 0.105	SS	41.55	
Ann Arbor, Mich., 3800 ac., lb/ac/yr	[31]		[NH_4 Org NO_3	0.7 0.4 0.8]	[PO_4	2.8]	[SS	1010]	
Chicago, Ill., mg/gm of soluble dust & dirt	5	40	TN	0.48	PO_4	0.05			
Ten cities, city streets, lb/curb-mi	18	95	NO_3 TN	0.043 2.4	PO_4	1.1	T	1188.0	1.66
lb/1000 ft							T	14.64	0.18
Southeast Michigan Residential, lb/ac/yr	[101]		[TN NH_3 NO_2+ NO_3	12.8 2.3 5.1]	[TP OPO_4	5.6 .71]	[SS	2990]	[5.4]

From Davis et al. [1977].

To estimate the rate of pollutant buildup on an urban watershed, between rainfall events, we assume the buildup to behave in the manner shown in Figure 4. The figure indicates that the solids load on the street accumulates on the watershed between rainstorms. However, the accumulation tends toward some maximum value as the time between rainstorms increases. If we call this maximum or ultimate accumulation SSU, we can approximate the curves shown of Figure 4 with the equation:

$$SSO(L, t_d) = SSU(L)\frac{(t_d)}{t_2 + t_d} \tag{1}$$

where $SSO(L,t_d)$ is the suspended solids load, lb/acre, on land use L after t_d days since the last storm or street sweeping, and t_2 is the time for $SSO(L, t_d)$ to equal 1/2 of SSU(L). The two parameters that determine the shape of this curve are SSU and t_2.

Washoff of Pollutants

Let us define SSL(t) as the amount of total suspended solids (TSS) remain on a watershed after a period t of rainfall. Let us also assume that the rate of washoff at time t is proportional to the load on the watershed that is available for washoff, i.e.,

$$\frac{d[SSL(t)]}{dt} = -K\,SSL(t) \times AVAIL \tag{2}$$

where K is the proportionality factor, a function of rainfall intensity, and AVAIL is the availability factor, also a function of rainfall intensity.

The proportionality factor K is assumed to be directly proportional to runoff rate. AVAIL is assumed to increase from some small value at low runoff intensities to 1.0 at the runoff intensity level at which essentially all the remaining load is available for washoff at the set decay rate.

The most common expression for AVAIL is

$$AVAIL = a + bR^c \tag{3}$$

where a, b, and c are constants and R is the runoff rate in in./h. The value of the constants used by different investigators ranges widely, however. For example, the original EPA SWMM program used 0.057, 1.4, and 1.1 for a, b, and c, respectively [Metcalf & Eddy, Inc., et al., 1971], while comparisons with measured data near Detroit, Michigan [Roesner et al., 1977] suggest values of 0.03, 3.3, and 2 for a, b, and c. In the original formulation, AVAIL was not allowed to exceed 0.75; in the Detroit study, AVAIL was set equal to 1.0 if R exceeded 0.17 in./h.

The decay rate K has previously been somewhat arbitrarily set at 4.6, representing a removal of 90% of the TSS load in 1 hour at a runoff rate of 0.5 in./h. For the Detroit area, a value of K equal to 2.0 gave better results. This implies a removal of 63% in 1 hour at 0.5 in./h, or 86% in 5 hours at 0.2 in./h. The values of K and AVAIL should be considered to be somewhat site-specific at this time, and caution should be exercised in applying them generally without measured data for verification.

Figure 5 illustrates the relation between time t and runoff R, TSS load remaining on watershed P and mass rate of removal M. For simplicity, the availability factor is assumed to be 1.0 The m versus t plot, known as a pollutograph, is one of the most informative methods of expressing the pollutant load carried by urban runoff. To determine the concentration of a pollutant in the runoff as a function of time, one simply divides the pollutograph value M by the discharge.

Washoff From Undeveloped Land Areas

The preceding discussion of buildup and washoff rates applies only to developed urban land uses. To estimate the mass rate of removal of suspendable solids from the undeveloped land uses, a modified form of the universal soil loss equation from Wischmeier and Smith [1965] is popular, i.e.,

$$A = (R)*(K)*(L)*(C)*(P) \tag{4}$$

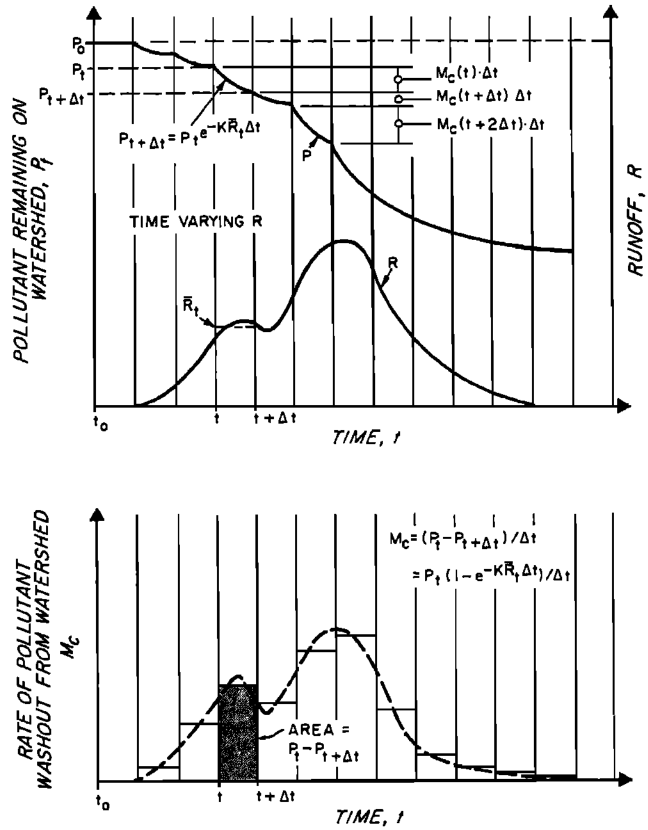

Fig. 5 Development of pollutograph (M_c versus t) from time history of P_t.

where

A soil loss per unit area, tons/acre/time step;

R rainfall factor;

K soil erodibility factor;

L slope length gradient ratio;

C cropping management factor;

P erosion control practice factor.

R, in turn, is given by

$$R = EI = \sum_i [(9.16 + 3.31 \log X_i)D_i]\, I \qquad (5)$$

where

E rainfall energy, hundreds of foot-tons/acre;

i rainfall hyetograph interval;

X_i rainfall intensity during time interval;

D_i inches of rainfall during the time interval;

I the maximum average 30-min intensity of rainfall.

The rainfall factor has been modified by some investigators who redefine D_i as the inches of runoff rather than rainfall. In addition, I is defined as the maximum 30-min intensity of rainfall up to the current time step. The value of L is given by

$$L = \lambda^{1/2} (0.0076 + 0.0053 S + 0.00076 S^2) \tag{6}$$

where λ is the length in feet from the point of origin of flow to the point at which sedimentation occurs or at which flow enters some defined channel and S is the average percent slope over the runoff length.

Washoff Rates for Pollutants Other Than Solids

It is often assumed for computational purposes that there is a relatively constant ratio between suspended solids load and other pollutants in stormwater runoff. This ratio R(I,L)--expressed as mg of constituent per gram of suspended solids--is assumed to vary by constituent I and by land use L. The concentrations of pollutants in the surface runoff from a particular watershed are thus computed as follows. For suspended solids, the contribution from each land use, both urban and rural, is summed, i.e.,

$$SS(t) = \sum_L SS(L, t) \tag{7}$$

where SS(t) is the concentration of suspended solids in the runoff from a particular watershed at time t and SS(L,t) is the concentration of suspended solids being contributed at time t by land use L in the watershed. The concentration C of a pollutant I at time t is expressed as the sum of the products of the suspended solids contribution from each land use in the watershed times the ratio of the constituent concentration to the suspended solids concentration. In other words,

$$C(I, t) = \sum_{L} [R(I, L) \times SS(L, t)] \tag{8}$$

The summation is taken over all land uses in the watershed, both developed and undeveloped.

The values of R (I, L) developed by Davis et al. [1977] for the Detroit Metropolitan area are shown in Table 7. Examination of the data on which these ratios were derived indicates that the assumption of a constant ratio between constituent concentrations and suspended solids concentrations in surface runoff works best for residential watersheds. The linear trend is also observed for most of the parameters in runoff from rural watersheds, but the scatter is greater than for residential areas. For commercial/industrial areas, Detroit data indicate that for the pollutants NO2+NO3, BOD, and oil and grease the use of a constant value for the concentration of these constituents in surface runoff would be a much better estimate than the use of the ratio technique. With the exception just noted, the Detroit data indicate that the ratio assumption for lead, iron, and total phosphorus is good to very good. It is fair to fairly good for the nitrogen series (TON, NH_3, and NO_2+NO_3), BOD, oil and grease, and fecal coliforms and very weak for dissolved orthophosphate. The assumption is totally invalid for chlorides and TDS whose concentrations were observed to bear an inverse relationshop to suspended solids.

Role of the Transport System

All of the preceding discussion has dealt with determination of the pollutant loads and quality of water washed off of the urban watersheds. Thus, the pollutographs shown in Figure 5 describe the rate of mass transport of a pollutant into the storm sewer system from a single watershed in an urban drainage area. The total drainage area will be composed of 20 to 100 watersheds (or subareas); thus for the total drainage area, there would be 20 to 100 such pollutographs formed. These individual pollutographs

TABLE 7. Ratio of Pollutant Concentrations to Suspended Solids Concentration

Constituent, I	Ratio R(I, L), Milligrams of Constituent per Gram of Suspended Solids			
	Residential	Commercial/ Industrial	Roads	Rural
BOD_5	34.	45.	10.	18.
Fecal coliforms[a]	87,000.	37,000.	200,000.	300,000.
Chlorides	0.	0.	0.	0.
Ammonia nitrogen	0.8	2.4	0.35	0.45
Nitrite + nitrate nitrogen	1.7	6.4	0.07	3.5
Total organic nitrogen	4.3	4.1	1.22	7.0
Total phosphorus	1.9	1.7	0.26	1.5
Dissolved orthophosphate	0.24	0.47	0.20	2.4
Oil and grease	25.	80.	100.	13.
Heavy metal (lead)	1.8	1.4	0.41	0.21
Chlorophyll a	0.	0.	0.	0.

Source of data: (1) Wet-weather sampling of October 6-7, 1976, for Smith Drain and Livonia Industrial Drain watersheds near Detroit provided the basis for residential and commercial industrial values; the Ridge Road small watershed provided the information used to generate the rural values [from Davis et al., 1977] and (2) values shown for roads are from Sartor and Boyd [1972].

[a](organisms/100 ml)/(gram/l TSS) or (organisms/gram TSS) x 0.1.

are then routed through the storm sewer or transport system using results from the hydraulic flow computation and the pollutant mass continuity equation to develop an outfall pollutograph at the lower end of the system. Depending upon the travel time in the transport system and the time to peak for the individual pollutographs, the resultant pollutograph at the outfall may have a high peak due to compounding of individual peaks from the tributary watersheds, or it may have a lower peak and broader base if the travel time in the sewer system is long compared to travel time on the individual watersheds.

The compounding effect is observed for stationary storms on steeper watersheds while the low-peak broad-base outfall pollutographs are observed in very flat systems. The compounding effect can also occur in a flat system, however, if the storm is moving down slope toward the outfall.

Environmental Assessment Considerations

Receiving waters serve multiple uses; thus procedures established for the planning of multiple purpose projects apply. We cannot simply chlorinate wastewater to maintain low coliform counts on a beach and, in turn, introduce a level of toxicity that will kill fish. Nor can we channelize the river for navigation or flood control without giving due consideration to the habitat of fish and fish-food organisms.

Figure 6 shows a conceptual diagram of an ecologic-water quality model. The interactions shown by the arrows are ecologic processes that transform chemicals such as carbon, nitrogen, and phosphorus between their abiotic state and the successions of organic biomass. It is apparent from this figure that if the water quality is changed as the result of a wastewater or stormwater discharge, there will be a resultant shift in the ecologic balance of the system. The severity and duration of the shift can be directly related to the severity and duration of the discharge.

The bases for environmental assessment are computations and value judgments. Computations are performed according to established knowledge or theory. Value judgment is then applied to extend the assessment beyond the state of the art of current computational technology.

Computation analysis should be carried out to depict effects with time and spatial detail. Along the time axis, there are long-term and short-term effects of urban runoff. In the spatial scale, the effect may be detected elsewhere downstream. Thus, stormwater runoff may create a transient increase in suspended solids and bacterial counts. Bacteria may die off rapidly, but

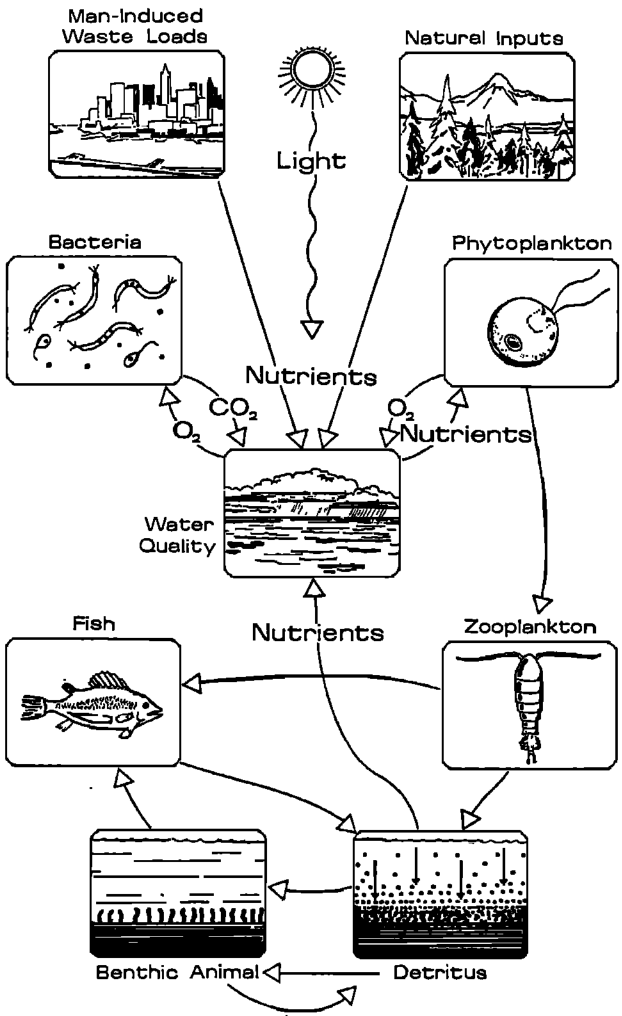

Fig. 6 Conceptual diagram of ecologic-water quality system.

suspended solids may settle and exert a long-term effect on the ecosystem.

The extent to which value judgment must be applied to the assessment of ecologic impact depends on the degree of sophistication used in the computational analysis. The receiving water model, RECEIV, documented in the EPA stormwater management model, simulates quality effects of BOD, dissolved oxygen, and suspended solids only. All other ecologic-water quality impacts must be inferred by value judgments. An expanded version of this model developed by Water Resources Engineers [1974] includes nine water quality parameters plus algae. Use of this model requires value judgments for evaluation of impact on higher trophic levels plus benthos.

Perhaps the most comprehensive computational tool presently available for ecologic-water quality assessment is the so-called ecologic model developed by Chen and Orlob [1972], which describes the interrelationship between some 23 water quality constituents and four biotic trophic levels. It too, however, requires the use of value judgments for interpretation of computed results and implication of these results on factors and processes that are not adequately accounted for in the models.

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7 DATA COLLECTION AND INSTRUMENTATION

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An important part of any urban stormwater investigation is a well-designed data collection effort including appropriate instrumentation. The data and instrumentation segment of an investigation is important for at least two reasons: (1) data collection costs are frequently high in relation to other project costs, and (2) the success of the investigation is highly related to a successful data collection effort. This chapter discusses various types of urban stormwater data collection efforts including data collection strategy, types of data needed, and typical instrumentation required for each type of study. The instrumentation mentioned here is in use by the U.S. Geological Survey (USGS) and is typical of a variety of available instrumentation. An important general reference for the information in this chapter is Alley [1977].

Data Collection Strategy

A data collection strategy which falls within the limits of economical, technical, and institutional constraints is an important planning prerequisite for an urban stormwater investigation. Consider the different types of urban stormwater investigations in Table 1, each of which requires a specific data collection strategy.

Each type of investigation requires a specific type of instrumentation and associated manpower and funding requirements. Each also requires the investigator to address the problems of catchment selection, gaging locations, and frequency and duration of data collection. A data collection strategy for each type of

TABLE 1. Types of Urban Stormwater Investigations

Water Quantity Emphasis	Water Quantity and Quality Emphasis
Storm drainage design or redesign Flood potential and profile delineation in urban streams Field research to define hydraulics of complex open channel and/or pressurized stormwater systems	Combined sewer design Real-time operation of storm drainage systems Water quality analysis of stormwater inputs to a receiving water System analysis of stormwater treatment alternatives Field research to define basic urban stormwater quality processes Field research to define benefits or management practices such as street sweeping, detention reservoirs and so forth

investigation is suggested later in this chapter. A few caveats concerning data collection strategy should be mentioned.

Caveat a. Because most investigations are performed by a research or consulting group for a planning organization, e.g., a municipal, county, state, or federal agency, the strategy should be well understood by both the performing and planning groups. In addition, an explicit statement of study objectives should be agreed upon and endorsed by both groups at the beginning of the study.

Caveat b. Because urban stormwater data include a significant component of natural event data, e.g., streamflow and rainfall, data collection strategy has to be somewhat adaptive and consistent with local hydrology. For example, it is not always practical to plan a priori to obtain urban stormwater data on x number of storms of given size in metropolitan area y during a z year (or month) study. The joint probability of this occurrence may be extremely small in many areas of the United States. The reality of natural hydrologic variability makes it necessary for the performing group to be in a state of preparedness as soon as the investigation is initiated to ensure that potentially valuable data are not lost due to lack of human effort or instrument malfunction.

Caveat c. During the data collection phase of the investigation it may be necessary to modify the data collection strategy for improved cost effectiveness as informational needs are met. For example, if a stormwater model is being used, calibration and verification should proceed as soon as data on the first few storms are available. This analysis may lead to a significant reduction in data collection costs. Thus, savings can be realized if modeling, or other kinds of data analysis, proceed interactively with the data collection effort. Another benefit of the interactive approach is the opportunity to identify unimportant varibles in the sampling program. Thus, if a particular chemical constituent is essentially constant based on early measurements, it may be possible to discontinue further measurements of that constituent.

Types of Data and Examples

This section describes the general types of data collected for the urban stormwater investigations listed in Table 1. The types of data are similar to other kinds of base hydrologic data. However, for urban studies, required data recording intervals are significantly smaller than for most other hydrologic studies and are generally less than 15 min and could be as small as 1 min. The short time scale of urban stormwater events also requires that synchronous recording be arranged between related data types such as rainfall and stormwater discharge.

Rainfall and Other Meteorological Data

Because rainfall is the basic driving variable of catchment response, its measurement and characterization is extremely important. Two kinds of rainfall data are important for urban stormwater studies: (1) at-site rainfall for calibration and verification of catchment response, and (2) long-term rainfall for use if long-term simulations are required. At-site rainfall records are generally short term, established for the duration of the study, and have preselected recording times. In some cases, existing gages can be used. Long-term rainfall data are available as a result of the excellent network of gages operated by the National Climatic Center (NCC). The NCC can supply hourly precipitation tapes for the nearest NCC or airport weather station and upon special request can mail available 5-min rainfall for use at a site.

The observed variability of rainfall in urban areas indicates a need for location of two or more rain gages on catchments that exceed a few hundred acres in size. Most stormwater models in use today are capable of accepting multiple rain gage inputs either as distinct input traces or as proportional or weighted single inputs. As a general rule, rain gages should be distributed over a given catchment so that equal areas and representative topography are sampled. Paramount in the location of new gages is the necessity of providing proper gage exposure. This can be a difficult problem on urban areas. Alley [1977] has several good suggestions for the proper location of rain gages. In general, rain gages should be located (1) near ground level rather than on top of buildings, (2) buildings and trees should be no closer to the gage than their height and preferably farther away and (3) sites on a significant ground slope or with the ground sloping sharply away from the gage should be avoided. Wind shields should be used where wind is expected to cause measurement errors.

Evaporation data are generally available from NCC on a daily basis and are of use in a few stormwater models which utilize moisture accounting procedures for infiltration computations during the inter-rain periods. It is generally unnecessary to install and operate an evaporation station for a stormwater investigation.

Maximum and minimum temperatures as well as several other meteorological parameters including snowfall, wind speed, and sunshine are available at some network NCC sites. If collected, the data would be available on the National Climatic Center's WBAN Summary of Data, Deck 345 for each day of the year at a given station. Many of these parameters are useful for snowmelt computations. Such data can be obtained from the National Oceanic and Atmospheric Administration, Environmental Data Information Services, National Climatic Center, Federal Building, Ashville, North Carolina.

Streamflow Data

A key variable in any urban stormwater investigation is stream discharge or, in some cases, the related variable stream stage. In any of the urban stormwater investigations listed in Table 1 the observation of stream discharge plays a major role. For example, in storm drainage design or flood potential mapping, a design discharge based on a series of actual discharge measurements is essential. In studies dealing with water quality parameters, accurate stream discharge is essential if accurate water quality loads and concentrations are to be computed.

In urban stormwater studies, stream discharge should be recorded at the same time interval as rainfall measurement if possible. For conventional USGS gaging stations, streamflow is available at hourly intervals for most gages and at 5-min intervals for some sites. At special USGS installations, 1-min data recording for stream discharge is available. Information on streamflow data at selected sites is available from the U.S. Geological Survey, Water Resources Division, Reston, Virginia.

The U.S. Geological Survey, Water Resources Division offices located in most states may be consulted for advice in locating new stream gaging stations. Several considerations are important in locating urban stream gaging stations. Among these are (1) favorable hydraulic conditions for either open-channel or pipe-flow gaging methods, (2) inclusion of desired land use conditions above the gaging point, (3) right-of-way acquistion, and (4) inclusion of representative water quality conditions above the gaging point when stream quality characterization is a study objective.

In some cases it may be advantageous to collect stream discharge at more than one site in a particular catchment in order to separate effects of different land use practices.

Chemical and Biological Data

In recent years, due to environmental emphasis, chemical and biological data are being collected as part of urban stormwater investigations. Such data collection greatly increases the cost of urban stormwater investigations due to the requirement of laboratory chemical analyses and expensive water quality sampling instrumentation. In addition, special care must be taken to ensure that samples (obtained by hand sampling or automatic sampling methods) are collected in a proper time relationship with stream discharge measurements. In general, this means that samples are well distributed over the discharge hydrograph with, perhaps, more samples taken on rising and peak segments of the hydrograph and fewer taken on the falling hydrograph. Figure 1 shows a reasonably well-sampled storm event for a small urban catchment near Miami, Florida. An automatic sampling device was used. The chemical constituent shown is total nitrogen and the calculated storm load, is 0.64 lb. This calculation was made using a load equation,

$$l_t = q_t c_t f \Delta t$$

where l_t is the incremental stormwater load for time period Δt; q_t is the stormwater discharge, in cfs, at Δt intervals; c_t is the total nitrogen concentration, in mg/l, interpolated at Δt intervals; f is a constant equal to 6.245×10^{-5} for 1-min data, in cfs and mg/l units; and Δt is time interval in seconds. Incremental values of stormwater load l_t are summed over the storm period to obtain the storm load.

A series of reports by the U.S. Geological Survey [Hardee,

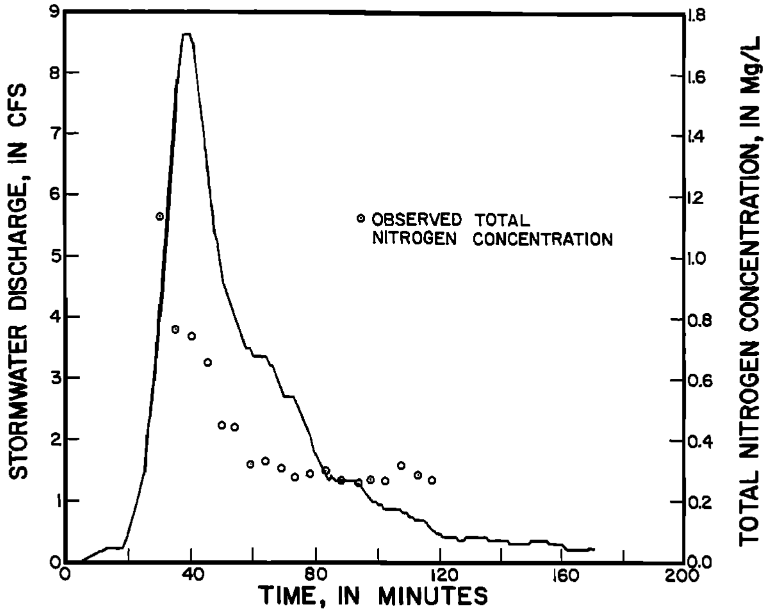

Fig. 1. Stormwater discharge and total nitrogen concentration for a storm on a 58.3-acre highway segment catchment in Broward County, Florida.

1979; Miller et al., 1979; Miller, 1979; Doyle, 1981] describing chemical and biological sampling and calculation methods for four small catchments near Miami, Florida, is available from U.S. Geological Survey, Water Resources Division, Tallahassee, Florida.

A core list of water quality constituents and associated USGS laboratory costs (1981) being analyzed in conjunction with an ongoing USGS-EPA National Urban Studies Program is given in Table 2. The constituents are listed in four categories: suspended sediment indicators, inorganic indicators, and bacteriological indicators.

Suspended sediment transport is of concern for several reasons including erosion on the catchment, sedimentation in the receiving water body, and aesthetics. Suspended sediment also serves as a transport mechanism for many chemical pollutants such as trace metals, nutrients, pesticides, and other organic compounds and oxygen-demanding substances. In some instances, chemical constituent concentrations may be related to suspended sed-

TABLE 2. Core List of Water Quality Constituents for the USGS-EPA National Urban Studies Program

Constituent and Laboratory Parameter Code	Laboratory Analysis Cost, 1981 dollars	Sample Preservation and Handling Recommendations
Sediment Indicators		
Suspended sediment (80154)	9.00	
Particle size analysis [a,b]	60.00	
Inorganic Indicators		
Specific conductance (00095)[a]	1.00	Less than 12 hours
pH (00400)[a]	1.00	Less than 12 hours
Dissolved solids (70300)	9.00	Filtered
Dissolved NO_2 + NO_3 as N (00631)	3.50	Filtered, chilled, less than 72 hours
Dissolved NH_3 as N (00608)	3.50	Filtered, chilled, less than 72 hours
Dissolved Kjeldahl as N (00623)	9.00	Filtered, chilled, less than 72 hours
Total Kjeldahl as N (00625)	9.00	Unfiltered, chilled, less than 72 hours
Dissolved phosphorus as P (00666)	10.00	Filtered, chilled, less than 72 hours
Total phosphorus as P (00665)	10.00	Unfiltered, chilled, less than 72 hours
Total lead (01051)	24.00	Acidified
Major cations and anions[b]	62.00	
Multielement scans for trace metals[d]	68.00	Acidified
Organic Indicators		
Dissolved organic carbon (DOC) (00681)	13.00	Chilled
Suspended organic carbon (SOC) (00689)	13.00	Chilled
Chemical Oxygen Demand (COD) (00340)	14.00	Acidified
Ultimate Biochemical Oxygen Demand (BOD)[a,b]		Chilled, less than 6 hours

5-day BOD (80082)[a,c]	Chilled, less than 6 hours
Priority pollutants such as pesticides, PCB's, oil and grease[b,e]	
Bacteriological Indicators	
Fecal coliform bacteria (31625)[a,c]	Chilled, less than 6 hours
Other Indicators	
Temperature (00010)[a]	Determined at site, once per storm

[a]Analyses performed on site or at a local laboratory. Other analyses at USGS Central Laboratories in Denver, Colorado, or Atlanta, Georgia.

[b]For selected samples (i.e., one or two per storm).

[c]Multiple samples during a few selected storms at each site.

[d]A few samples from each site during first year.

[e]Selected samples sent to EPA contract laboratory for anlaysis of 129 priority pollutants.

iment concentration using regression relations. Alternative sediment indicators, such as volatile suspended solids, total volatile solids, settleable solids, and settling velocities, might be considered depending on the project objectives. In addition, particle size analysis and determination of constituent concentrations by particle size may be an important consideration.

Inorganic chemical constituents, including nutrients, trace metals, and road salts, may be transported in solution in association with suspended material. Trace metals of concern will probably vary from study to study depending on land use practices. The USGS-EPA study scans for about 25 trace metals; however, principal interest should focus on lead and cadmium, chromium, copper, zinc, iron, manganese, and perhaps arsenic. Specific conductance and pH, easy indicators to obtain, should be measured on all samples along with major ions.

Organic chemical indicators include oxygen-demanding substances and toxic substances such as pesticides and industrial organic compounds. Because of the influence of section 307 of the Clean Water Act and the Toxic Substances Control Act, consideration should be given in any study to dangerous toxic substances such as pesticides and PCB's. Additional analyses should include ultimate BOD and COD as well as dissolved organic carbon and suspended organic carbon.

Bacteriological indicators, such as fecal coliform bacteria (which indicate the possible presence of pathogens or disease-causing organisms), may be sampled using automatic samplers if extremely high values typical of urban runoff are found. However, if low values are found, significant contamination may have occurred in the sampling mechanism.

In addition to storm water sampling, water quality analyses are also performed on debris collected from street surface [Pitt, 1978] and on atmospheric wet and dry fall deposition samples.

Land Use Characteristics

Land use characteristics are an extremely important and often neglected data type used in urban stormwater investigations. In typical applications, land use characteristics are related statistically to catchment water quality loads and are used as parameters in hydraulic and/or distributed runoff quality models. Land use characteristics may include catchment physiographic information, climatological or hydrologic factors, and environmental practices depending on the type of study.

A list of frequently used physiographic characteristics compiled for the USGS-EPA National Urban Studies Program [USGS, 1980] is shown below. Depending on the scope of the study, a selection of these or similar land use characterisitcs should suffice for most studies. Land use characteristics should be updated during the course of the study in order to account for changes occurring on the catchment. Catchment physiography information can be obtained from maps describing land use, soils, topography, and storm drainage as well as from aerial photography. The list of 22 physiographic, land use, and water quality characteristics in the National Urban Studies Program is as follows:

1. Total drainage area, in square miles (exclude noncontributing areas).
2. Impervious area in percentage of drainage area.
3. Effective impervious area in percentage of drainage area. Include only impervious surfaces connected directly to a sewer pipe or principal conveyance.
4. Average basin slope, in feet per mile, determined from an average of terrain slopes at 50 or more equispaced points using best available topographic map.
5. Main conveyance slope, in feet per mile, measured at points 10 and 85% of the distance from the gaging station to the divide along the main conveyance channel.
6. Permeability of the A horizon of the soil profile, in inches per hour.
7. Available water capacity as an average of the A, B, and C soil horizons, in inches of water per inch of soil.
8. Soil water pH of the A horizon.

9. Hydrologic soil group (A, B, C, or D) according to U.S. Soil Conservation Service methodology. Use numeric codes, A = 1, B = 2, etc.

10. Population density in persons per square mile.

11. Street density, in lane miles per square mile (approximately 12-ft lanes).

12. Land use of the basins as a percentage of drainage area including:

    a. Rural and pasture
    b. Agricultural
    c. Low-density residential (1/2 to 2 acres per dwelling)
    d. Medium-density residential (3 to 8 dwellings per acre)
    e. High-density residential (9 or more dwellings per acre)
    f. Commercial
    g. Industrial
    h. Under construction (bare surface)
    i. Idle or vacant land
    j. Wetland
    k. Parkland

13. Detention storage, in acre-feet of storage.

14. Percent of watershed upstream from detention storage.

15. Percent of area drained by a storm sewer system.

16. Percent of streets with curb and gutter drainage.

17. Percent of streets with ditch and swale drainage.

18. Mean annual precipitation, in inches (long term).

19. Ten-year 1-hour rainfall intensity, in inches per hour (long term).

20. Mean annual loads of water quality constituents in runoff, in pounds per acre.

21. Mean annual loads of constituents in precipitation, in pounds per acre.

22. Mean annual loads of constituents in dry deposition, in pounds per acre.

In many studies, especially those having a water quality element, climatologic and hydrologic factors which affect stormwater runoff quantity or quality are useful. For example, observed water quality constituent accumulation and washoff processes can often be explained by use of combinations of hydrologic, climatologic, and physiographic characteristics as well as existing drainage patterns and environmental practices effective on the catchment. A list of 18 typical storm and dry weather characteristics being compiled for the USGS-EPA study mentioned above is as follows:

1. Total precipitation, average for the basin in inches.
2. Maximum 5-min rainfall rate in inches per hour.
3. Maximum 15-min rainfall rate in inches per hour.
4. Maximum 1-hour rainfall rate in inches per hour.
5. Number of dry hours prior to storm, counting backwards to storm event with precipitation greater than 0.2 inches.
6. Depth of precipitation accummulated during previous 24 hours, in inches.
7. Depth of precipitation accummulated during previous 72 hours, in inches.
8. Depth of precipitation accummulated during previous 168 hours, in inches.
9. Total runoff, in inches over the basin.
10. Peak discharge, in cubic feet per second.
11. Base flow prior to storm, in cubic feet per second.
12. Duration of storm runoff used to calculate load, in minutes.
13. Duration of precipitation, in minutes.
14. Time from beginning of precipitation to hydrograph peak, in minutes.
15. Time since last street cleaning, in days.
16. Storm-runoff loads of individual constituents, in pounds per acre.
17. Dry deposition load of individual constituents since previous storm in pounds per acre (interpolated from monthly dry deposition rate, based on number of dry days, i.e., from characteristic 5 above).

18. Precipitation load of individual constituents, in pounds per acre.

In studies where water quality is a key element, environmental practice data must be collected for each watershed to establish cause and effect relationships and management techniques. Because such data are difficult to collect for large areas, a spatial sampling procedure is frequently used. A generalized list of recommended environmental practice data is listed in Table 3. Specific types of studies may have unique environmental practices which impact water quantity and quality. These practices should be identified and documented in order to support both modeling and statistical techniques of analysis. If best management practices (BMP's) are to be tested, it is necessary to obtain cooperation of local agencies to (1) select most appropriate BMP's (2) implement and manage the selected BMP's, and (3) document the type, location, and frequency of each BMP.

Examples of Instrumentation in Urban Stormwater Investigations

It is beyond the scope of this monograph to compare or recommend urban stormwater instrumentation. Reports on instrumentation comparison are available [see Shelley and Kirkpatrick, 1975 a, b; Shelley 1977]. The types of instrumentation presented here are presently in use by the U.S. Geological Survey and are typical of a variety of available instrumentation. Two instrumentation systems are discussed: (1) a conventional urban (small catchment) gaging system and (2) the USGS urban hydrology monitoring system (UHMS).

Conventional Urban Hydrology Gaging System

Conventional stream gaging depends on collection of stage measurements and occasional current meter discharge measurements upstream of an open channel discharge control on a stream or river. Stage readings are converted to discharge estimates by use of a stage—discharge rating. Conventional USGS gaging

TABLE 3. Environmental Practices Inventory Recommended for the National Urban Studies Program

Numeric Data	Descriptive Data
Average frequency of street sweeping, days	Method or type of street sweeping equipment
Estimated annual fertilizer load applied to watershed, lb/acre of nitrogen	Grading and agricultural ordinances in effect
Estimated annual fertilizer load applied to watershed, lb/acre of phosphorus	Refuse collection practice
Average time interval between sewer flushing, days	Solid waste disposal areas in watershed
Average time interval between catch basin cleaning, days	Flood retarding features such as gravel filter strips
Estimated average daily vehicle traffic, vehicle miles per day	Leaf disposal practice in watershed
	Identify major sediment source(s)
	Street pavement and condition
	Deicing chemicals

Fig. 2. Conventional urban hydrology gaging system, Alazan Creek, San Antonio, Texas. Photo U.S. Geological Survey.

methods are described by Buchanan and Somers [1968, 1969].

More than 10 thousand conventional gaging systems are in operation by the U.S. Geological Survey. However, for urban gaging situations, where the range in stage is not excessive, specialized gaging systems have been developed. Figure 2 shows a typical conventional urban hydrology gaging system being used in a concrete-lined urban stream in San Antonio, Texas. This type of system is composed of a stage recording mechanism (float type), a recording rain gage, a cork-dust crest stage gage indicator, and two independent staff gages. The gaging system is equipped with two automatic digital recorders, one for stage and one for rainfall, with data recorded on punched tape at a 5-min time interval. Both recorders are programmed for one battery-operated crystal timer. The two recorders are housed in separate shelter covers, each mounted on a 2-inch-diameter steel pipe reservoir which serves as one leg of the tripod base. The third leg is used as the crest stage gage. In order to discourage vandalism, no permanent ladders or platforms are usually attached. Access to the recorders requires the use of a portable ladder.

Such a gaging system is flexible, not highly expensive, and may be used at any site where the range in stage is not excessive. If a larger float-well is desired, a 4-inch pvc pipe may be attached to the 2-inch steel pipe supporting the stage recorder. If high velocities are expected, the three 2-inch pipes may be arranged in a straight upstream-downstream line instead of the typical tripod arrangement. If rain gage exposure is a problem, the rainfall recorder may be located several hundred feet from the stage recorders. Conventional USGS urban gaging systems similar to that described above are in routine operation throughout the United States.

USGS Urban Hydrology Monitoring System (UHMS)

Two recent aspects of urban instrumentation are worth noting. First, as a result of increased interest in water quality aspects of urban stormwater, a variety of new and elaborate instrumentation is available. These instruments generally incorporate features of advanced electronic design including microprocessor technology. Second, the available opportunity for adequate conventional gaging controls in urban areas is limited making it necessary to gage in underground storm drains. Not suprisingly, these and other aspects of urban instrumentation have led to the concept of packaged instrumentation [Shelly, 1977].

Figure 3 shows a schematic of an urban stormwater instrumentation package in use by the U.S. Geological Survey. The gaging system, called the Urban Hydrology Monitoring System (UHMS) was specifically designed for flow gaging in underground storm sewers, using a flow constriction as a discharge control. However, some components of the UHMS also can be used with conventional urban flow gaging systems such as discussed above.

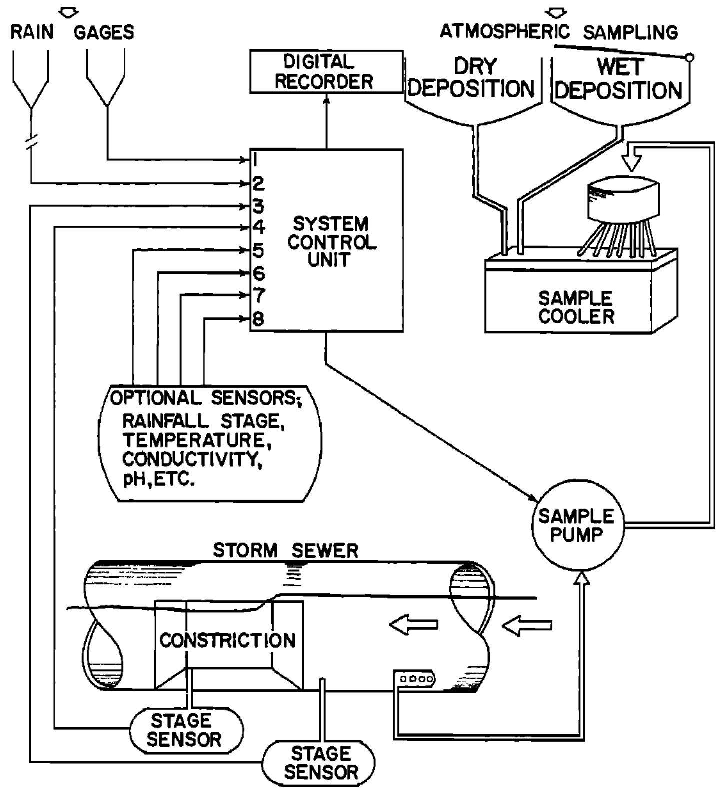

Fig. 3. Typical installation of the USGS Urban Hydrology Monitoring System.

Basically, the UHMS is designed to acquire storm rainfall and runoff quantity and quality data.

The UHMS is composed of five subsystems: The system control unit (SCU), the rain gage sampling subsystem, the atmospheric sampling subsystem, the stage (or flow) sensing subsystem and a water quality sampling subsystem.

System Control Unit (SCU)

The SCU is a microprocessor based unit which records data at a central site, controls an automatic water-sampling device, records one or more rain gages via low-grade telephone lines, and continuously monitors stage. Optional water quality parameters such as conductivity, turbidity, and temperature can also be monitored.

The SCU operates in a standby mode between storms. In standby, data are recorded only if rainfall occurs (except for a single daily recording of all parameters). A threshold value of stage corresponding to a selected discharge switches the system to the storm mode which activates continuous recording of data at a preselected time interval ranging from 30 seconds to 1 hour.

Water quality samples are taken at each recording interval or at multiples of recording intervals based on an algorithm programmable into the microprocessor circuitry at each site. Sampling options include sampling with stage, discharge, change-in-stage on rising and falling hydrograph segments, and/or time. Pre-event quantity and quality model simulations using a distributed routing rainfall-runoff quality model [Dawdy et al., 1978; Alley et al., 1980] or the judgment of an experienced hydrologist can be used to provide initial sampling settings. These settings are then refined using poststorm reassessment. A refrigeration unit is used to preserve up to 24 runoff samples.

The SCU causes data to be recorded on 16-channel punched paper tape. Each data record includes the following information: (1) Time in hours, minutes, and seconds, (2) Julian day, (3) stage and discharge parameter(s), (4) accumulated rainfall (one or more sites) and (5) sequential sample number if sample was taken. Multiples of parameters (items 3, 4, and 5 above) are limited to any combination not exceeding a total of eight parameters. To avoid unnecessary site inspections, the SCU can be interrogated with a telephone answering system. The answering system reports whether the equipment is in a recording, sampling, or off mode.

Rain Gage Sampling Subsystem

The rain gages typically used with the UHMS are the remote recording rain gage, P 501-I by Weather Corporation [1977]. (The brand names used are for identification purposes only and do not imply endorsement by the U.S. Geological Survey.) The gage uses an 8-inch-diameter orifice and a tipping bucket mechanism coupled to a mercury switch. The buckets are calibrated to tip after

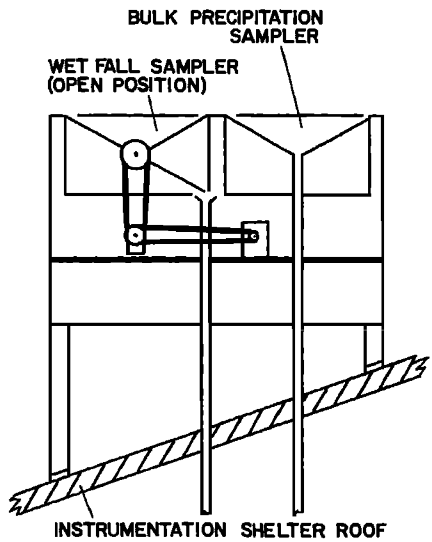

Fig. 4. Schematic of USGS roof-mounted atmospheric sampling subsystem.

each 0.01 inch of rainfall. Typically three or four accumulated rainfall records will be recorded on the UHMS.

Atmospheric Sampling Subsystem

Figure 4 illustrates the atmospheric sampling subsystem used in earlier studies by the USGS. The instrument, described in a report by Hardee [1979], collects rainfall and bulk precipitation samples. Dry fallout can be calculated from rainfall and bulk precipitation data by subtraction. Two rectangular teflon-coated collectors, one fixed in position to collect bulk precipitation, the other movable so as to collect only rainfall, were used for precipitation sampling. The movable collector is turned upright by a 12-Volt DC motor which is activated by the first tip of the rainfall gage. This places the collecting surface of the rainfall collector in the open (up) position. The collector remains open after 0.01 inch of rainfall occurs within a preset time period. If no additional rainfall occurs in this period, the motor rotates the collecting surface back to the closed (down) position. In this position no dry deposition is collected.

Control of the movable collector includes a sample mode switch set for single or multiple collections, and a second switch to set the length of time the collector is to stay open. With the sample mode switch set for single sample, the collector will open and close one time only and will not reopen until reset manually. In the multiple collection mode, the collector will reopen if rainfall begins again. Samples are preserved (see Figure 3) in the instrumentation house by refrigeration.

An advanced design atmospheric sampling subsystem, recently interfaced to the UHMS, is available from Aerochem Metrics [1979] Miami, Florida. The Aerochem Metrics model 301 is designed to collect rain and snow in a container which is open only during precipitation events; a second container is uncovered between precipitation events and collects only dry deposition material. Thus dry deposition and wet deposition are directly measured for both snow and rain using this instrument. More than 200 of these units are operating in field situations by various organizations.

Stage-Sensing Subsystems

Because conventional gaging controls are often not applicable in urban stormwater situations, specialized gaging methods have been developed. As shown in Figure 3, the UHMS has an optional underground storm sewer gaging control or constriction. Note that two manometer-type transducers are used to monitor differential water pressure representing stage. Dry nitrogen gas is bubbled through tubes to the two piezometer taps at a constant rate. One tap is located somewhat upstream, and the other is located within the constriction. The USGS constriction, which is U-shaped when viewed longitudinally, acts as a venturi meter at higher flow rates and as a critical depth meter at lower flow rates. A similar type constriction has been developed at the University of Illinois [Wenzel, 1975].

At less than full-pipe flow conditions, only the stage in the constriction is used for discharge computation. For full-pipe flow or pressure flow conditions, both stage sensors are used, and the constriction behaves as a modified venturi meter. Details of flow computation from stage readings are given in a report by Miller et al. [1979].

Based on laboratory experiments, the constriction method of gaging has an accuracy of approximately ±5% for steady flow tests under both open-channel and full-pipe flow conditions. However, the method has some problems. For example, the transition range rating, as flows pass to full or pressurized flow, is very sensitive, requiring the analyst to make a choice of possible ratings. In addition, the constriction method is unsuitable, or at least untested, for cases of variable submergence. Finally, laboratory ratings may be invalid for complex upstream pipe alignment and/or slope and for the rapidly varied unsteady effects associated with urban stormwater flow.

A promising new instrument, based on a velocity-sensing technique using the electromagnetic principle and called the VMFM flow meter has been developed by Marsh-McBirney, Inc. [1979], Gaithersburg, Maryland. The instrument, which can be used with the UHMS, offers advantages over the constriction method. For example, no flow constriction is involved, no empirical equations are necessary, the instrument installs easily in existing pipes, it is capable of monitoring reverse flow and surcharged conditions, and use does not depend on knowledge of pipe alignment, slope, or roughness. The VMFM monitors point flow velocity (which is then related to average velocity) and water depth (for full or partially full pipe conditions), using a bubble-type stage sensor. These two measurements are combined by internal electronic circuits to produce continuous discharge values. VMFM outputs (flow values) are recorded under control of the SCU.

Independent laboratory tests of the model 250 VMFM performed at the U.S. Geological Survey's Gulf Coast Hydroscience Center in 15- and 30-inch-diameter pipes through a range of flows and slopes indicate that the calibrated flow meter maintains an accuracy of ±10%. Because the velocity sensor is a point meter,

TABLE 4. Characteristics of Samplers

Sampler	Sample Size	Sampling Method
DH 48	1 pint	wading
DH 59	1 pint	rope
D 77	3 liter	cable and reel

the flow meter must be calibrated by an independent means such as standard current meter measurement or dye dilution gaging.

Water Quality Sampling Subsystem

Water quality samples taken by automatic pump sampling devices are increasingly being used in field investigations. Previously water quality samples were taken by hand or cable sampling methods using various techniques adapted from fluvial sediment sampling methodology [Guy and Norman, 1970]. These methods include the discrete 'grab' methods; the composite method, in which several samples are composited into a single sample based on time or flow-volume weighting; and stream depth or width integrated methods, in which special sampling procedures and equipment are used [Guy and Norman, 1970]. Table 4 lists various samplers available from the Federal Interagency Sedimentation Project, St. Anthony Falls Hydraulic Laboratory, Minneapolis, Minnesota.

The sample collection times of automatic devices should be synchronized with the recording of rainfall and discharge data. The minimum sampling interval of the automatic device should be programmed to equal the data-recording interval of other continuous hydrologic data if feasible. Where possible, manual depth-integrated samples should be taken occasionally to check the representativeness of point samples taken by the hand sampling method or by automatic sampler.

The particular sampling subsystem used in the UHMS is a modification of a Manning 4050 sequential sampler having 24, three-liter samples. The sampler sits atop an adapter plate on top of a commercially available freezer. The freezer is modified with an external thermostat to maintain sample temperatures of approximately 5°C.

Examples of Data Collection Strategy

Having introduced the types of data collected and typical instrumentation used for urban stormwater investigations, a possible data collection strategy for each of the nine typical urban stormwater investigations listed in Table 1 can now be discussed.

Storm Drainage Design and Analysis

Conventional urban gaging systems with associated rain gage network will suffice for most studies. In a few detailed studies, especially those involving pressured flow, use of the UHMS with a VMFM flow meter may be warranted. The number of gaging sites is dictated by the study approach. In some studies the design catchments are gaged; in others, representative catchments of comparable size to design catchments are gaged and the results summarized to produce general design procedures. In general, gaging is continued at least 2 years on catchments of about 1 mile^2 or less.

Flood Potential and Profile Delineation in Urban Streams

This kind of urban stormwater study requires two kinds of data that are obtainable from conventional urban gaging systems, i.e., flood hydrograph and maximum flood stage information. Most studies, depending on the size of area, utilize a base network of 10-40 complete flood hydrograph and rainfall stations and as many as 100 nonrecording crest stage gages. In general, a few long-term gaging stations also exist in a given urban area. The network of gages should be operated at least 5 years or to collect information on about 20 storm events on catchments from less than 1 mi^2 to as large as 50 mi^2 in size.

Combined Sewer Design

Combined sewer problems generally involve both water quantity and water quality aspects. In addition, a typical study also involves water quality aspects of receiving water. In most cases, UHMS-type instrumentation would be used with most studies being 2 to 4 years in duration. Depending on the scope of study, 2 to 20 UHMS-type sites on catchments from less than 1 to 20 mi^2 would be used. Traditional full range gaging stations equipped with USGS water quality monitors would be established at receiving water sites.

Real-Time Operations of Storm Drainage System

In several U.S. cities, automatic computer control systems for handling urban stormwater are being studied and, in some cases, implemented [Grigg et al., 1976]. For such studies, real-time interrogation of gaging systems of the conventional and UHMS type is the basis of part of the data collection strategy. The number and location of gaged sites is wholly dependent on the size and complexity of the area served. In general, the network is operated indefinitely but with frequent operational modification. Both water quantity and water quality considerations are involved.

Water Quality Analysis of Stormwater Inputs to a Receiving Water

This type of study requires instrumentation similar to that for combined sewer assessments. However, the focus on the study is on the receiving water; thus, major stormwater inputs would be gaged using UHMS-type instrumentation.

Research to Define Urban Runoff Pollutant Build-up and Washoff Mechanism

Because of uncertainties in the knowledge of pollutant accumulation and natural washoff processes in urban areas, planning for management of urban stormwater pollutants has been hindered. Questions concerning specific pollutants, their association with given urban land use types of mechanisms of pollutant accumulation and build-up rates, the mechanisms of natural and man-made removal, and mass transport phenomena through an urban catchment need to be answered definitively. Such questions are being addressed nationally and internationally [McPherson and Zuidema, 1977]. A national study program in the United States has recently been initiated by agreement between the U.S. Geological Survey and the U.S. Environmental Protection Agency [USGS, 1980]. Instrumentation systems will typically be of the UHMS type with a carefully planned network of 6 to 20 gaging points per metropolitan area located to sample land use characteristics on catchments 20 to 300 acres in size. Gaging should continue for at least 2 years and emphasis will be placed on identification of water quality processes and their relationship to parameters in physically based deterministic urban stormwater planning models.

Research to Define Storm Sewer Hydraulics

In many cities, older stormwater systems operate at full or pressurized flow much of the time or with flows moving in both directions in a given pipe or storm drain system depending on flow levels. Flow phenomena in such systems is not fully understood, but given adequate data, numerical network models using the dynamic equations of flow can be used for analysis. A system of UHMS-type gages at critical locations and junctions within the storm drain system can provide the data necessary to calibrate such models. The gages should be operated for several stormwater events and possibly indefinitely in order to define hydraulic processes required to understand and enhance design options.

Research to Define Benefits of Urban Stormwater Management Practices

Owing to the recognition of the magnitude and importance of urban stormwater quality, much attention has been directed to management practices such as street sweeping, litter control, use of detention reservoirs, and so forth. Because of a lack of data and controlled experiments the effectiveness of such practices is not known for many areas of the United States. A data collection strategy with UHMS-type instrumentation, along with street surface sampling, must be coordinated with typical management controls during the gaging period.

Analysis of Stormwater Treatment Alternatives

Many cities are considering the possibility of treatment of urban stormwater. Such a possibility requires accumulation of considerable urban stormwater and receiving water data in order to justify outlays of large expenditures associated with treatment alternatives. Data collection strategies utilizing UHMS-type instrumentation in a metropolitan network with gaging of small and moderate-sized catchments for at least a 2-year period is implied. Both flow and water quality records should be collected.

The above data collection strategies are meant to serve as examples only. Much planning and interaction must go into a data collection strategy before a workable approach can be developed. Care should be taken to avoid the caveats mentioned previously. Above all, a qualified data collection group well acquainted with local hydrologic processes should be selected during the early planning stages of the investigation.

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8 OVERVIEW OF URBAN STORMWATER MODELS

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Introduction

This chapter dwells on the fundamental issues in stormwater modeling and presents an overview of the more widely used among the currently available models. Modeling generally denotes the mathematical description of a physical phenomenon or process. Models of urban hydrologic processes were presented in Chapters 5 and 6 for the quantity and the quality of urban runoff. The analysis of hydrologic processes from a practical standpoint is useful to the extent that it permits the determination of adequate measures to mitigate the adverse effects of urbanization, which are mainly the increase in flood hazard and the pollution of receiving waters. Design measures originally revolved around networks of collectors or channel improvements which conveyed runoff from the site as rapidly as possible. They were soon expanded to include detention and retention devices such as natural or artificial ponds, groundwater recharge, and treatment facilities, to name but a few. These new techniques resulted in the development of complex systems which necessitated a large-scale basin-wide stormwater approach. The evolution of this approach has been described previously in Chapter 1. This chapter addresses the topic of large-scale urban stormwater models and presents the most important among the existing large-scale simulation models, so as to identify the tools that are readily available for case-by-case application, along with their characteristics, advantages, and limitations.

Role of Urban Stormwater Models and Levels of Analysis

Urban storm drainage models are designed to help solve practical problems and in fact, every real-life problem has its own peculiarities requiring its own modeling effort. Most storm drainage related problems can be solved with existing models, though sometimes imaginative interpretations are required. However, caution must be exercised to avoid the tendency to distort the physical world so that it fits the simulation capabilities of a given model. Instead, the imaginative interpretation referred to above should extend a model's capabilities to make it fit actual situations. Examples of such applications are given in Chapter 9.

A variety of problems exist pertaining to urban storm drainage. For example, we have come to realize that urban runoff and, particularly, combined sewer overflows are pollutant sources of significant magnitude to be considered in their impacts upon receiving waters along with other point sources, or what mix of sources should receive what degree of treatment. Is it better to treat several sources at one facility or rather consider separate facilities? What portion of the runoff should be conveyed away from the drainage basin and what portion should be diverted to permanent or temporary storage? Should one central storage facility be considered more cost-effective than many smaller facilities distributed over the basin? What is the appropriate type and size of drainage system that reduces the risk of street flooding and pollutant overflows to acceptable levels?

Clearly, all above questions are interrelated to some extent. Likewise, many of the existing models perform equivalent or similar functions. Therefore it is useful to classify storm drainage problems and models into three levels of analysis: namely, the planning level, the analysis/design level, and the operations/control level. At the planning level of analysis, one is concerned with future conditions, the effect of various land use patterns, and choosing among storm drainage alternatives (for instance, retention storage versus conveyance or detention storage versus treatment). At the design level the type of system is more or less already decided, and the concern is for the actual sizing of the facilities. This activity in general, will require more detailed models.

Finally, for combined sewer systems in large metropolitan areas, the concern is for improving the performance of the system by appropriate gate and pumping operations that result in advantageous routing through sewer mains, diversion to storage and treatment facilities, so as to minimize street flooding and pollutant spills to the receiving waters. This level of analysis requires the most detailed among the available models, often including real-time forecasting capabilities.

Planning Models

The planning level of a storm drainage study is typically concerned with conditions of future urbanization. The problem is to screen the major alternatives for effective stormwater management. Prominent among the planning models is STORM, developed for the U.S. Army, Corps of Engineers, Hydrologic Engineering Center [1976]. The program was originally developed to analyze runoff quantity and quality from urban basins as part of large-scale planning. It is intended to aid in the selection of storage and treatment facilities to control the quantity of stormwater runoff and land surface erosion. Conceptually, the runoff and pollutant washoff from an urbanized basin is collected and transported to a treatment facility, conveyed to temporary storage or discharged to receiving waters as depicted in Figure 1. The quantity of overflow depends on the amount of runoff and the capacity of the treatment and storage facilities according to the flow chart of Figure 2.

Nonurban areas can also be considered. The infiltration is estimated by a weighted runoff coefficient or by the SCS method. Snowmelt is included. A triangular unit hydrograph is used to

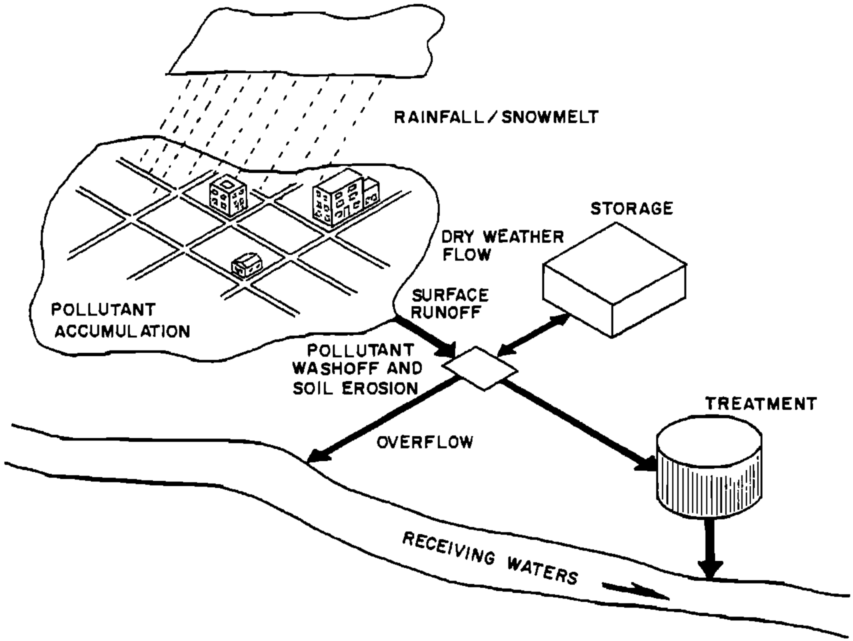

Fig. 1. Planning level basin modeling conceptualization (STORM).

produce the subbasin flows. The time step is fixed at 1 hour. Empirical equations are used to estimate the runoff quality. Land erosion for both urban and nonurban areas can be computed.

The program is a stormwater management model and does not design sewer systems. It is designed to perform continuous simulation by using years of continuous hourly rainfall data. However, selected individual storm events may also be analyzed.

The continuous simulation option eliminates the need to account explicitly for correct antecedent conditions and permits one to perform individual statistics on the simulated treatment, storage, and overflow events. The fixed time interval of 1 hour is compatible with the availability of rainfall data from the National Weather Service and with the level of accuracy required by the simulation. It also restricts model use to basins with time of concentration greater than 1 hour. The output provided by the STORM program includes (1) quantity analysis, (2) quality analysis, and (3) pollutograph analysis.

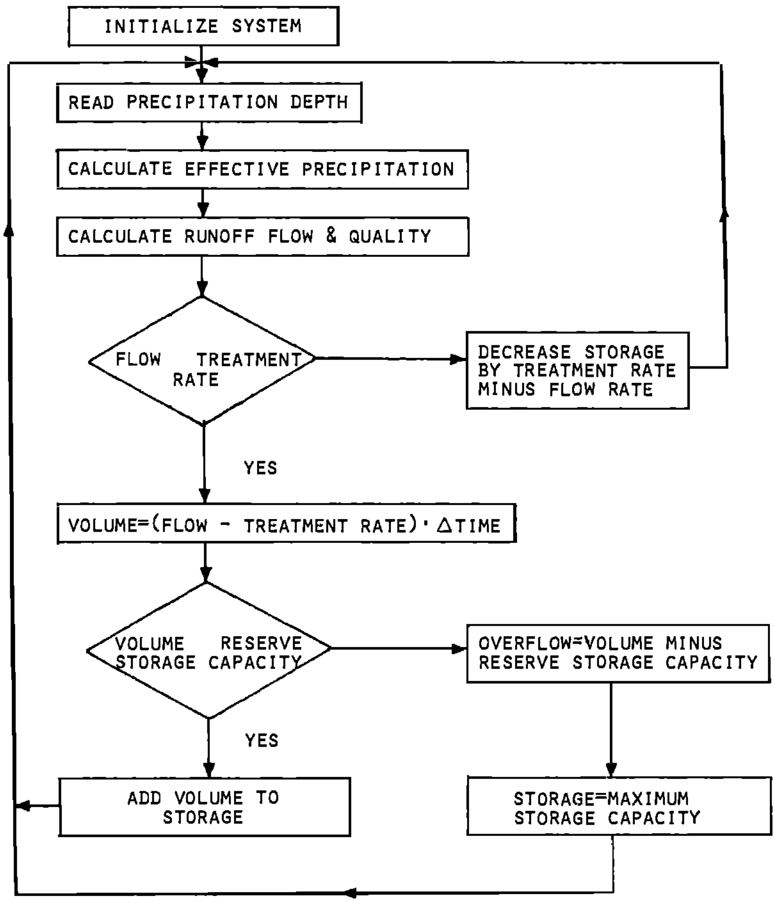

Fig. 2. Simplified flowchart of STORM.

The quantity analysis reports rainfall, rainfall duration, runoff, and information about overflows. The quality report includes rainfall, runoff, total pounds of pollutants in runoff, and pounds of pollutants in overflows. The pollutograph analysis gives the pollution loading (in lb/h) and pollutant concentrations (in mg/l) for individual events. The quality parameters include suspended and settleable solids, BOD, total nitrogen, and orthophosphate. The erosion analysis gives the amount of sediment washed from the watershed during individual events and shows average annual values.

The computer program is operational on the IBM 360/50, UNIVAC

7108, and CDC 6600 and 7600 computer systems. Program and user's manuals are available from the U.S. Army, Corps of Engineers, Hydrologic Engineering Center [1976, 1977] at Davis, California. A new receiving water model has been recently developed by Medina [1979], which takes STORM output as input and provides analysis of combined sewer overflow (CSO) impact. The level III receiving model is available from EPA's Center for Water Quality Modeling in Athens, Georgia.

Shubinski et al. [1977] developed a new version of the model STORM called SEMSTORM for a 208 study for the Southeast Michigan Council of Governments. Though retaining the important features of the original model, significant new features incorporated in SEMSTORM permit its proper classification as a new program. They stem from the recognition that an urbanized watershed considered as a study area is composed of many subbasins, which represent basic hydrologic units defined by surface topography, delineating land area lying between water course divides and sharing the same drainage system. Thus, the program is divided in three interdependent phases as schematically illustrated in Figure 3. Phase 1, the major component of the program, uses precipitation, land use, and hydrologic data to simulate continuous stream flow quantity and quality data during dry and wet weather periods for each subbasin under study. Phase 2 of the program either combines the individual subbasins directly or allows a time lag to be introduced. Finally, phase 3 is a statistical package that analyzes the various events generated by phase 1 or phase 2. SEMSTORM can be used for separate storm drain areas, combined sewer areas, and nonurban or unsewered areas. Major limitations inherent in the planning level of analysis are that the maximum size of the subbasin is limited to a time of concentration on the order of 1 or 2 hours, reliability of the program results is based on the analysis of a large number of events, all quality constituents being simulated are significantly correlated to total suspended solids, the effect of settling on the quality constituents being simulated is negligible and treatment and storage are completely mixed.

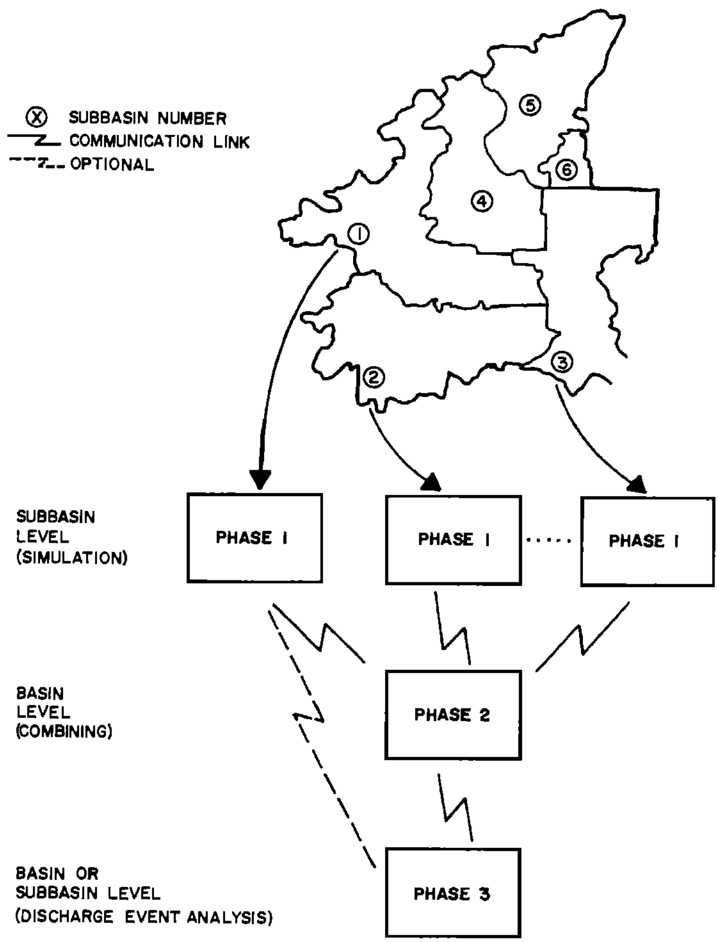

Fig. 3. Three phases of SEMSTORM.

A simplified stormwater management model was also developed for the EPA [Lager et al., 1976] and applied in the San Francisco area to estimate on an hourly basis, quantity and quality of runoff, dry weather flows, and other lateral inflows, pumped withdrawal from storage and overflows as they may occur.

An important limitation of the above models is the consideration of a single storage device per basin. Oftentimes a distribution of many storage devices within the basin may be advantageous. A model that emphasizes the timing of subarea flow contributions to peak rates at various points in a watershed is the Penn State runoff model. Thus, this model can be used to analyze the effectiveness of stormwater detention structures as a function of location within a watershed drainage system. The user's manual by Aron and Lakatos [1979] is available to the public. Two types of reservoirs can be analyzed. The first will accommodate the overland flows from the subwatershed in which it is located; the second, located at the subwatershed outlet, is intended for diversion and attenuation of the entire outflow of the watershed. The application of the model to the drainage system in Aberdeen, Maryland, has been described by Lakatos and Wiswall [1978]. A study of the parameter sensitivity of the model was performed by Kibler and Aron [1978]. Unlike the models STORM and SEMSTORM, the Penn State runoff model is a single event quantity model.

Design and Analysis Models

Many models exist that offer both planning and design capabilities. They highlight the fact that the planning-design-operations model classification has basically academic merits. An essential component of any design model is the capability to route runoff flows through a sewer network. The Illinois Urban Drainage Area Simulator (ILLUDAS) [Terstriep and Stall, 1974] offers such a capability. It is a single event model adapted from the British Road Research Laboratory (RRL) storm sewer design model which treats the paved and grassed areas separately, as depicted in the flowchart of Figure 4. The capability to determine design storms according to the Illinois State Water Survey (ISWS) method as a function of total rainfall depth and duration is also built into ILLUDAS. Infiltration in conjunction with the evaluation of initial abstractions is calculated by the modified Horton formula. The model is currently being expanded to include a water quality function and to accommodate a continuous rainfall input.

Rainfall-runoff and sewer routing processes are only part of the urban hydrology picture. Other processes are the diffusion, dispersion, and decay of various pollutant constituents in the

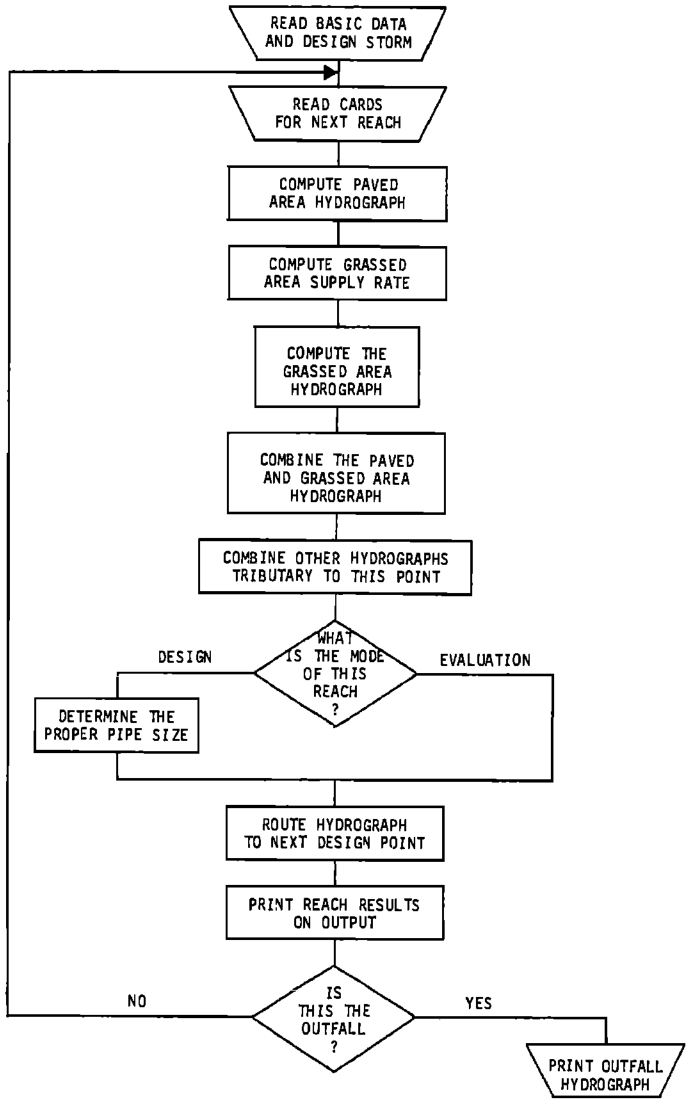

Fig. 4. Flowchart for ILLUDAS.

network of collectors, the effect of internal and external storage and treatment devices, and finally, the quality of the receiving waters. A model capable of simulating these processes is the EPA storm water management model (SWMM), developed for the U.S. Environmental Protection Agency by Metcalf & Eddy, Inc., the University of Florida, and Water Resources Engineers, Inc. [Huber et al., 1975]. The hydrologic model presented in Chapter 5, simulates single storm events on the basis of rainfall (hyetograph) inputs and system (catchment, conveyance, storage/treatment, and receiving water) characterization. The Horton equation is used for infiltration, and the kinematic wave method is used for routing the flows. The time step is variable. Empirical equations are used for runoff quality. These have been presented in detail in Chapter 6.

SWMM is a large FORTRAN program which models the complete urban rainfall/runoff cycle, including flow overland and in the sewage system, in-line and off-line storage, treatment (including costs) of stormwater flows, and which includes a receiving water module to assess water quality impacts. It requires a computer hardware system equivalent to the IBM 360/65. Program outputs consist of tables, hydrographs, and pollutographs, which can be displayed at points within the system as well as in the receiving waters. Water quality parameters include BOD, suspended solids, and coliform count. SWMM has six major computational blocks as shown in Figure 5.

1. The EXECUTIVE block provides control and interfacing of other five computational blocks.

2. The RUNOFF block computes stormwater runoff and its associated pollution loadings for a given storm and stores the results in the form of hydrographs and pollutographs at the inlets to the main sewer system.

3. The TRANSPORT block routes flows through the sewer system and calculates dry-weather flows (for combined sewer systems). Routing is based on a finite difference solution of the St. Venant equations.

4. The EXTRAN (extended Transport) block performs the same functions as TRANSPORT except that the routing scheme is based on a link-node representation of the sewer system and includes the complete continuity and momentum equations, allowing modeling of surcharged systems.

5. The STORAGE AND TREATMENT block simulates the operation of

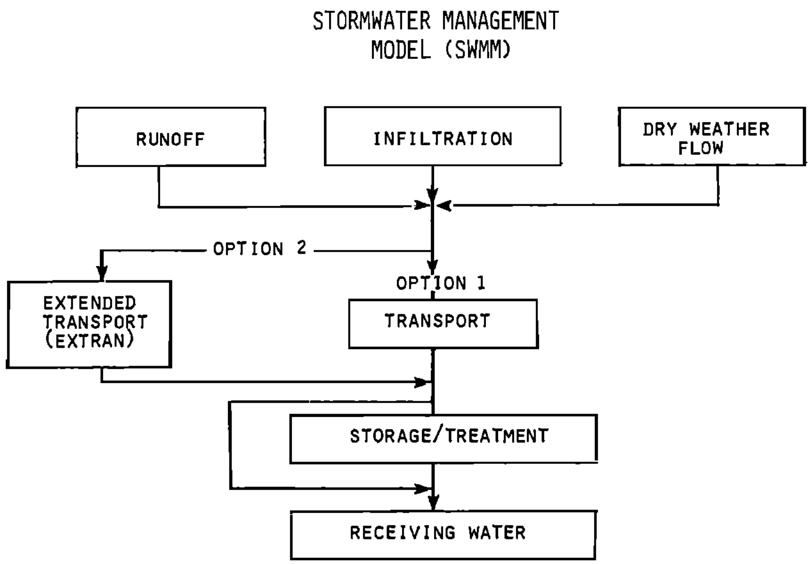

Fig. 5. EPA stormwater management model subroutines.

a treatment facility with one or more processes and calculates the capital and operating costs of processes chosen.

6. The RECEIVE block computes the impact of discharge upon the receiving water.

The SWMM model is well-documented, the computer program is available without charge [U.S. EPA, 1977], and EPA is providing for continual maintenance and updating of the program. A users group, consisting of over 400 members, meets semiannually to discuss SWMM applications and modifications and the use of other models as well [Torno, 1978]. A few of the more recent changes and additions of the SWMM model include prediction of urban erosion, modeling of new treatment devices and biological treatment facilities, flexibility in modeling new types of watersheds, new and improved cost functions for treatment and storage options, and inclusion of the St. Venant's routing equations (EXTRAN block).

Oftentimes, the comprehensiveness of the SWMM model makes its use somewhat difficult. The need has arisen for simplified versions of SWMM and/or simplified stormwater models. Thus, for example, SWMM, which is basically a single-event simulation model, also can be run in a continuous mode with 1-hour time intervals for runoff simulation over extended periods.

Many of the functions and components of the SWMM model can be used independently and interchangeably according to the varying requirements of given real life situations. Such an example is the development and use of the model RUNQUAL [Roesner et al., 1977] for the Southeast Michigan Council of Governments. RUNQUAL is a wet weather model designed to give detailed information for individual storm events. For a given rainfall, it generates runoff from up to 12 separate land use categories and the attendant pollution load picked up by the runoff. Runoff and pollution loads are routed through the channel network represented by the model, and hydrographs and the histories of the concentrations of various water quality parameters for each channel in the network are produced. The model is useful for assessing the relative impacts of point versus nonpoint sources of pollution loads, for assessing the contribution of various land uses to pollution, and for determining the reduction in pollution loads achievable by various surface runoff management alternatives. RUNQUAL is composed of two blocks identified as the surface runoff block and the runoff quality block. The two blocks are linked computationally by tape transfers in which the output from the runoff block is provided as input to the quality block. The major input data categories, tape transfer, and output products are shown in Figure 6. A given watershed is subdivided into homogeneous subareas, which are modeled as overland flow areas according to the kinematic wave approximation method. No backwater dynamic effects are accounted for with this method.

The MITCAT model is a modular quantity model developed by P.E. Eagleson and his associates at MIT for single rainfall events [Harley et al., 1970]. Two basic runoff elements are used to model an urban watershed. The first element is a simple plane which is used to model overland flow. The second element is the stream segment. Only the quantity of runoff is modeled. The

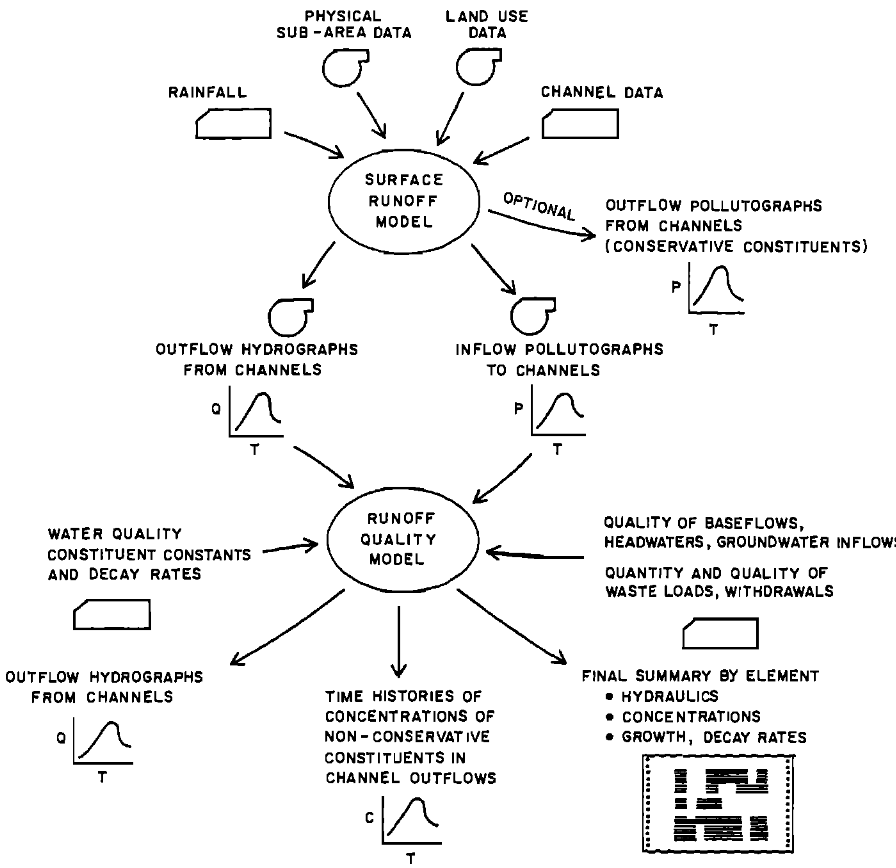

Fig. 6. Structure of RUNQUAL.

overland flows are treated by the kinematic wave method, and channel flows are routed by means of a linearized form of St. Venant's equations. Several infiltration equations (Horton, Holtan, SCS) may be used. The time step is variable. It is a model of moderate complexity but does not have a water quality module.

The kinematic wave approximation for overland flow has also been used in more recent models. The U.S. Geological Survey has recently completed a computer program for routing urban flood discharges through a branched system of pipes and natural channels. The user's guide by Dawdy et al. [1978] is available to the public. The model is divided into four components: a soil moisture accounting component, a rainfall excess component, a routing component, and an optimization component. The soil moisture is modeled as a two-layered system. Moisture is added to the upper layer in accordance with Philip's infiltration equation. On impervious surfaces the only abstraction from rainfall is the impervious retention. The routing component includes four types of segments. These are the overland flow, the channels, the resevoirs, and the nodes or functions. The kinematic wave theory is applied for both overland and channel routing utilizing a four-point finite difference mesh. Both the linear reservoir or the simple reservoir routing may be used. The optimization component is used to determine the optimum parameter values in the infiltration and soil moisture accounting.

A common characteristic of the above design and analysis models is that they simulate the performance of an urban storm drainage system under various rainfall conditions. Thus, different designs can be compared on a case by case basis by simulating their performance under similar conditions. A formal methodology was developed at the University of Illinois for the design of least cost storm sewers. The optimization is accomplished by discrete differential dynamic programming. Three computer models were developed.

The first model designs crown elevations, slopes, sewer diameters, and detention storage for a predetermined sewer system layout. There are two versions. In the simple version the user specifies the location of the reservoirs and the maximum allowable downstream flow, and the program computes the necessary storage volume. In the second version the user specifies the location and maximum storage capacity of the reservoirs. The program then determines the optimum storage volume and the pipe sizes and elevations so as to minimize the total cost. The detention storage cost is computed using a unit storage cost provided by the user. The second model incorporates the risk-based damage cost of flooding in the design procedure. The risk is introduced through risk-safety factor curves corresponding to the service life of the sewer. The third model includes the layout as part of the design. As an option, the user can utilize the hydrograph generation portion of ILLUDAS (See above, this section) to compute the hydrograph at any inlet [Wenzel et al., 1979].

Operation and Control Models

A model that permits the evaluation of the optimal operation of control regulators is the Urban Wastewater Management Model developed by Battelle Memorial Institute [Brandstetter, 1976; Brandsetter et al., 1973]. The basic operation is depicted by the flow diagram in Figure 7. The model uses portions of a mathematical model developed by the University of Minnesota and is intended to simulate efficiently major components of a sewer system. It can evaluate the performance of a planned or existing sewerage system during a variety of rainfall conditions.

The Urban Wastewater Management Model can minimize wastewater overflows to receiving streams by determining the required operation of control regulators. The flow rate and quality of sewerage and availability of storage and treatment capacities are all considered in order to determine when and where to discharge into receiving streams. The model can also be used to design new sewer systems or evaluate existing systems. The least cost combination of alternatives for improving the performance of an existing system can also be determined.

Central monitoring and control is particularly useful in regulating urban combined sewers. Presently, there are several U.S. cities that have used central monitoring and computer control to regulate urban combined sewer systems. The level of automatic control or the degree to which a computer is utilized varies from city to city. In Cincinnati a system monitors and detects overflow events and has been reported to be successful. In Minneapolis/St. Paul a central computer system is used to monitor rain level and flow data and assist the operator in routing and storing stormwater flows. In Seattle, a computer system monitors

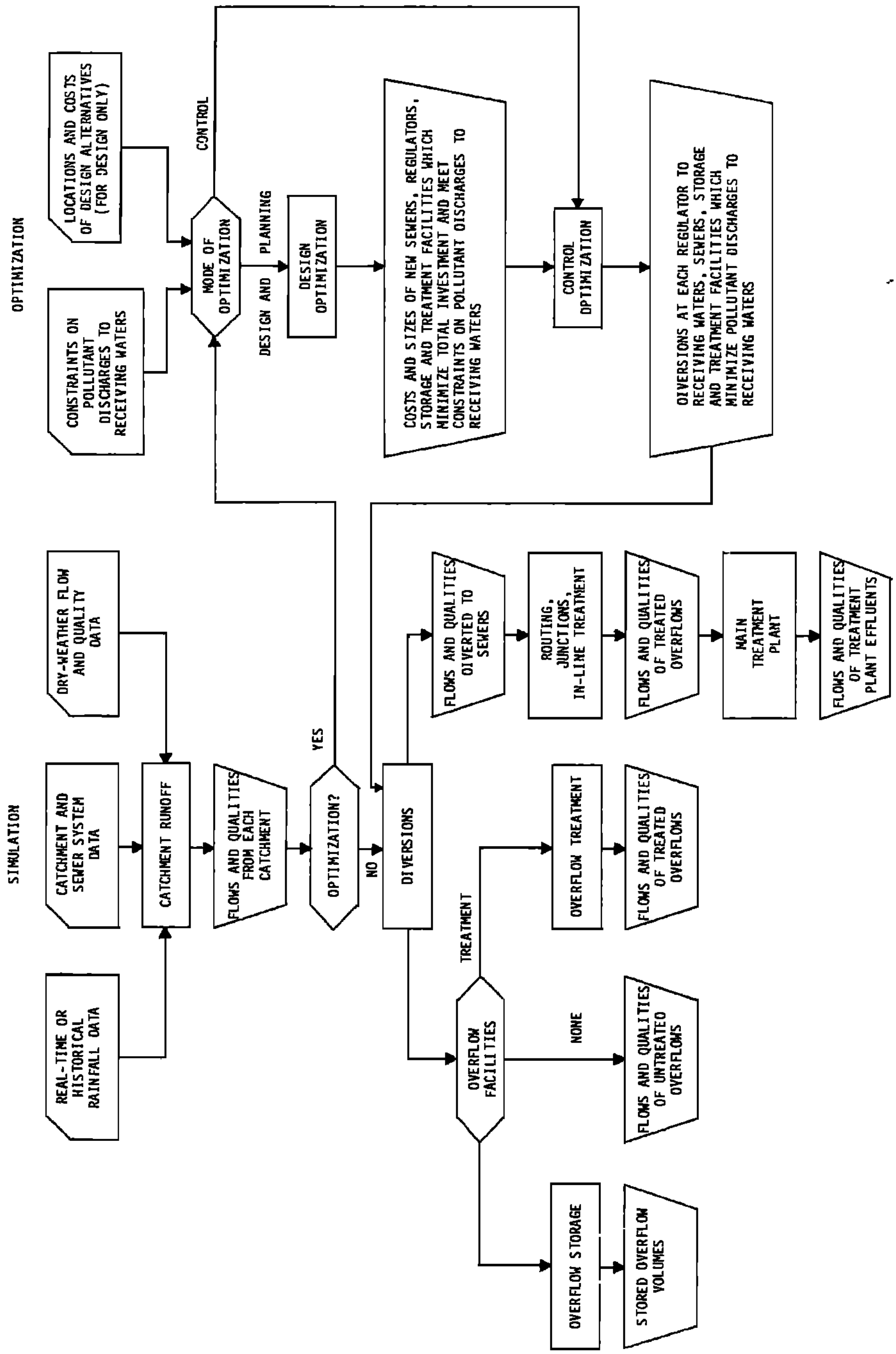

Fig. 7. Operational flowchart of Urban Wastewater Management Model.

rain level, flow, and water quality data and allows for three levels of control, one of which is automatic computer control. In the city of Detroit, a computer is utilized to monitor rainfall overflow and wet weather pump station information, while control is provided by an operator via a manual control panel. The city of Omaha has just finished the planning stage of a real-time automatic control system that will divert the highest strength combined wastewaters to the treatment plant and release more dilute wastewaters directly to the Missouri River. The city and county of San Francisco is in the planning stages of a completely automatic distributed computer control system for storing and routing stormwater flows. It is clear that computer technology will find an increased application to the problem of combined sewer system monitoring in the near future.

Other Models

There exist other hydrologic models that are widely used in urban hydrologic studies. Some of them exceed the narrow scope of urban hydrology. Others are more suitable for large watershed studies of flood control or irrigation. They may be grouped in the family of programs available from the Hydrologic Engineering Center of the U.S. Army, Corps of Engineers and the Soil Conservation Service, U.S. Department of Agriculture. This family of programs includes the original Stanford model that is geared toward a complete water balance of a watershed. It also includes European models that are based primarily on the detailed hydraulic behavior of sewer systems. These models are presented briefly below.

The Hydrologic Engineering Center of the Corps of Engineers has developed a complete package of hydrologic models other than STORM, such as the series HEC-1, the model SSARR, and the model WQRRS. The continuous flood hydrograph model HEC-1C Hydrologic Engineering Center, [U.S. Army, Corps of Engineers, 1973] is a continuous event extension of the original HEC-1 model. Both models estimate the infiltration by means of a simple nonlinear function of precipitation and losses. Snowmelt is taken into account. The Clark unit hydrograph method is used to produce the subbasin flows which are routed in the channels either by a modified storage routing or a Muskingum routing. The time step is variable. This is a simple model to use and was primarily developed for nonurban basins. However, difficulty may be encountered in calibrating it for urban basins, as some of the parameters may vary from storm to storm. It does not simulate water quality.

Also developed by the U.S. Army, Corps of Engineers [1972] is the streamflow synthesis and reservoir regulation model (SSARR) which was originally developed for application to undeveloped basins. This continuous simulation model estimates the infiltration by means of a variable runoff coefficient which is a function of soil moisture. Multiple reservoirs are used for simulating the basin and channel routings. The time step is variable, but the model does not include runoff quality estimation. A model that evaluates water quality conditions in a river or a reservoir system is WQRRS [U.S. Army, Corps of Engineers, Hydrologic Engineering Center, 1974]. It is a one-dimensional model of the vertical stratification in a reservoir or lake. For a river it models the quality variation in the downstream direction.

Finally, a model that has enjoyed a wider use in urban hydrology studies is the SCS-TR20 model [U.S. Department of Agriculture, Soil Conservation Service, 1965]. It uses the SCS curve number method for runoff and has been used to estimate flood peaks for developing watersheds. A simplified version of this model has been incorporated in TR55, which is described as a desk-top method in Chapter 4.

Complete Water Balance Models

Most models presented so far consider processes such as infiltration, interflow, and evaporation as secondary to surface runoff. These processes were introduced as initial abstractions necessary for the determination of proper starting conditions. However, there exists a class of models that focuses on the complete hydrologic cycle and computes a continuous moisture balance within a watershed. One such model is the Hydrological Simulation Program (HSPF).

The Hydrological Simulation Program (HSPF) is the FORTRAN successor of the Stanford watershed model [Crawford and Linsley, 1966]. It is a continuous simulation model extended to include water quality constituents and is available from the EPA Center for Water Quality Modeling in Athens, Georgia. HSPF estimates infiltration by means of a complex accounting of the basin moisture. The kinematic wave method is used to obtain both the subbasin flows and to perform the channel routing. The time step is variable. Empirical equations are used to estimate the runoff quality parameters. The flowchart shown in Figure 8 illustrates the complete water balance approach used in developing the HSPF model.

HSPF is a modular program which performs deterministic simulations of a variety of aquatic processes which occur on or under land surfaces, channels, and reservoirs. One of the modules of particular interest in urban hydrology is the nonpoint source (NPS) model. NPS and the agricultural runoff management (ARM) model were forerunners of HSPF and also are available as separate models from EPA.

NPS is a continuous simulation model of the generation of nonpoint pollutants from pervious and impervious land surfaces [Donigian and Crawford 1976, 1979]. It simulates the surface and subsurface hydrologic processes, pollutant accumulation, and pollutant transport for any selected period of input meteorologic data. The NPS model does not include flow routing. It also does not include the processes that occur after the pollutants have reached the receiving waters. Thus, NPS alone is limited to small areas, probably not exceeding 2 mi^2. Larger watersheds require NPS to be interfaced with other modules available in the HSPF package.

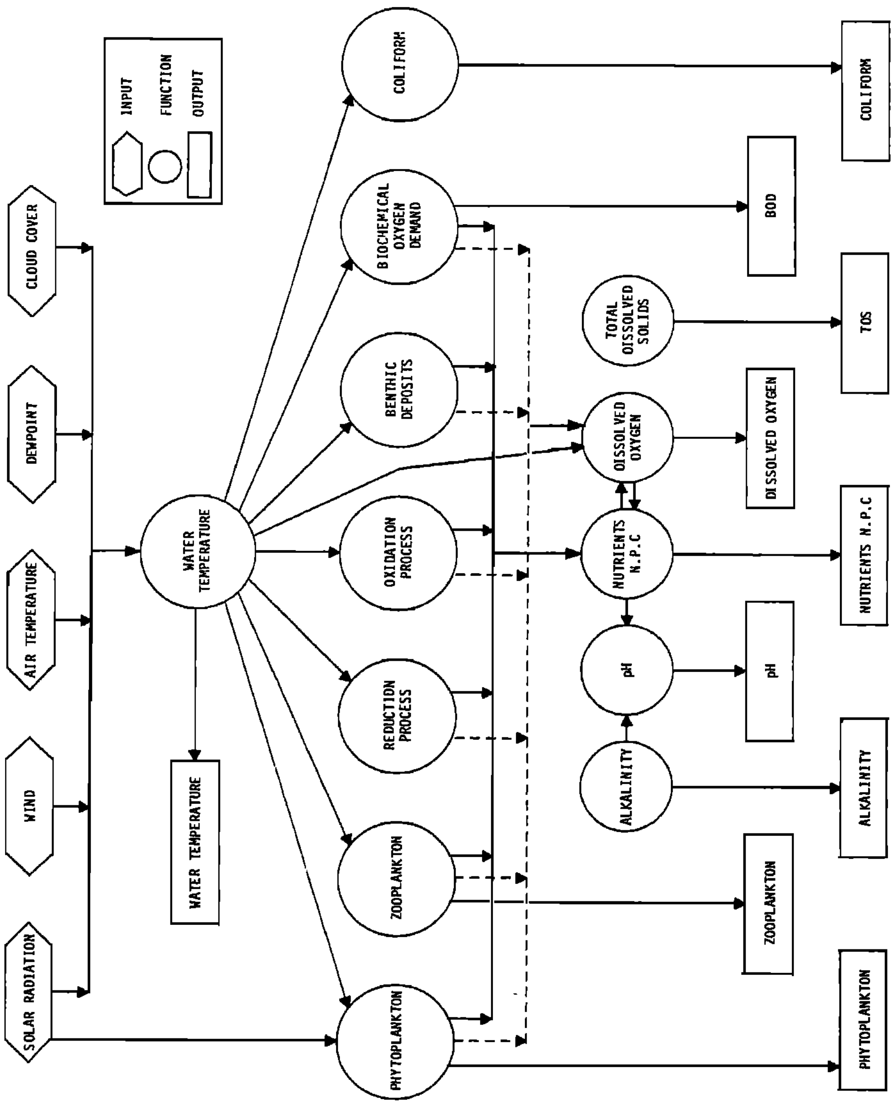

Fig. 8. HSPF quality flowchart.

Hydraulically Based Models

All the models presented above concentrate on hydrologic or water quality aspects of urban runoff to the exclusion of hydrodynamic effects. There exist models, however, which focus on the hydraulic behavior and the hydraulic design of the sewer system. Examples of such models are CAREDAS and the HVM-QQS model, developed by DORSCH CONSULT.

The CAREDAS model developed by SOGREAH is a sophisticated model of the unsteady flow hydraulics of looped or branched drainage networks subject to both pressurized and free-surface flows. The St. Venant's equations are used for the unsteady flow. The system includes modules designed to simulate hydrologic inflows to the drainage system and to simulate the dispersion of pollutants within the system. Chevreau and Holly [1978] described the model and its application to the extremely complicated network of Seine Saint Denis, northeast of Paris. The system drains approximately 100 km^2 divided into 200 subbasins. It includes more than 400 sections totaling 280 km of pipes (with cross sections larger than 0.75 m^2) and 2000 calculation points. The model includes gates, dams, reservoirs, siphons, and flap valves. Hydraulic calculations include backwater pressurization, overflow, and special structures. For large systems the correct evaluation of runoff volumes is essential, but the precise time lag or hydrograph shape is of lesser importance when the number of subbasins is large.

The quantity-quality simulation (QQS) program developed by DORSCH in Germany [Geiger, 1975] produces runoff hydrographs and pollutographs and evaluates the frequency and duration of rainfall-runoff in the urban drainage system. Three precipitation records of up to 20 years can be used as input. The drainage network consists of a sewer network and a receiving water system. The sewer system may contain up to 450 nodes, 70 of which may represent overflows, pumping stations, detention basins, and treatment plants. The receiving water system contains a maximum of 150 elements connected by up to 125 nodes. The time increment normally used in the program is 5 min, although time intervals of 10 or 15 min can also be used. The surface runoff from small catchments including small underlying sewers is treated by the instantaneous unit hydrograph after allowing for evaporation, surface depression storage, and infiltration. The flows through interceptor and trunk sewers are routed by the dynamic wave equations using the St. Venant's equations. A continuity equation for the pollutants is used at the nodal points. Retention functions dependent on basin slope allow for the retention of pollutants by basins, and the purification in treatment facilities is handled by a treatment efficiency factor which depends on the pollutant and on the flow rate. The QQS model has been used in Toronto, Rochester, and Vancouver in North America and has been widely used in Germany.

Comparative Studies

In the presence of so many models with similar capabilities, the selection of one model for a given application can be an imposing task. This task can be facilitated by the comparative studies reported in the literature.

Jennings and Mattraw [1976] compared the predictive accuracy of SWMM, ILLUDAS, and the MIT model for single event simulation. Four catchments were studied with areas varying from 48 to 613 acres, imperviousness varying from 19 to 39% and slope from 0.2 to 20 ft per 100 ft. Only one catchment had water quality records. Preliminary testing with SWMM gave poor water quality results and motivated further studies on constituent-accumulation functions using quantity-quality field data. All the models produced good prediction of the average peak flow.

Abbott [1978] compared three continuous simulation models (STORM, HSPF, and SSARR) and three single event models (HEC-1, SWMM, and MITCAT) on a single basin, the Castro Valley Watershed. The drainage area is 5.5 mi^2, 80% of which is residential, 5% is a strip commercial development, and the remainder is the undeveloped headwater of the valley. Another assessment of storm and combined sewer models was made by Brandstetter [1976]. Numerous comparative studies were also performed in Canada. MacLaren, Ltd. [1975] and MacLaren, Ltd. and Proctor and Redfern, Ltd. [1978] made a review of Canadian design practice and compared various urban hydrologic models, and Wisner et al. [1975] initiated a study to standardize the capabilities of urban stormwater management models.

Finally, additional information on urban hydrologic models can be found in the publications of the Urban Water Resources Research program of the American Society of Civil Engineers (ASCE) [McPherson, 1977, 1978] and in a critical review of the runoff process in urban areas [Delleur and Dendrou, 1980]. Others have been referred in Chapter 1.

Data Requirements

A common denominator among large urban storm drainage simulation models is the amount and complexity of required input data. As the models grow larger and more comprehensive in scope, so do the requirements for reliable data.

The input data for large-scale hydrologic simulation models can be grouped into the following categories: physiographic characteristics of the basin, precipitation characteristics, and specifications of the man-made drainage system itself.

The first group includes such information as the land use and population density of the basin, the average slope, and most importantly the percent imperviousness of the area. Large volumes of data are usually compiled which are extracted from soil survey maps, land use zoning plans, and aerial photographs. The task of planimetering discrete subareas is time consuming and error prone. As a consequence, the land cover data management process has received special attention in recent years, which coincided with an effort at utilizing Landsat data for commercial uses. Early applications showed that Landsat had potential as an operational tool in hydrology [Rango et al., 1974, 1975; Jackson et al., 1977].

The main conclusions of the latter study indicate that Landsat data do provide a suitable source of land cover data for planning level studies in developing areas. Moreover, significant reductions in the costs and man-hours associated with the development of land cover and parameter estimates for hydrologic models can be achieved using the Landsat approach, their magnitude being a function of the watershed size and the type of conventional approach being used.

Finally, the field observation of urban runoff quality parameters has been sporadic with the result that much less is known about the pollution parameters of the runoff than the quantity aspects. The U.S. Geological Survey has recently developed a specialized instrumentation package to measure and collect stormwater runoff [Smoot et al., 1974]. This is described in detail in Chapter 7. The interface of the USGS instrumentation with statistical and deterministic models has been described by Wilson et al. [1978]. Another series of stormwater quality data was obtained for seven basins near Portland, Oregon, and their statistical analysis was reported by Miller and McKenzie [1978]. Finally, information on worldwide sources of runoff quantity/quality data may be obtained from EPA's Urban Rainfall-Runoff-Quality Data Base, compiled by Huber et al. [1979].

Model Calibration, Validation, and Verification

Urban stormwater models are basically simulation models that mathematically mimic the performance of urban storm drainage systems under various conditions. They were developed to aid in arriving at better decisions regarding problems that may be viewed as planning problems, design/analysis problems, and operations problems. The correct use of stormwater models is based on the premise that they reproduce the real-life behavior of the storm drainage system within acceptable bounds of accuracy. This usually involves the following three steps: calibration, validation, and verification of the model.

Calibration of the model is performed by tuning various parameters of the model (for example Manning's n, time of concentration) until the model reproduces measured data. Validation of the model is a further test of the calibrated model's capability to reproduce other measured data from storms outside those used in calibration. Finally, verification of the model consists of investigating whether the model is representative of all possible conditions that may occur in the prototype, for example, snowmelt and frozen soil conditions. Only after such a thorough testing can the model be used in a predictive mode to help in making decisions about the effect of future urban growth, or the required temporary storage and/or treatment rate in a combined sewer system.

There are many difficulties in performing the above described process. Foremost is the scarcity of data [Huber et al., 1979]. Other difficulties, however, are even more important for the proper use of existing models, in that they stem from inherent modeling assumptions built in the program. These difficulties have slowed the movement toward standardization of stormwater management modeling. Results from some models are very sensitive to the schematization of the watershed or the selection of characteristics of the system. The same model under different schematizations may lead to vastly different results. Also, many models are very sensitive to space discretization (or lumping) in the watershed and the time discretization of the storm input hyetograph. However, sensitivity studies performed on various models [Kibler and Aron, 1978] are helpful in properly setting up case by case applications. Finally, it is important to recognize that the ultimate goal in using sophisticated models is to help achieve better decisions. Therefore, promotion of better modeling should run parallel with improvements in the decision making process. A sample of studies which have employed stormwater models to assist the decision-making process is presented in Chapter 9.

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9 EXAMPLE MODEL APPLICATIONS

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Introduction

Recent studies by Donigian and Linsley [1978] and Hansen [1979] have clearly shown that computer-based mathematical models of urban hydraulic and hydrologic systems are in wide use and that this use is increasing rapidly as practicing engineers and planners become comfortable with such design and analysis tools. Perks [1978], Delleur et al. [1978], and others have further indicated that engineers experienced in the application of urban hydrologic models employ a hierarchy of models, ranging from simple to complex, choosing the appropriate model(s) to fit the problem being studied.

Table 1, which shows the respective suitabilty of five different models to a variety of typical problems, demonstrates the point. It should be obvious that no single model is uniquely suited for a specific problem nor is there a "universal" model. One should further note that the models selected were for example purposes, and other models could have been equally well included in the table.

The model applications in this chapter are divided into three general categories:

1. Planning, the evaluation of future alternatives. Such applications usually start with the use of a simple model and get progressively more complex, as necessary.

2. Analysis, the evaluation of existing systems or the detailed study of a planned alternative.

TABLE 1. Model Applicability to Different Urban Drainage Problems

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

3. Design, the final sizing or configuring of a system for which planning or analysis is complete.

The dividing lines between these categories are by no means clear, and many applications could fall into more than one category, depending on the point of view of the model user.

The emphasis in this chapter will be in providing examples of the range of problems which have been addressed. Practical, as opposed to research, examples will be stressed. Models will be described only if they were not covered in the preceding chapter. There will be no comparative evaluation of models, since the literature is replete with such comparisons. Furthermore, the "accepted" models work well on problems for which they are suited, and little is served by providing comparisons of how different models perform on the same data base. In some cases, models of widely varying complexity will produce equivalent results on a watershed; in others they will not. This does not reflect a deficiency in the models but rather that all models are formulated using a large number of assumptions, and it is essential that a model user understand these assumptions, the model formulations, and their implications when the models are applied to a particular problem.

It will be noticed in the following examples that a few models are mentioned repeatedly. This does not imply that these models are better but rather that they are among the relative few which receive wide use. Torno [1979] has identified some of these models and has postulated reasons for their selection.

Four Mile Run: Planning and Analysis

The Four Mile Run watershed is a small catchment (see Figure 1), slightly less than 20 mi^2, draining into the Potomac River near Washington, D. C. About the turn of the century, several railroad and highway bridges were constructed on the downstream end of the catchment, and subsequent urbanization of the watershed has resulted in severe flooding problems, particularly in the Arlandria area immediately upstream of the bridges. The U.S.

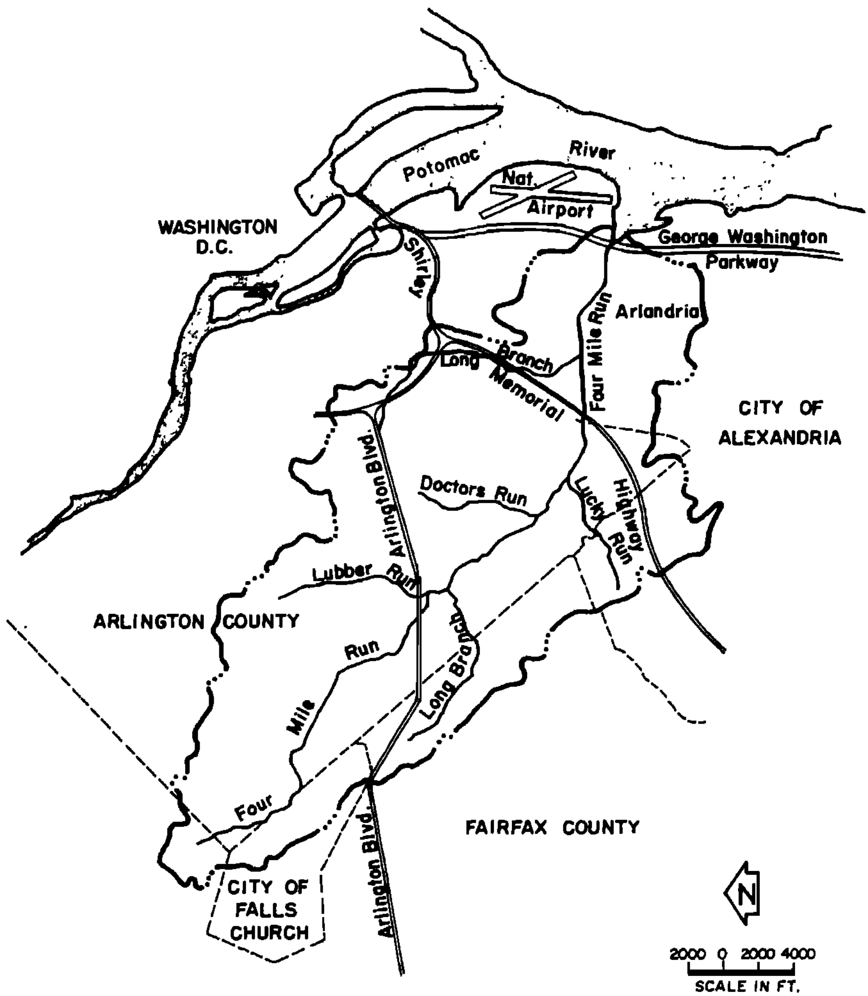

Fig. 1. Four Mile Run study area. Figure adapted from Water Resources Engineers [1976].

Army Corps of Engineers designed a flood protection project for Four Mile Run, which was approved by the Congress as a part of P.L. 93-251, contingent upon a joint study by the political jurisdictions in the watershed to insure that future land use changes did not jeopardize the project. In September 1974 the Northern Virginia Planning District Commission (NVPDC), acting on the behalf of the local jurisdictions, conducted a study of the relationship between urban development and flooding in Four Mile Run.

The objectives of the study were to (1) estimate runoff from the various jurisdictions under current conditions, (2) evaluate the effectiveness of runoff control methods, and (3) establish a common procedure for the evalution of the effects of land use changes and runoff control measures.

In the first phase of the study the STORM model, developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center [1977], was calibrated on six recent runoff events and then was used to develop flood frequency curves, based on local rainfall records for a 51-year period [1911-1973]. The flood frequency so developed compared favorably with the flood frequency developed by the Corps of Engineers based on historic stream flow data. A design flow, at the 100-year interval, was selected by increasing historic flows to account for urbanization using a method formulated by Anderson [1970]. A watershed design storm was then developed, using the method of Kiefer and Chu [1957], based on an analysis of local rainfall records and modified so as to provide the design flow when used as input to the STORM models. A site design storm, similar to the watershed design storm but having greater intensity, was also developed to account for the adjustment to smaller areal coverage.

The remainder of the study was conducted with the WATERSHED model, developed by Water Resources Engineers [1976], which is an extensively modified version of portions of the stormwater management model (SWMM). An important facet of this study was the establishment of the baseline condition, which was defined as a combination of the design storm, the land use, and drainage system existing or authorized on April 30, 1975, and the verified WATERSHED model.

The WATERSHED model consists of three main elements: (1) RUNOFF, a set of computer routines which simulates the surface flow from individual subcatchments and in some pipes and open channels, (2) land use management program (LUM), an auxiliary data program to RUNOFF, and (3) TRANSPORT, a set of computer routines which simulates the flow through the most significant part of the drainage system.

The method of assessing impacts of potential land use changes or drainage modifications is as follows:

1. The subcatchments affected by the proposed study are analyzed (often with a more detailed model than WATERSHED) to generate the runoff hydrograph under design conditions.

2. The resulting hydrograph is incorporated into WATERSHED to examine effects on the whole basin.

3. If the proposed modifications will cause flooding increased over the baseline case, alternative runoff control measures are then evaluated, and a set of control measures is selected so that the baseline runoff will not be exceeded.

This process insures that future modifications in the Four Mile Run watershed will not lead to future flooding problems.

Bucyrus, Ohio: Analysis and Design

The application of urban hydrologic models is not confined to large cities. Bucyrus, Ohio, a town with slightly over 13,000 residents, was required to abate pollution of the Sandusky River resulting from combined sewer overflows. Cole et al. [1978] have described a project in Bucyrus which examined alternative techniques for abating this pollution source during the facilities planning stage. Alternatives considered included sewer separation, on-site treatment of excess combined flows, storage in aerated lagoons, and the selected alternative, on-site detention of combined flow for subsequent treatment at the main wastewater treatment facility. In the initial analysis, the decision was made to size the intercepting sewer to allow for future expansion of detention structures if subsequent monitoring indicated that receiving water quality could be improved. Modeling for this initial analysis utilized an early version of the Stormwater Management Model (SWMM) developed by the U.S. Environmental Protection Agency and described by Huber et al. [1975]. The existing sewer system operated under surcharged conditions, and the proposed design included surcharging as well. Since this early version of the SWMM could not model these surcharged

TABLE 2. Sewer Size Comparisons

	Preliminary		Final	
Pipe Size (diameter) Range, in.	Length, ft	Percent of Total Length	Length, ft	Percent of Total Length
18-30	1,200	10.7	4,555	42.2
36-60	7,585	67.5	6,255	57.8
72-78	2,455	21.8	--	0
Total	11,240[a]		10,810[a]	

[a]Minor changes in total length due to adjustments in alignment after completion of route surveys.

conditions, preliminary interceptor design was based on peak flows at each of the 24 overflow points. Flows were also assumed to be occurring under surcharged conditions resulting from maximum levels in the two detention structures.

In 1977, a new version of the SWMM was released which included the capability to model extensive system surcharging. This new version was applied to this design. As a result, the preliminary design was found to be very conservative. The system was redesigned, allowing increased interceptor surcharge, as long as ground level was not reached and as long as the water level in the existing collectors was not affected. Table 2 shows a comparison of the preliminary and final sewer sizes.

The shift to smaller pipe size as a result of the revised analysis resulted in an estimated 25% savings in interceptor system construction costs. Such an analysis would not have been economically feasible without the use of such a model.

Edmonton, Alberta: Analysis and Design

The City of Edmonton, Alberta, Canada, has taken a leading role in the application of new stormwater management methods in the design and analysis of their urban drainage systems. Fok et al. [1979] have described a project in Edmonton involving the

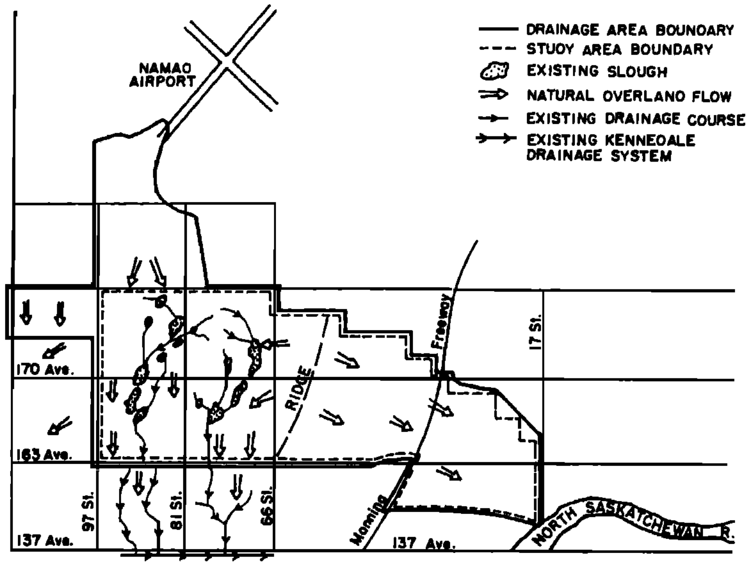

Fig. 2. Study area drainage. Figure taken from Fok et al. [1979].

preparation of a master drainage plan for a large (7350 acre) new development. The total study area is shown in Figure 2. One of the features of the study was that it involved an application of the major-minor concept, developed in Canada. This concept recognizes that the storm drainage (minor) system can only carry a limited flow and that for extreme occurrences there will be some flow in the overland (major) system. Drainage design for the eastern watershed was fairly straightforward. The western watershed, however, was more complex due to natural drainage features and the presence, to the south, of limited sewer capacity in the developed areas adjacent to the study area. Three basic stormwater management alternatives for controlling runoff from this watershed were distilled from a number of options for more detailed consideration. Preliminary hydrologic estimates clearly demonstrated that any drainage scheme for the western watershed has to plan and provide for detention storage for feasibility, economy, and viabilty. These alternatives are depicted in Figures 3, 4, and 5. Alternative 1 involved diversion of runoff eastward through a tunnel using gravity flow. Alternative 2 is

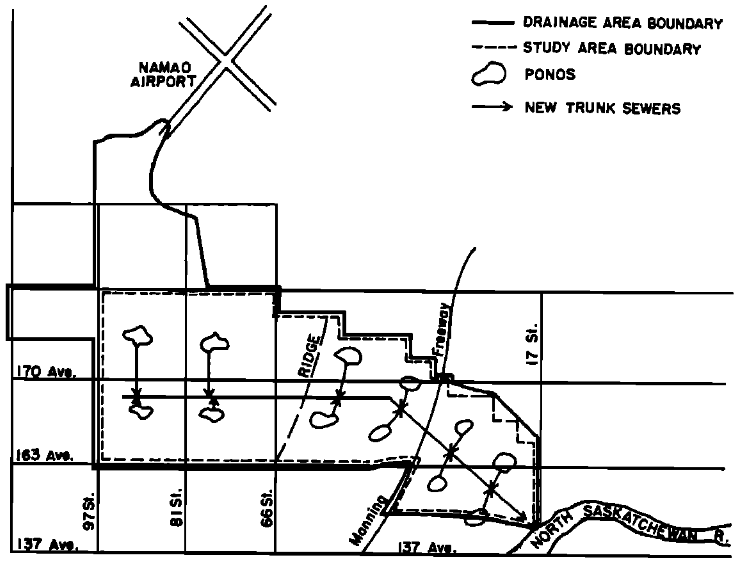

Fig. 3. Schematic of alternative 1, gravity-induced flow through tunnel.

similar to alternative 1 except that storm flows are pumped over the ridge. Alternative 3 maintained the natural southerly drainage, incorporating storage lakes and connections to the existing sewer system.

Synthetic rainfall events, encompassing return frequencies of 5 to 100 years and durations of 6 to 24 hours, and actual rainfall data were used in the study. The method of Kiefer and Chu [1957] was used for short duration synthetic events. For long-duration (12 hours or longer) design storms, a uniform rainfall distribution was assumed since such long storms tend to have relatively low average intensities and the resulting total runoff volume is much more critical than the peak flow for designing a lake storage system. Rainfall data since 1880 at Edmonton Municipal Airport were obtained for the simulation of real storms. Three sets of rainfall data, consisting of continuous hourly rainfall data from 1960 to 1977, largest storm data as given by hourly rainfall data for the storm of July 14-15, 1937, and largest monthly total rainfall as given by daily rainfall data for the month of July 1901, were used in the analysis.

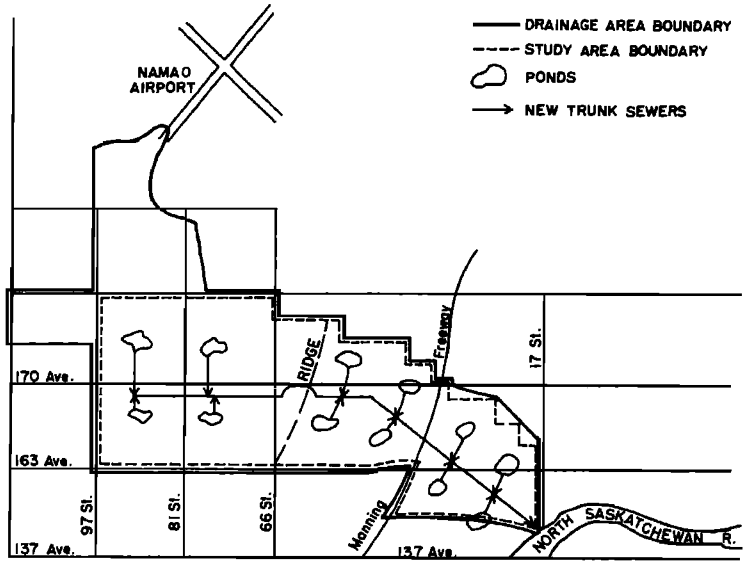

Fig. 4. Schematic of alternative 2, pumping over the ridge through a piped system.

Three models were used in the analysis. The HYMO model, developed by Williams and Hann [1973] of the U.S. Soil Conservation Service, was used to simulate single storm events. HYMO uses a unit hydrograph procedure to transform rainfall data into runoff hydrographs, a variable storage coefficient routing technique to route flows through streams and valleys, and an SCS storage indication technique to route floods through reservoirs. Sediment yields can be calculated using the Universal Soil Loss Equation. STORM was used to simlulate runoff from a continuous rainfall record (1960-1977) in which closely spaced consecutive storms occurred. The SWMM was used to analyze the hydraulic response of the existing sewer system to the flows released from the study area in conjunction with other local flows. Snowmelt runoff was evaluated, but it was determined that summer rainfall events were more critical regardless of return frequency. Table 3 indicates the estimated flows for each of the three alternatives.

Using the estimated flows as shown in Table 3, preliminary cost estimates were prepared for all alternatives. Alternative 3 was

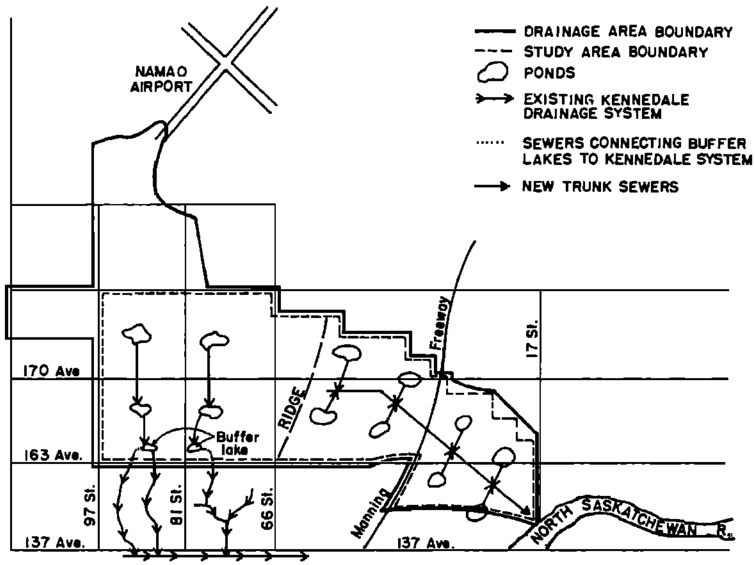

Fig. 5. Schematic of alternative 3, controlled release to the 137 Avenue trunk sewer.

most cost effective as well as permitting staged construction as development proceeds. In addition, the storage ponds improve aesthetics, enhance groundwater recharge, and improve water quality. Alternative 3 was selected and then analyzed further to arrive at the final system configuration.

Bloomington-Normal, Illinois: Planning and Analysis

A chronic problem faced by those planning or designing urban stormwater facilities is the shortage of data, particularly when water quality considerations predominate. The Illinois Environmental Protection Agency [1978] has described an interesting model application directed at extending a limited data set in a study done by the Illinois State Water Survey (ISWS) in Bloomington-Normal, Illinois. Most of the urban area involved is drained by Sugar Creek, which has a small base flow and often approaches zero flow in late summer. The effects of storm runoff, point sources, and combined sewer overflows are significant during low flow periods. It was therefore important to establish the variability of constituent levels over time, yet

TABLE 3. Estimated Flows for Alternatives

Design Storm	Alternative	Required Storage Volume ac-ft	Pumping Rate cfs	Overland Flow South, cfs
5-year 6-hour	1- Tunnel plus ponds	360	-	-
(minor system design)	2- Ponds plus pumping	360	50	-
	3- Ponds plus controls	360	-	-
100-year 6-hour	1- Tunnel plus ponds	360	-	-
	2- Ponds plus pumping	720	50	20
	3- Ponds plus gated controls	720	-	0
	4- Ponds without control	720	-	50
	1- Tunnel plus ponds	720		-
	2- Ponds plus pumping	1030 940	50 50	0 50
	3-Ponds plus gated controls	720 940 1130	-	220 100 0

Table adapted from Fok et al. [1979, Table 1].

the data were not available, and the cost and time requirements to collect such data were prohibitive.

A version of the ILLUDAS model [Terstriep and Stall, 1974] which included water quality modeling capabilities (QUAL-ILLUDAS) was used to extend the data set. The procedure is as follows:

1. QUAL-ILLUDAS was applied on a limited number of watersheds for which calibration and verification data were available.

2. Relationships between model parameters were explored, including results from watersheds outside the study area. It was

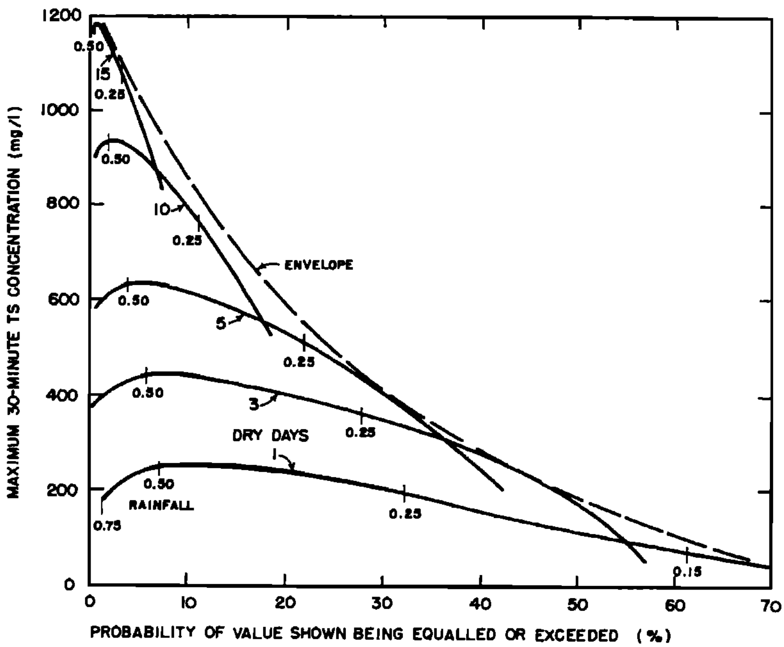

Fig. 6. Typical relationship between dry days, rainfall, and 30-min maximum concentration. Figure adapted from Illinois Environmental Protection Agency [1978].

apparent that concentrations and loads generated were related to total area, paved area, slope, soil type, loading rate, length of dry period, and rainfall. Since they were able to determine the first five of these variables for any location in the study area, the length of dry period and rainfall were used to relate concentrations to frequency of occurrence. Figure 6 shows that the concentration increases with the interval between rainfall events (dry days) and that for any given dry period the maximum concentration occurs at about 0.50 inches of rainfall. By drawing these curves for dry day periods from 1 to 20 days, an envelope curve was drawn representing the maximum concentration for any rainfall/dry day combination. Statistical tests showed that the length of dry period was independent of the following rainfall. Hence, the probability of each point in Figure 6 is the product of the individual probabilities of dry days and rainfall.

Model results were extended to unmodeled areas by use of envelope curves like Figure 6. Since each envelope curve represents a single point in a watershed (i.e., slope, drainage area, and directly connected paved area are constant), a family of curves, representing 'typical' locations in the study area, was developed. These were then subjected to log transformation and multiple-regression analysis, and an equation for maximum 30-min solids concentration was developed of the form

$$MC30 = 4716\ SL^{-0.1014} A^{-0.0264}\ DCPA^{0.532}\ E^{-4.196(PROB)} \tag{1}$$

where

MC30 30 min maximum concentration, mg/l;

SL basin slope, ft/mi;

A watershed area, acres;

DCPA directly connected pave area, %;

E base for Naperian logarithms;

PROB probability of occurrence.

Borough of East York (Toronto, Canada): Analysis

A problem facing many cities with older combined sewer systems is that of basement flooding, usually caused by excessive wet-weather flows in combined sewer interceptors and trunks. The traditional way of ameliorating this problem has been to separate combined sewers into storm and sanitary components, or to construct some sort of relief sewer (called 'road' sewers in Toronto). In the Borough of East York, early studies had indicated that sewer separation was the desired alternative. Various political and physical constraints, however, led the Borough, in 1975, to reassess the validity of these early studies.

Eicher [1978] has described this assessment, which involved the application of the EXTRAN (Extended Transport) block of the SWMM program. This routine, which provides the capability to analyze flow in complex sewer systems which are subject to extensive surcharging, flow reversal, and flow in looped or interconnected sewers, now permits analyses which heretofore were not possible. The consultant was asked, as a part of the assessment, to evaluate interim solutions which would provide early benefits from the large trunk sewers which form the backbone of the new sewer system. While at the time the report was written the assessment was incomplete, several significant findings emerged:

1. The locally looped and interconnected systems have noticeably better performance than single-ended branches due primarily to interaction between sewers making use of the dynamic character of the runoff.

2. The trunk/collector system is in many areas the primary cause of flooding, contrary to earlier findings which were based on rational method analyses and which indicated that the trunk would have the larger capacity. It should be noted here that a similar study in Edmonton, Alberta, indicated that the major problem was in the local sewers. The point is that there is no consistency in the response of different systems, and that each must be analyzed. There are no easy or typical answers.

3. Relieving the existing combined sewer at key locations can significantly improve the flooding protection provided by the existing system. It should be noted here that relief structures are constructed so that the combined trunk will operate at maximum capacity before overflowing and so that these interim structures can easily be adapted to provide necessary receiving water protection.

4. The construction of relief sewers can be optimized so that the worst surcharging can often be relieved without implementing the entire scheme. Roesner et al. [1980] have recently described improvements to the EXTRAN model.

Dorchester Bay: Planning and Analysis

Camp Dresser and McKee [1980] have described the use of stormwater models in a combined sewer overflow project in the Dorchester Bay area of metropolitan Boston, Massachusetts. Three

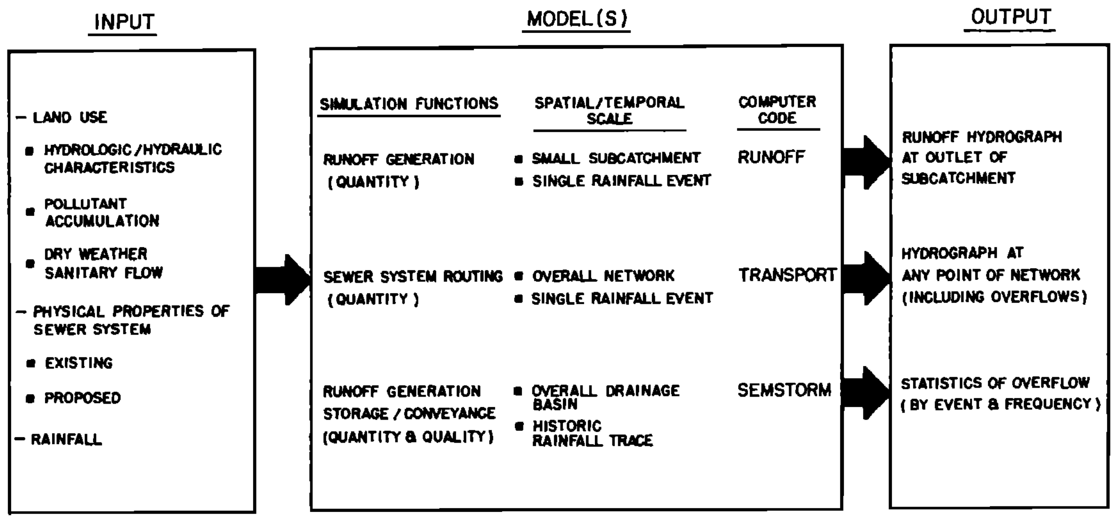

Fig. 7. Computer modeling of Dorchester Bay study area: Input, simulation, functions, computer code, and output.

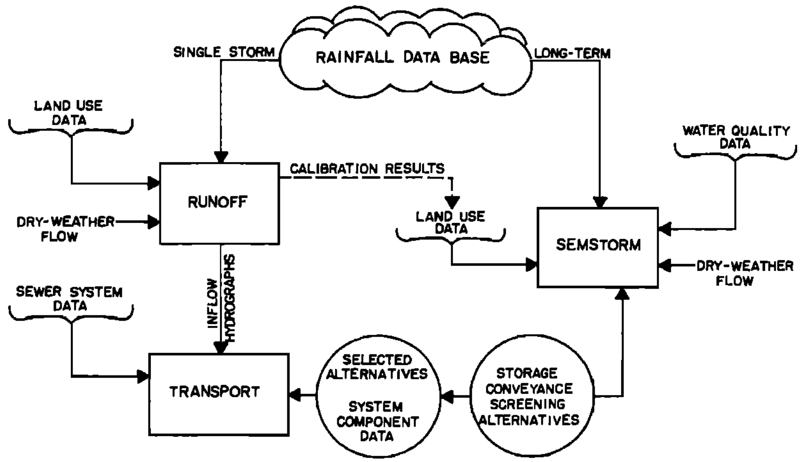

Fig. 8. Interrelation of modeling components.

models, SEMSTORM, RUNOFF, and TRANSPORT, were used to assess the nonpoint source (NPS) contribution to the pollution of the bay from combined sewer overflows. The latter two are improved and expanded versions of computational blocks of the original EPA-SWMM model described in Chapter 6. Their use is shown in the functional diagram of Figure 7 and in the flow diagram of Figure 8. Selecting an integrated 'package' of programs which would meet the needs of this facilities plan involved consideration of three related issues: (1) the nature and availability of input data associated with the Dorchester Bay study area, (2) the 'simulation functions' that would supply the desired planning-level information for rational decision making, and (3) the type of model output that would be most useful for this planning problem. Figure 7 illustrates these related issues for the three simulation models which were selected for this work. Simulation capability was required to generate annual statistics of overflow quantity and quality to be used as input to the 'Harbor Model' for the long-term assessment of combined sewer overflow (CSO) impacts on water quality in Boston Harbor. The same statistical capability was used for screening possible CSO alternatives for the mitigation of overflow problems. SEMSTORM provided this capability. Also, a capability to study the short-term hydraulic sewer system response to design storm rainfall events was needed for the detailed evaluation of those alternatives which appear to be attractive, based on a screening process. RUNOFF and TRANSPORT were used for this purpose. Figure 8 illustrates in schematic form how the three selected models were actually interrelated to form a single package for the purpose of the Dorchester Bay area CSO facilities planning process.

Other Applications

The previous examples have stressed the direct application of models to urban drainage problems, however they are also extremely useful as tools to aid in other forms of analysis. The following should provide some insight into the possibilities open to the creative model user.

Planning for storm drainage systems often amounts to determining the most economical treatment-storage combinations to meet system performance constraints. Comparisons can be made using some indicator of the expected level of utilization of both storage and treatment facilities, such as cost, and some measures of the performance of the system, for example, the frequency of overflows, the average or maximum volume of overflow, and the associated levels of pollutants in the receiving waters, such as BOD, nitrates, coliform, or the average value of a generalized water quality index. Many combinations of storage and treatment capacities can achieve the same level of performance, as illustrated in the graph of Figure 9. These combinations form curves known as isoquants. The least cost combination for given levels of performance form the so-called expansion path. Decisions as to the appropriate storage and treatment capacities can be made on the basis of such functional analyses [Shubinski, 1974; Heaney et al., 1976]. Closed-form analytical expressions of isoquants are not, however, commonly encountered in the literature and are usually mathematically cumbersome, and their use is limited by restrictive, simplifying assumptions [see, e.g., Chan and Bras, 1979]. Recourse thus has to be made to simulation models such as STORM to derive the storage treatment isoquants.

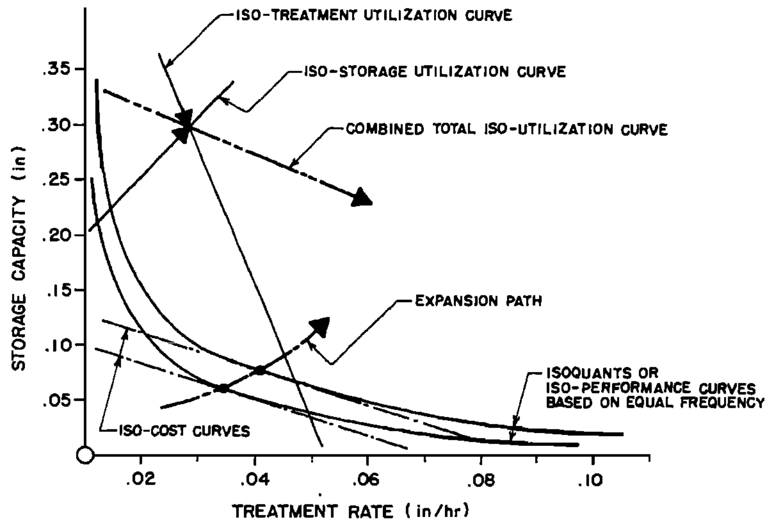

Fig. 9. Most economical treatment-storage combinations to meet system performance constraints.

From the standpoint of the practicing engineer, the sole use of economic criteria in arriving at design decisions leaves a lot to be desired. If one wishes to determine most efficient designs independently of cost minimization arguments, simulation models such as STORM can again be used to advantage. As shown in the upper part of Figure 9, iso-utilization curves may effectively replace the iso-cost curves in the previous example. An index showing the degree of utilization of a storage facility (iso-storage curve) depends generally on both storage capacity S and treatment capacity T. Such an index could be taken, for example, as the area under the 'storage utilization' curve provided in the STORM output. Likewise, a treatment utilization index would also depend on both decision variables, S and T. A vectorial combination of the iso-treatment and iso-storage curves provides a total iso-utilization curve parallel to the traditional iso-cost curves, so that both indexes could be used systematically to determine optimal (or most efficient) designs for a given level of performance (in terms of number and quality of overflows, street flooding, etc.).

A final example of the use of stormwater models relates to a computer program package developed by Dendrou et al. [1978a] that integrated and interfaced an urban growth model and an urban hydrology model, a slightly modified version of STORM. In this package, alternate growth scenarios can be directly related to the corresponding storm drainage systems. If these are designed to achieve specific standards of performance, then a useful comparison among several possible urban growth patterns can be made. The planning of a global storm drainage system for a conglomerate of urban basins is accomplished by a coordination of the interactions among the different basins. The planning variables are the drainage network capacity, the placement and size of the storage facilities, and the size of a central treatment facility. A multilevel coordination problem is recognized among the basins of the watershed. The storm drainage planning model is defined at the watershed level and the optimization is achieved by a multilevel feasible decomposition scheme, where the land use based hydrologic simulation is used locally and separately for each basin. A constrained cost minimization scheme is used for the solution procedure.

Conclusion

The preceding should give the reader some idea of the range of problems that can be addressed using urban drainage models. It should also be clear that models are receiving wide use. In spite of their complexity and the need for skilled personnel to interpret model results, there are a number of significant advantages in using models:

1. They are a powerful tool in the assessment of hydraulic and hydrologic impacts of future growth on existing and planned facilities.

2. They result in a better understanding on the part of the model user of the system being analyzed. The physical system must be accurately represented for the models to give good results, and the models frequently reveal details of system performance which are not intuitively obvious.

3. They clarify the relationship between land use decisions, mitigative actions, and costs.

4. Once set up, a model becomes an available tool of great power and versatility, allowing rapid updates as conditions change, and at low cost.

5. Models allow the joint consideration of water quality/quantity issues.

6. Models are an aid in identifying deficiencies in existing facilities and management programs.

Models can be quite expensive to use, though costs vary widely and are generally lower when experienced personnel are doing the modeling. Computer modeling studies are labor intensive. Computer costs seldom exceed 20% of total project cost and are frequently in the 10% range. Since in any good modeling study skilled personnel are needed to evaluate model output, it should be obvious that the most effective cost-cutting technique for those doing modeling work is to acquire competent staff.

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