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A Statistical Method for the Assessment of Urban Stormwater

A STATISTICAL METHOD
FOR ASSESSMENT OF URBAN STORMWATER
LOADS - IMPACTS - CONTROLS

FOR

EPA - NON POINT SOURCES BRANCH

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SUBJECT: Transmittal of Document Entitled "A Statistical Method for Assessment of Urban Runoff"

FROM: Swep T. Davis, Deputy assistant Administrator
Office of Water Planning and Standards ~~(WH-554)~~

TO: All Regional Water Division Directors
ATTN: All Regional 208 Coordinators
All Regional NPS Coordinators
All Nationwide Urban Runoff Prototype Projects
All State and Areawide Water Quality Management Agencies
Other Concerned Groups

TECHNICAL GUIDANCE MEMORANDUM-TECH- 49

Purpose

This document "A Statistical Method for Assessment of Urban Runoff" has been prepared to provide technical assistance to the Nationwide Urban Runoff prototype projects and other interested groups in assessing the impact of urban stormloads on the quality of receiving waters, and to evaluate the cost and effectiveness of control measures for reducing these pollutant loads.

Guidance

The enclosed report is provided in accordance with the Nationwide Urban Runoff Program established under Section 208 of the Clean Water Act of 1977. This methodology is appropriate for use at the planning level where preliminary assessments are made to define problems, establish the relative significance of contributing sources, assess feasibility of control, and determine the need for and focus on additional evaluations. It can also be used effectively in conjunction with detail studies, in evaluating the most cost-effective alternatives for controlling urban runoff.

Attachment

STORMWATER MANUAL

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CHAPTER 1

INTRODUCTION

This manual describes a simplified methodology which can be used to assess the impact of urban stormloads on the quality of receiving waters, and to evaluate the cost and effectiveness of control measures for reducing these pollutant loads. The methodology is particularly appropriate for use at the planning level where preliminary assessments are made to define problems, establish the relative significance of contributing sources, assess feasibility of control, and determine the need for and focus of additional evaluations. It can also be used effectively in conjunction with detailed studies, by providing a cost-effective screening of an array of alternatives, so that the more detailed and sophisticated techniques can examine only the more attractive alternatives.

The methodology is based on the determination of certain statistical properties of the rainfall history of an area. From these statistics, the desired information on loads, performance of controls, and receiving water impacts is generated directly. Procedures are quite simple to apply, using charts and graphs which facilitate screening alternate types or levels of control, testing sensitity to assumptions concerning drainage area characteristics, stormwater contaminant levels and similar variable factors.

The theoretical basis for the methodology is presented, although the reader need not be familiar with statistical theory or procedures to utilize it effectively. The user need not read the manual from cover to cover and understand and apply each part of it in a rigorous sequence, in order to benefit from it. While separate chapters are mutually supporting, each essentially stands on its own for the particular aspects which are addressed.

Chapter 2 - Presents an overview of the urban stormwater problem, and a perspective in which water quality problems caused by storm loads can be considered.

Chapter 3 - Presents a description of the statistical methodology, its theoretical basis, and its application for characterizing storm loads, receiving water impacts and the performance of selected control measures.

Chapter 4 - Presents a description of a simulation model which calculates receiving water impacts for streams and estuaries. It is designed to operate on input consisting of time variable storm loads as generated by various load generating simulators (e.g. STORM). As an alternative to the statistical calculations, it will provide the user with both the time history of receiving water concentrations and their statistics for the period analyzed, at a number of receiving water locations.

Chapter 5 - Presents a condensed summary and analysis of available information and data on numerical estimates of parameters required for performing a stormwater impact analysis. The information is of value for analyses utilizing simulation methods as well as for application of the statistical procedures described in this manual.

Chapter 6 - Presents considerations for the design of effective monitoring programs, applicable for either preliminary assessments or more intensive programs.

Chapter 7 - Provides examples which illustrate the applications of the methodology to specific problem settings.

Bibliography - An extensive bibliography is provided which can direct the reader to additional sources of information on aspects which he may wish to pursue in greater depth.

Although the statistical methodology presented is essentially designed to be an estimating technique, and of maximum applicability and value in preliminary assessments performed at the planning level, it provides results quite similar to those generated by simulation techniques (such as STORM) when similar basic data inputs are used, and comparable levels of spatial and/or temporal definition are employed. The validity of calculations performed by the statistical methodology has been "established" by comparisons with simulation results both as reported in this manual, and by others.

The statistical methodology is not proposed as a substitute for simulation techniques, other than for preliminary assessments. For preparation of formal facility plans and especially for final design, the optimum distribution, location and individual size of controls will normally require a definition of spatial and temporal detail which can be more effectively provided by detailed simulation techniques.

CHAPTER 2

STORMWATER RUNOFF - A REVIEW OF THE PROBLEM

The most fundamental issue in urban stormwater control studies is determining the degree of control which is justifiable from the standpoint of benefits returned on investment. While this basic issue is quite clear, the best methods for addressing it within the context of a planning study are the subject of continuing debate. Numerous analytical approaches for evaluating urban stormwater needs have been developed and applied (1,2,3). Most of these tend to be strongly oriented toward load estimation, treatment performance, or cost estimation, and useful assessment procedures for these problems have been presented. However, the other side of the equation, that of evaluating benefits, has generally been approached with subdued enthusiasm. This is principally due to the difficulty in quantifying the long term water quality improvements associated with urban stormwater control.

The principal purpose of this chapter is to develop a framework within which stormwater control requirements can be evaluated from the standpoint of water quality improvements. This perspective is essential to the planning study, since stormwater control is a questionable investment if it does not result in long term enhancements to legitimate water uses.

From this perspective, stormwater control studies must begin with an evaluation of water quality problems. What are the existing problems in an area? To what extent does urban stormwater runoff contribute to identifiable problems? Is stormwater treatment a reasonable alternative to effectively control existing water quality problems?

Once the potential benefits of stormwater control have been defined, the planning process can proceed to more specific planning questions such as: How much treatment is enough? What are the benefits of alternative levels of treatment? Are there particularly effective treatment devices or other controls which should be considered? What are the costs of achieving alternative water quality objectives?

Chapter 2 will explore problems and opportunities in evaluating the needs for urban stormwater control. The principal water quality problems in urban areas will be discussed in terms of the resulting limitation of beneficial water uses. Various analysis methods for evaluating stormwater problems and control requirements will also be discussed. A more detailed presentation of the analysis methods required for effective planning will be made in subsequent chapters.

2.1 Major Water Quality Problems in Urban Areas

The nature of water quality problems associated with the receiving waters in and around metropolitan areas is quite varied. Although a list of common potential problems can be drawn up readily, local factors have a predominant influence in determining both the class of problem, its severity, and the specific source or sources which are most critical. The local factors which affect water quality include climate, geography, population, population concentration, the nature and degree of industrialization in the area, the receiving water system, its nature, size and hydrology; and the nature of the surrounding area - both upstream and downstream of the urban area itself.

Water quality problems, either current or potential, will be generated by waste loads which enter the receiving water from the area in question. There are a number of different types of sources which contribute waste loads, including discharges of domestic sewage, either treated or untreated, industrial waste discharges, storm runoff from urban, agricultural, or undeveloped land areas, surface returns from irrigated agriculture, subsurface seepage of groundwater, and leachate from land disposal sites.

Waste loads are generally classified according to their temporal variability as either continuous or intermittent, and according to their spatial extent as either point or non-point (distributed) sources. Continuous sources, such as municipal sewage treatment plants or industrial facilities, produce a relatively constant load over time, although daily or seasonal variations are often present. Intermittent loads, often associated with wet weather conditions, occur infrequently and at random intervals. Some types of pollutant loads, such as those due to groundwater seepage, may occur with an intermediate degree of temporal variability.

The classification of waste loads as point or distributed sources is also somewhat ambiguous in certain cases. Storm runoff, for example, is essentially a non-point source in that it is generated by precipitation falling over a wide area. In terms of the actual load to the receiving water, however, the point source classification may be more appropriate for urban areas where storm runoff is collected and enters the stream at specific locations. In developing a solution oriented approach to water quality problems, storm water loads may be treated as either intermittent point sources, or as distributed loads in order to fit the simplest and most effective approach to analysis.

Each of the individual source types has distinctive characteristics. The type of pollutants which predominate can differ radically between sources, as can the absolute quantity of pollutant. Further, controls which may be applied, with either treatment facilities or management practices, usually modify both the total quantity of pollutant in the source and the predominant type present. The method of analyzing the effect of all sources on water quality, and the impact of alternative control measures must be able to handle these variations effectively.

2.2 Urban Runoff

Human activity as well as natural processes result in conditions which cause contaminants to be mixed with stormwater. Automobiles cause oils and other hydrocarbons as well as certain heavy metals to accumulate on streets and parking lots. Lawn fertilizers and pesticides are applied in a manner which makes them suseptible to erosive processes. Pets contribute to organic pollutant buildups in the urban environment. Construction activity often leaves unconsolidated soils exposed to the elements. Natural processes such as the decomposition of animal and vegetative materials and wind erosion similarly cause potential pollutants to buildup on the land surface where they are susceptable to transport by storm runoff.

Urban runoff may reach receiving waters via storm sewers, overland runoff, or combined sewer overflows. The quantity of storm related pollutants entering a receiving water is largely influenced by which of these conveyance systems are present.

Storm sewers are designed to reduce ponding and flooding problems in a drainage area by conveying runoff away from the area into the receiving stream. Sources of the runoff flows include street runoff, roof drains, drainage from large paved areas (such as parking lots and industrial complexes), and runoff from parks and vacant lands. The quantity of storm sewer flow is influenced by drainage basin characteristics such as the percent impervious area, soil types, and land slope, which will be discussed in more detail in Chapters 3 and 5.

Overland runoff has essentially the same quality and flow characteristics as stormwater discharges with one exception; the flow and load enter receiving waters as distributed sources rather than at discrete stormwater discharge points.

Combined sewer systems convey both dry weather sewage flows and stormwater runoff, normally to a wastewater treatment plant. The systems are designed to accommodate a design flow which is periodically exceeded. When a storm runoff volume exceeds the design capacity of the combined sewer system, the excess flow, a mixture of raw sewage and stormwater, is bypassed to nearby receiving waters. The quantity of flow bypassed is a function of interceptor capacity and regulator operational procedures.

Combined sewer overflow is generally higher than storm runoff in its concentration of most pollutants, due to the higher concentration of these materials (BOD, nutrients, bacteria, etc,) in raw sewage. This generalization should, however, be borne out by site specific measurements since certain materials, such as suspended solids, are frequently higher in storm runoff than in domestic sewage. Monitoring programs for making evaluations of this type are discussed in Chapter 6.

2.3 Contaminants in Urban Runoff

The previous discussion of urban runoff sources describes some of the factors which influence the types and concentrations of contaminants which are present in storm related loads from an urban area. Materials transported by runoff constitute a problem when their discharge into receiving streams cause violations of water quality standards or limit legitimate water use. The definition of a problem may be either quantitative, as in the case of comparing measured or computed water quality with standards, or as is often the case, a qualitative description. This later type of problem definition often focuses on factors such as aesthetic concerns (floatables, turbidity, or surface slicks) or on quantitative measurements for which federal, state and local standards do not exist.

The contaminants in urban runoff generally fall into the seven categories itemized in Table 2-1.

TABLE 2-1 - CONTAMINANTS IN URBAN RUNOFF

1. Floatables and visual contaminants
2. Degradable organics
3. Suspended solids
4. Nutrients
5. Bacteria, virus
6. Toxicants
7. Dissolved solids

Each of these contaminant categories can, when present in sufficient quantities, contribute to water quality problems. Although stormwater related loads generally contain measurable amounts of materials in all seven classes, the total load may or may not constitute a water quality problem depending on the magnitude of the instream impact to the load. Table 2-2 lists some typical classes of water quality impacts which can result from urban runoff loads.

TABLE 2-2 - SELECTED INSTREAM IMPACTS

1. Aesthetic deterioration
2. Dissolved oxygen depression
3. Sediment deposition
4. Excessive algal growth
5. Public health threats
6. Impaired recreational value
7. Ecological damage
8. Reduced commercial value

The relationships between the contaminants generally found in urban runoff and the resulting instream impacts are discussed in the following paragraphs.

1. Floatables and Visual Contaminants - Aesthetic deterioration is caused either by the general appearance of water bodies (dirty, turbid, cloudy), or the actual presence of specific objectionable conditions, including odors, floating debris or films, scums or slimes, etc. These conditions may make the receiving water unattractive or repugnant to those in its proximity. Ecological problems might also result in fish, water fowl, or lower levels

of the food chain due to impairments of physiological functions.

2. Degradables Organics - Degradable organic materials stimulate the growth of bacteria which may consume oxygen more rapidly than it can be replenished by natural reaeration processes. This condition may or may not be visually apparent. In its extreme stages, excessive oxygen depressions can cause discoloration and the formation of gas and odors. However, before this extreme is reached, the environmental stress may be sufficient to cause respiratory damage to fish and other lower aquatic organisms. Species diversity shifts may result if conditions prevail for more than a few days. Similar conditions can occur due to the presence of reduced organic and ammonia forms of nitrogen which utilize oxygen as they are stabilized.

3. Suspended Solids - Particulate matter may contribute to a variety of problems, such as objectionable aesthetic conditions and the formation of sediment deposits which smother bottom dwelling aquatic organisms, impede navigation, and restrict river flows, thus increasing flooding potential. Organic sediment deposits can also react to form a benthal oxygen demand.

4. Nutrients - The discharge of materials which fertilize or stimulate excessive or undesirable forms of aquatic growth can create significant problems in some receiving water systems, particularly lakes and impoundments. Overstimulation of aquatic weeds or algae (eutrophication) can be aesthetically objectionable, cause dissolved oxygen problems, and in extreme cases, interfere with commercial and recreational uses by impeding small boat navigation, creating odors, and heavy mats of floating material at shorelines.

5. Bacteria/Virus Concentrations - The presence of excessive concentrations of objectionable microorganisms can impair the ability to utilize the receiving water for water supply, recreational purposes. or shell-fish harvesting. Excessive bacteria concentrations are generally taken as an indication of a potential public health problem.

6. Toxicants - Toxicity problems can fall into either of two categories: chronic bioinhibition or acute toxicity. Chronic effects may be exhibited by relatively low concentrations of metals, pesticides or persistent organics which tend to accumulate in the tissue of aquatic organisms over long periods of time. Their effects can be manifested at all levels of the food chain and may occur in areas well removed from the point of discharge. In excessive concentrations these materials, and others such as ammonia nitrogen and effluent chlorine bi-products, can exhibit acute toxic impacts in a local area surrounding a discharge. These effects are typified by fish kills or shifts in biological diversity.

7. Dissolved Solids - A number of beneficial uses can be impaired by excessive concentrations of dissolved solids. Both domestic and industrial water uses are sensitive to dissolved solids concentrations. Irrigated agricultural is quite sensitive to the salt content of the applied water. On a practical day to day basis, farmers must compensate for high salinity in irrigation water by increasing the quantity of irrigation flow. This imposes additional demands on water development and conveyance programs, and

further contributes to drainage problems. Effective agriculture can be destroyed if irrigation rates required to compensate for high salinity exceed the percolation and drainage capacity of the soil.

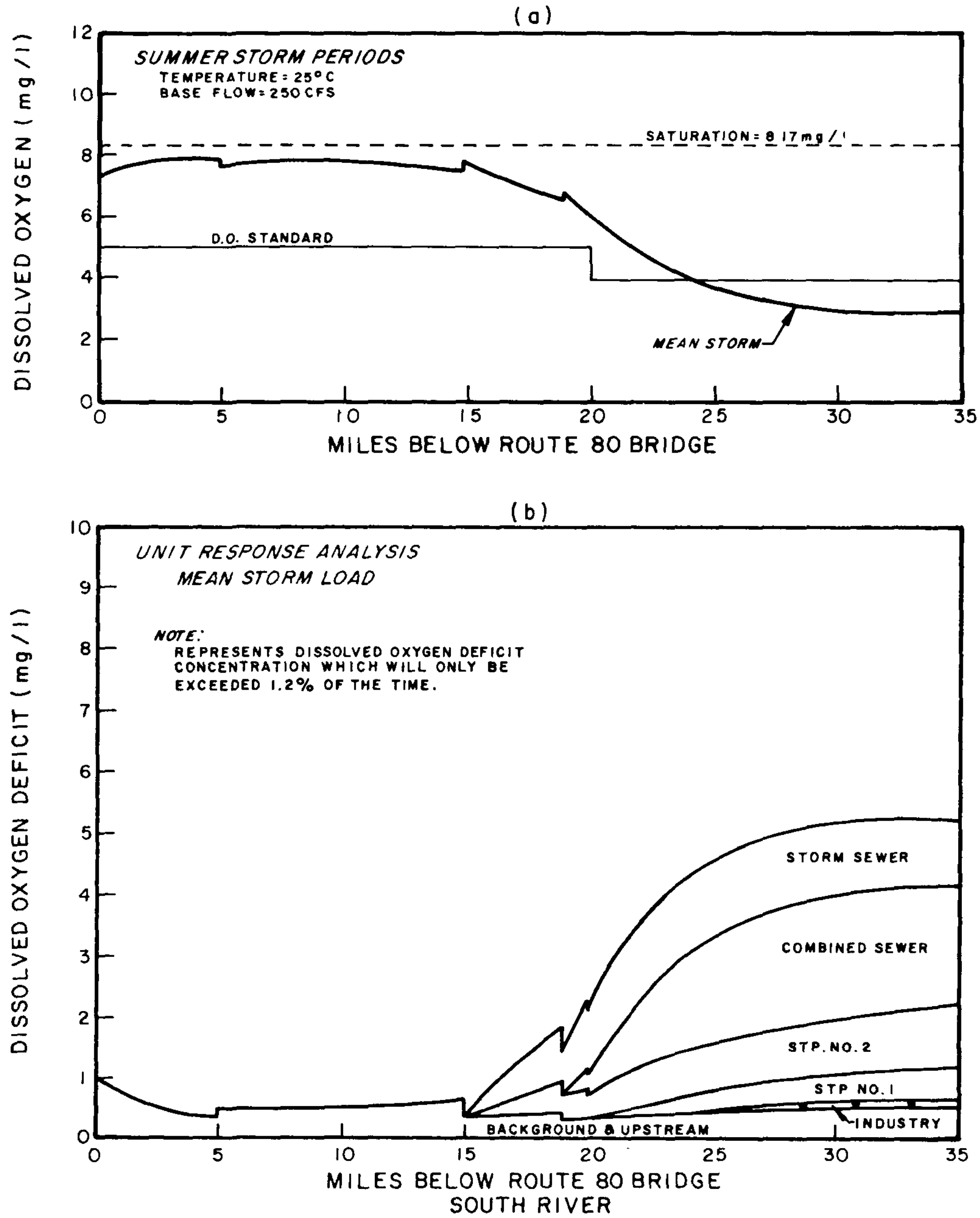
2.4 Water Quality Problem Definition

The previous section summaries the potential problems which can result from a variety of storm related water impacts. The severity of each problem is defined by the degree to which water use interferences occur. From this perspective, "the stormwater problem" can be defined only in terms of specific water quality impacts in a study area. Site specific water quality problems are related to specific contaminants, often originating from identifiable sources. Each contaminant, and possibly each of its sources, will respond differently to control measures. Therefore, effective control practices must concentrate on reducing the specific contaminants which contribute to identified water quality problems, and not those which have negligible or insignificant water quality impacts. Thus, effective control of a stormwater related problem requires an accurate definition of the problem.

The definition of a problem should begin with a stream walk for visual identification of conditions, and be followed by a quantitative of water quality impacts. This is often a difficult task, since water quality is also influenced by factors other than stormwater runoff and it is necessary to separate the existing water quality impact into its component parts. This can be accomplished using a mathematical analysis of the receiving water system. An illustration of such an analysis is shown in Figure 2-1.

Figure 2-1(a) shows the result of an analysis of urban point and non-point source impacts on dissolved oxygen in a hypothetical river. The dissolved oxygen example is used to illustrate one type of impact, and to show an approach for identifying the relative impact of storm loads versus input from other sources. Similar comparisons can be made for other contaminants, some of which may have more significant impacts than dissolved oxygen in specific cases.

The computed dissolved oxygen concentrations are given as a function of downstream distance. The dissolved oxygen saturation concentration, and a dissolved oxygen standard are also shown. This analysis is for the impact of a particular event, the mean summer storm, on the river. It indicates a violation of the 4.0 mg/l dissolved oxygen standard between Milepoints 24 and 35. The causes of the dissolved oxygen depression may be seen more clearly in Figure 2-1(b) which converts the dissolved oxygen concentration to the corresponding dissolved oxygen deficit value (amount below saturation) and displays the components of the dissolved oxygen deficit profile that make up Figure 2-1(a). It is apparent that the most significant factors contributing to the problem (violation of the water quality standard during the average summer storm) is the storm related loads from storm sewers and combined sewer overflows within the urban area. Point sources from two sewage treatment plants, and various industrial discharges contribute to the problem to a lesser degree. Similar analyses for other water quality indicators would yield similar insight into the causes of potential or actual problems.



NOTE.
REFERENCE [4]

FIGURE 2-1

TYPICAL ANALYSIS OF WET WEATHER DISSOLVED OXYGEN

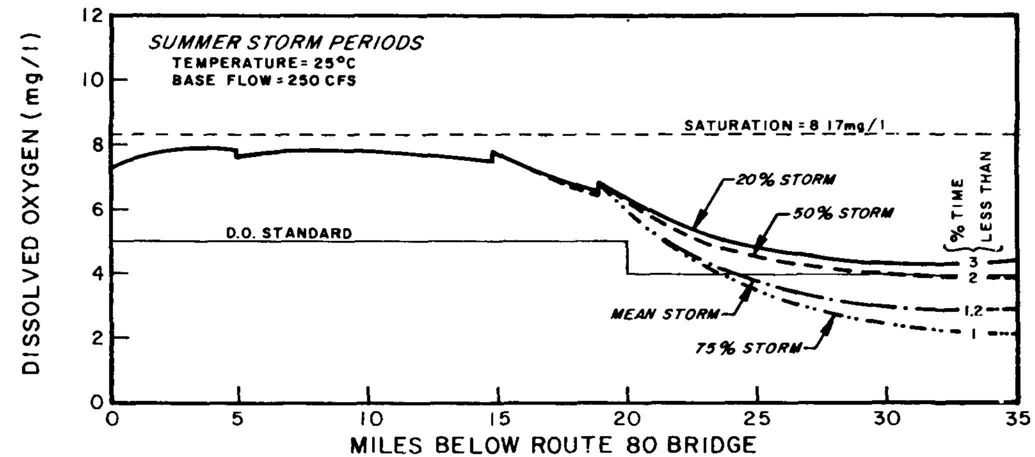
This type of analysis may be used to develop certain conclusions regarding storm related dissolved oxygen problems. For example, stormwater appears to be a major contributor to the problem, where the problem is defined in terms of a stream standard specifying dissolved oxygen concentrations to never be less than some value (in this example, 4.0 mg/1). Control of storm sewer flow and/or combined sewer overflow will yield the largest single improvement to the indicated problem. But there are other important questions which Figure 2-1 cannot answer, and which can have a significant bearing on the cost of controls required to protect the beneficial use which the stream standard is designed to preserve. These relate to: How often will stormwater events cause violations of standards? This is a final refinement to the definition of the problem which provides major additional insights.

Figure 2-2 presents these results for the same urban area indicated in Figure 2-1. In this case, however, the water quality is associated with storms having different frequencies of occurrence. This analysis provides additional insights into the problem. It indicates that the summer storm, shown in Figure 1, is a large storm; one which is exceeded only 35 percent of the time. The degree to which other storm frequencies impact summer water quality is also apparent. The analysis indicates that 50 percent of the storms are small enough not to cause water quality standard violations.

Furthermore, the analysis permits an assessment of how frequently dissolved oxygen concentrations fall below prescribed levels due to storm impacts. These frequencies are shown on the right side of the figure and include both dry and wet weather periods. For example, summer dissolved oxygen concentrations fall below the 4.0 mg/1 standard 2 percent of the time due to stormwater impacts. This analysis applies for the conditions selected to characterize the other significant elements of the analysis, principally the magnitude of other contributing waste loads and the stream flow assigned. However, where uncertainty exists in the selection of these factors, additional calculations of the same type can be made to provide the desired understanding and perspective on the problem. For example, the analysis could be repeated for a different stream flow, selected such that the two analyses would bound the range of possible responses.

An alternative method of presenting probabilistic water quality results is shown in Figure 2-3, reproduced from the "Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume II, Cost Assessment and Impacts" (5), which shows the result of wet weather mathematical modeling studies for the Des Moines River. The computed minimum dissolved oxygen concentration during wet weather periods is indicated as a function of the storm frequency. The components of the dissolved oxygen impact are also indicated. Storm related loads from separate runoff and combined sewer overflows are shown to be the primary source of the dissolved oxygen deficit during runoff periods. Under existing conditions, the model indicates that the minimum dissolved oxygen concentration falls below the desired level (4.0 mg/1) following 42 percent of the runoff events.

The planning question which relates to problem definition at this point is: does violation of the standard 2 percent of the time (from Figure 2-2) or following 42 percent of the storms (in the case of Figure



LEGEND.

20% STORM = 20% OF STORMS HAVE A LOWER RUNOFF RATE

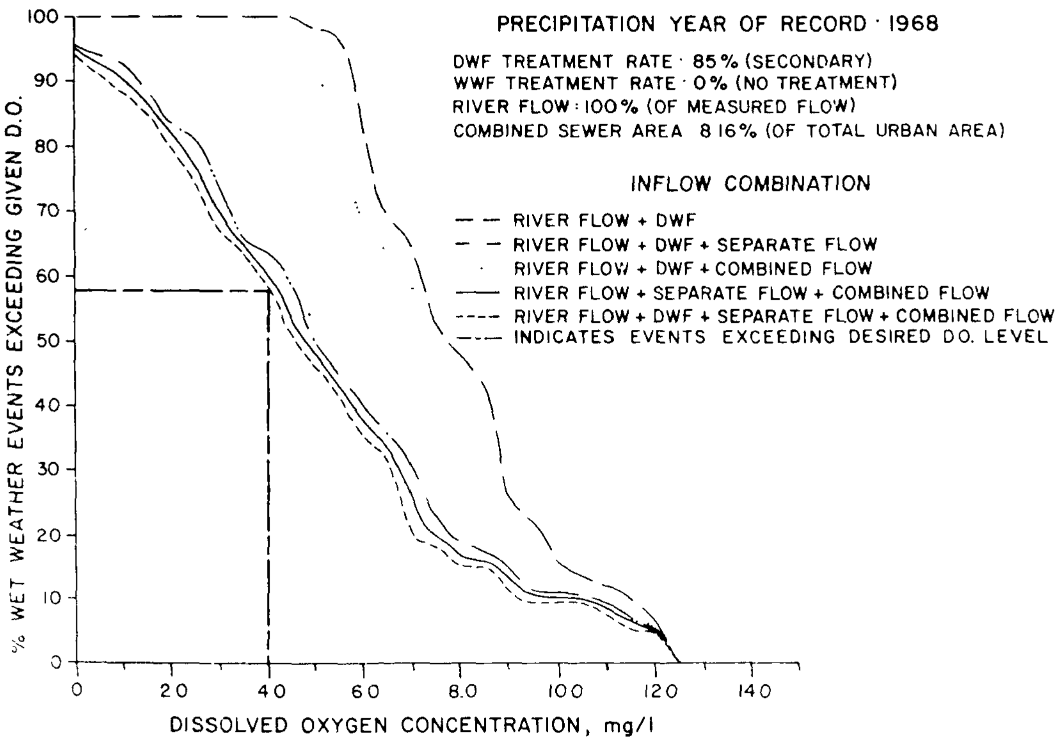
3% OF TIME LESS THAN - STREAM DISSOLVED OXYGEN CONCENTRATION WILL BE BELOW INDICATED VALUE 3% OF THE TIME

NOTE:

REFERENCE [4]

FIGURE 2-2

PROBABILISTIC ANALYSIS OF WET WEATHER DISSOLVED OXYGEN



NOTE:
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FIGURE 2-3

MINIMUM DISSOLVED OXYGEN FREQUENCY CURVES FOR EXISTING CONDITIONS IN THE DES MOINES RIVER

2-3) constitute a water quality problem? This type of question is at the heart of meaningful analyses of stormwater related problems and will be discussed in detail in subsequent chapters. Those discussions will focus on the frequency analysis of stormwater loads, treatment, and receiving water impacts. For the purposes of this section a more detailed discussion of the factors influencing the definition of water quality problems is appropriate.

The principal factors involved in defining stormwater problems have, thus far, been related in a very general way to water quality. There are, however, a series of factors which must be considered for an adequate review of the potential for water quality problems in a site specific urban setting. These are:

1. Relevant time and space scales of the problem
2. Water use objectives and criteria
3. Characteristics of the particular study area

These factors are discussed below.

2.4.1 Relevant Time and Space Scales

A basic step in identifying potential water quality problems is the definition of the time and space scale of the problem. How large an area might be affected by a particular class of contaminant? How long does it take for the problem to become manifest and over what interval will the impact be felt?

The definition of proper time and space scales is important in a number of stormwater problem analysis tasks. The time and space scale of a potential problem will determine the time and space scale of the models or other analyses which are used to address it. Short term transient phenomenon are analyzed using different analysis frameworks than those used to evaluate longer term steady state buildups of less reactive contaminants.

Time and space scales of receiving water impacts are also important in the determination of effective monitoring programs and for the characterization of waste loads. If a particular problem is due to continuous point source loads, the monitoring program which addresses it will be entirely different than a monitoring program evaluating stormwater loadings to the same river.

2.4.1.1 Nature of Contaminants

Each of the classes of water quality contaminants in Table 2-1 and the potential problems in Table 2-2 must be viewed in a specific time and space scale. For example, bacterial contamination is particularly relevant in the time scale of a few days and generally occurs in a localized area. This is due to the high rate of decay of coliform bacteria in natural water systems which typically result in their reduction to background levels within a few days. The space scale over which they are relevant is similarly small and is governed by the distance that they are transported before they die-away. By way of contrast, toxic substances such as pesticides, persistent organics, and heavy metals are normally viewed in longer time scales, and space scales which may extend many hundreds of miles. These substances tend to be persistent (i.e., they do not readily decay in the environment). The time scale in this case may be as large as decades, and contaminants can have effects at locations which are remote from their point of introduction into the environment.

Certain contaminants may fall into multiple time and space scales. For example, the persistent toxicants discussed above may exhibit acute toxic effects in the immediate vicinity of discharge if local concentrations exceed tolerable limits of sensitive aquatic species. The appropriate time and space scale chosen for the analysis should reflect judgement based on local conditions, the type of water body, particularly sensitive areas, local transport conditions, and types of waste loads. A guideline in developing the proper time and space scales for suspected water quality problem is contained in Figure 2-4 and 2-5.

2.4.1.2 Nature of the Receiving Water System

The classes and appropriate time and space scales of storm related water quality problems in a particular urban setting is often related to the nature of the receiving waters. Streams and rivers are particularly sensitive to intermittent short term discharges (storm loadings) because the mass is transported as an identifiable pulse with relatively little dispersive mixing. Sensitive downstream locations, such as water intakes or bathing areas, may be affected by these identifiable pulses. Other systems such as estuaries tend to spread and dilute the impacts of storm loads, and short term temporal definition may not be necessary.

Specific types of contaminants are assimilated differently in different water bodies. For instance, sediment inputs may contribute to high suspended solids concentrations in a swiftly moving river. Classes of water quality problems which are important in this case are normally related to aesthetic concerns or interferences with physiological functions of aquatic organisms. In estuaries and lakes where velocities are not as great the sediment problem is largely due to deposition which causes solids buildup in navigable channels or increased organic bottom activity. Similarly, nutrient enrichment is normally more critical in lakes of long detention times than in free flowing streams. In lake systems the problem is often related to discoloration, floating algal mats, and depressed oxygen levels, while in streams and rivers aquatic weeds may be the major problem.

2.4.2 Water Use Objectives and Criteria

Water quality problems should be defined in terms of their limitation of beneficial uses of the water body. Thus, an inventory of present and planned water use is an integral part of the planning process. An example inventory of beneficial water uses is presented in Table 2-3.

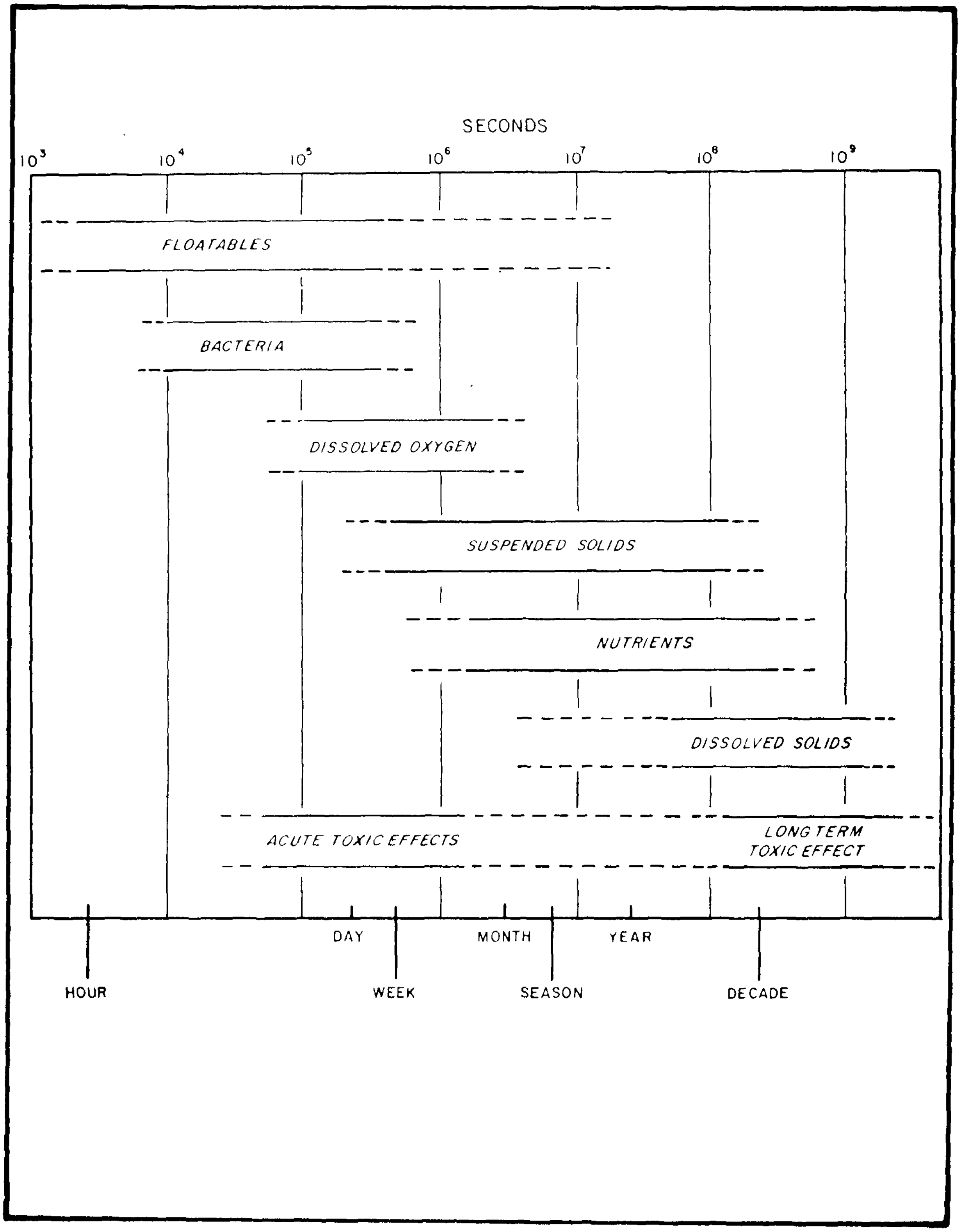


FIGURE 2-4
TIME SCALES
STORM RUNOFF WATER QUALITY PROBLEMS

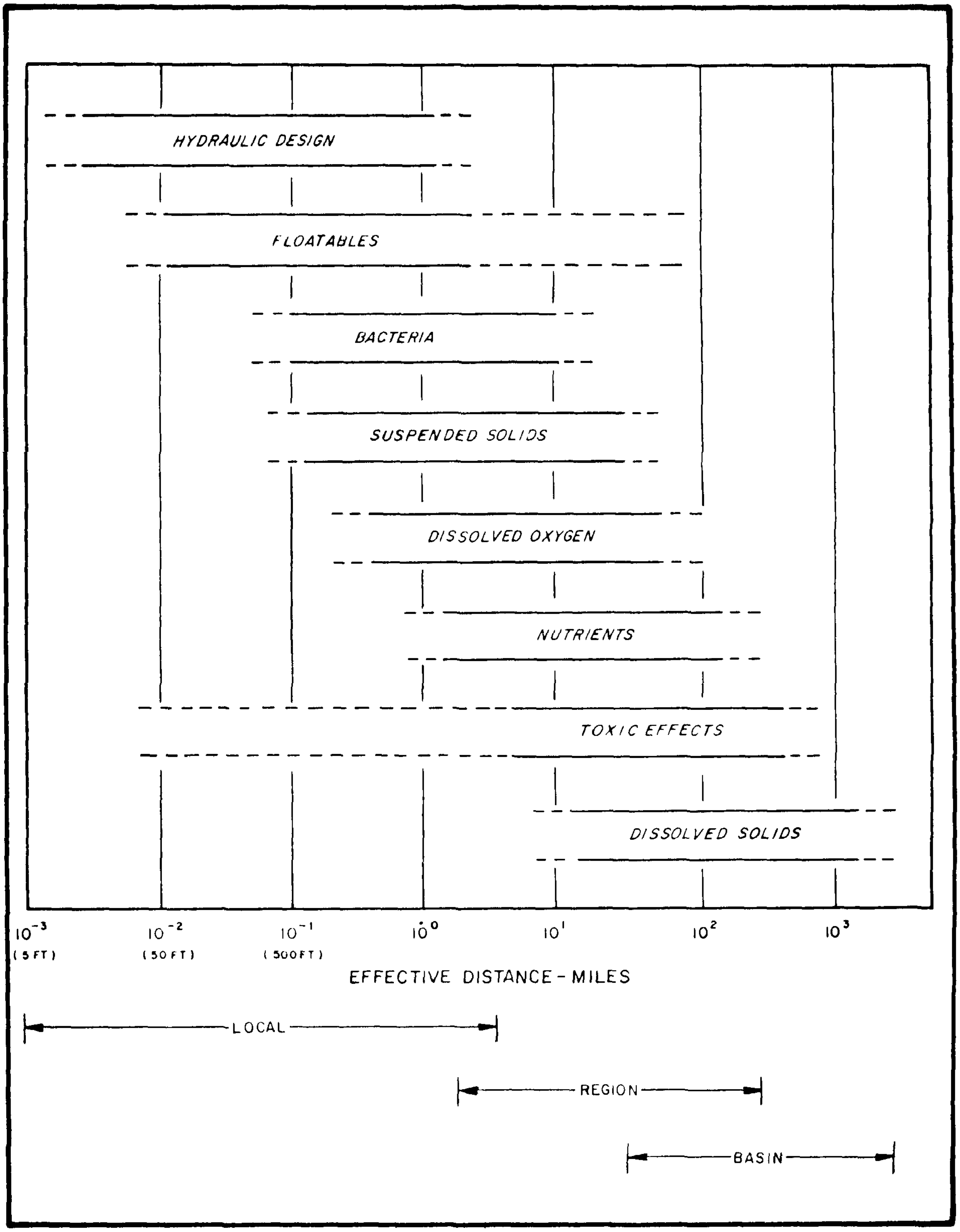


FIGURE 2-5
SPACE SCALES
STORM RUNOFF WATER QUALITY PROBLEMS

TABLE 2-3 EXAMPLE BENEFICIAL WATER USE INVENTORY

1. Water Supply
   a. Domestic Water Supply
   b. Industrial Water Supply
   c. Irrigation
   d. Livestock and Wildlife Use

2. Maintenance and Propagation of Fish and Other Aquatic Life
   a. Commercial Fisheries
   b. Recreational Fishing
   c. Aesthetic Value

3. Recreation
   a. Water Contact Sports, Swimming
   b. Boating
   c. Aesthetic Value

4. Navigation

5. Power Generation

6. Transport and Assimilation of Treated Wastes

Each of these should be screened for potential impacts early in the planning process. The protection and enhancement of these beneficial water uses is the primary goal of any stormwater control plan.

Water quality criteria and standards should be developed to protect specific beneficial water uses. Comparing current or projected pollutant concentrations in the receiving water with regulatory standards thus provides a basis for defining the stormwater problem in terms of beneficial water use limitations.

2.4.3 Characteristics of Particular Study Area

Planning studies are particularly meaningful when their scope goes beyond the "standard" dissolved oxygen and suspended solids problems, and explores other potential problem classes which are important in terms of water use in the study area. This does not mean that every potential water use interference must be explored in detail. But, the potential for significant water use limitations should at the very least be screened.

In this regard the planner must think beyond documented problems in their study area and identify areas where storm loads might contribute to as yet undocumented problems. These can include concerns related to oils and greases, heavy metals, pesticides, herbicides, carcinogens, and viruses. In identifying the potential for water use interferences in any of these classes two factors are normally evaluated: 1) existing or potential water uses which might be effected, and 2) potential sources of the contaminant. Local knowledge of the study area is required for both evaluations. For example, downstream water supplies suggest a class of potentially harmful contaminants.

A review of upstream study area characteristics will indicate whether storm loads could potentially contribute harmful contaminant loads. Agricultural areas are a potential source of pesticides and herbicides. Landfill leachate is a potential source of heavy metals and oils and grease. Combined sewer overflows are sources of viruses. Any one of these areas may be a potentially important contaminant source. Crude but conservative estimates of loads from each source, using local data where possible, and simple mass balance computations are generally sufficient to determine the potential for water quality problems from each of these sources. Where these simple estimates indicate a high problem potential, more detailed analyses using sampling data will serve to further document the problem.

Planning develops sequentially and planners must continually expand their technical base of information in order to provide input to the water resource management decision process. The current trend towards higher water quality standards and increased water use will undoubtedly continue. Planning studies must therefore be responsive to this increased demand.

2.5 References

1. Field, Richard, Anthony N. Tafuri, Hugh E. Masters, *Urban Runoff Pollution Control Technology Overview*, U.S. Environmental Protection Agency, EPA-600/2-77-047, Cincinnati, Ohio, March 1977.

2. Brandstetter, Albin, *Assessment of Mathematical Models for Storm and Combined Sewer Management*, Battelle, Pacific Northwest Laboratories for U.S. Environmental Protection Agency, EPA-600/ 2-76-175a, Cincinnati, Ohio, 1976.

3. Lager, John A., W.G. Smith, W.G. Lynard, R.M. Finn, E.J. Finnemore, *Urban Stormwater Management and Technology: Update and Users Guide*, Metcalf Agency, EPA-600/8-77-014, Cincinnati, Ohio, September 1977.

4. Hydroscience, Inc., *208 Areawide Assessment Procedures Manual*, Volume I, Chapter 5, U.S. Environmental Protection Agency, EPA-600/9-76-014, Cincinnati, Ohio, July, 1976.

5. Heaney, James P., et al., *Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume II: Cost Assessment and Impacts*, University of Florida for U.S. Environmental Protection Agency, EPA-600/2-77-064, March, 1977.

CHAPTER 3

THE STATISTICAL METHOD FOR THE ASSESSMENT OF RUNOFF AND TREATMENT

Once site specific urban stormwater problems are identified, suitable analysis techniques are required for evaluating the magnitude of the receiving water impact and the effectiveness of alternative control options. This task is particularly complex because of the random nature of the problem's principal forcing function, the rainfall-runoff process.

Chapter 3 explores a particularly effective methodology for evaluating urban stormwater problems. It is the result of a growing awareness on the part of individuals involved in stormwater impact analysis that the real benefits in stormwater control can only be determined by evaluating the long term response characteristics of treatment devices and receiving water quality (1,2,3,4,5,6). The techniques discussed in this chapter accommodate this need by analyzing storm related problems in terms of the long term statistical properties of rainfall, runoff, treatment performance and receiving water quality. The analysis produces results which address critical urban stormwater questions such as: How often will stormwater runoff cause water quality and water use objectives to be violated? What is the long term performance efficiency of various control options? What are the marginal costs associated with alternative water quality objective sets? Subsequent chapters will develop analytical techniques for evaluating model coefficients in a specific urban setting. These incorporate monitoring program design as well as various numerical estimates.

The predominant analytical tool currently used to evaluate stormwater problems is rainfall-runoff simulation. The emphasis in simulation models has evolved from the more detailed description of a particular storm event (7,8,9,10) towards the more general analysis of long term stormwater impacts with "continuous versions" (4,5,11,12). Examples of these more general, long term simulators are discussed in Chapter 4.

Recently, analytical methods which utilize the statistical properties of rainfall and runoff to investigate stormwater problems have been developed (2,13,14,15). This chapter presents a general methodology for assessing stormwater problems analytically, without a computer based simulation model. The method requires minimal computer use and can be implemented with an electronic calculator. Both the statistical method and computer simulation models are analysis tools which involve simplifying assumptions about the mechanisms of the real stormwater process. Many of the assumptions are the same in both approaches, and as demonstrated in Section 3.6.7.7, the results are similar. The primary advantage of the statistical method is that it allows a relatively quick, simple, and inexpensive screening of stormwater problems and a wide range of control alternatives. This is particularly useful in the early stages of the planning process. The use of the techniques and curves of the statistical method gives planners a better understanding of their stormwater problems. Insight is gained by following the problem through, step by step, rather than developing a list of input factors, running a complex simulation model, and receiving the final output.

The statistical method is particularly well suited to assessment studies where the principal issues involve broad planning questions such as: What are the major loads contributing to the problem? What treatment devices are particularly effective? What are the levels of costs associated with alternative controls or alternative water quality objectives? While the statistical method may also have utility in more detailed siting studies, such as 201 facilities plans, it is often advisable to supplement the statistical method with additional detailed modeling using one of the more sophisticated stormwater simulators. Refined simulation is particularly useful at the design stage, where the planner is interested in the behavior of control alternatives and the response of the receiving water during a specific sequence of critical storms. In this respect, the statistical method and simulators should be viewed as complementary, each appropriate at different stages in the analysis and each providing insight and direction for the use of the other.

Chapter 3 presents a sequential development of the statistical method. The chapter begins with a discussion of the random nature of storm events and provides the basic framework for the characterization of the rainfall-runoff process. Stormwater loads are developed through the analysis of rainfall, runoff quantity, and quality. Methods for determining receiving water impacts are presented, and techniques for assessing the benefits of stormwater control alternatives are developed.

3.1 Storm Runoff Events as Random Occurrences

The most basic source of variability in the rainfall-runoff process is the fact that sometimes it is raining, and sometimes (usually) it is not. The rainfall-runoff process consists of a series of events occurring randomly in time. A more precise definition of the random nature of storm event occurrences is presented in this section.

Consider the rainfall event process and let T be a fixed length of time, e.g., one month. For each successive period of length T, let n be the number of events which occur in that period. Since the occurrence of rainfall is a probabilistic event, the number of occurrences during a fixed length of time is a random variable. The critical assumption which allows a comparatively straightforward analysis is that n is a Poisson random variable with discrete probability density function:

$$f_n(n) = \frac{(T/\Delta)^n \, e^{-T/\Delta}}{n!} \quad n = 0, 1, \ldots \qquad (3\text{-}1)$$

with parameter Δ. Further, n is assumed to be an independent random variable, that is, the number of rainfall events in any period of length T is independent of the number that occurred in any other period. A direct consequence of this assumption is that the storm interval, δ (measured from the temporal midpoints of the events), is a random variable with an exponential probability density function. To see this, consider the probability that $\delta \leq T$, that is, that at least one rainfall event occurs during the period T:

$$\Pr(\delta \leq T) = 1 \quad \Pr(\delta > T) \tag{3-2}$$

and

$$\Pr(\delta > T) = \Pr(n = 0) = e^{T/\Delta} \tag{3-3}$$

since if $\delta > T$ no rainfall event has occurred within the period T and $n = 0$. Therefore, the cumulative distribution function of the random variable δ is

$$\Pr(\delta \leq T) = 1 \quad e^{-T/\Delta} \tag{3-4}$$

so that δ has an exponential probability density function:

$$p_\delta(\delta) = \frac{1}{} e^{-\delta/\Delta} \qquad \delta \geq 0 \tag{3-5}$$

Further, the average value of δ is:

$$\bar{\delta} = \int_o^\infty \frac{\delta}{\Delta} e^{-\delta/\Delta} \, d\delta = \Delta \tag{3-6}$$

The parameter of the Poisson density function is thus seen to be the average time between storms.

This characterization of the rainfall events as a Poisson process is the most convenient available and is also surprisingly realistic. Analysis of rainfall records (Chapter 5) indicate that the storm intervals is well described by the exponential distribution and that the storm event definition (number of consecutive dry hours needed to terminate an event) may be chosen to more accurately approximate an exponential distribution for δ. The assumption that the interval between two storms is independent of the interval between any other two storms is also quite reasonable.

3.2 Characterization of Runoff Events

The storm runoff process may now be characterized as in Figure 3-1 as a series of independent events occurring randomly in time. The intrastorm variability depicted in Figure 3-1(a) is ignored for the time being and each event is characterized in Figure 3-1(b) by its duration (d_r), runoff volume (v_r), time since the previous storm (δ) (defined from the midpoints of the successive storms), and the average runoff flow ($q = v_r/d_r$). The transformation from rainfall to runoff is not yet directly addressed. Rather it is assumed that the relevant runoff characteristics are available either from direct observations, or from suitable modifications of the rainfall record. This is discussed in more detail in Section 3.2.3.

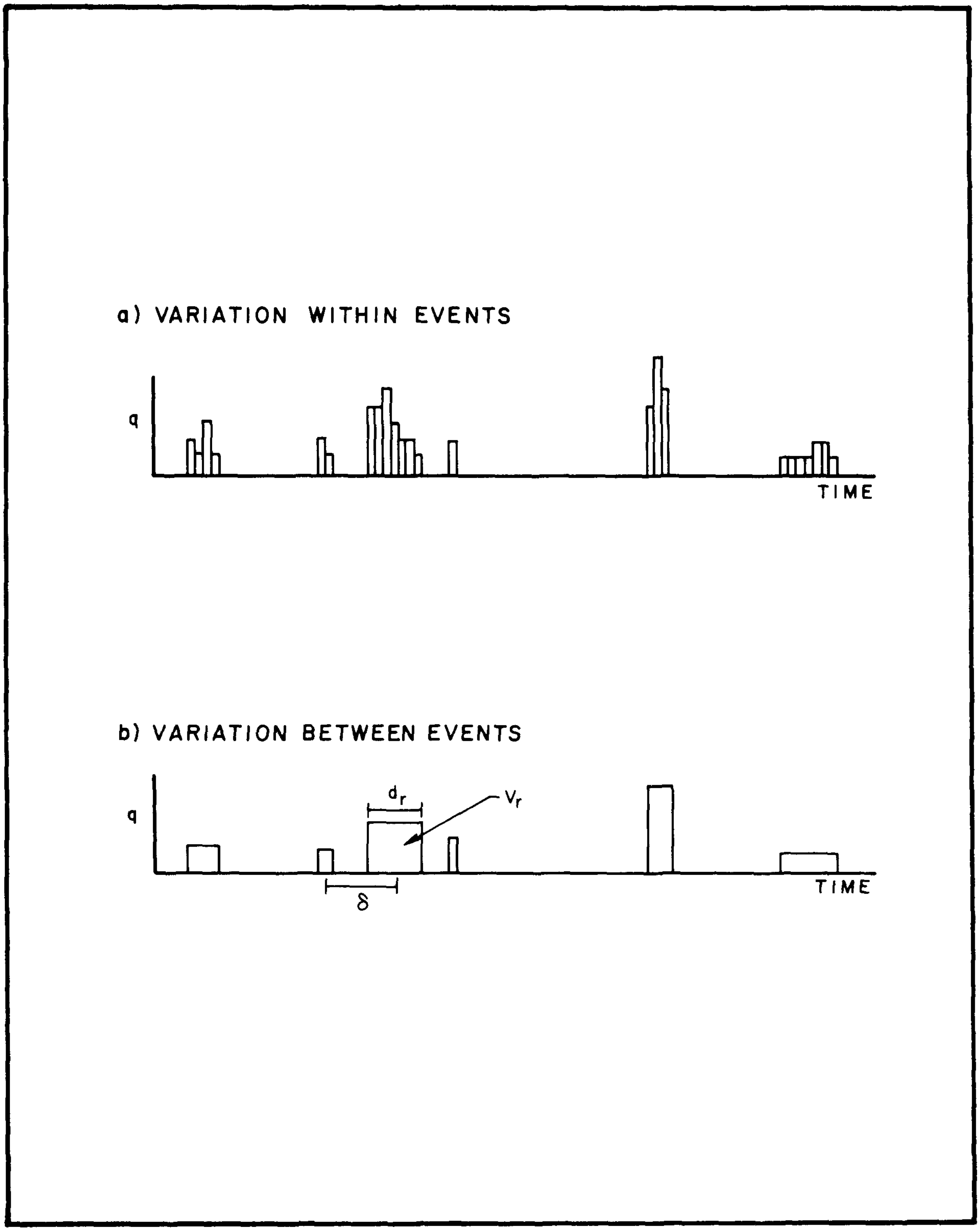


FIGURE 3-1
REPRESENTATION OF STORM RUNOFF PROCESS

3.2.1 Statistical Properties of Runoff Parameters

The runoff event characteristics may be treated as random variables, each with an associated probability density function. For a given period of record, the mean and coefficient of variation (standard deviation divided by the mean) of each variable are calculated. The required statistics are summarized in Table 3-1.

TABLE 3-1

STATISTICS FOR STORM CHARACTERIZATION

Parameter	For Each Storm	Mean	Coefficient of Variation
Runoff Flow	q	Q_R	ν_q
Duration	d_r	D_R	ν_d
Runoff Volume	v_r	V_R	ν_R
Time Between Storms	δ	Δ	ν_δ

Storm flows and durations are assumed to be independent and gamma distributed. A gamma distribution is defined by a mean, in this case, Q_R and D_R; and a coefficient of variation ν_q and ν_d. The probability density function for runoff flows is then:

$$p_q(q) = \left(\frac{\kappa}{Q_R}\right)^{\kappa} \frac{q^{\kappa-1}}{(\kappa)} \exp(-\kappa q/Q_R) \qquad (3\text{-}7)$$

in which $\kappa = 1/\nu_q^2$. A similar expression describes the probability density function of storm durations. The gamma distribution is effectively a more generalized version of the exponential distribution. The gamma distribution allows the coefficient of variation to vary; the exponential distribution is a special case of a gamma distribution in which the coefficient of variation is equal to 1.

As described in the previous section, the probability density function of the time between storms (δ) is well described by an exponential distribution; that is, the coefficient of variation is approximately equal to one. If this is not the case, the gamma distribution may also be used to describe the time between storms.

The runoff volume of a particular event is equal to the product of the runoff flow and the duration. The runoff volumes may thus be represented by the product of two independent, gamma distributed, random variables. If this were completely true, the mean runoff volume, V_R, would equal the product of the mean runoff flow and mean runoff duration ($V_R = D_R Q_R$). This may not always be valid, however, particularly in areas where shorter, intense summer storms and longer, less intense winter and spring showers are typical and runoff flows and durations are not completely independent. Thus the statistics of runoff volumes must be obtained in addition to those on flows and durations. In fact, the assumption that storm runoff volumes are themselves gamma distributed is probably quite reasonable.

The cumulative distribution function for the gamma distribution is shown in Figure 3-2. Figure 3-2 is used to determine the percent of storms with runoff characteristics less than or equal to the given value. For example, if the variation of storm runoff flows is $\nu_Q = 1.25$, from Figure 3-2(a) the 90th percentile runoff flow is 2.6 times the mean runoff flow, Q_R. In other words, ten percent of the storms have average runoff flows greater than 2.6 Q_R. Interpolation may be used for intermediate values of the coefficient of variation. Note that when the coefficient of variation (ν) is large, events with very small and very large values relative to the mean become more likely. That is, there is more spread around the mean in the probability density function.

Once the percent of storms larger than a given value is determined, the expected number of storms greater than a given value may be estimated. The average number of storms occurring during a given period is first calculated:

$$\text{Average number of storms} = \frac{\text{Length of Period}}{\Delta} \qquad (3\text{-}8)$$

Then, for example, if the period of interest is one year and the statistical analysis indicates that the average time between storms, Δ, is 70 hours:

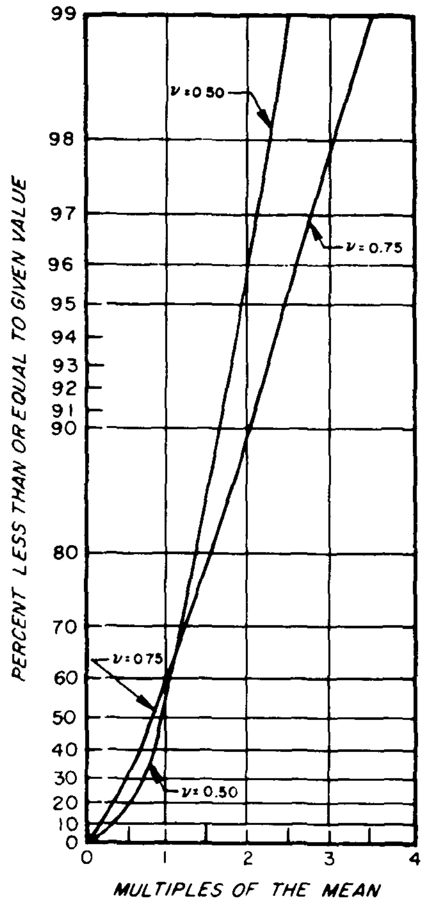
$$\text{Average number of storms} = \frac{1 \text{ year} \cdot 8766 \text{ hr/year}}{70 \text{ hr}} = 125$$

The expected number of storms greater than a given value is then the fraction of storms greater than the given value times the average number of storms. From the previous example, there will be (on the average) (0.10)125 = 12.5 storms per year with average runoff flows greater than 2.6 Q_R.

3.2.2 Long Term Runoff Process

Once the expected number of events exceeding a given criterion is estimated, the final step in the statistical characterization is the determination of how often this will occur during the entire interval of interest, including both the rain and nonrain periods. This estimate is most strongly influenced by the fact that it is usually not raining.

The transformation from a probability distribution function for storm events to a probability distribution function for both rain and non-rain periods is depicted in Figure 3-3. The fraction of the time during which storm runoff is occurring is estimated as D_R/Δ. The remaining portion of the time (generally between 70 and 98 percent) there is no storm runoff. The



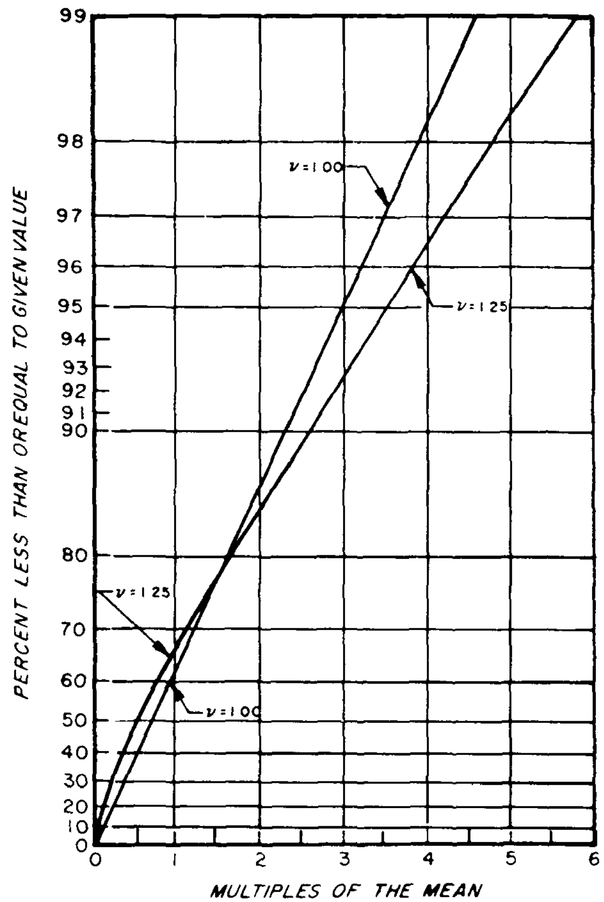


FIGURE 3-2(a)
CUMULATIVE DISTRIBUTION FUNCTION FOR GAMMA DISTRIBUTION

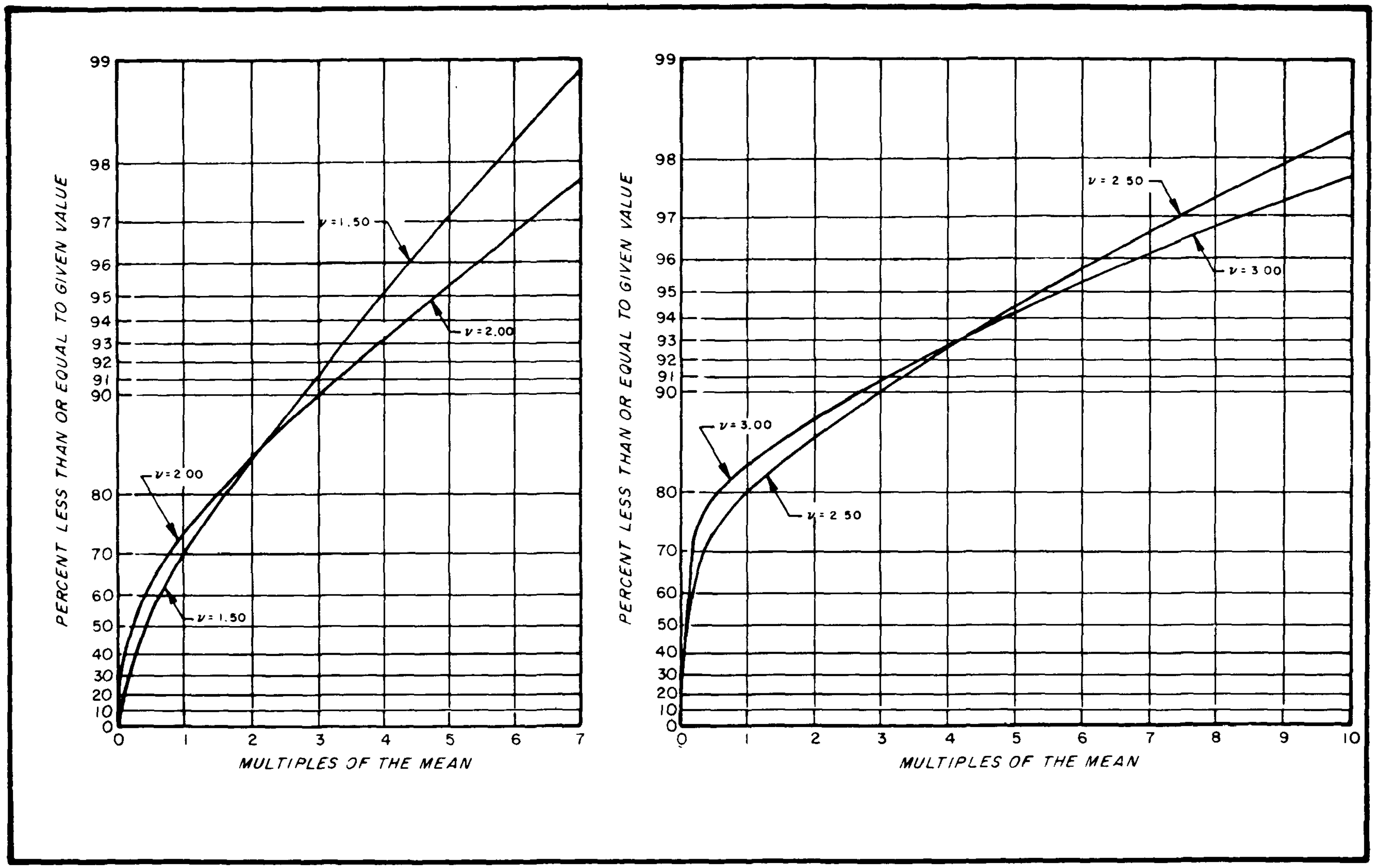


FIGURE 3-2(b)
CUMULATIVE DISTRIBUTION FUNCTION FOR GAMMA DISTRIBUTION

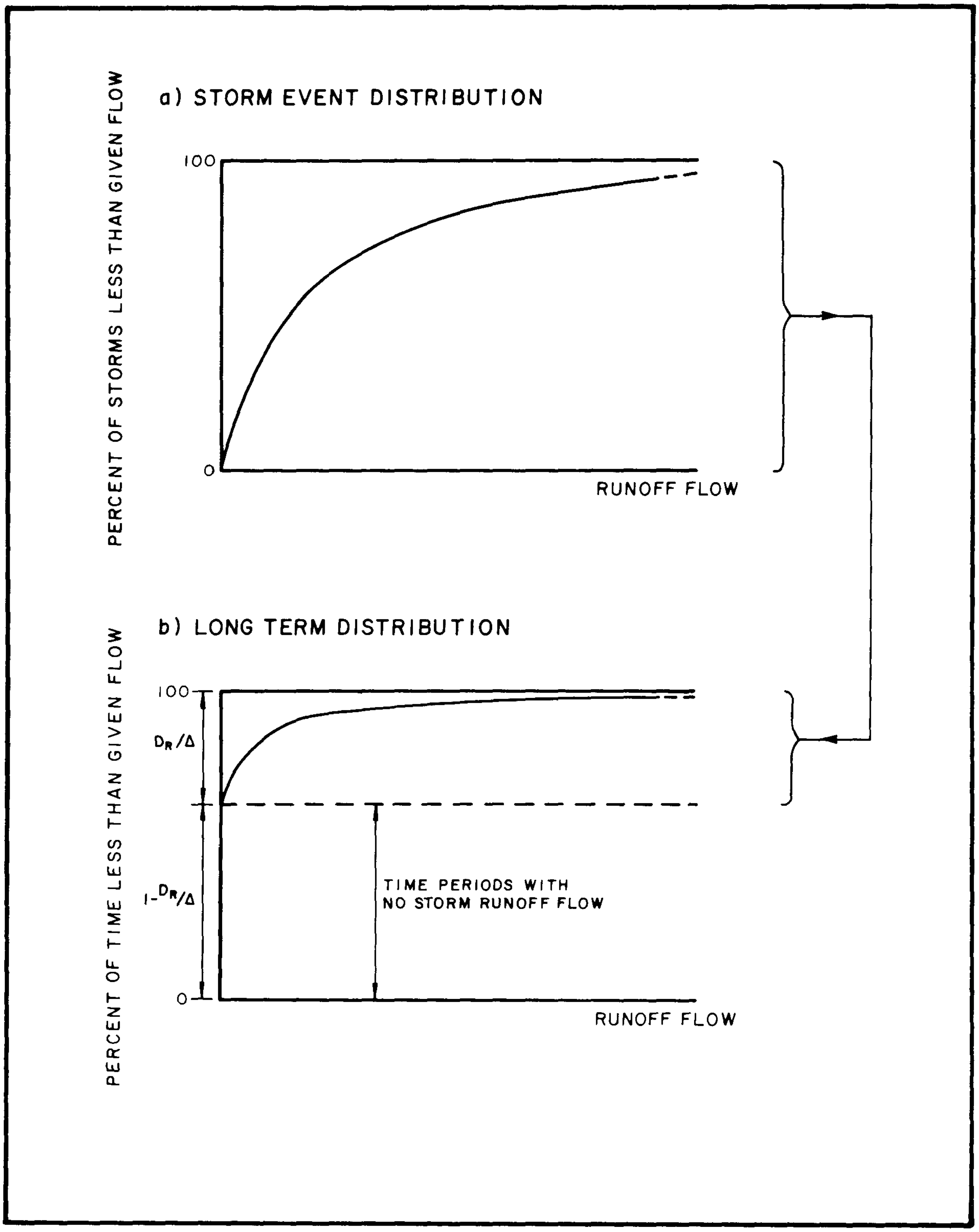


FIGURE 3-3

STORM EVENT AND LONG TERM FREQUENCY DISTRIBUTIONS probability of a given runoff flow being exceeded is the probability that a storm will have an average runoff equal to or greater than the given flow times the fraction of the time that runoff occurs. Given the assumption that storm runoff flows are gamma distributed, the percent of time (including rain and nonrain periods) when runoff flows exceed a given flow may be determined from Figure 3-2 by knowing Q_R, ν_q, and D_R/Δ. For example, assuming D_R = 8 hours, Δ = 70 hours, Q_R = 4.0 cfs, and ν_q = 1.25; Figure 3-2(a) indicates that 10 percent of the storms have average runoff flows greater than 2.6 Q_R = 10.4 cfs. The fraction of time when average runoff flows exceed 10.4 cfs then equals:

$$0.10\ (D_R/\Delta) = 0.10\ (8/70) = 0.0114$$

In other words, considering the entire time history, runoff flows are estimated to exceed 10.4 cfs about 1.1 percent of the time.

The mean and variance of the long term runoff process shown in Figure 3-3(b) may be calculated from the generalized Cambell's Theorem for random (i.e., Poisson) pulse processes (17,18). The long term average runoff flow and its coefficient of variation (including rain and non-rain periods) are:

$$Q_o = Q_R\ (D_R/\Delta) \tag{3-9}$$

$$\nu_{qo} = \sqrt{\frac{\nu_q^2 + 1}{(D_R/\Delta)}} \tag{3-10}$$

Note that the long term average runoff flow is calculated by assuming that the storm runoff is effectively spread out over the entire time history, and Q_o is thus smaller than Q_R. The coefficient of variation of the long term process (ν_{qo}) is larger than the variation of storm flows (ν_q) due to the many increments of time when runoff flows equal zero. The smaller the fraction of the time runoff is occurring (D_R/Δ), the smaller is the long term average flow (Q_o), and the larger is the variability of the long term process (ν_{qo}).

3.2.3 Determination of Runoff Parameters

The basic theoretical framework for analyzing the statistical properties of runoff events has been presented. The actual method for determining the runoff characteristics (i.e., the runoff volume, duration, etc., for a series of storms) has not yet been addressed. A number of approaches may be taken to determine runoff event characteristics. The most direct method is to monitor the runoff flows each storm for a period of interest. Although a monitoring program is useful and necessary for certain aspects of the study plan, time and budgetary constraints limit the applicability of monitoring for a complete long term characterization, and estimating techniques are required.

Rainfall is the primary driving force in the generation of stormwater runoff. Analytical models may be used to transform rain event characteristics to runoff event parameters. These may range from simple models using a direct linear conversion of the precipitation falling over an area to its runoff; to sophisticated models employing varying infiltration rates, depression storage, overland flow and flow routing through the conveyance system.

A simple method for converting rainfall records to runoff records is presented in the following sections. The techniques for evaluating the long term statistical properties of the stormwater induced receiving water responses and the long term efficiency of control alternatives are not dependent upon the use of these simple rainfall - runoff conversions. More sophisticated models may be appropriate, particularly during the later stages of the planning study. The general statistical methodology is flexible, and more refined estimates may be used at different stages in the analysis, depending upon the specific problem setting and the data available.

3.3 Rainfall, the Driving Force

Precipitation is the driving force in the generation of stormwater runoff and its associated pollutant loadings. Precipitation may occur as rainfall, snow, or hail; and the characteristics of the resulting runoff are very different, depending upon which of these types of precipitation has occurred. Stormwater impacts generated by rainfall are the primary focus of this manual. The techniques presented in this manual may be suitable for evaluating impacts caused by snowmelt on a long term (i.e., yearly) time scale, however, more sophisticated modeling techniques are required for a more detailed analysis (19,20,21). Rainfall and snow impacts may be separated by a seasonal assessment, aided by the monthly characterization of precipitation described later in this section.

Rainfall occurs as a series of random events, with the amount of precipitation varying spatially as well as temporally. The approach for analyzing rainfall presented in this section is directed towards a long term characterization of rainfall at a point, i.e., a raingage. A discussion of the areal variability of rainfall, and techniques for transforming point to areal rainfall are presented in Section 5.1.4.

The method used to describe point rainfall is based on a statistical characterization of event properties for a long period hourly rainfall record. Such records are collected by the U. S. Weather Bureau at weather stations through the United States, and are available on cards or magnetic tape through:

The National Climatic Center
National Oceanic and Atmospheric Administration
Federal Building
Asheville, North Carolina 28801

The length of rainfall record required to adequately define the statistical variability of the rainfall record is generally 20 to 30 years. Shorter records may contain atypical numbers of dry or wet years, but may be used if longer data records are not available.

The development of rainfall event statistics begins with a grouping of the discrete hourly records into a series of storm events. Table 3-2 demonstrates the implied transformation. Table 3-2(a) shows one month of hourly rainfall data from Minneapolis, Minnesota. Only days with some precipitation, and the first day of the month, are shown. The characteristics of the events for this period are shown in Table 3-2(b). The storm duration is calculated from the first hour of rainfall until the last hour of rainfall which is followed by 6 consecutive dry hours. The number of consecutive dry hours required to end a storm event is the basic parameter which determines how the hourly records are grouped into storms. Considerations for selecting this parameter in a particular study area are presented in Section 5.1.2. The intensity is the average intensity during the storm and the volume (also referred to as the depth) is the total storm volume in inches. The interval between storms (calculated from the temporal midpoint of the previous storm) is also shown. This analysis thus characterizes each storm for the period of record.

The event characteristics are statistically analyzed for all storms or for groups of storms in the period of record. These statistics are used to evaluate the long term properties of storm events in the urban area. The mean and coefficient of variation of each of the event characteristics are calculated. This is accomplished using a synoptic rainfall analysis which has been described in the Areawide Assessment Procedures Manual (2). A typical output from such an analysis is presented in Figure 3-4 which displays the rainfall statistics from Central Park, New York City, summarizing the mean and coefficient of variation of the four rainfall parameters on a monthly basis. The statistical summary is for all storms which occurred in each of the months during the 27 year period (1948-1975).

Some interesting properties of the rainfall process are evident in Figure 3-4. For example, the mean intensity and mean duration show marked seasonal variability. The summer months are generally characterized by short, high intensity storms (an indication of thunderstorm activity), while the winter months have longer, less intense events. Note that the mean storm volume is less variable seasonally (0.3 to 0.4 inches) and the mean storms interval is approximately 80 hours, or slightly more than three days. Storms occur least frequently in October, when the mean time between events is approximately 110 hours. Storm intensities and volumes tend to be more variable in the summer, while the coefficient of variation of duration and the time between storms (ν_d and ν_δ, respectively) are relatively constant throughout the year and very nearly equal to one.

The rainfall statistics for a particular month or seasonal period may be estimated from Figure 3-4. For example, for the period of June - August, the storm characteristics are given below:

TABLE 3-2

MINNEAPOLIS RAINFALL ANALYSIS

A. HOURLY PRECIPITATION (Hundredths of an Inch)

Date		1	2	3	4	5	6	7	8	9	10	11	12
5/ 1/74	am	0	0	0	0	0	0	0	0	0	0	0	0
	pm	0	0	0	0	0	0	0	0	0	0	0	0
5/ 4/74	am	0	0	0	0	0	0	0	0	0	0	0	0
	pm	0	0	0	0	0	0	0	0	2	0	0	0
5/ 7/74	am	0	0	0	0	0	0	0	0	0	0	0	0
	pm	0	0	0	0	0	0	0	1	0	0	1	1
5/ 9/74	am	0	0	0	0	0	5	6	2	0	1	1	0
	pm	0	0	0	0	0	0	0	0	0	0	0	0
5/10/74	am	0	0	0	0	0	0	0	0	0	0	0	0
	pm	0	1	0	1	1	15	2	3	8	12	23	13
5/11/74	am	0	0	0	0	0	6	0	0	0	0	0	0
	pm	0	0	0	0	3	0	0	0	0	0	0	0
5/13/74	am	0	0	0	0	0	5	19	6	1	0	0	0
	pm	0	0	0	0	0	1	0	0	1	3	0	0
5/14/74	am	0	0	0	0	0	0	0	0	0	0	0	0
	pm	1	1	0	0	0	0	0	0	0	0	0	0
5/15/74	am	0	0	0	0	0	0	0	0	0	0	0	0
	pm	0	0	0	0	0	0	1	1	0	0	0	0
5/16/74	am	0	0	0	0	0	0	0	0	0	0	0	0
	pm	0	0	0	0	0	0	0	0	0	0	0	0
5/21/74	am	0	0	0	0	0	0	0	0	0	0	0	0
	pm	0	1	0	0	5	2	0	0	0	0	0	0
5/30/74	am	0	0	0	0	5	0	0	0	0	0	0	0
	pm	0	0	0	4	4	10	27	0	0	0	0	0

TABLE 3-2

MINNEAPOLIS RAINFALL ANALYSIS (Continued)

B. CHARACTERISTICS OF STORM EVENTS

Storm No.	Date	Beginning Hour	Duration (hr)	Intensity (in/hr)	Volume (in)	Interval Between Storms (hr)
2688	05/04/74	21	1	0.020005	0.02	186.5
2689	05/07/74	19	6	0.005000	0.03	72.5
2690	05/09/74	6	6	0.025000	0.15	35.0
2691	05/10/74	14	17	0.050000	0.85	37.5
2692	05/11/74	17	1	0.030005	0.03	19.0
2693	05/13/74	6	4	0.077501	0.31	38.5
2584	05/13/74	18	5	0.010001	0.05	12.5
2695	05/14/74	13	2	0.010002	0.02	17.5
2696	05/15/74	19	2	0.010002	0.02	30.0
2697	05/16/74	4	1	0.020005	0.02	8.5
2698	05/21/74	14	5	0.016000	0.08	132.0
2699	05/30/74	5	1	0.050005	0.05	205.0
2700	05/30/74	16	4	0.112501	0.45	12.5

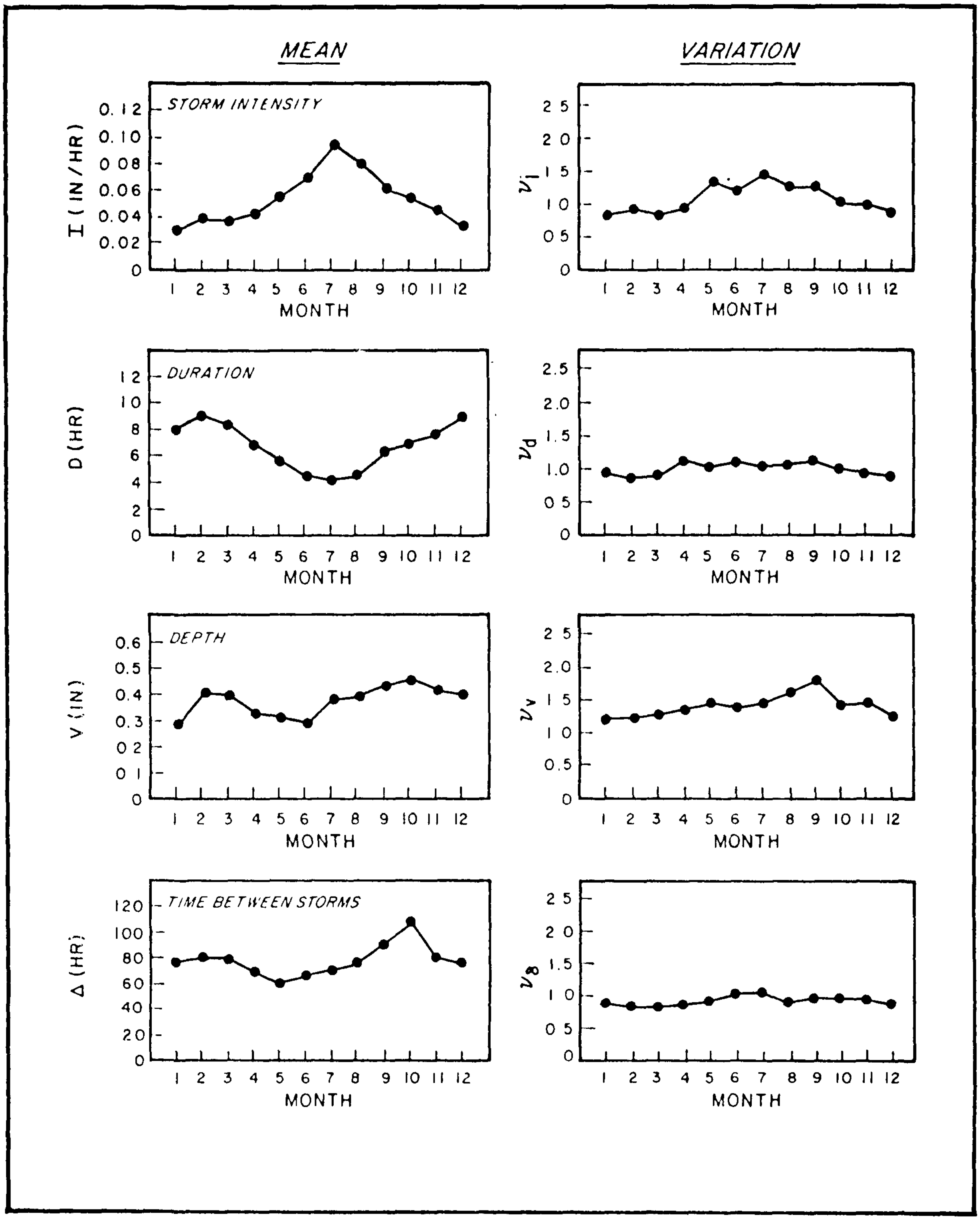


FIGURE 3-4

MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
CENTRAL PARK

STATION 305801

Estimated Central Park Storm Characteristics

June - August

	Mean	Coefficient of Variation
Storm Intensity	I = 0.08 in/hr	$\nu_i = 1.4$
Duration	D = 4.5 hr	$\nu_d = 1.1$
Unit Volume (Depth)	V = 0.35 in	$\nu_v = 1.5$
Time Between Storms	Δ = 70 hr	$\nu_\delta = 1.0$

(Note that some error is introduced by simply averaging monthly statistics for a given season, particularly for the coefficient of variation where a pooled variance calculation is necessary. This error is usually quite small, however, and may be ignored for the purposes of this manual.)

After the mean and coefficient of variation of the rainfall properties are determined, the frequency distribution is compared to the gamma distribution in Figure 3-2. Examples are presented in Section 5.1.3, along with a more detailed discussion of the applicability of the gamma distribution to storm characteristics.

Once the rainfall properties are adequately characterized, the estimation of stormwater loads may proceed. This is outlined in the following sections.

3.4 Development of Stormwater Loads

This section discusses estimating techniques for stormwater loads of various time scales, consistent with the problem definition time scales presented in Chapter 2. These include the average annual load, various seasonal loads, and transient loads occurring from individual events. The technique for making these estimates utilizes a direct and simple transformation from rainfall to runoff quantity and quality. It is a basic and very effective method for long term load characterizations when applied with reasonable estimates of the various model coefficients.

The specific techniques for estimating loads are presented here. Detailed discussions of model coefficients and useful numerical estimates are presented in Chapter 5.

3.4.1 Runoff Quantity

Stormwater runoff is generated by the rainfall on an area, which flows across the surface and gravitates toward the natural outlet from the area either through natural drainage courses or through sewers or other collection systems. The amount of runoff is related to the quantity of rain. Not all rainfall reaches the outlet of the drainage area. Some percolates into the ground or is retained in natural depressions. The net result is that only a fraction of the rain falling on an area ultimately becomes runoff.

An important influence on the fraction of rainfall which runs off is the degree of imperviousness of the area. The major effect is due to artificial surface cover such as pavement, roof tops, sidewalks, and street surfaces. Soil types in the other areas (i.e., sandy vs. clay type) also exert an influence, as do ground slopes and soil moisture. Regardless of the significance of these factors, it is well to bear in mind that the predominant influence on the amount of runoff to be expected at any particular time is the amount of rainfall. A convenient approach is to define the average runoff to rainfall ratio for a study area:

$$R_V = \frac{\text{Volume of Runoff}}{\text{Volume of Rainfall}} \tag{3-11}$$

This assumes a linear relationship between rainfall and runoff volume, and assumes that this relationship is the same for all storms in the period of record. The fraction of the rainfall volume which becomes runoff actually varies from storm to storm, depending upon the antecedent conditions, the storm intensity and patterns, etc. In using a single ratio of runoff volume to rainfall (R_V), the runoff volume is overestimated for some storms and underestimated for others. The long term characterization of the storm runoff properties (i.e., mean, variability, and frequency distribution) is, however, well estimated, and the simple runoff to rainfall ratio is used in other long term, initial assessment methodologies employing simulators (4).

The best way to determine the runoff coefficient for a particular study area is to compare raingage data with the runoff monitored during corresponding storms. Sufficient data of this type are often not available, and in such cases estimates must be made based upon land use characteristics, either from land use surveys of the drainage area, or inferred from the population density. Methods for estimating the average volumetric ratio of runoff to rainfall (R_V) based on drainage basin characteristics are presented in Section 5.2.1.

The average volumetric ratio of runoff to rainfall is used to convert the rainfall statistics to runoff statistics. The mean runoff volume (V_R) is determined:

$$V_R = 0.027(R_V VA) \tag{3-12}$$

where:

V_R = mean runoff volume (million gallons, MG)

V = mean storm volume (in)

A = drainage area (acres)

0.027 = conversion factor (MG/acre-in).

To estimate the mean runoff flow (Q_R), a transformation from rainfall intensity is required:

$$Q_R = R_V IA(D/D_R) \tag{3-13}$$

where:

Q_R = mean runoff volume (cfs)

I = mean storm intensity (in/hr)

D = mean storm event duration (hr)

D_R = mean runoff event duration (hr)

The conversion factor from (acrein/hr) to (cfs) equals one. The term (D/D_R) is a correction factor to account for the attenuation of runoff beyond the end of a rain event. This is particularly necessary for a large catchment area with a long equilibrium time (the time it takes for the entire basin to contribute runoff at a particular point in the receiving water) where D_R will be somewhat larger than D. A method for estimating D_R as a function of drainage area size and the degree of urbanization, using unit hydrograph analysis, is presented in Section 5.2.2.

The storm to storm variation in runoff volumes and flows is due primarily to differences in the amount of rainfall, although variability in the ratio of runoff to rainfall and the amount of runoff attenuation experienced during storms, caused by changing antecedent conditions, may also contribute variability. However, for most planning purposes, the estimate of runoff variation may be based solely upon the variation of the measured rainfall parameters:

$$\nu_{vR} = \nu_v \tag{3-14}$$

$$\nu_q = \nu_i \tag{3-15}$$

The procedure described does not specifically correct for runoff attenuation due to depression storage in the drainage basin. Where the user considers it important to take this into account in his analysis, the long term effect of depression storage on runoff loads reaching the receiving water can be estimated by using the treatment curves presented later (Figure 3-18) which define the level of control provided by storage basins.

Since the statistical analysis does not depend directly on the method used to estimate runoff statistical properties more sophisticated models or data for estimating the basic runoff parameters may be incorporated where appropriate in the later stages of the planning process, without loss of continuity.

3.4.2 Runoff Quality and Resulting Loads

Runoff flows may be translated into stormwater loads by multiplying by the appropriate pollutant concentration, c (mg/l). The concentration of contaminants in stormwater runoff and overflows varies within storms, between storms, and from location to location as a function of drainage basin characteristics, land use, conveyance type, season, storm type. Since the variation is often, random and unpredictable, it is useful to begin the loading assessment in a particular area using a simple average pollutant concentration ($\bar{c}$, mg/l) in the runoff or overflows. The significance of concentration variation between and within storms will be presented as the methodology is developed.

The best way to estimate average runoff pollutant concentrations is with a wet weather monitoring program. When sufficient data of this type are unavailable, however, estimates may be made based on drainage area characteristics, although significant errors are possible due to the wide variability observed in stormwater concentrations. Further guidance and estimates for determining runoff quality are presented in Sections 5.3.1 and 5.3.2.

Once appropriate estimates of the average runoff concentration are made, storm loads may be determined. Equations for estimating the mean runoff loading rate during storms (W_R, lbs/day), the mean load per storm (M_R, lbs), and the long term average mass discharge rate (W_o, lbs/day) are presented in this section. The relevance of each of these loading characterizations is discussed in Chapter 2. Estimates for the variability and frequency distribution of stormwater loads are also presented.

Advective streams and rivers are usually sensitive to instantaneous loading rates of pollutants, and the analysis of transient stormwater impacts thus requires a characterization at this time scale. If storm runoff flows and concentrations are statistically independent, the mean runoff loading rate, (W_R, lbs/day), will simply equal the product of the mean concentration, ($\bar{c}$, mg/l), and the mean runoff flow, (Q_R, cfs).

$$W_R = 5.4\ \bar{c}\ Q_R \qquad (3\text{-}16)$$

where 5.4 is a conversion factor to make units consistent (lbs/day)/(cfs-mg/l) 1. It is reasonable to begin by assuming c and q_R are independent; if data collected for specific pollutants indicate they are not, the following refinement can be employed:

$$W_R = 5.4\ \bar{c}Q_R\ (1 + \nu_c \nu_q \rho_{cq}) \qquad (3\text{-}17)$$

where:

ν_c = the variation of the pollutant concentration (between storms)

ν_q = the variation of the runoff flow (between storms)

ν_{cq} = the linear correlation coefficient between the pollutant concentration and the storm runoff flow (ranging from -1 to +1)

Positive correlation between the pollutant concentration and the storm runoff flow will yield a higher average loading rate, while negative correlation will yield a lower average loading rate. Positive correlations generally occur where higher flows are necessary to cause a significant scouring of suspended solids in the sewer system or to dislodge and transport solids from the ground surface. Negative correlations may occur when the diluting effects of the larger runoff flows dominate.

Equation 3-16 calculates mass loadings based on mean runoff concentration ($\bar{c}$), when there is no flow-concentration correlation. Where a flow-concentration correlation does exist for a particular study area, the correct mass loading will be calculated either by equation 3-17 (in which $\bar{c}$ is the unweighted mean concentration), or by equation 3-16 when $\bar{c}$ is taken to be a flow-weighted concentration.

The variation of the runoff loading rate at a particular site is due to variation in both the flow and the concentration. An equation is available for estimating loading rate variability as a function of ν_q and ν_c by assuming flow and concentration are independent (2). Because the technique for evaluating instream responses to runoff loads requires information on both the pollutant load and the flow associated with it (except in large rivers where the flow contributed by the runoff is insignificant compared to the base flow already present), a consistent probabilistic analysis requires that all the variability in the runoff load be associated with flow variation. This is discussed further in section 3.4.2.1 on impacts in streams and rivers. The coefficient of variation of the loading rate during storms (ν_w) is thus estimated to be equal to the coefficient of variation of the runoff flow:

$$\nu_w = \nu_q \qquad (3\text{-}18)$$

This is equivalent to assuming the runoff concentration is constant, as if often done with simplified simulation models (4).

Highly dispersive receiving waters such as estuarine rivers and bays require only that the total mass entering the water due to the storm runoff be properly identified. The high degree of mixing makes them insensitive to the actual loading rate which occurs during the storm. For these systems, an estimate of the total mass discharged per storm and its variability from event to event is sufficient. If storm runoff volumes and concentrations are independent, the mean storm runoff load (M_R, lbs) is estimated as the product of the mean concentration ($\bar{c}$, mg/l) and the mean runoff volume (V_R, MG).

$$M_R = 8.34\ \bar{c}V_R \qquad (3\text{-}19)$$

where 8.34 is a conversion factor to make the units consistent [lb/(MG-mg/l)].

If storm runoff volumes and concentrations are not independent, a correction similar in form to Equation 3-17 may be used to adjust the estimate of M_R. This correction requires large amounts of data to estimate the correlation coefficient between storm volumes and concentrations with statistical significance. When a first flush effect is present, however, there is a negative correlation inherent between runoff concentrations and storm volumes as shown below.

The first flush phenomenon is the condition, often occurring in stormwater discharges, in which a disproportionately high fraction of the runoff load is carried in the first portion of the discharge or overflow. The pollutant concentration in the runoff at the beginning of a rainfall event is relatively high, and as the rainfall continues, the subsequent runoff concentration decreases. The temporal profile of the pollutant concentration during storms is approximated by an exponential decrease, as depicted in Figure 3-5. Given this assumption, and the assumption that runoff durations and flows are independent and exponentially distributed ($\nu_q = \nu_d = 1$, and $\nu_{vR} = \sqrt{3}$) the correction in the estimate of M_R due to the first flush effect can be calculated. The average runoff concentration ($\bar{c}$) is calculated as the expected value of time integral c(t) for the varying storm durations:

$$\bar{c} = E\ c(t) = \int_{d=o}^{\infty} \int_{t=o}^{d} \frac{c(t)}{d}\, p_d(d)\ dt\ dd \qquad (3\text{-}20)$$

The result is:

$$\bar{c} = c_o + \frac{\beta}{D_R} (c_p c_o) \ln(1 + D_R/\beta) \qquad (3\text{-}21)$$

Where the calculated average concentration is between c_o and c_p, depending upon the rate of subsidence of the first flush peak. This provides an estimate of the time averaged runoff concentration observed at a site (without flow weighting). The average runoff volume is the product of the mean duration and runoff flow:

$$V_R = D_R Q_R \qquad (3\text{-}22)$$

The average runoff load is calculated as the expected value of the storm load for the varying flows and durations from event to event; and the decaying concentration profile within storms:

$$M_R = E\ c(t)q\ d = \int_{q=o}^{\infty} \int_{d=o}^{\infty} \int_{t=o}^{d} \frac{c(t)}{d}\, q\ d\ p_d(d)\ p_q(q)\ dt\ dd\ dq \qquad (3\text{-}23)$$

This calculation accounts for the fact that storms of longer duration have a lower average runoff concentration.

The result is:

$$M_R = D_R Q_R \left[c_o - \frac{(\beta/D_R)(c_p - c_o)}{(D_R/\ ) + 1} + \frac{\beta}{D_R} (c_p - c_o) \right] \qquad (3\text{-}24)$$

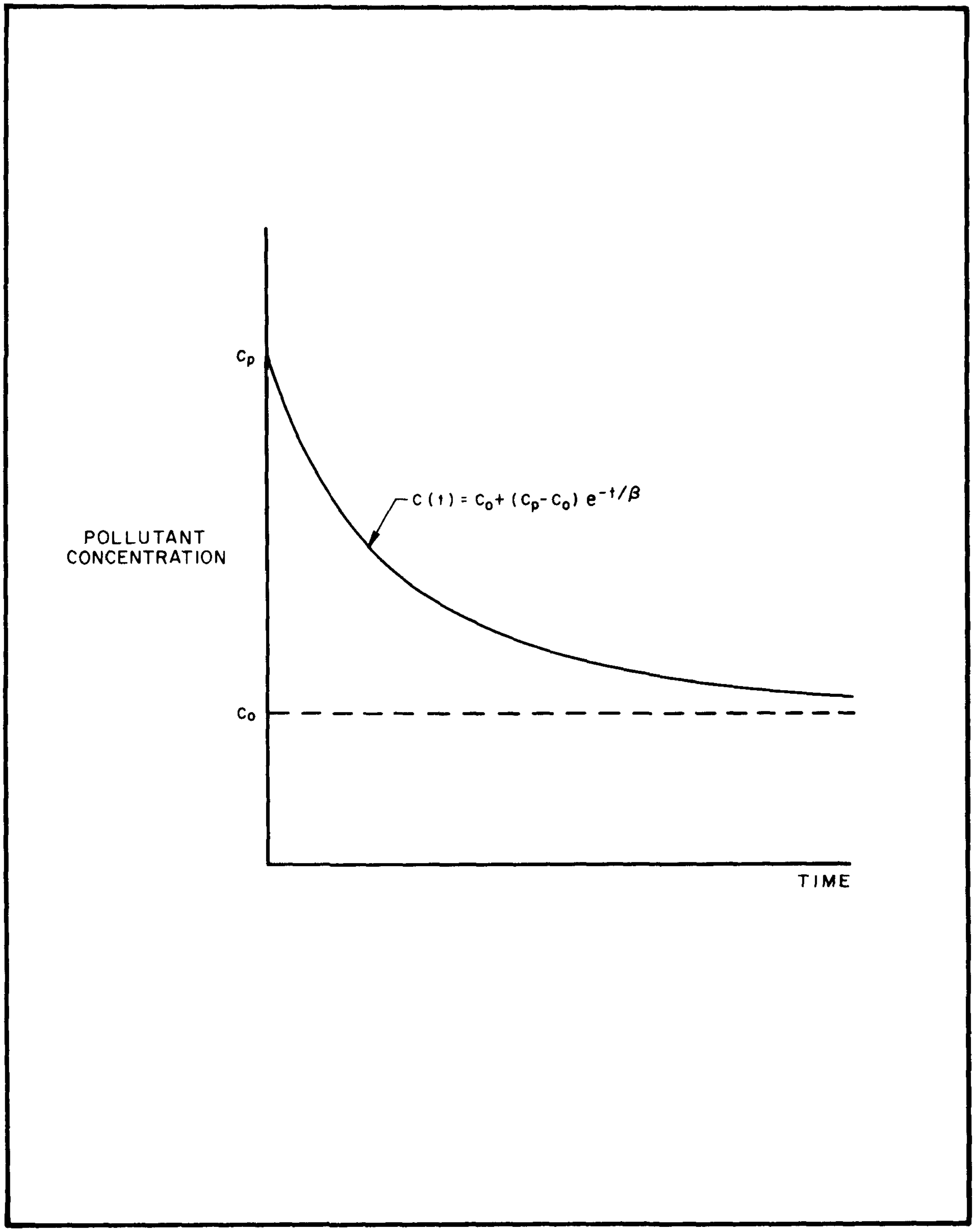


FIGURE 3-5
IDEALIZATION OF FIRST FLUSH EFFECT

The quantity of interest is $M_R/\bar{c}V_R$, which is the ratio of the actual mean storm load to the storm load predicted by using $\bar{c}$ and V_R. $M_R/\bar{c}V_R$ is plotted in Figure 3-6 as a function of c_p/c_o, the ratio of the peak concentration to the concentration after the first flush has subsided, which indicates the magnitude of the first flush; and D_R/β, the ratio of the average runoff duration to the first flush decay time, which indicates the rate of first flush subsidence. The effect is seen to be generally small, with the correction factor for M_R near 1.0 although it can approach 0.7 for large, rapidly subsiding first flush effects. Note that these curves will be somewhat different when durations and flows are not independent and exponentially distributed, i.e., $\nu_d \neq 1$, $\nu_q \neq 1$, and $\nu_{vR} \neq \sqrt{3}$). However the results in Figure 3-6 are reasonable for a first estimate, and may be used to adjust the estimate of M_R when a first flush effect is present.

Theoretically, a first flush should always exist, since materials will accumulate on land surfaces and in sewer lines during the periods between storms. Local factors, such as the size of the drainage basin and the staggered time interval during which first flushes from different parts of the basin reach the overflow or monitoring location, may suppress this effect. Generally, local monitoring of a sufficiently large number of storm events will be necessary to reliably characterize the first flush effect actually present in a study area. However, as shown in Figure 3-6, the influence on the estimate of the mean runoff load (M_R) may not be of major importance. The first flush effect also has an impact on the performance of storage devices for stormwater control, as discussed in Section 3.6.1.2.3.

The variability of storm runoff loads is due to variation in both the volume and the concentration. Because of the negative correlation sometimes present between storm volumes and concentrations, it is reasonable to associate all of the load variation with volume. The coefficient of variation of the runoff load (ν_m) is thus estimated to equal the coefficient of variation of the runoff volume:

$$\nu_m = \nu_{vR} \tag{3-25}$$

The probability distribution of storm loads and loading rates is expected to be similar to the distribution of volumes and flows. A gamma distribution describes the frequency characteristics and Figure 32 may be used to predict the fraction of storms and the expected number of times per year, month, or season that a given storm load or loading rate is exceeded, as outlined previously.

The average storm load (M_R) and loading rate (W_R) are representative of pollutant loads during storm periods. The long term average mass discharge rate, W_o, is calculated by determining the total storm load during the year (pounds), and assuming that it occurs continuously (during both rain and nonrain periods). If the period of interest is a particular month or season, rather than the entire year, W_o may be calculated by determining the total storm load during the particular month or season, and by assuming that the storm load occurs continuously. W_o is also estimated from the mean storm load (M_R) as follows:

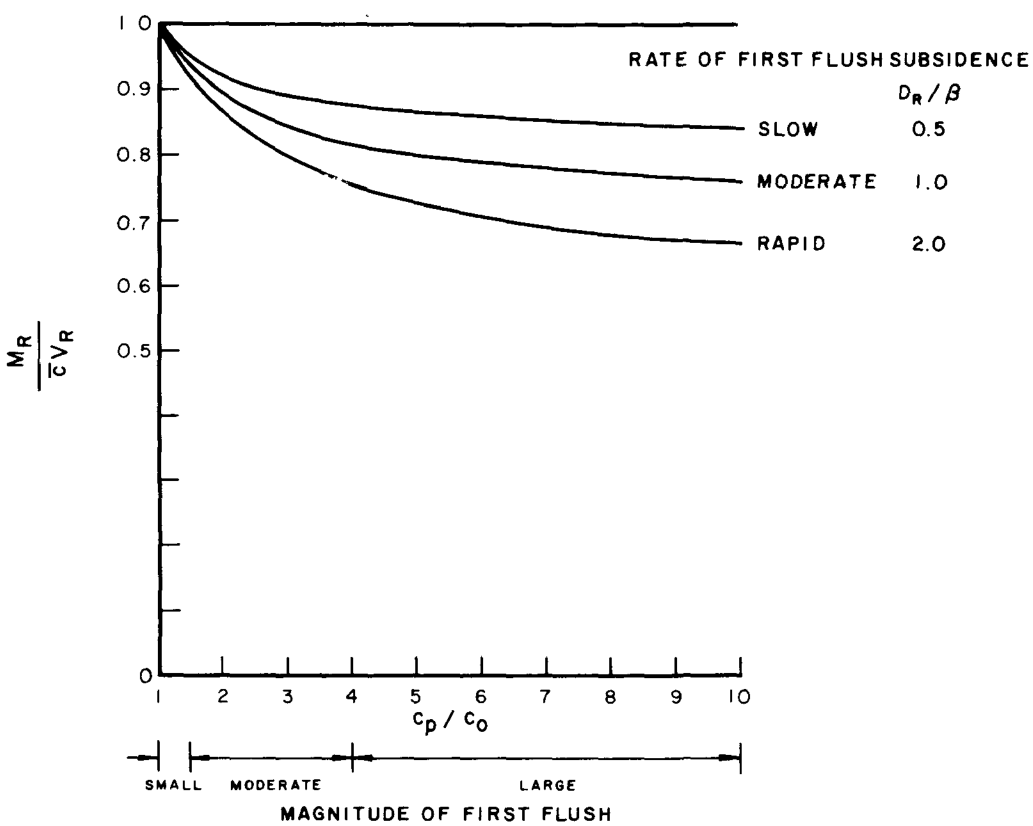


FIGURE 3-6

CORRECTION IN ESTIMATE OF M_R WHEN FIRST FLUSH IS PRESENT

$$W_o = W_R/\Delta \qquad (3\text{-}26)$$

The total pollutant mass occurring over a long term period is the product of the length of the period and W_o. For example, the average yearly load (Y_m, lbs/year) is:

$$Y_m = 365.25 \text{ (day/year)} \times W_o \text{ (lbs/day)} \qquad (3\text{-}27)$$

W_o is the loading rate which is used to assess the cumulative long term stormwater effects and may be compared with the continuous municipal and industrial point source loadings to determine the relative magnitude of each source. At least five years of raingage data, either for the entire year, or during the particular month or season of interest, should be used to provide adequate confidence in W_o.

For pollutants which impact the receiving water in a transient fashion, such as coliforms or BOD_5, the long term loading rate, W_o, may not fully indicate the severity of the problem. For example, stormwater loads may contribute only a small part of the total yearly BOD_5 load entering a receiving water in a particular area, but the occurrence of this load only during storm periods may lead to violations of dissolved oxygen standards during or immediately following a number of rain events. It is for such cases that the actual mean storm load (M_R), mean loading rate during storms (W_R), and their variations, ν_m and ν_w, respectively, become the important indicators of stormwater loadings.

To demonstrate a typical loading table developed for a study area, assume the following factors are determined for stormwater BOD_5 loadings in the summer (July - September).

$Q_R = 10$ cfs, $\quad \nu_q = 1.20, \quad V_R = 2.2$ MG

$\Delta = 85$ hr $= 3.54$ days, $\quad \bar{c} = 40$ mg/l BOD , $\quad$ Moderate first flush

The mean storm loading rate (W_R) is

$$W_R = 5.4\ \bar{c}Q_R$$

$$= 5.4 \text{ (lbs/day)/(cfs-mg/l)} \quad 40\text{(mg/l)} \ 10 \text{ (cfs)}$$

$$= 2{,}160 \text{ lb } BOD_5\text{/day}$$

The coefficient of variation of the loading rate (ν_w) is estimated to be:

$$\nu_w = \nu_q = 1.20$$

The mean storm load (M_R) calculated from equation 3-19 is:

$$M_R = 8.34\ \bar{c}V_R$$

$$= 8.34 \text{ lb/(MG-mg/l)} \quad 40\text{(mg/l)} \ 2.2\text{(MG)}$$

$= 734$ lbs BOD_5

Correcting the estimate of M_R for a moderate first flush ef assuming $D_R/\beta = 1.0$, Figure 3-6 indicates $M_R/\bar{c}V_R$ equals about 0.85. The corrected estimate of M_R is therefore:

$M_R = 0.85(734)$

$= 624$ lbs BOD_5

To calculate the frequency of occurrence of different storm flows and loading rates, Figure 3-2 is used. These are translated into the expected number of summer storms which will exceed a given flow and loading rate by knowing the average number of storms per summer:

$$\text{Average Number of Storms} = \frac{\text{Length of Period}}{\Delta} = \frac{92 \text{ days}}{3.54 \text{ days}} = 26$$

The results are summarized in Table 3-3, together with the long term average loading information. The probabilistic loading rate information (W, lbs/day) is appropriate for an advective stream or river, whereas a table constructed for an estuarine area would include frequencies of storm loads (M, lbs) in multiples of the mean load (M_R). The long term average loading rate during the summer (W_o) is:

$W_o = M_R/\Delta$

$= 624$ (lb)/3.54 (days)

$= 176$ lb BOD_5/day

Table 3-3 summarizes the essential loading information necessary for the instream response estimate.

3.5 Impacts in the Receiving Water

The quantification of stormwater related loads in an urban area provides insight into the importance of runoff and overflows relative to other waste loads. It is not, however, the final step in the assessment since contaminant loadings from urban runoff are evaluated in terms of their impact on the adjacent receiving water. The range of water quality problems which may be due to stormwater runoff is quite large. This section presents methods and equations for making quantitative assessments of receiving water impacts including both long term and transient effects. Different estimating techniques are developed for streams, rivers, estuaries, lakes and impoundments. Guidance for selecting particular model coefficients and parameters is presented in Section 5.4 of Numerical Estimates. Note that a basic understanding of receiving water modeling is assumed for the users of this manual. A more detailed description of mathematical water quality models and their important components may be obtained from the following references:

1. *Areawide Assessment Procedures Manual*, Chapters 2 and 5 (2)
2. *Simplified Mathematical Modeling of Water Quality* (22)

TABLE 3-3

EXAMPLE STORMWATER LOADING TABLE FOR ADVECTIVE STREAM
(JULY - SEPTEMBER)

1. Loading during storms:

Q_R = 10 cfs W_R = 2,160 lbs BOD_5/day

ν_q = 1.20 ν_w = 1.20

Flows		Loading Rate		Percent of Storms Greater Than	Expected Number Summer Storms Greater Than
Q (cfs)	Q/Q_R	W (lbs/day)	W/W_R	(%)	(No.)
5.5	0.55	1190	0.55	50%	13.0
12.0	1.20	2590	1.20	30%	7.8
17.0	1.70	3670	1.70	20%	5.2
25.0	2.50	5400	2.50	10%	2.6
35.0	3.50	7560	3.50	5%	1.3

2. Long term loading rate

W_o = 176 lbs BOD_5/day

3. *Mathematical Modeling of Natural Systems* (23)
4. *Systems Analysis and Water Quality Management* (24)

3.5.1 Prediction of Long Term Impacts

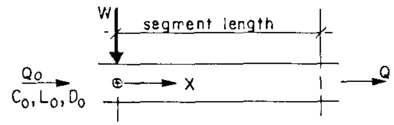
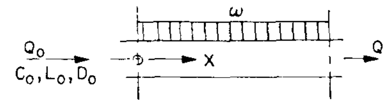
Although stormwater loads occur intermittently in the order of hours, their important receiving water impacts may be manifest over longer time periods. As discussed in Chapter 2, this is particularly true for problems such as sediment or nutrient buildups in lakes and impoundments, and the increase in levels of persistent organics or metals. These problems may be addressed by determining the portion of the average, steady state receiving water concentration which is due to stormwater loads. The long term average loading rate (W_o) is the appropriate input for such an analysis. In addition to these long term problems, some understanding of transient effects, such as coliform bacteria increases and dissolved oxygen depressions, may be gained by examining the average, long term contributions of stormwater runoff. Again, a steady state analysis using long term average loads is appropriate. Finally, results from steady state mathematical models may also be used to infer information about the transient nature of these impacts, as will be demonstrated in the following sections. Steady state representations are particularly useful at the assessment stage because of the relative simplicity of the calculation and the ability to respond rapidly and relatively inexpensively to specific planning questions.

3.5.1.1 Streams and Rivers

The simplest type of receiving water is a one-dimensional flowing stream or river where the mixing characteristics are such that the dispersion of the mass of material can be neglected in comparison to the flow. In this case, the river flow is the major mass transport mechanism. This simplification is significant in terms of computational complexity and the amount of information required for water quality analysis.

For a complete specification of stream responses to pollutant loads, the initial concentration, the reaction rate, and the river flow and cross-sectional areas are required. For some variables, there may be a coupling effect where the solution of one equation feeds forward into a second equation and acts as an input. For example, the interaction between the biochemical oxygen demand and dissolved oxygen is represented by a coupled set of equations. A summary of the basic Streeter-Phelps solutions for steady state pollutant concentrations in a stream is presented in Table 3-4. Critical seasonal effects are estimated by assuming constant waste and stream characteristics for the particular season. Concentrations are assumed to be constant throughout the depth and across the width of the receiving water. The receiving water geometry is, therefore, approximated by a series of constant geometry and constant flow segments. The governing differential equations for the receiving water concentrations are linear so that the effects of the individual waste sources can be calculated separately and, at a given location, added together to give the total instream concentration.

It is recommended for the first assessment that a single segment model be used with a spatially aggregated stormwater load (W_o). If more spatial

	POINT SOURCE	DISTRIBUTED SOURCE
		
Conservative C	$C = C_0 + W/Q$	$C = C_0 + wX/Q$
Reactive L	$L = L_0 e^{-K_r X/U} + (W/Q) e^{-K_r X/U}$	$L = L_0 e^{-K_r X/U} + \frac{w}{A.K_r} (1 - e^{-K_r X/U})$
Coupled D	$D = D_0 e^{-K_a X/U}$ $+ L_0 \cdot \frac{K_d}{K_a - K_r} [e^{-K_r X/U} - e^{-K_a X/U}]$ $+ (W/Q) \cdot \frac{K_d}{K_a - K_r} [e^{-K_r X/U} - e^{-K_a X/U}]$	$D = D_0 e^{-K_a X/U}$ $+ L_0 \cdot \frac{K_d}{K_a - K_r} \cdot [e^{-K_r X/U} - e^{-K_a X/U}]$ $+ \frac{w}{AK_r} \cdot \frac{K_d}{K_a - K_r} [\frac{K_r}{K_a} e^{-K_a X/U} - e^{-K_r X/U} + \frac{K_a - K_r}{K_a}]$

NOTE

Q = FLOW
X = DISTANCE
C = CONSERVATIVE SUBSTANCE CONCENTRATION
L = REACTIVE SUBSTANCE CONCENTRATION (BOD)
D = COUPLED SUBSTANCE CONCENTRATION (D O DEFICIT)
U = VELOCITY
A = CROSS-SECTIONAL AREA

K_r = BOD REMOVAL RATE
K_d = BOD OXIDATION RATE
K_a = D O REARATION RATE
C_o, L_o, D_o = CONCENTRATION AT X=0

TABLE 3-4

SUMMARY OF STEADY STATE SOLUTIONS FOR POLLUTANT CONCENTRATIONS IN RECEIVING STREAMS detail is required, it is recommended that the stream be segmented into a maximum of five reaches. The purpose of limiting the segmentation of the stream is to simplify the number of calculations required in the impact analysis and to keep the level of detail of the impact analysis consistent with the accuracy of the load estimation. In general, stream segments are constructed for areas of approximately constant flow, cross-sectional areas, depths, and velocities. Additional segments are formed at the location of important point source load inputs. If less than five stream segments are required for the particular basin, then the analysis is more manageable.

An important factor in the segmentation of the model is the effect of the spatial detail of the stormwater load characterization on the accuracy of the predicted instream response. Various levels of spatial detail which may be employed are demonstrated in Figure 3-7. To limit the error in the predicted downstream water quality concentration to 5 percent, loading aggregations should be limited to a distance X (miles), such that $X = 0.05 \frac{U}{K}$, where U is the stream velocity (miles/day) and K is the reaction rate (per day). The resulting load is the sum of the individual loads.

Note that as the reactivity of the water quality parameter increases, the distance over which load aggregation can take place is reduced. Therefore, an analysis of coliform bacteria loads will usually result in considerably smaller aggregation distances than an analysis of BOD or suspended solids loads. Judgment should be used in aggregating loadings of conservative materials as there is a practical limit to the aggregation distance.

3.5.1.2 Estuaries and Coastal Waters

An estuary is that portion of a coastal river where the tidal action from the ocean is a significant hydrodynamic parameter. There are two broad sections of estuaries, the tidal river portion where the water body ebbs and floods, but is entirely freshwater; and the lower estuarine portion where, in addition to the ebbing and flooding of the tide, a significant intrusion of sea salts occurs. One or two spatial dimensions (e.g., the longitudinal and vertical dimensions) may be of importance in estuaries, although initial assessments may simplify the problem to a one-dimensional analysis. The primary difference between estuaries and the one-dimensional river flow situation is the dispersive mass transport due to the tidal mixing. This forms an important transport phenomena in addition to the net freshwater flow through the estuary and is included in the analysis. The steady state equations for pollutant concentrations in one dimensional estuaries are presented in Table 3-5. The additional parameter of interest is the dispersion coefficient (E) due to the tidal action. The long term stormwater loading rate (W_o) is used to determine the average response of the estuary. The variability of the response for transient impacts is addressed in Section 3.5.2.2.

Coastal waters such as tidal embayments and near-shore areas usually require more complex analysis with two or three-dimensional specification of geometry, hydraulic regimes, circulation, etc. Some simplified analysis techniques for evaluating ocean outfalls and localized near-shore areas are presented in Chapter 5 of the Areawide Assessment Procedures Manual (2).

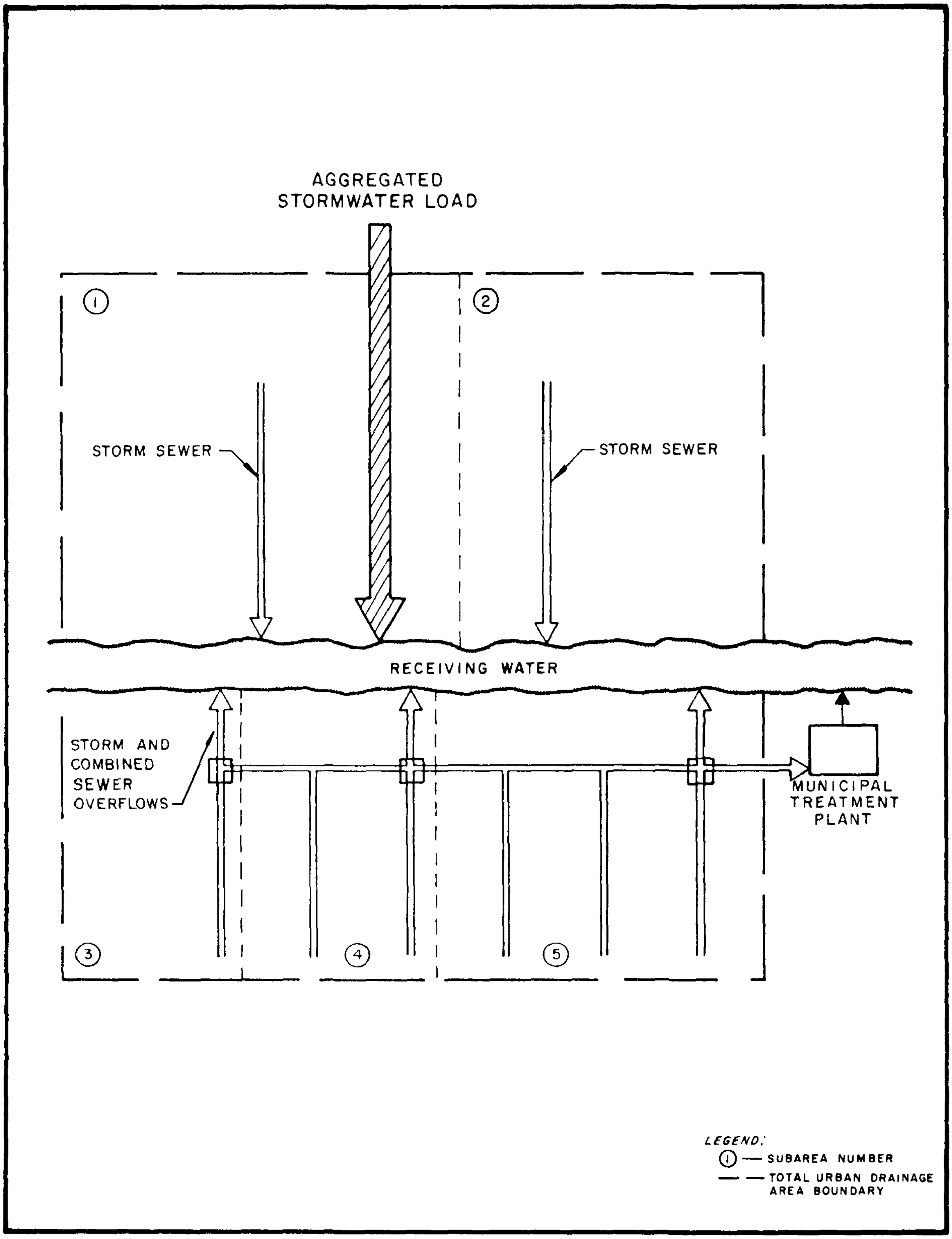


FIGURE 3-7
COMPARISON OF SPATIAL DETAIL
IN STORMWATER LOADING CHARACTERIZATION

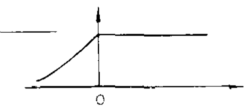
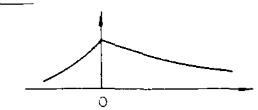
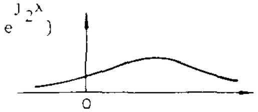
TABLE 3-5

STEADY STATE EQUATION FOR WASTE CONCENTRATIONS IN TIDAL RIVERS AND ESTUARIES DUE TO POINT SOURCE

"long" estuary

W

Q ⟶ 0 ⟶ x L, A, K_1 constant

Type Waste	$x \leq 0$	$x = 0$	$x \geq 0$	
Conservative	$C = C_o e^{Ux/E}$	$C_o = W/Q$	$C = C_o = W/Q$	
Reactive (System 1)	$L = L_o e^{g_1 x}$	$L_o = W/(Qm_1)$	$L = L_o e^{J_1 x}$	
Sequentially Reactive (System2)	$D = \frac{K_{12}}{K_2 - K_1} L_o \left(e^{g_1 x} - \frac{m_1}{m_2} e^{g_2 x}\right)$	$D_o = \frac{K_{12}}{K_2 - K_1} L_o \left(1 - \frac{m_1}{m_2}\right)$	$D = \frac{K_{12}}{K_2 - K_1} L_o \left(e^{J_1 x} - \frac{m_1}{m_2} e^{J_2 x}\right)$ $x_c = n\left(\frac{m_1}{m_2} \cdot \frac{1-m_2}{1-m_1}\right) \div \frac{U}{2E}(m_2 - m_1)$	

DESCRIPTION OF SYMBOLS WITH TYPICAL UNITS

Q = flow (cfs)

U = velocity = $\frac{Q}{A}$ (fps)

A = cross-sectional area (ft^2)

E = dispersion coefficient (mi^2/day)

K_1 = decay rate, system 1 (e.g. BOD) (day^{-1})

K_2 = reaction rate, system 2 (e.g. reaeration rate)

K_{12} = reaction rate between systems 1 & 2, (e.g. deoxygenation rate K_d) (day^{-1})

W = mass discharge rate (lb/day)

$m_1 = \sqrt{1 + 4K_1 E/U^2}$

$J_1 = (U/2E)(1 - m_1)$

$g_1 = (U/2E)(1 + m_1)$

TABLE 3-5

STEADY STATE EQUATION FOR WASTE CONCENTRATIONS IN TIDAL RIVERS AND ESTUARIES DUE TO POINT SOURCE

3.5.1.3 Lakes and Reservoirs

Lakes and reservoirs can involve either two or three spatial dimensions. The flow regime in these bodies of water can be quite complex since there are usually no dominant mechanisms which determine the advective flow and mixing, in contrast to the case of estuaries and rivers. The stratification which can occur due to the absence of intense advective or mixing forces, complicates the distribution of water quality constituents in a vertical direction. Thus, lakes and reservoirs can encompass a broad spectrum of complexity, ranging from completely mixed water bodies to highly stratified water bodies.

Initial assessments of long term stormwater impacts in lakes and reservoirs may be made with a few simplifying assumptions. To determine the average concentration of conservative or slowly reactive constituents (i.e., dissolved solids, persistent organics, etc.), the water body may be assumed to be completely mixed. Equations for a large, completely mixed impoundment are presented in Table 3-6. Because lakes with long detention times require a long time period to reach steady state, equations for estimating the pollutant buildup (or reduction after treatment) over time are also presented. Note that these equations are appropriate only for an initial assessment, and not for more detailed planning.

The eutrophication of lakes and impoundments due to excessive nutrient load is a problem which has received a considerable amount of attention (25). Simplified techniques for estimating the potential for lake eutrophication have been recently developed (26, 27). These techniques are particularly applicable for estimating the long term impact of stormwater related nutrient loads. Because eutrophication of a water body is very complex, simplified analysis techniques should be used with extreme caution.

The equation used for the model developed by Dillon (27) considers the hydraulic flushing time, the nutrient loading, the nutrient retention ratio, the mean depth, and the nutrient concentration of the impoundment:

$$\frac{L\ (1 - R)}{\rho} = HN \qquad (3\text{-}28)$$

where:

L = nutrient loading divided by the surface area of the lake (gm/m /yr);

R = fraction of nutrient retained;

ρ = hydraulic flushing rate (1/yr);

H = mean depth (m); and

N = nutrient concentration (mg/1)

The nutrient loading due to stormwater runoff is calculated from the total yearly load (Y_m). The graphical solution to this equation is a log-log plot of L (1R)/ρ versus H. Figure 3-8 is a reproduction of Dillon's work on

CONCENTRATIONS IN LARGE, COMPLETELY MIXED IMPOUNDMENT

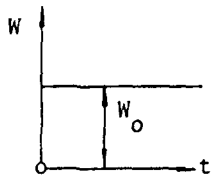
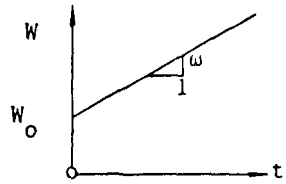
	Constant Load	Linearly Increasing or Decreasing Load
	 $W = W_o$	 $W = W_o + \omega t$
Conservative - Concentration vs. Time	$C_o e^{-\frac{Q}{V}t} + \frac{W_o}{Q}\left(1 - e^{-\frac{Q}{V}t}\right)$	$C_o e^{-\frac{Q}{V}t} + \frac{W_o}{Q}\left(1 - e^{-\frac{Q}{V}t}\right) \pm \frac{\omega V}{Q^2}\left(\frac{Q}{V}t + e^{-\frac{Q}{V}t} - 1\right)$
- Steady State	$\frac{W_o}{Q}$	
Slowly Reactive $(K_r << \frac{Q}{V})$ Note: $\alpha = \frac{Q}{V} + K_r$ - Concentration vs. Time	$L_o e^{-\alpha t} + \frac{W_o}{\alpha V}\left(1 - e^{-\alpha t}\right)$	$L_o e^{-\alpha t} + \frac{W_o}{\alpha V}\left(1 - e^{-\alpha t}\right) \pm \frac{\omega}{\alpha^2 V}\left(\alpha t + e^{-\alpha t} - 1\right)$
- Steady State	$\frac{W_o}{\alpha V}$	Note Q = Flow through Impoundment V = Volume of Impoundment

TABLE 3-6

CONCENTRATIONS IN LARGE, COMPLETELY MIXED IMPOUNDMENT

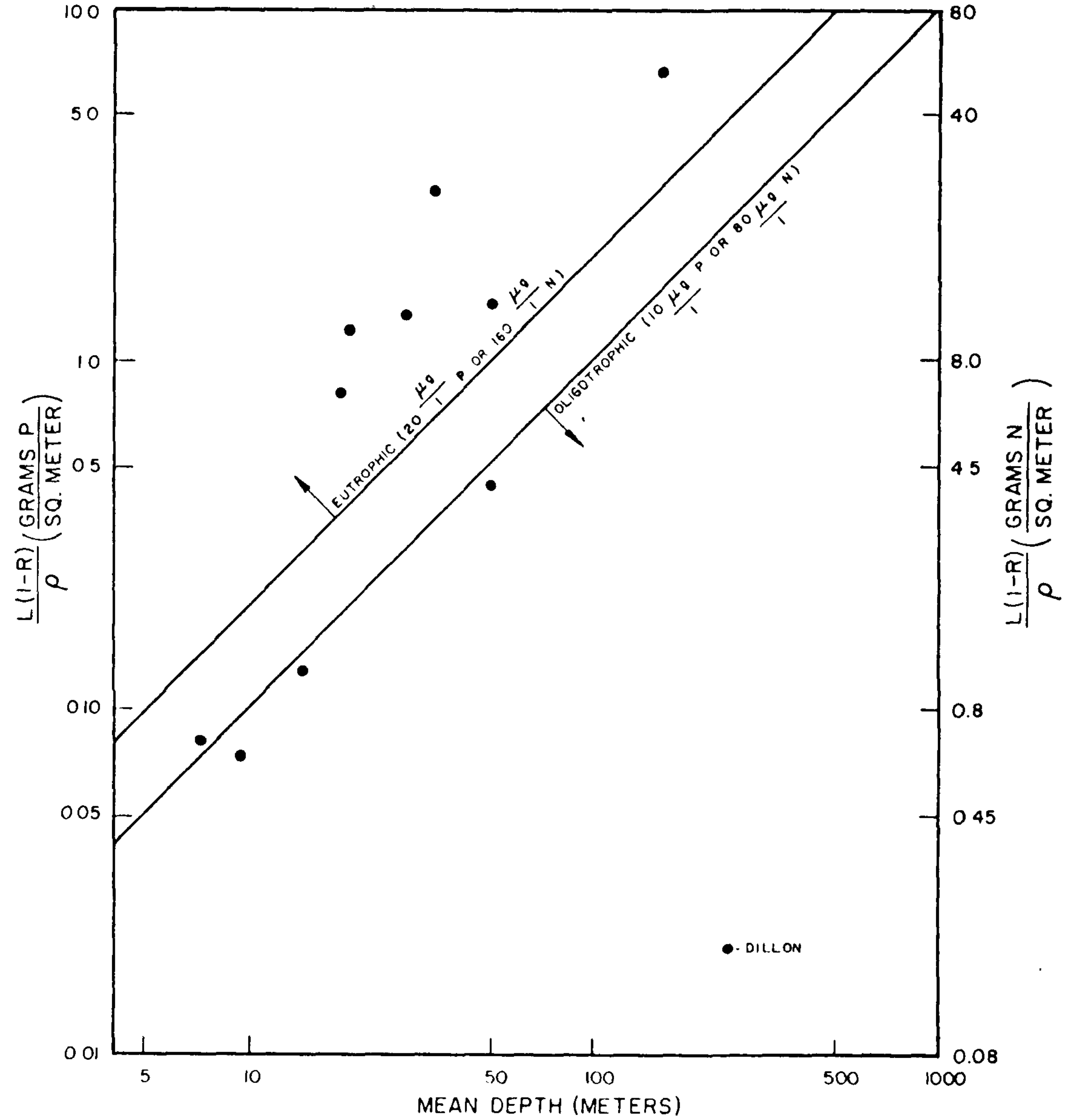


FIGURE 3-8
GRAPHICAL SOLUTION TO THE DILLON APPROACH

several lakes in Canada, based on phosphorous loadings. The nitrogen axis has been added to the graph based on a stoichiometric relationship of the mass of nitrogen to phosphorous in algae. This relationship does vary; a range of 3 to 15 mgN/mgP has been reported (28) with an average of 8 being typically used. Lakes or impoundments which fall above the 20 μgP/l or 160 μgN/l concentration for total phosphorous or total inorganic nitrogen respectively tend to be eutrophic while those below the 10 μgP/l or 80 μgN/l concentration line for total phosphorous or total inorganic nitrogen respectively tend to be oligotrophic. Dillon's work should be referred to for a more detailed description and development of the method.

3.5.2 Prediction of Transient Impacts

Steady state mathematical models are useful for determining the long term average concentration of pollutants in the receiving water. For many water quality problems, however, this information is insufficient for a complete evaluation. Significant stormwater impacts leading to violations of receiving water standards and criteria may only occur during, or immediately following, storms. A method is needed for estimating the variability of the receiving water response and the frequency with which stormwater related problems occur.

The most direct method for evaluating the variation of receiving water quality is with a time variable simulation model. The hourly (or any other suitable time interval) stormwater flows and loads are input into the model, and the resulting pollutant concentration is calculated for each hour during the period of interest, such as a season or a given year. The next step is to statistically evaluate the continuous temporal concentration profile calculated by the model to determine its mean, variability and frequency characteristics. Because time variable receiving water simulations are complex and costly, methods have been developed for directly estimating the pertinent characteristics of the receiving water response from steady state models using information on the mean and variability of the stormwater loads.

3.5.2.1 Streams and Rivers

Streams and rivers are characterized by a predominantly advective transport. Storm loads from an urban area enter the river and are transported downstream. In the idealized case, there is no interaction between storm events in the river, and the response to each storm may be calculated independently of any other event. The frequency distribution of instream concentrations is thus directly related to the frequency characteristics of the stormwater loadings.

In the special case of constant flow advective systems, the variability characteristics of the response function as a function of load variability have been investigated (24). In particular, it can be demonstrated that the coefficient of variation of the water quality response at any location is equal to the coefficient of variation of the input loads. Thus, knowing the mean load and its variability, one can compute the mean response using a steady state water quality model and then calculate the variability of the water quality response based on the variability of the load. This is a valid and recommended approach for analyzing variable load impacts on streams where the constant flow assumption is reasonable. However, in situations where intermittent storm related loads are important, the impact of the runoff on the advective flow is often a major factor. A calculation of the impact of each storm event, with a defined frequency of occurrence, is thus required.

The concept is shown diagrammatically in Figure 3-9. Two loads are considered: a continuous steady state load and an intermittent load. The continuous loading rate is characterized completely by its mean, $\overline{W}$, the intermittent loading rate is characterized completely by its mean, W_R, its coefficient of variation, ν_w, and its probability distribution function. The runoff flow associated with each of the loadings is also required. The instream concentrations calculated by the water quality model are peak concentrations which pass a particular location during or following a given storm. The 90th percentile receiving water response is induced by the 90th percentile storm loading and flow, determined by a loading table such as that shown in Table 3-3.

This is a considerably simplified representation of the probabilistic nature of pollutant concentrations in rivers and streams. The frequency distribution of instream concentrations is in reality also affected by variations in the base flow (which may be partially correlated with storm events, depending on the size of the upstream drainage area and areal rainfall patterns) and temperature; both of which are assumed to be constant in this simplified analysis. Variations in these and other factors may be included in a sophisticated continuous simulation by incorporating them as stochastic inputs. For initial planning studies, however, it is felt that a simplified representation based on the frequency characteristics of storm loads and flows provides an adequate basis for estimating and assessing stormwater impacts.

There are two steps in determining the impacts of a particular storm. First, the steady state Streeter-Phelps equations are used to estimate the spatial profile which would result if the given storm load and flow occurred continuously. If there is no dispersion in the river, this would be the concentration observed at each location as the storm pulse passes (23). The second step is to adjust the result to account for attenuation of the pulse due to dispersion.

To understand the effect of dispersion, one may first view the stream as a purely advective, plug flow system. If such a system is loaded with a series of pulse loads of a conservative tracer as indicated in Figure 3-10(a), measurements of a downstream point would yield a series of pulse responses as indicated in Figure 3-10(b). The time between the measured pulses and their magnitude would be directly related to the characteristics of the input loading function and the pertinent stream characteristics such as river flow, channel characteristics, etc.

In natural water systems, there is normally some longitudinal mixing taking place as the pulses move downstream. The effect of such mixing, or dispersion as it is commonly called, is to spread the pulses out, as indicated in Figure 3-10(c). The effects of longitudinal dispersion on wet weather

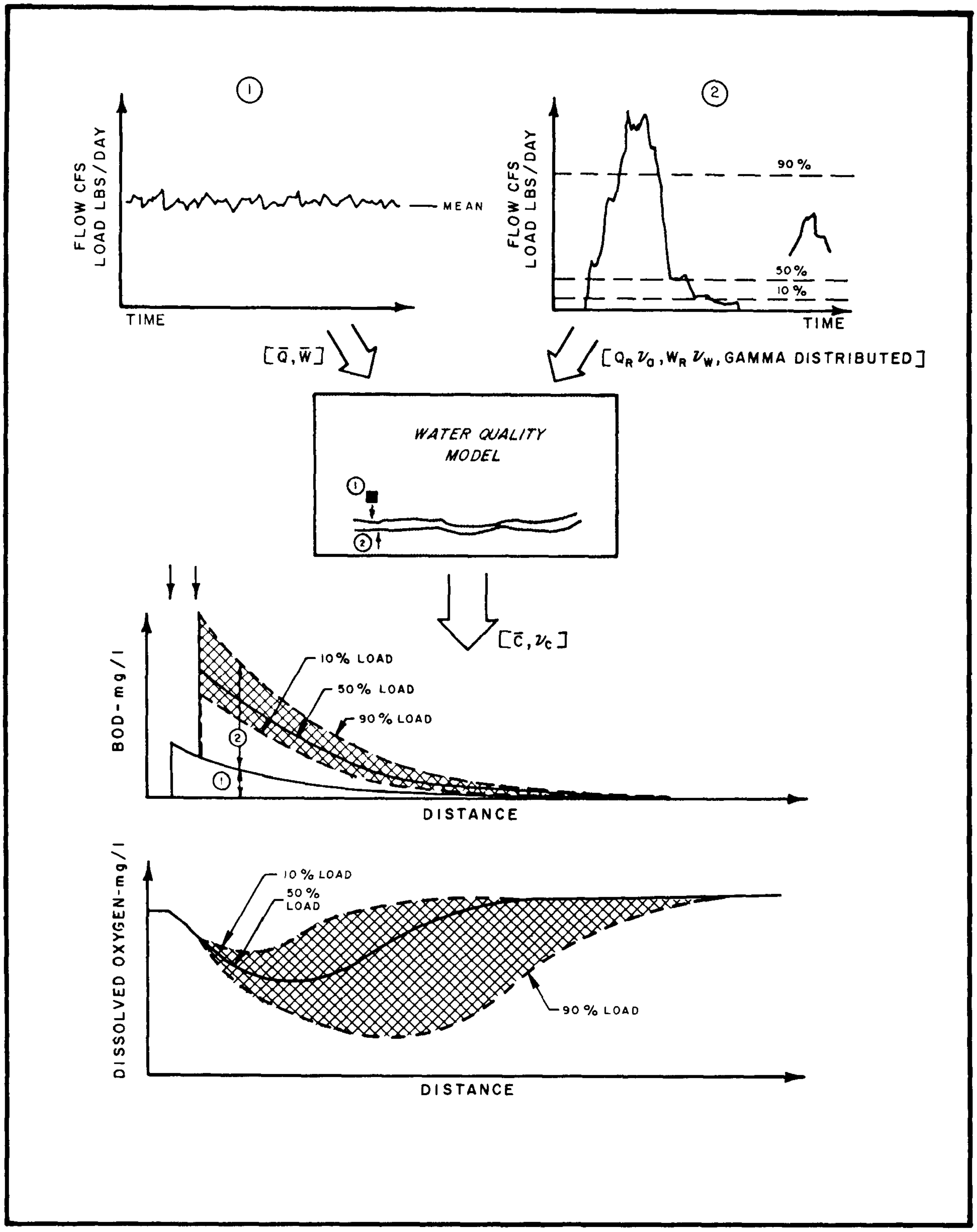


FIGURE 3-9

WATER QUALITY RESPONSE SIMULATOR
BOD-DISSOLVED OXYGEN EXAMPLE

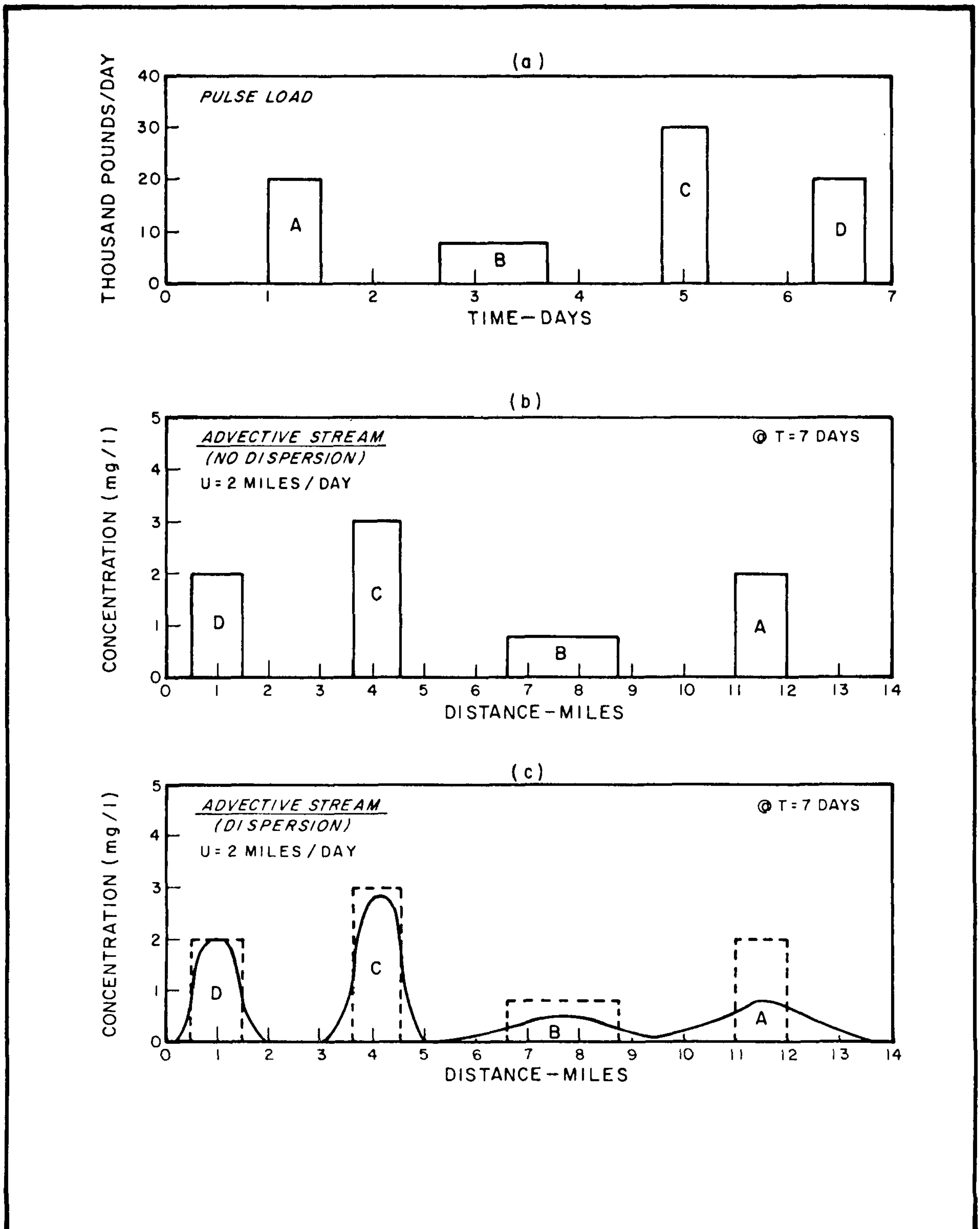


FIGURE 3-10

STREAM RESPONSE CHARACTERISTICS TO PULSE LOADS receiving water quality can be quite dramatic. Under certain conditions peak concentrations in a storm related pulse can be attenuated by 30-60% within 15 miles of the point of stormwater discharge. Figure 3-11 presents a simple graphical solution for determining the degree to which model results should be corrected to account for dispersion. Figure 3-11 was developed based on simulation calculations with and without dispersion (29). The figure indicates the reduction in peak concentration as a function of a dispersive transport factor, α:

$$\alpha = \frac{2Et}{d_r^2 U^2} \tag{3-29}$$

where:

E = longitudinal dispersion coefficient (miles^2/day)

t = time of travel from the discharge point (days)

d_r = duration of the runoff event (hours)

U = river velocity (miles/day)

An α is computed for discrete distances downstream using the time of travel to that point from the discharge location, the average stream velocity through the river segment, and an estimate of the instream longitudinal dispersion. Typically, dispersion coefficients for streams and rivers vary between 0.01 and 1.0 miles^2/day. The site specific value is dependent upon a number of factors which influence velocity gradients. For example, the existence of impoundments or dense aquatic weed growths lead to high dispersion coefficients while narrow, free flowing streams generally have low dispersion coefficients. The best method for determining the dispersion characteristics of a specific stream is through the analysis of dye study results (30). A technique for estimating E in the absence of dye studies is presented in Section 5.4.1.2. As an example of the use of Figure 3-11, assume the following stream characteristics:

E = 0.3 miles^2/day

t = 10 hours = 0.42 days

d_r = 3 hours = 0.125 days

U = 10 miles/day

$$= \frac{2Et}{d_r^2 U^2}$$

$$= \frac{2(0.3)\ (0.42)}{(.125)^2(10)} = 0.16$$

From Figure 3-11, α = 0.16 corresponds to a reduction in the peak concentration of a 3 hour loading pulse to 78% of its initial value at the point of discharge. This will occur a distance:

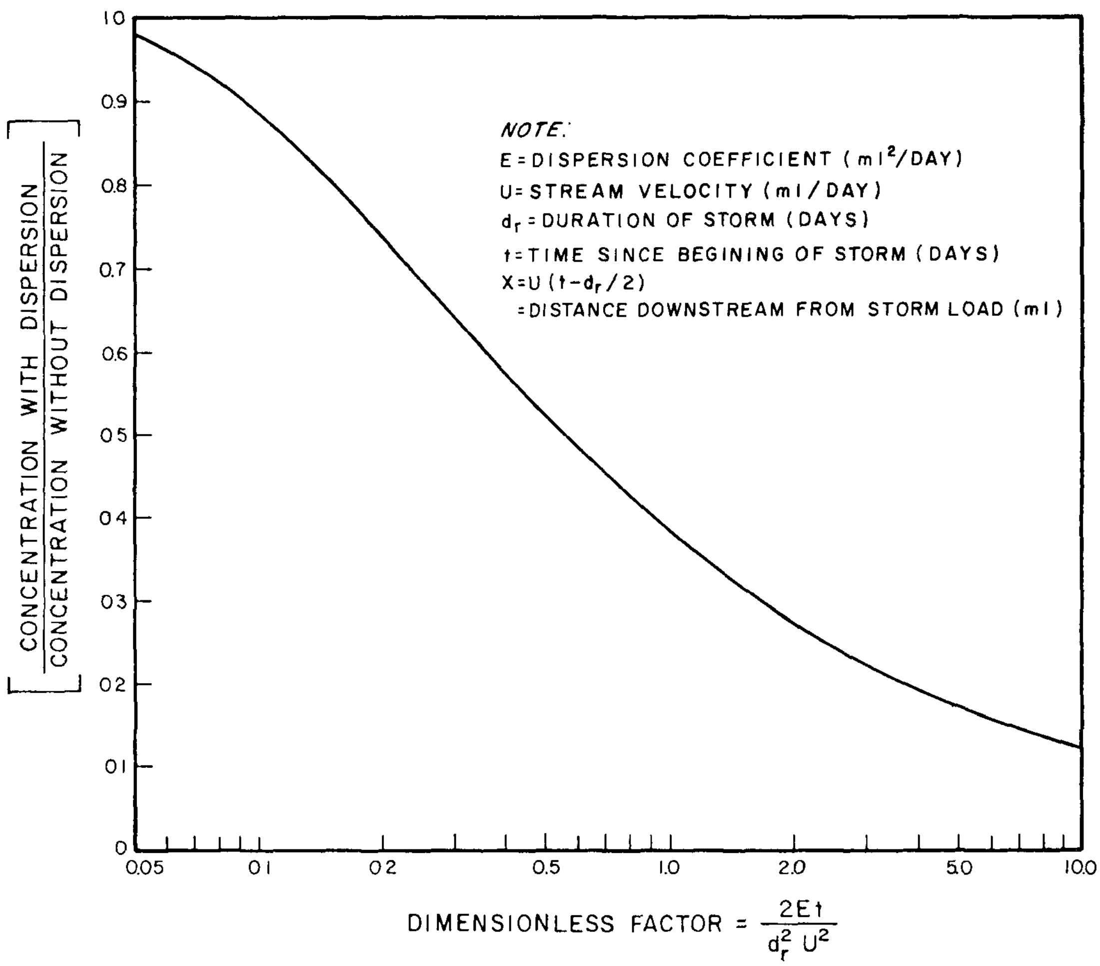


FIGURE 3-11
EFFECT OF DISPERSION ON POLLUTANT CONCENTRATION AT MIDPOINT OF STORM PULSE

$$x = U(t - d_r/2)$$

$$= 10\ (0.42 - 0.125/2)$$

$$= 3.6 \text{ miles downstream of the loading point.}$$

The concentrations calculated represent the maximum concentration of each water quality indicator that will occur as the diluted and dispersed pulse load moves downstream. This minimum concentration is calculated for each storm with a given frequency of occurrence. Note that for a direct mapping of the storm frequency onto the stream imput frequency, the resultant pollutant concentration in the river must increase monotonically with larger storm sizes. The applicability and limitations of this assumption are discussed in Chapter 7 where a detailed example of the computation of probabilistic water quality in streams using the statistical method is presented for Salt Lake City.

3.5.2.2 Estuarine Systems

The high degree of mixing in estuarine systems makes the separate analysis of individual storm events inappropriate. The effects of previous storms may still be prevalent when the current storm occurs, and the impact of each of the storms must be superimposed to determine the total stormwater response. This may be accomplished with a time variable simulation of the loadings and system response.

It is possible however, to estimate the mean and variability of pollutant concentrations in an estuary directly, without continuous simulation (18). The equations are derived from the response shape of a single loading pulse to an advective-dispersive system, and the assumption that these pulses occur as a Poissen process. The mean and standard deviation of the concentration are estimated as:

$$\bar{c} = \frac{M_R/\Delta}{UAm} \exp\left[\frac{Ux}{2E}\ (1 \pm m)\right] \tag{3-30}$$

$$\sigma_c = \frac{\sigma_m^2 + M_R^2}{A\sqrt{2\pi E\Delta}} \quad \exp \quad \frac{Ux}{2E} \quad K_o^{\frac{1}{2}} \quad \frac{Uxm}{E} \tag{3-31}$$

where:

$m = \sqrt{1 + 4KE/U^2}$

M_R = mean storm load (lb/storm)

σ_m = standard deviation of storm load

U = freshwater velocity = (freshwater flow)/ a

A = cross sectional area

E = dispersion coefficient

K = reaction rate

x = distance from storm load (The sign in Equation 3-30 is negative when $x \geq 0$, positive when $x < 0$. Positive x is in the direction of freshwater flow).

Δ = mean time between storms

K_o = modified Bessel function (Note $K_o(-b) = K_o(b)$). Figure 3-12 or tables of modified Bessel functions may be used for evaluating this term (31).

The solution for the mean concentration is the same as the steady state solution which would be calculated using the long term average loading rate, $W_o = M_R/\Delta$. This is reasonable given the fact that the total mass of pollutant entering the estuary is the same, whether it occurs as discrete pulses, or evenly distributed in time. The standard deviation of the receiving water response increases as the storm loads increase (increasing M_R) and more variable (increasing σ_m). The variability within tidal cycles is not included in this analysis.

The simplicity of Equations 3-30 and 3-31 is remarkable. The equations allow the simulation procedure to be by-passed in the calculation of $\bar{c}$ and σ_c. To illustrate the results of this calculation, an example using stormwater coliform loads into a simple, one dimensional model of the lower Hudson River Estuary is summarized in Figure 3-13. The theoretical results are calculated using Equations 3-30 and 3-31. The simulator results are from an hourly simulation using Central Park raingage data transformed into runoff loads. Note the close agreement between the theoretical and the simulated results. The highly variable nature of the response is reflected in the fact that the standard deviation is from one to two times the mean concentration. The final step in the analysis is to determine the frequency distribution of the estuary response to predict, for example, what the 90th percentile coliform concentration is at a given location. Further research is currently under way to examine this problem.

3.6 Assessment of Stormwater Control Alternatives

Once the magnitude of stormwater impacts on receiving water quality are estimated, control strategies for the reduction of these impacts may be analyzed. A variety of stormwater control alternatives are available. These are generally grouped into two types of approaches:

1. Structural, end-of-pipe treatment devices.
2. Management practices.

Structural, end-of-pipe alternatives include devices which capture and store runoff, such as interceptors and retention basins, and devices which reduce the pollutant concentration of runoff or overflows, such as screens, filters, concentrators, and disinfection systems. Management practices include source

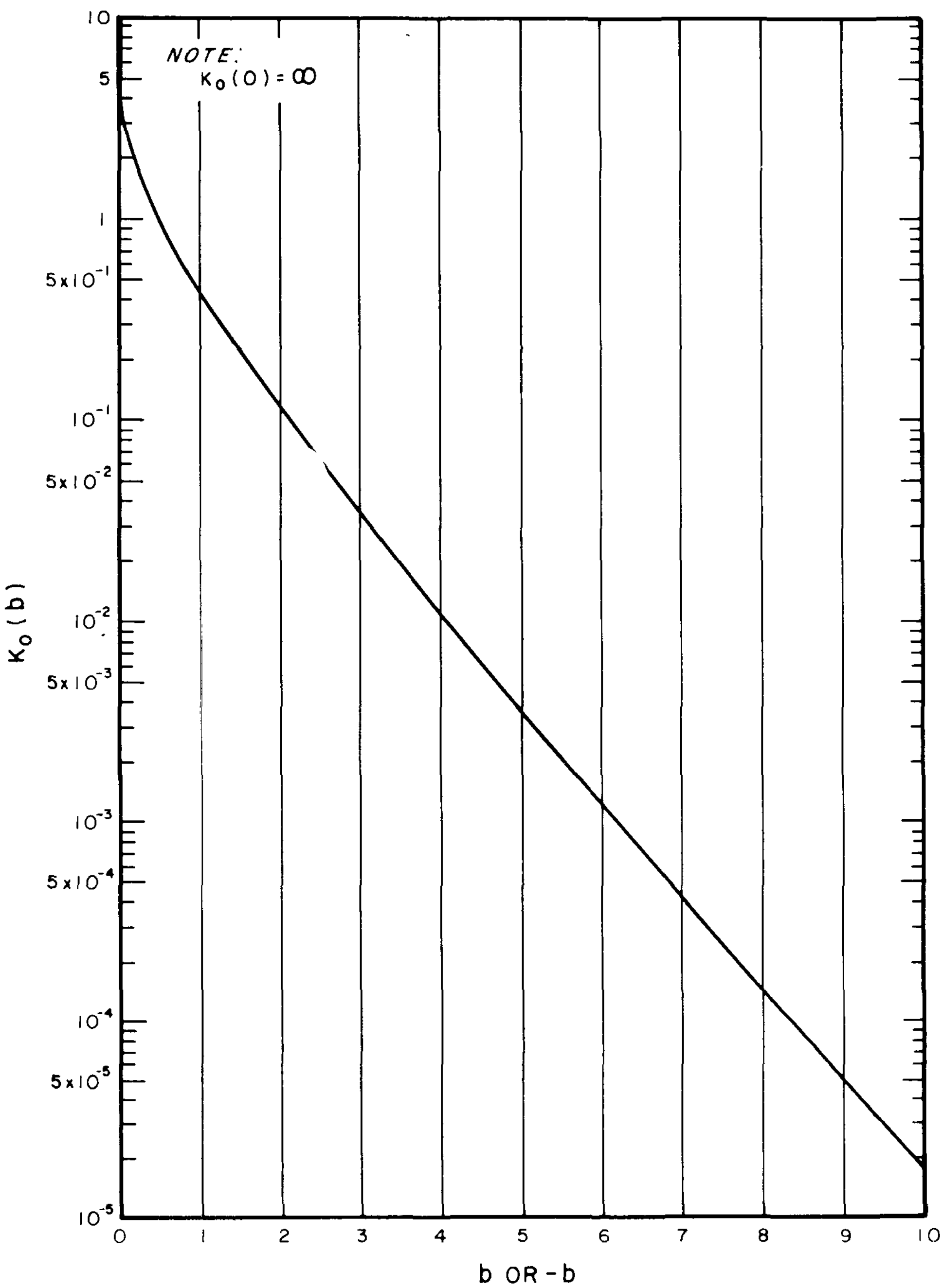


FIGURE 3-12
GRAPHICAL SOLUTION TO K_0 BESSEL FUNCTION

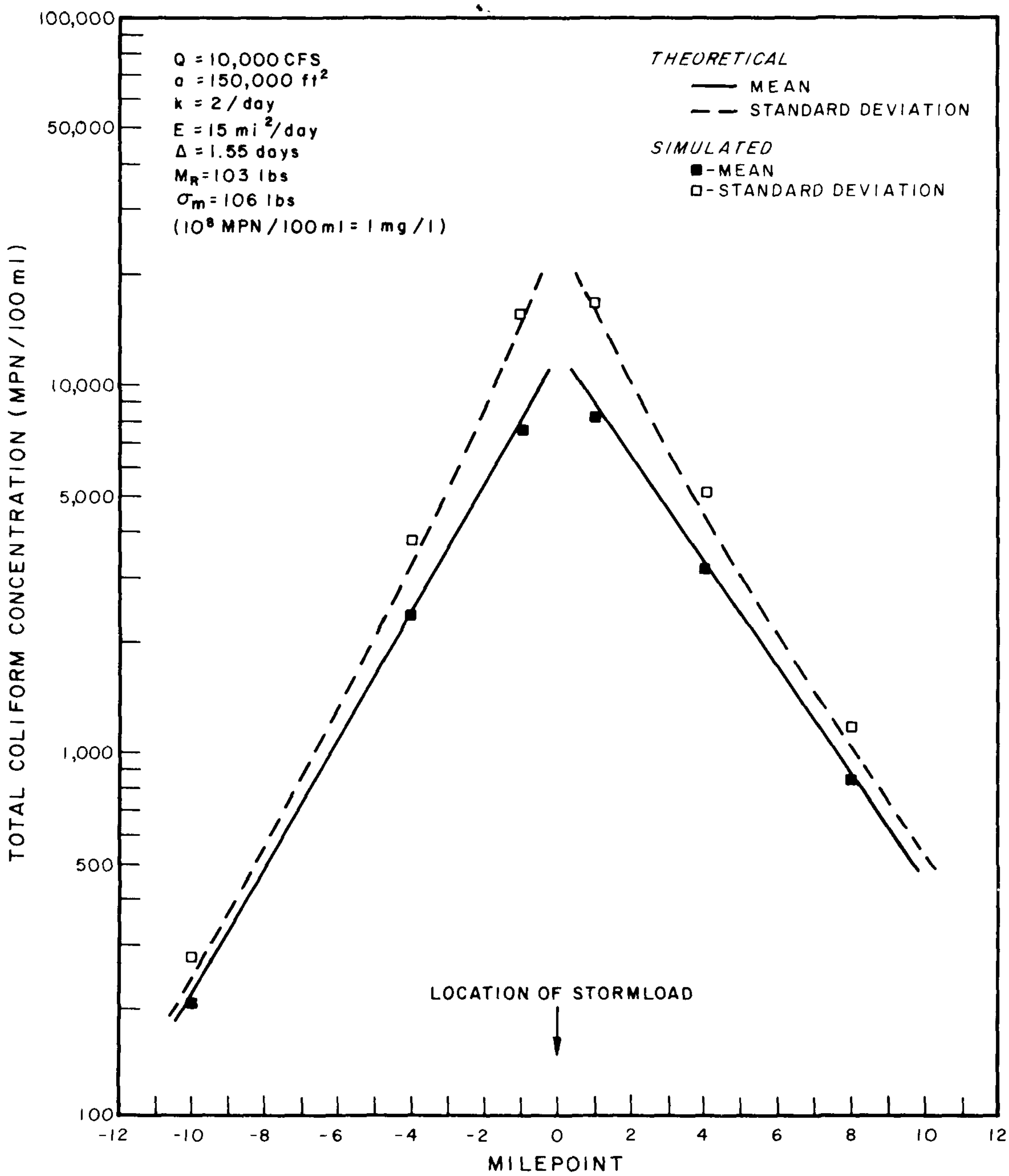


FIGURE 3-13
COMPARISON OF THEORETICAL AND SIMULATED
ESTUARY RESPONSE TO STORMLOADS
HUDSON RIVER, RAINGAGE 305801, JULY-AUGUST 1969, HOURLY RECORDS

controls, such as street sweeping and erosion control; and collection system management techniques, such as sewer flushing and polymer injection to increase the flow capacity of the sewerage system.

A brief description of each control alternative is presented in this section together with quantitative methods for estimating their effectiveness. The basic statistical properties of the runoff loads and the characteristics of the treatment alternatives are used to determine modified stormwater loads to the receiving water. The projected improvement in receiving water quality due to the modification of stormwater loads then represents the benefit of potential stormwater control actions.

3.6.1 Structural Treatment Devices

Stormwater control devices may be constructed to provide a given level of treatment for a fixed runoff flow, storm duration and influent concentration. Treatment performance will change, however, as the storm runoff characteristics vary from storm to storm. A statistical method of analysis is described in this section, which focuses on the determination of the long term performance efficiency of devices subjected to the varying rainfall-runoff process.

The structural control devices considered are grouped into two basic categories: (1) those which capture and store runoff, and (2) those which reduce the pollutant concentration of the stormwater. The first group is typified by interceptors and retention basins or tanks.

The operation of interceptors and storage devices is depicted in Figure 3-14. The storm runoff process is represented as a series of independent events, as shown in Figure 3-14(b). The interceptor removes a constant flow rate, Figure 3-14(c), the storage device captures a fixed volume, Figures 3-14(d), and the combination of interception and storage removes a constant flow rate and captures a fixed volume of the interceptor overflow (Figure 3-14(e)). The unshaded areas in Figure 3-14 represent the uncaptured portion of the storm runoff. This provides the basic theoretical framework for the analysis of the long term performance of devices which capture and store runoff.

3.6.1.1 Interception

An interceptor captures up to a flow rate, Q_I, the available treatment plant capacity. Thus, Q_I is the total capacity minus the dry weather flow. The portion of the runoff in excess of Q_I overflows into the receiving water through established relief points in the system. The performance of the interceptor is important because it captures a portion of the runoff which may subsequently receive treatment; either at the municipal treatment plant of a combined sewer system, or at special stormwater treatment facilities for separate storm sewer systems.

The long term fraction of the runoff load, f_I, not captured by an interceptor is calculated as the expectation of the runoff load that overflows $[\bar{c}d(q-Q_I)]$ divided by the total runoff load:

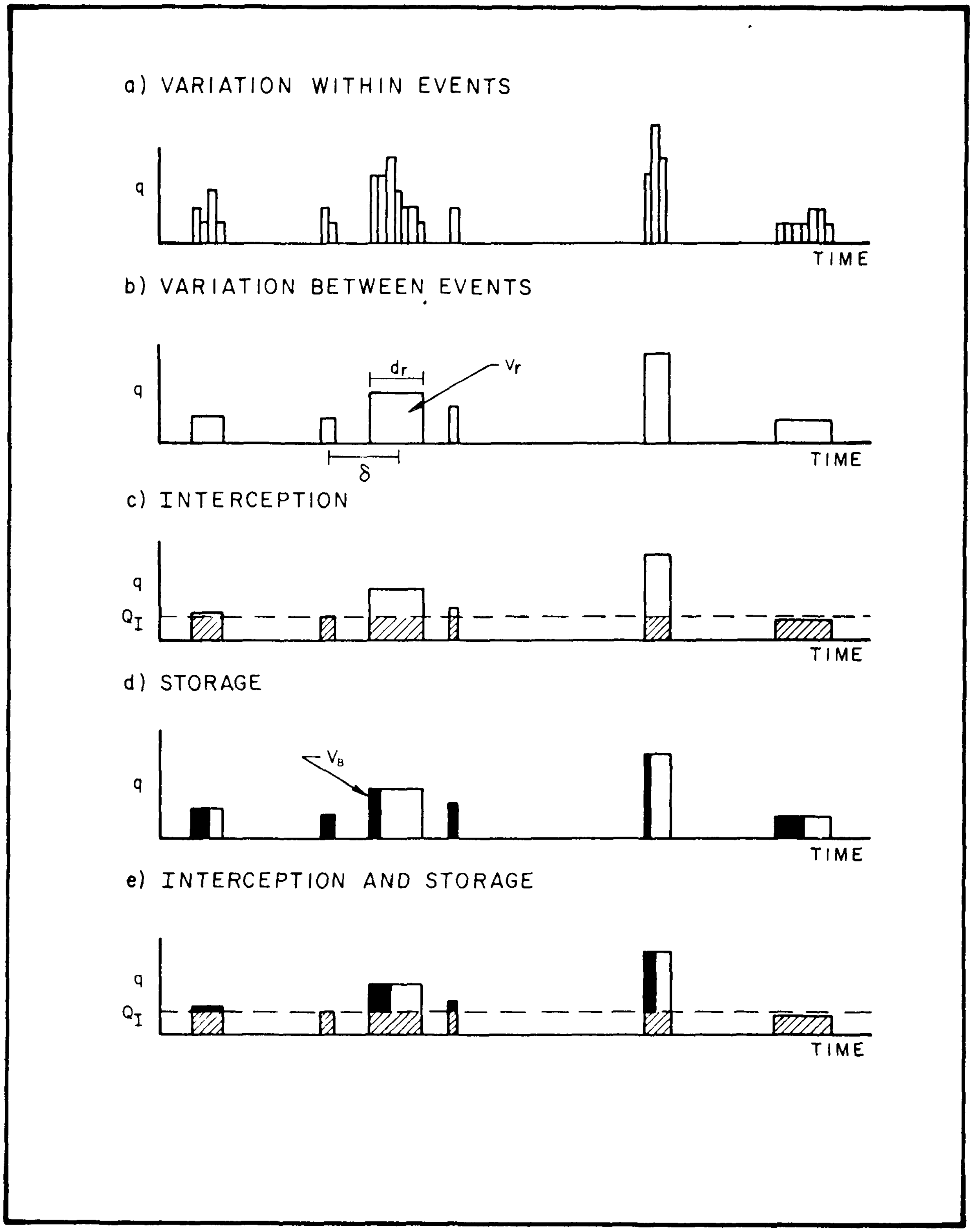


FIGURE 3-14

REPRESENTATION OF
STORM RUNOFF PROCESS, INTERCEPTION AND STORAGE

$$f_I = \frac{\overline{M}}{M_R} = \frac{\bar{c} \int_{q=Q_I}^{\infty} \int_{d=0}^{\infty} (q-Q_I) d \; p_d(d) \; p_q(q) \; dd \; dq}{\bar{c} Q_R D} \qquad (3\text{-}32)$$

where $\overline{M}$ is the mean overflow load per storm, and $p_d(d)$ and $p_q(q)$ are the probability distribution functions of storm durations and flows respectively, which are assumed to be independent and gamma distributed. The integrals in Equation 3-32 are evaluated numerically, and the results are shown in Figure 3-15. The fraction of the runoff load which is uncaptured, f_I, is a function of the normalized interceptor size, Q_I/Q_R, and the coefficient of variation of the runoff flows ν_q. *The greater the variation in runoff flow,* the more poorly the interceptor performs on average. Note the diminishing increases in the amount of runoff captured for each increment in interceptor size.

The use of Figure 3-15 may be demonstrated with an example. Assume a drainage area with an interceptor has the following characteristics:

Mean runoff flow	$= Q_R = 10$ cfs
Variation of runoff flow	$= \nu_q = 1.15$
Available interceptor capacity	$= Q_I = 12.5$ cfs

For $Q_I/Q_R = 1.25$, and interpolating between the curves for $\nu_q = 1.00$ and $\nu_q = 1.25$ for the case when $\nu_q = 1.15$, Figure 3-15 indicates that the fraction of the runoff load not captured is: $f_I = 0.33$.

The analysis has assumed that the variation of flow within storms is small compared to the variation of flow between storms. When this is not the case, the performance level of the interceptor will be further reduced beyond that shown in Figure 3-15. The analysis has also assumed that the concentration of the runoff is independent of the flow. If this is not the case, and higher flows tend to have higher concentrations, the interceptor will perform more poorly than predicted in Figure 3-15. If higher flows tend to have lower concentrations, the device will perform better than predicted in Figure 3-15. Analysis of runoff data have thus far indicated that these effects are not of major importance for a first estimate.

3.6.1.2 Storage

A storage device captures up to a capacity volume, V_B, and the remaining flow from the storm is by-passed. The captured runoff may then be discharged at greatly reduced flow rates, with or without treatment, to the receiving water, or pumped to the interceptor for treatment at the municipal or industrial treatment plant. The storage capacity allows a significant reduction in the size of the treatment facilities required and provides a reduction in the magnitude of the shock load delivered to the receiving water.

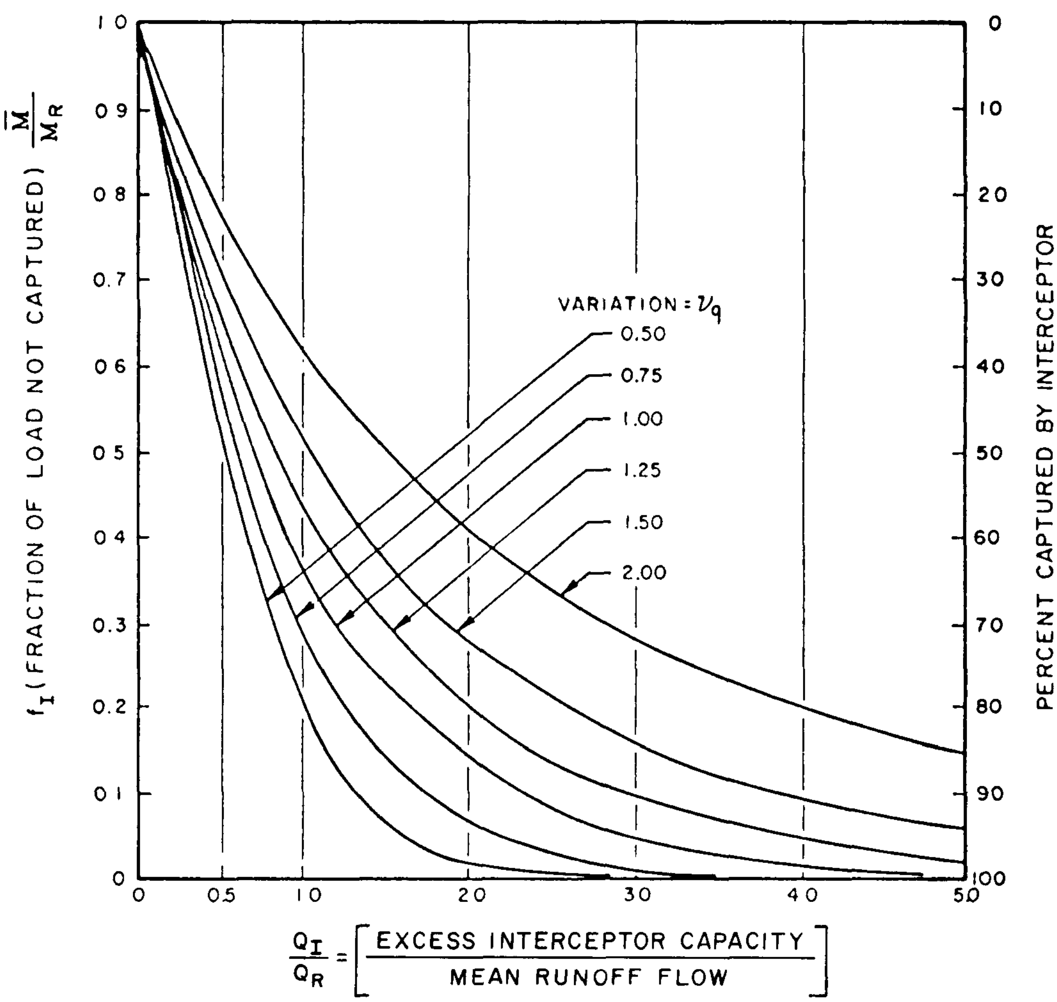


FIGURE 3 - 15

DETERMINATION OF LONG TERM INTERCEPTOR PERFORMANCE

3.6.1.2.1 Effect of Previous Storms

The total storage may not always be available at the beginning of a particular storm. The basin may still have leftover stored runoff from previous storms. The storage that is available on the average, termed the effective storage capacity, V_E, will determine the long term performance of the retention basin. The effective storage capacity, V_E, is a function of the actual size of the basin, V_B, the mean runoff volume, V_R, the rate at which the basin is emptied, Ω, and the average time between storms, Δ.

The derivation of the solution for the effective storage capacity is outlined in Figure 3-16. Storm 1 is assumed to begin with the long term effective storage capacity available. It rains and a volume, v, further fills the basin. Between storm 1 and storm 2 the tank is emptied at a rate Ω. The basin then has an available storage capacity of V_e at the beginning of storm 2. The problem is to find the expectation of V_e over the possible values of v and δ. This expectation of V_e is the long term effective volume of the basin, V_E.

The integral of Figure 3-16 is solved for the special case when $\nu_q = \nu_d = \nu_\delta = 1$ (i.e., runoff flows durations, and the time between storms are exponentially distributed and independent). The results, normalized by the mean runoff volume, V_R, are displayed in Figure 3-17.

This analysis provides a useful guideline for estimating the effect of previous storms and determining an adequate treatment rate for the storage volume. The expression $\Delta\Omega$ may be thought of as the average captured volume processed (i.e., emptied from the tank) between storms. The smaller this value is relative to V_R, the more likely the basin will still contain leftover runoff with a storm begins, and the effective storage capacity, V_E, is lower. When $\Delta\Omega/VR > 2$, there is very little loss in effective storage volume. When $\Delta\Omega/V_R < 2$, however, the effective storage drops rapidly. The drop is more pronounced in large basins where large storms may be accumulated rather than by-passed or overflowed.

Note that some error in the estimate of the effect of previous storms may be introduced due to deviations from the assumptions used to derive the curves of Figure 3-17; for example, the coefficient of variation of storm volumes may not equal $\sqrt{3}$ as assumed in the calculations, or the actual storage device may be operated with a variable emptying rate. The estimate is still useful, however, for an initial assessment and screening of storage treatment.

3.6.1.2.2 Storage Effectiveness

Once the effective storage capacity has been determined, the analysis of basin performance may proceed. The long term fraction of the runoff load, f_V, not captured by a storage device is calculated as the expectation of the by-passed runoff load, $\bar{c}q(d-V_E/q)$, divided by the total runoff load:

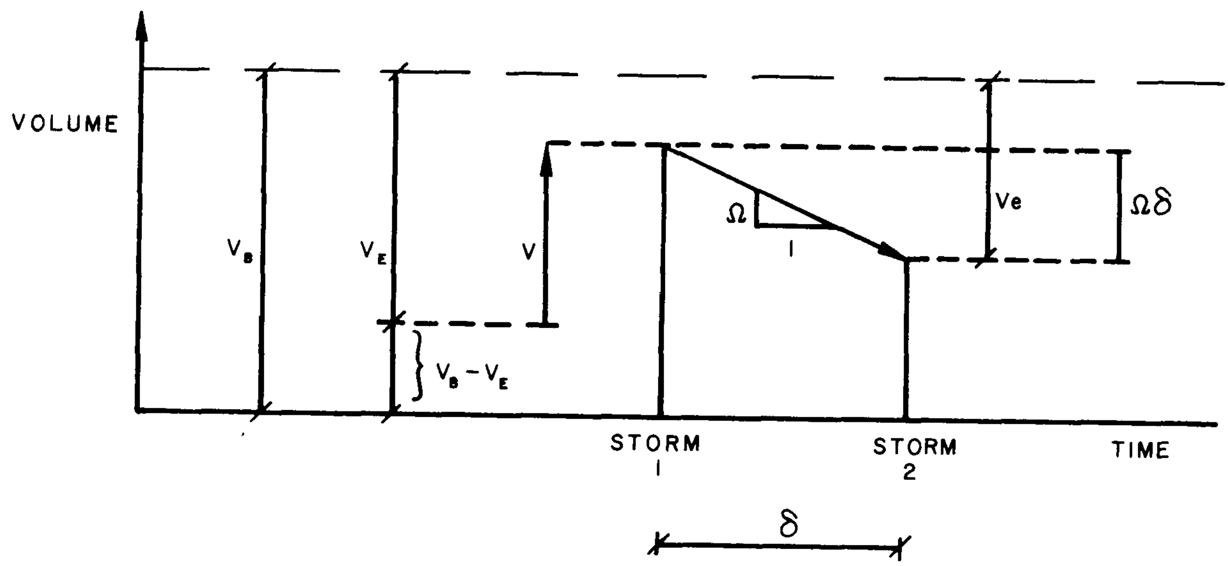


FIGURE 3-16

DERIVATION OF SOLUTION FOR EFFECTIVE TANK VOLUME

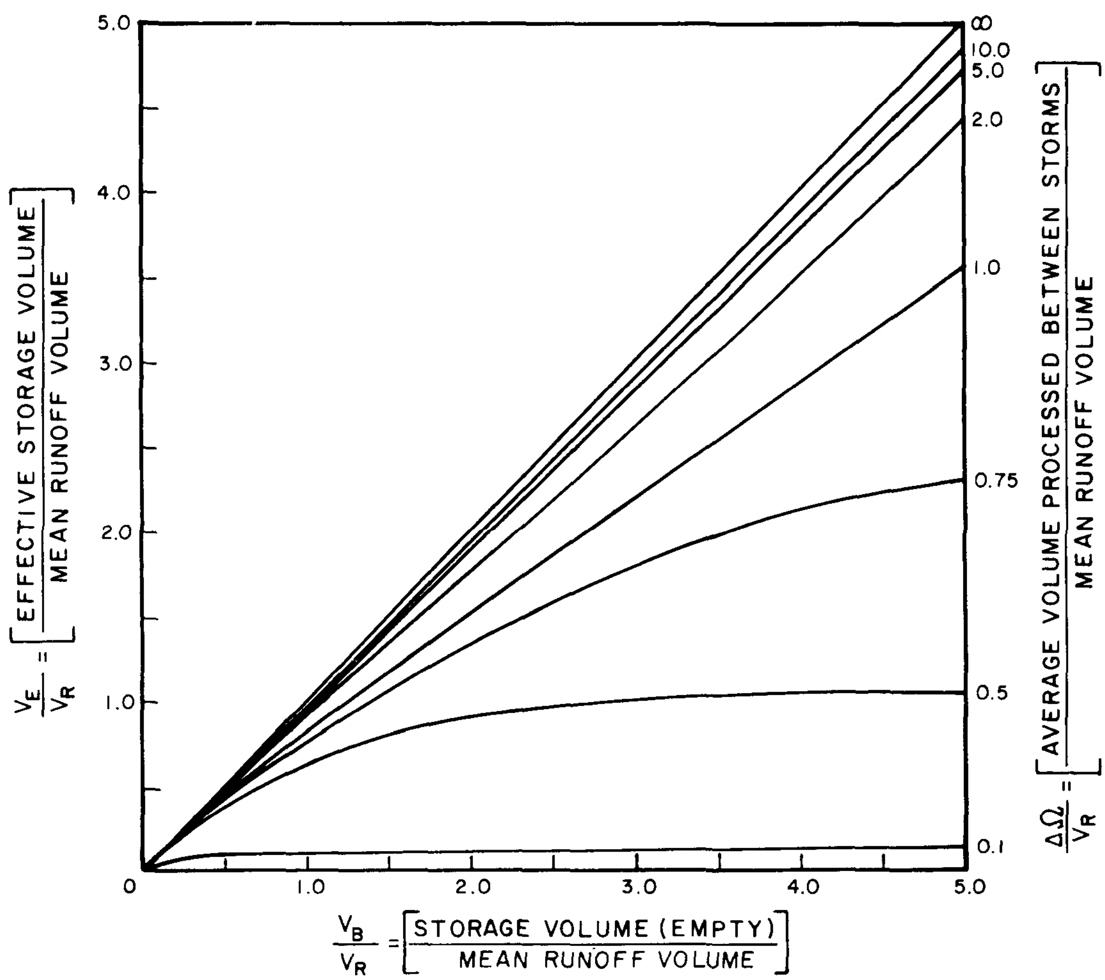


FIGURE 3-17
EFFECT OF PREVIOUS STORMS
ON LONG TERM EFFECTIVE STORAGE CAPACITY

$$f_V = \frac{\overline{M}}{M_R} = \frac{\bar{c} \int_{q=0}^{\infty} \int_{d=V_E/q}^{\infty} q(d-V_E/q)\, p_d(d)\, p_q(q)\, dd\, dq}{\bar{c}\, Q_R D} \qquad (3\text{-}33)$$

where $\overline{M}$ is the mean by-passed load per storm. Equation 3-33 is evaluated numerically, and the results shown in Figure 3-18. The fraction of the runoff load which is not captured, f_V, is a function of the normalized storage capacity, V_E/V_R, and the coefficient of variation of the runoff volumes, ν_{VR}. The use of volume statistics makes the curves applicable to situations in which runoff flows and durations are not independent, as is often the case. Note that for a given coefficient of variation, the curves in Figure 3-18 are very similar to the curves in Figure 3-15. This suggests that there is a similarity between a random variable which is gamma distributed (runoff flows) and a random variable which is the product of two independent, gamma distributed, random variables (runoff volumes). The assumption that storm runoff volumes are themselves gamma distributed is probably quite adequate (16).

3.6.1.2.3 First Flush Effect

The curves in Figure 3-18 are developed without consideration of a first flush effect. Therefore, the curves actually represent the fraction of the runoff volume captured, rather than the runoff load. Adjustments should be made to account for the first flush effect when it exists. Because a first flush effect results in a disproportionately high fraction of the runoff load in the first portion of the discharge or overflow, a correspondingly high fraction of the runoff load is captured by a storage device.

The improvement in storage device performance associated with the first flush effect is evaluated by assuming that the temporal concentration profile of the runoff has an exponential shape as shown in Figure 3-5. The results of this analysis are simplified and presented in Figure 3-19, which may be used to adjust the performance curve selected on Figure 3-18, with the corresponding new values of f_V. The magnitude of the first flush effect to be used in Figure 3-19 may be estimated from the ratio of the peak pollutant concentrations generally found at the beginning of storms, c_p, to pollutant concentrations observed after the first flush subsides, c_o. Table 3-7 provides general guidelines for estimating the magnitude of the first flush effect. As previously stated, local monitoring of a sufficiently large number of storm events will be necessary to reliably characterize the first flush effect actually present in a study area. However, as shown in Figure 3-19, the influence on device performance may not be of major importance.

The analysis of the first flush effect assumes that the storage device is operated in a by-pass mode, that is, by-passing the later storm runoff flows after the device is full. The device may also be operated in an overflow mode; accepting all storm runoff flows in one end, and overflowing from the other when the basin becomes full. This will negate the storage improvements related to the first flush effect, however, some treatment may be provided within the storage device, such as sedimentation, which may make the overflow mode more favorable. These factors should be considered and weighed when designing a storage-treatment system.

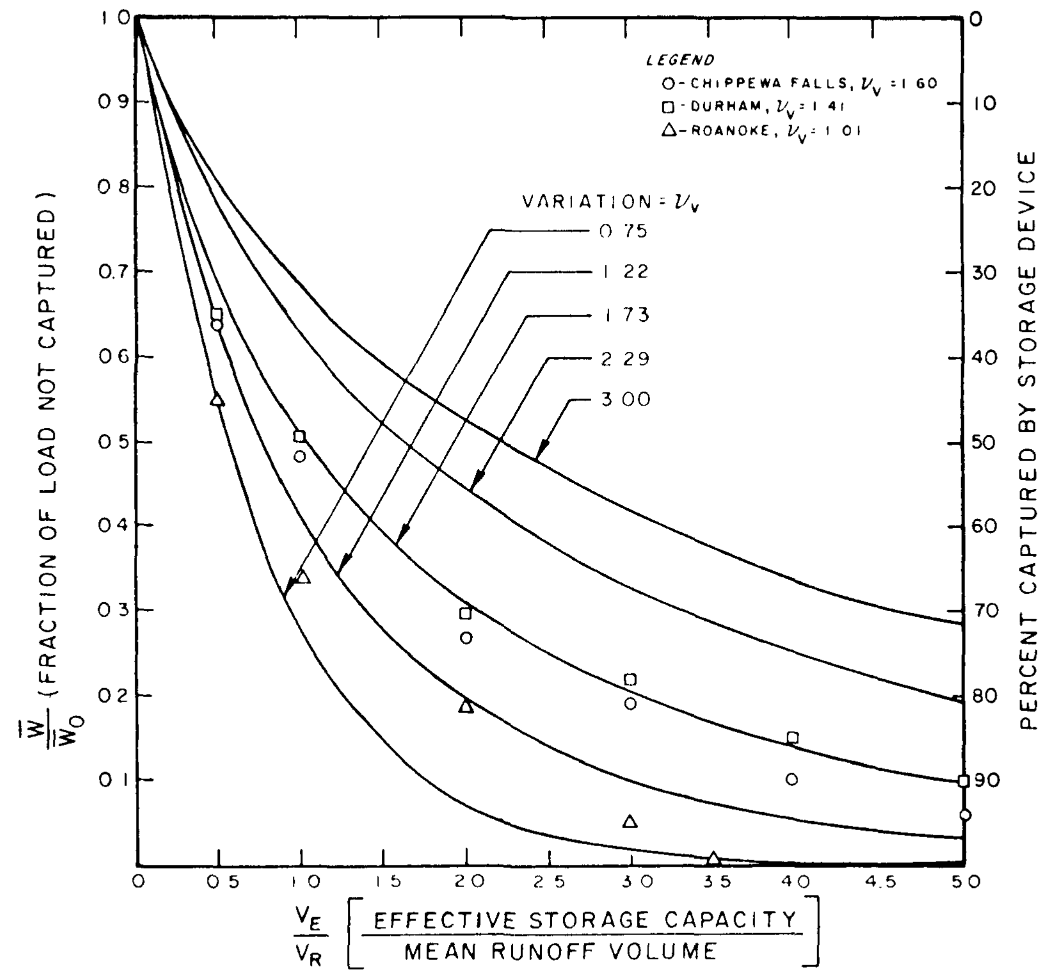


FIGURE 3-18

DETERMINATION OF LONG TERM STORAGE DEVICE PERFORMANCE

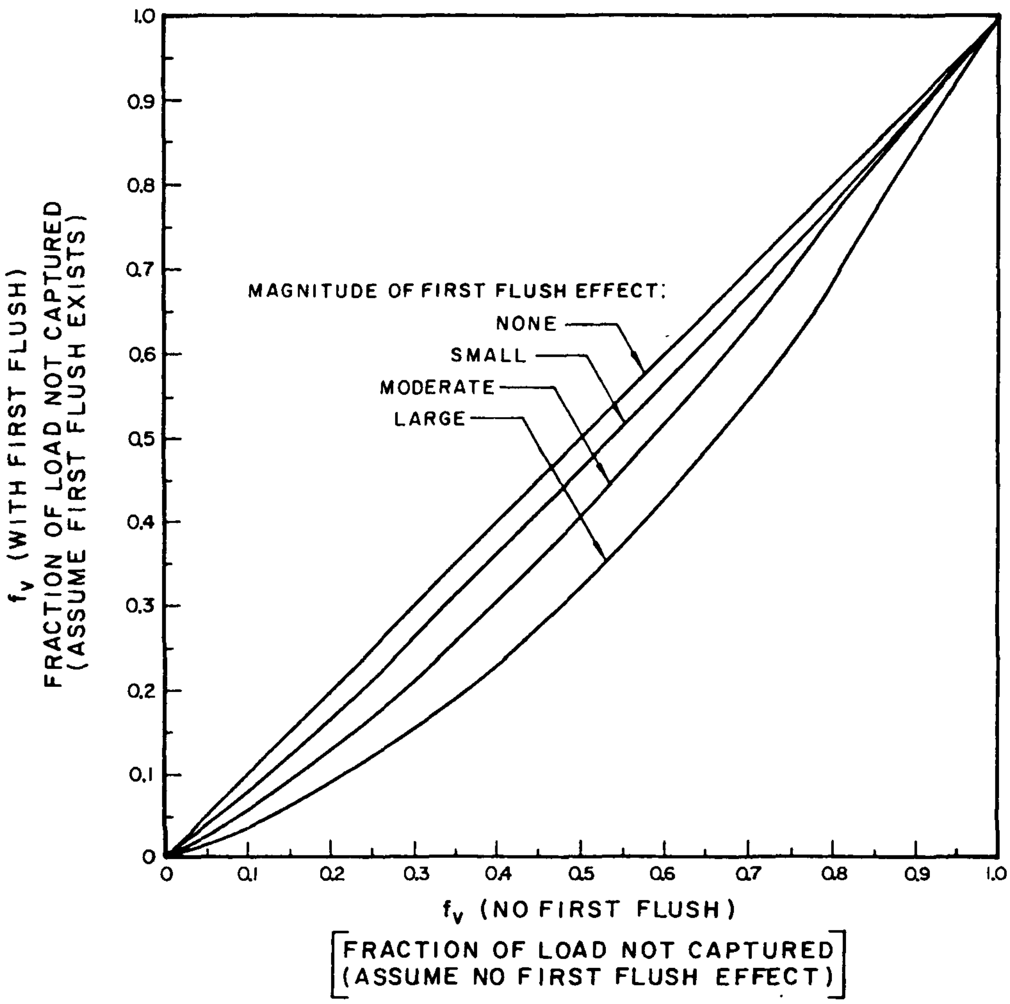


FIGURE 3-19
IMPROVEMENT IN LONG TERM STORAGE DEVICE PERFOMANCE
DUE TO FIRST FLUSH EFFECT

TABLE 3-7

GENERAL GUIDELINES FOR ESTIMATING MAGNITUDE OF FIRST FLUSH EFFECT

Ratio of Peak to Final Concentrations (c_p/c_o)	Magnitude of First Flush Effect
1.0 - 1.5	Small
1.5 - 4.0	Moderate
> 4.0	Large

Assumed $D_R/\beta = 1.0$

3.6.1.2.4 Treatment of Stored Runoff

Once the fraction of the runoff load captured by a storage device has been determined, the effects of treating the stored portion of the runoff (off-line treatment) may be incorporated. Treatment may occur either through settling or reaction in the basin itself or by pumping the stored stormwater through a treatment device. The treatment rate should be controlled to maximize the overall treatment benefits. A lower treatment rate is desirable to improve the pollutant removal and to attenuate the release of stormwater into the receiving stream. If the rate is too low, however, the effective storage capacity may be reduced, as described previously. Assuming the captured runoff is treated with a percent removal (r), the modified average runoff load (M_R^*), will be:

$$M_R^* = f_V M_R + (1 - f_V) M_R (1 - r) \qquad (3\text{-}34)$$

Again, note that this modified load will enter the receiving water over a longer time period than the original storm runoff.

3.6.1.2.5 Example of Storage Device Evaluation

The determination of storage device performance may be demonstrated with an example. Assume a drainage area served by a storage device has the following characteristics:

Mean runoff volume = V_R = 4 MG (.536 x 10^3 ft^3)

Variation of runoff volume = ν_{vR} = 1.75

Mean runoff load = M_R = 2,000 lb BOD

First flush effect = moderate

Average time between storms = Δ = 84 hr = 3.5 days

Storage volume (empty) = V_B = 6 MG (.804 x 10^6 ft^3)

Emptying rate from storage = Ω = 3 MGD (4.6 cfs)

Percent removal of BOD_5 in stored runoff = r = 50%

To determine the effective storage capacity, the ratio of the average volume processed between storms to the mean runoff volume is calculated:

$$\Delta\Omega/V_R = (3.5 \text{ days})\ (3 \text{ MGD})/(4 \text{ MG}) = 2.63$$

The ratio of the empty storage volume to the mean runoff volume is:

$$V_B/V_R = (6 \text{ MG})/(4 \text{ MG}) = 1.5$$

Figure 3-17 is then used to determine the effective storage capacity:

$V_E/V_R = 1.35$

$V_E = 5.4$ MG

The long term performance, assuming no first flush effect, is determined from Figure 3-18. For $V_E/V_R = 1.35$ and $\nu_{VR} = 1.75$, Figure 3-18 indicates the long term fraction of the runoff not captured:

$f_V = 0.45$

The improved treatment due to the moderate first flush effect is then incorporated, using Figure 3-19. Adjusting $f_V = 0.45$ for a moderate first flush effect, Figure 3-19 indicates the revised fraction of the runoff load not captured is:

$f_V = 0.35$

Given the average runoff load $M_R = 2{,}000$ lb BOD_5, and the percent BOD removal of the captured runoff load $r = 50\%$, the modified average runoff load is:

$$M_R^* = f_V M_R + (1 - f_V) M_R (1 - r)$$

$$= 0.35(2{,}000) + (0.65)(2{,}000)(0.50)$$

$$= 1{,}350 \text{ lb } BOD_5$$

To demonstrate the long term average pollutant reduction achieved by the storage treatment system, the yearly stormwater BOD_5 load (Y_m) may be calculated with and without the control:

Before storage:

$$Y_m = (M_R/\Delta)(365.25 \text{ days/year})$$

$$= (2{,}000 \text{ lb } BOD_5/3.5 \text{ days})(365.25 \text{ days/year})$$

$$= 208{,}700 \text{ lb } BOD_5\text{/year}$$

After storage:

$$Y_m = (M_R^*/\Delta)\ (365.25 \text{ days/year})$$

$$= (1350 \text{ lb } BOD_5/3.5 \text{ days})\ (365.25 \text{ days/year})$$

$$= 140{,}900 \text{ lb } BOD_5\text{/year}$$

A 32% reduction in the long term average BOD_5 load is indicated. In addition, because of the storage device, part of the load discharged to the receiving water can be spread over an extended period, rather than just during or immediately following storm events.

3.6.1.3 Interception and Storage

Interceptors and storage devices may be operated in combination. The interceptor captures up to a flow rate, Q_I. The overflow from the interceptor is stored until the basin capacity is reached, after which overflows from the interceptor are by-passed. This is equivalent to the "storage-treatment" system commonly analyzed with simulators. The treatment component can be thought of as available interceptor capacity.

The long term fraction of the runoff load, f_{IV}, not captured by an interceptor in combination with a storage device is calculated as the expectation of the runoff load both overflowed and by-passed, divided by the total runoff load:

$$f_{IV} = \frac{\overline{M}}{M_R} = \frac{\bar{c} \int_{q=Q_I}^{\infty} \int_{d=V_E/(q-Q_I)}^{\infty} (q-Q_I)\ d-V_E/(q-Q_I)\ p_d(d) p_q(q) dd\ dq}{\bar{c}\ Q_R D} \qquad (3\text{-}35)$$

Equation 3-35 is evaluated numerically and the fraction of the runoff load not captured by both the interceptor and the storage device is well estimated by the product of the individual fractions remaining for each device:

$$f_{IV} = f_I f_V \qquad (3\text{-}36)$$

Equation 3-36 is exact when $\nu_q = 1$, slightly underestimates f_{IV} when $\nu_q > 1$, and slightly overestimates f_{IV} when $\nu_q < 1$. The error of the estimate is small compared to the overall uncertainties involved in stormwater treatment analyses.

The effect of treating the captured portion of the runoff may again be incorporated for the combination interceptor-storage device system. This is typified by a combined sewer system where the interceptor routes runoff to the municipal treatment plant and storage is added to capture overflows which are subsequently pumped back to the interceptor and the treatment plant. Assuming the treatment plant provides a percent removal, r, for both flows captured by the interceptor and for stormwater retained by the storage device and subsequently returned, the modified runoff load, M_R^*, is:

$$M_R^* = f_{IV} M_R + (1 - f_{IV}) M_R (1 - r) \qquad (3\text{-}37)$$

This assumes that the percent removal (r) obtained at the treatment facility is similar for both the storm flows which reach it during the rainfall event and for the captured stormwater returned later at controlled rates. This is not usually the case, however, and it may be prefereable to analyze such a system as an in-line treatment device whose percent removal decreases with flow, as will be described in the following section.

3.6.1.4 In-Line Treatment Devices Which Reduce Pollutant Concentration

A number of treatment devices have been applied as control measures to reduce the concentration of stormwater overflows and runoff. The pollutant removal efficiency will vary from one treatment device to another, and may also vary in any one unit operation due to variations in the flow rate and the influent waste load characteristics. In general, as more flow passes through a device of a given size, the efficiency of removal will decrease. Furthermore, the removal efficiency (expressed as percent removal) of some treatment devices, in particular screens and sedimentation tanks, will be enhanced when the influent suspended solids concentration increases.

The performance of most treatment devices may be approximated by an exponential decrease in the percent removal as the flow increases. Most treatment devices cannot be subjected to an excessive large flow rate, however, and runoff beyond a certain flow rate must be by-passed. Hydraulic capacity limitations as well as process considerations will determine the maximum flow accepted. The effect of this by-pass may be approximated as a continued exponential decrease in the overall treatment efficiency (including processed and by-passed stormwater). This is demonstrated for dissolved air flotation in Figure 3-20. The assumption that the percent removal decreases exponentially as the flow increases is an approximation and may not describe the performance of some devices as well as it does others; however, this approach is a useful simplification and is reasonable for initial assessments.

For an exponential performance curve the relationship between flow and percent removal is described analytically by the following equation:

$$r(q) = Z \exp(q \ln(\tfrac{F}{Z})/Q_R) \qquad (3\text{-}38)$$

where:

$r(q)$ = percent removal as a function of q

q = influent flow

Q_R = mean runoff flow

Z = best percent removal obtainable at very low flows

F = percent removal at the mean runoff flow, Q_R.

The relationship is shown graphically in Figure 3-21.

The long term average reduction, P_F, obtained by a treatment device which is subject to varying runoff flows is calculated from the expectation of the portion of the load remaining:

$$1 - P_F = \frac{1}{M_R} \int_{q=o}^{\infty} \int_{d=o}^{\infty} \int_{c=o}^{\infty} 1\text{-}r(q)\ qdc\ p_q(q)\ p_d(d)\ p_c(d)\ dq\ dd\ dc \qquad (3\text{-}39)$$

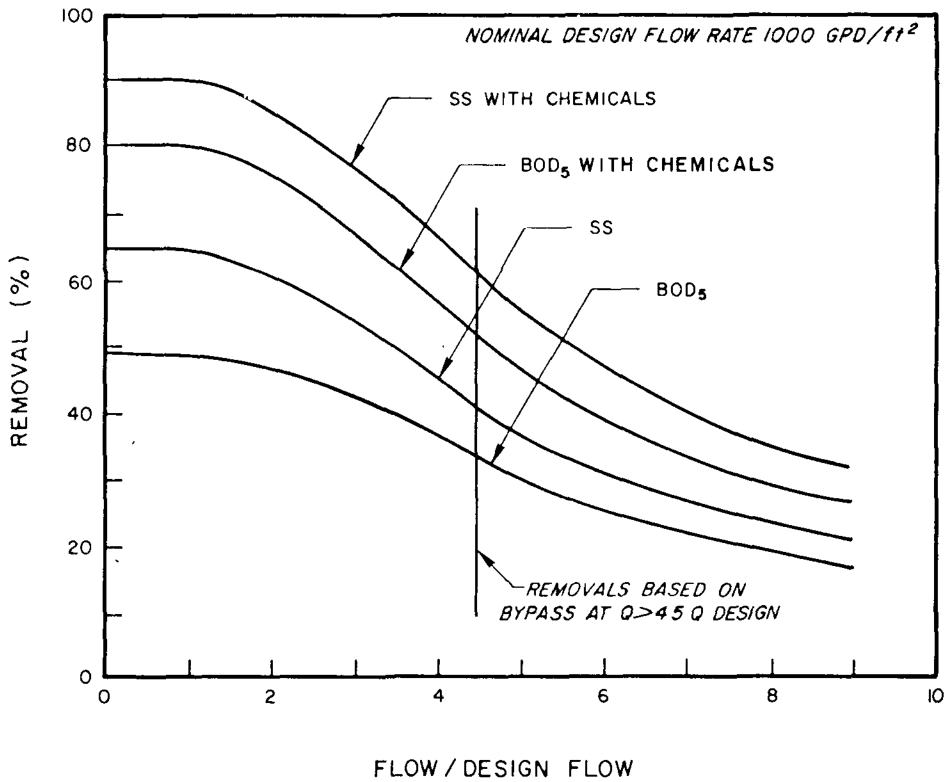


FIGURE 3-20

DISSOLVED AIR FLOTATION PERFORMANCE RELATED TO DESIGN RATE

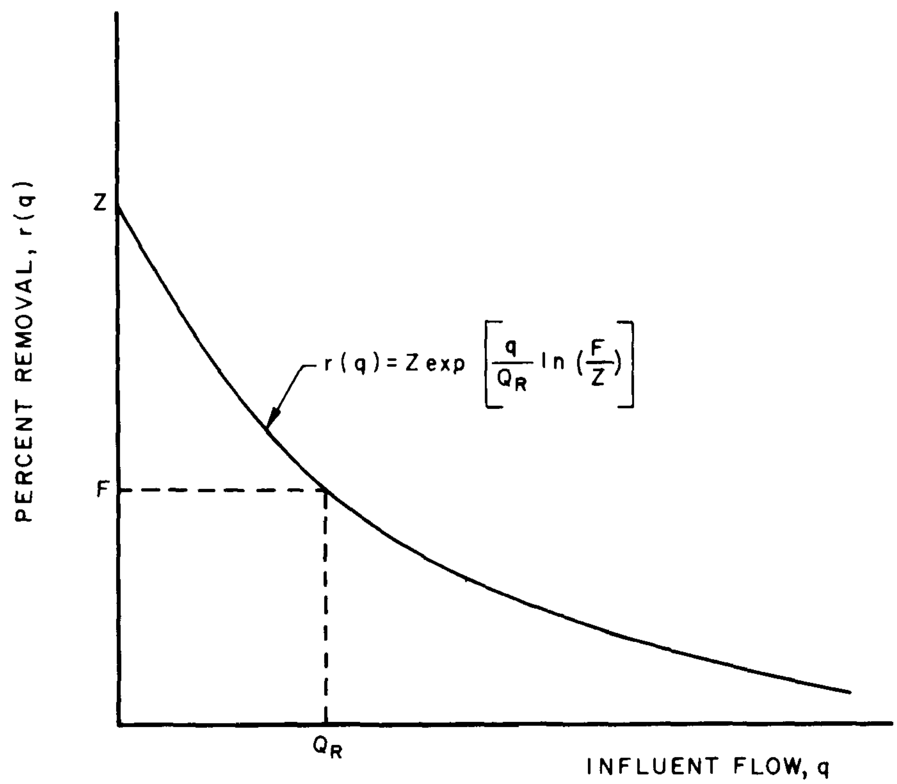


FIGURE 3-21

IDEALIZED REMOVAL EFFICIENCY CURVE
FOR A FLOW SENSITIVE TREATMENT DEVICE

where storm flows are assumed to be gamma distributed and independent of c and d, and r(q) is expressed as a fraction. The result is:

$$P_F = Z \left[\frac{\kappa}{\kappa - \ln(F/Z)}\right]^{\kappa+1} \qquad (3\text{-}40)$$

where: $\kappa = 1/\nu_q^{\,2}$

Equation 3-40 is plotted in Figure 3-22 which demonstrates the long term performance which can be achieved by in-line treatment devices. Note that the greater the variation in the runoff flow (ν_q), the more poorly the device performs on average.

To use Equation 3-40 or Figure 3-22, an efficiency curve for the particular device is required. These are presented for many devices in Figures 5-43 - 5-54 in Section 5.5.2 of the Numerical Estimates Chapter. Also included in Section 5.5 is a discussion of the amount of field and experimental data used to develop the curves, their applicability to separate versus combined runoff, and the extrapolation of information on suspended solids removal to predictions for the removal of other pollutants. If other information is available to develop an efficiency curve for a particular treatment device, this curve may be used in the same manner. Once the percent removal versus flow curve is chosen, the following procedure is employed to determine the average long term performance:

Step 1. From the efficiency curve, determine F, the percent removal of the given size device at the mean runoff flow (Q_R).

Step 2. From the efficiency curve, determine Z, the largest percent removal of the given size device at very low flows. The idealized removal curve (from Equation 3-38) should be drawn over the actual curve to insure a reasonable representation.

Step 3. Given the variation of the runoff flow, ν_q, use Equation 3-40 or Figure 3-22 to determine P_F.

3.6.1.4.1 Example: Analysis of In-Line Treatment Device

To demonstrate the evaluation of the long term performance of a treatment device, assume a drainage area served by an in-line dissolved air flotation system with chemicals (see Figure 3-20 for efficiency curve) has the following characteristics:

Mean runoff flow = Q_R = 4 MGD = 4,000,000 GPD (6.2 cfs)

Variation of runoff flow = ν_q = 1.15

Mean runoff load = M_R = 2,000 lb BOD_5

Surface area of air flotation device = 1,000 sq. ft.

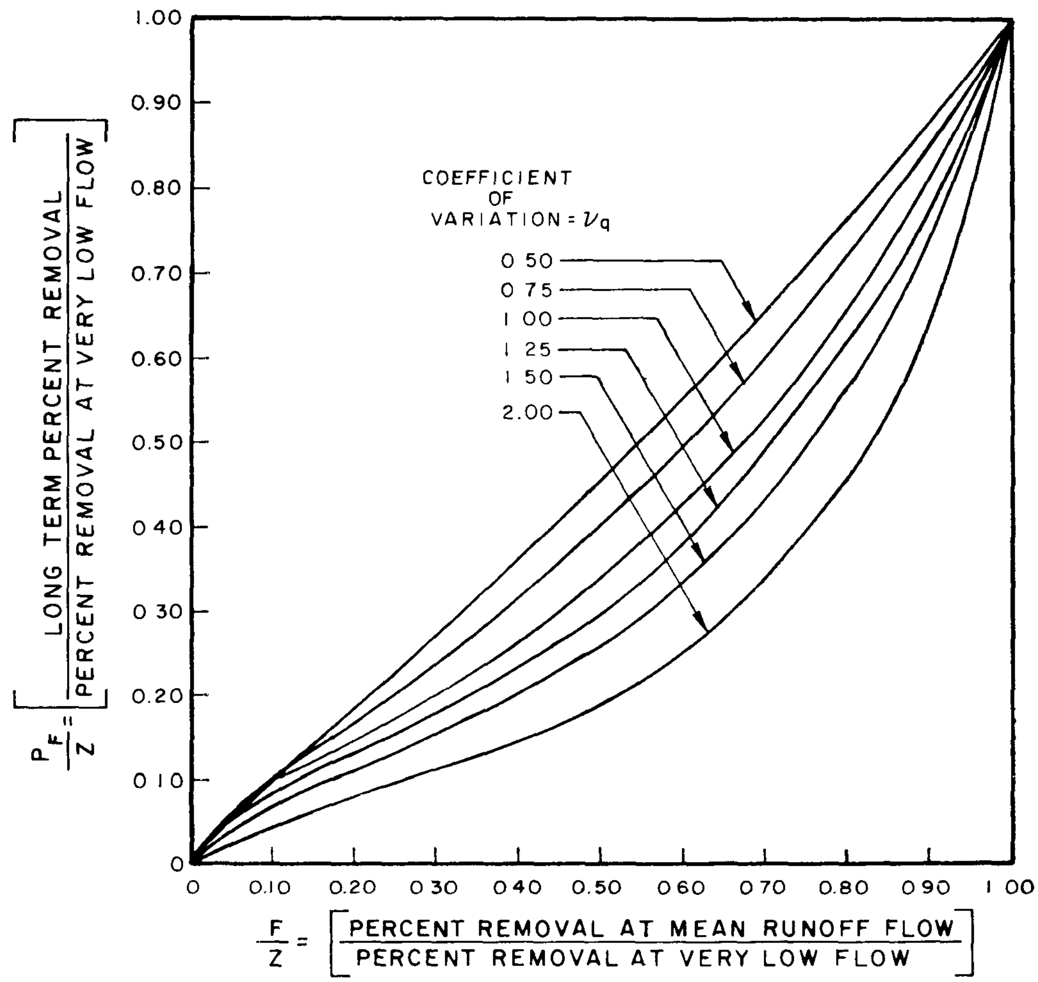


FIGURE 3-22

LONG TERM PERFORMANCE OF
A FLOW SENSITIVE TREATMENT DEVICE

The surface loading rate at the mean runoff flow is (4,000,000 GPD)/(1,000 ft^2) = 4,000 GPD/ft^2, which is 4 times the nominal design flow. From Figure 3-20 (BOD_5 with chemicals), the values of F and Z are selected. The first inclination is to select F = 57 and Z = 80. Because of the flat shape of the actual performance curve at low flows, however, the idealized exponential curve with F = 57 and Z = 80 does not provide a good match to the actual performance. Selecting F = 55 and Z = 100 provides a much better overall match. The idealized curves are compared to the actual performance curve in Figure 3-23. For these values the long term average reduction in the runoff load is calculated from Equation 3-40.

$$\kappa = 1/\nu_q^{\,2} = 1/1.15^2 = 0.76$$

$$P_F = Z\left[\frac{\kappa}{\kappa - \ln(F/Z)}\right]^{\kappa+1} = 100\left[\frac{0.76}{0.76 - \ln(55/100)}\right]^{0.76+1}$$

= 36 percent reduction of long term BOD_5 load.

Using Figure 3-22, F/Z = 55/100 = 0.55. Given that ν_q = 1.15, P_F/Z = 0.36, and P_F is determined as P_F = 100(0.36) = 36 percent. Note that either the numerical or the graphical method may be used, yielding the same result. The modified runoff load, M_R^*, is then:

$$M_R^* = (1 - 36/100)\ M_R = 0.64\ (2{,}000) = 1{,}280\ \text{lb}\ BOD_5$$

As with the interceptor, the long term performance of the flow sensitive treatment device is adversely affected by large within storm flow variations and positive flow-concentration correlations. Again, these effects are usually not large enough to be significant over the long term (see Section 3.6.1.7.4).

3.6.1.4.2 Concentration Sensitive Treatment Devices

The percent pollutant removal obtained by a number of treatment devices, including screens and sedimentation tanks, increases with higher influent concentrations. The relationship between the influent concentration and the percent removal is assumed to be increasing to a limit. An analysis similar to the one for the flow sensitive treatment device has been performed to determine the long term load reduction as a function of the percent removal at the mean runoff concentration, the best percent removal obtainable at very high concentrations, and the variability of the runoff concentration (32). The long term performance curves are similar in form to those of Figure 3-22, except the performance improves with a higher coefficient of variation of runoff concentration. Similarly, high within storm concentration variations and positive flow concentration correlations improve the long term treatment.

Most treatment devices which are sensitive to influent concentration are also sensitive to influent flow. These dually sensitive devices have also been analyzed (32), and an intermediate long term performance is obtained, depending upon the degree of sensitivity to either flow or concentration. The improved treatment at higher concentrations generally

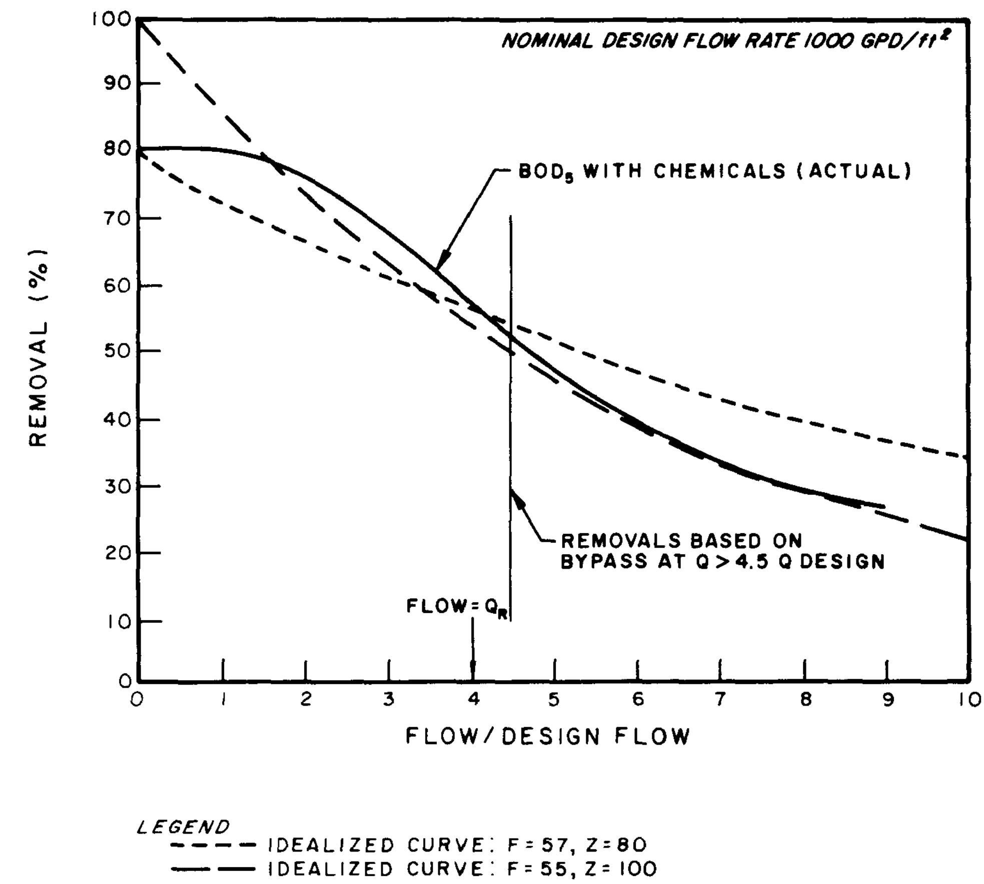


FIGURE 3-23

COMPARISON OF IDEALIZED
AND ACTUAL DISSOLVED AIR FLOTATION PERFORMANCE

to an improvement in the long term performance of the device subjected to varying influent concentrations. This improvement may be ignored, however, for an initial, conservative assessment, and the methodology developed for the flow sensitive treatment device may be used, with the average influent pollutant concentration determining the appropriate curve selected in Section 5.5.2 to define the treatment device performance at the mean runoff flow and low flows.

3.6.1.4.3 Disinfection

Disinfection of pathogenic organisms in stormwater overflows is often necessary to protect public health, protect water supplies, bathing beaches, and other water uses. Conventional disinfection of wastewater generally uses chlorine and chlorine compounds, and most of the investigations on the disinfection of combined sewer overflows have been conducted with chlorine disinfectants. These and other disinfection systems are discussed in more detail in Section 5.5.2.7 of the Numerical Estimates Chapter.

The disinfection process is quite complicated and the approach outlined in Figure 3-22 is not readily applicable to the analysis of disinfection. A unique methodology is required and the basic assumptions and mathematical development of the analysis are presented in Section 5.5.2.7.

The purpose of the analysis is to provide a method of transforming information about the bacterial removal of disinfection systems under controlled conditions to estimates of the long term performance of systems subjected to varying runoff or overflow rates. The results are displayed in Figure 3-24. The abscissa is the product of the effective kill rate (k), the mean contact time ($\bar{t}$), each determined with the influent flow equal to the mean runoff flow, (Q_R). The long term bacteria reduction is shown for a device treating a constant influent flow (equal to Q_R) and devices subjected to varying influent flow ($\nu_q = 1$); one with a constant disinfectant feed rate and one with a feed rate directly proportional to the influent flow.

To demonstrate how Figure 3-24 is used, assume a proposed disinfection system is designed to achieve nearly plug flow and gives a 99.99 percent bacteria removal (10^{-4} remaining) at the mean runoff flow. This is equivalent to $(k\ \bar{t})_R = 20$. The long term average reduction for the proportional feed device is 99.85 percent (1.5×10^{-3} remaining) and only 97 percent (3×10^{-2} remaining) for the constant feed device. Changes in the size of the device may be estimated by changing the contact time ($\bar{t}$) while changes in the amount of disinfectant used may be estimated by changing the kill rate (k). Improvements in long term performance may be compared to the cost of flow metering and proportional feed equipment, larger disinfection systems, and the use of more disinfectant.

The analysis of disinfection systems, as well as other types of stormwater treatment, has been directed towards the determination of the long term pollutant load reduction. While this provides much insight as to the effectiveness of the treatment alternative, and may be adequate for many planning purposes, it may provide an incomplete picture. This is particularly true for disinfection controls, where the fact that most storms are treated

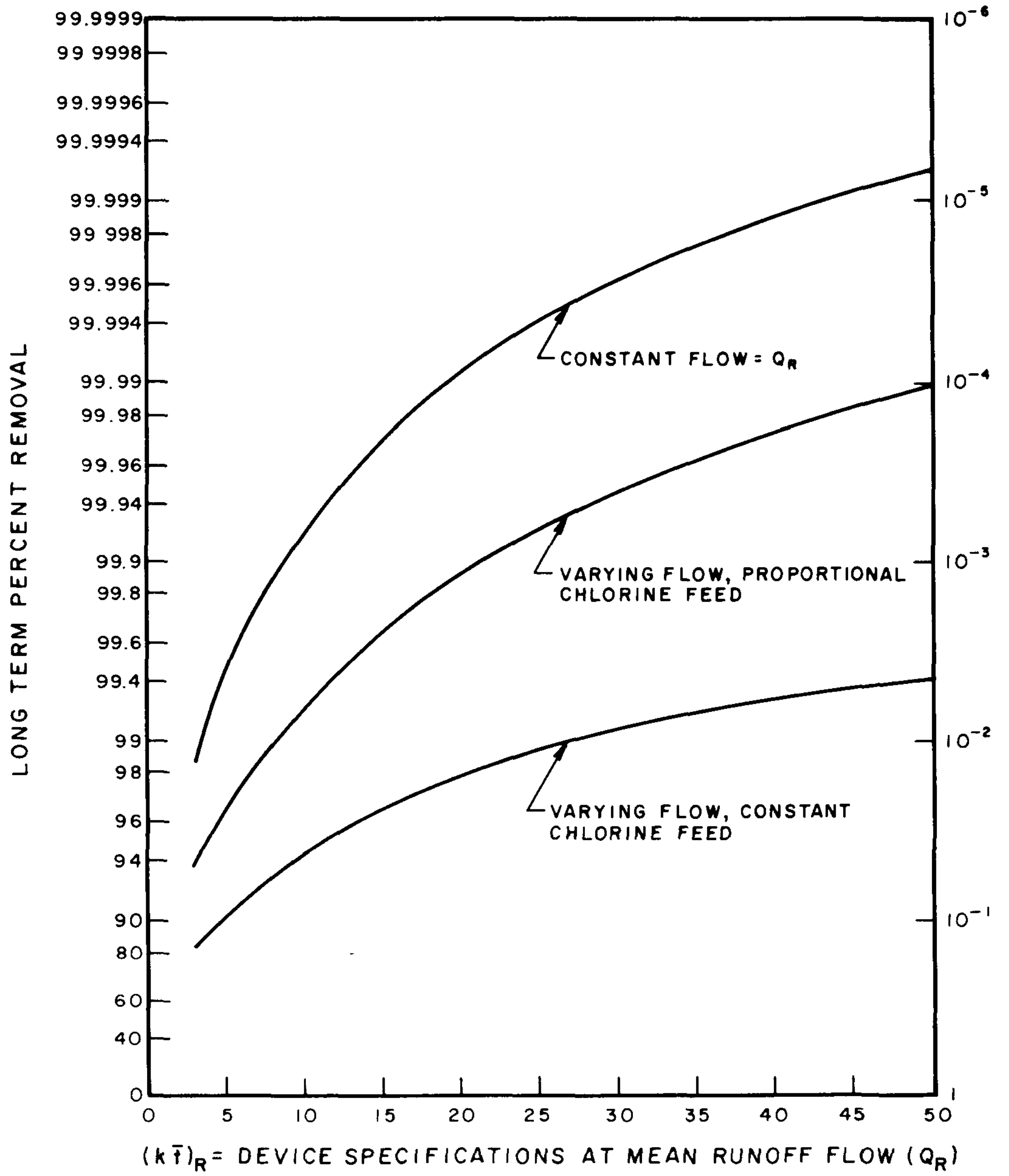


FIGURE 3-24

EFFECT OF STORMFLOW VARIATION
ON PERFORMANCE OF EMPIRICAL DISINFECTION DEVICE

adequately is more important than the fact that a few large storms are overflowing with little treatment, thereby having an adverse effect on the long term average reduction. The important consideration is the frequency of storm load occurrences after treatment, rather than the long term average. This is discussed in more detail for disinfection and other stormwater control alternatives in Section 3.6.1.6, The Effect of Stormwater Control Devices on the Frequency of Loadings.

3.6.1.5 Combined Treatment Systems Which Capture, Store, and Treat Runoff

To estimate the long term performance of a combined treatment system which captures, stores, and provides in-line treatment for stormwater overflows, the approximation developed for a combination of interceptors and storage devices may be employed: the fraction of the load remaining for the combined system equals the product of the fractions remaining from each of the individual components.

To illustrate the evaluation of combinations of control strategies, assume the storage and the in-line facility previously presented are both used. The storage device captures runoff until it is full, when the bypassed flows are treated by the in-line dissolved air flotation system. The runoff captured by the storage device is then treated at a controlled rate such that there is 50 percent treatment (r = 50%). For these examples, the following loads were determined:

M_R = Mean runoff load = 2,000 lb BOD_5

M_R^* predicted after storage and treatment of captured runoff alone = 1,350 lb BOD_5

M_R^* predicted after in-line treatment alone = 1,280 lb BOD_5

The resulting runoff load with both the storage and in-line treatment is then estimated as:

$$M_R^* = \left(\frac{1,350}{2,000}\right) \times \left(\frac{1,280}{2,000}\right) \times 2,000 = 860 \text{ lb } BOD_5$$

3.6.1.6 The Effects of Stormwater Control Devices on the Frequency of Loadings

The techniques, curves, and equations presented in the previous sections for estimating the reduction in the long term average stormwater load due to various control devices provide useful information to the planner; and for stormwater impacts which are long term in nature, such as sediment desposition and eutrophication due to nutrient runoff into an impoundment, the knowledge of the long term removal is completely adequate. For transient stormwater impacts which occur primarily during, or immediately following storm events, however, planning decisions may require information on how treatment devices modify the frequency of stormwater loadings.

Research has been conducted to determine the effect of treatment on the variance of stormwater loads (33). A general observation is that control devices reduce the variance of the runoff load less than they reduce the mean. The majority of the smaller storms are treated very well, and in the case of storage or interception, they may be completely captured. The larger storm loads, however, may only be marginally reduced by the treatment system. The long term output from the treatment system thus has more variation around the mean than does the input.

Assuming one can estimate the effect of treatment on the variance of the runoff load, problems still arise because the modified storm loads may no longer be gamma distributed. Different control devices affect the frequency distribution of storm loads in different ways. A few simple examples of this are presented for different control devices. When a single, simple device is used some estimates may be made of the resulting frequency of modified storm loads. More complex cases involving combinations of control devices require simulation for reliable estimation.

3.6.1.6.1 Interception

Interceptors capture a portion of the flow from a storm runoff event and their effect on the frequency distribution of storm loading rates (i.e., pounds per hour during the storm) may therefore be approximated. All storms with flows less than or equal to the available interceptor capacity (Q_I) are completely captured while larger storms (higher average flows) have their flow rates reduced by Q_I. Assuming a constant runoff concentration, this effect on the flow rate may be transformed to an equivalent effect on the loading rate.

The effect of an interceptor on the frequency distribution of storm loading rates is demonstrated in Figure 3-25. The original runoff loading rates are assumed to be gamma distributed, and Figure 3-2 is used to draw the frequency distribution given the coefficient of variation of the runoff flows, $\nu_q = 1.15$. An interceptor with $Q_I/Q_R = 1.25$ completely captures 71 percent of the storms. The overflow rates from the remaining 29 percent of the storms are reduced as indicated. For example the loading rate during the 90th percentile storm (with a loading rate exceeded by only 10 percent of the storms) is reduced from about 2.4 W_R to 1.15 W_R. Knowing the average number of storms per year or season, this result may be transferred to the expected number of occurrences during the year or during the particular season.

3.6.1.6.2 Storage

Storage devices capture a portion of the volume from a storm runoff event, and their effect on the frequency distribution of storm loads (i.e., total pounds per storm) may therefore be approximated. All storms with volumes less than or equal to the effective storage capacity (V_E) are completely captured while larger storms (higher total runoff volumes) have their runoff volumes reduced by V_E. Assuming a constant runoff concentration, this effect on the runoff volume may be transferred to an equivalent effect on the runoff load.

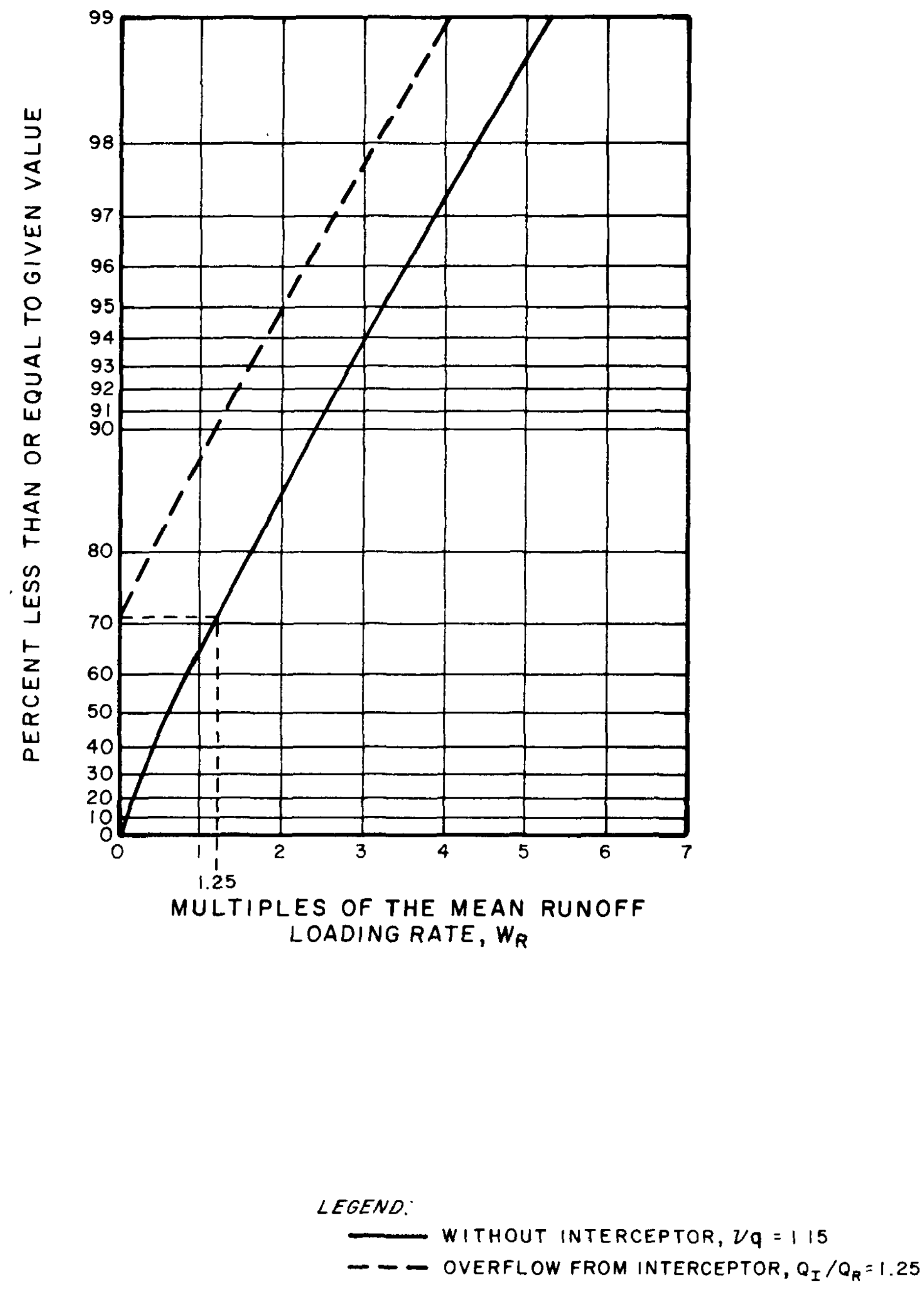


FIGURE 3-25

FREQUENCY DISTRIBUTION OF STORM OVERFLOW LOADING RATES BEFORE AND AFTER INTERCEPTION

The effect of a storage device on the frequency distribution of storm loads is demonstrated in Figure 3-26. The original runoff loads are assumed to be gamma distributed and Figure 3-2 is used to draw the frequency distribution given the coefficient of variation of the runoff volumes, $\nu_{vR} = 1.75$. A storage device with $V_E/V_R = 1.35$ completely captures 77 percent of the storms. The by-pass loads from the remaining 23 percent of the storms are reduced as indicated. For example, the total runoff load from the 90th percentile storm (with a total load exceeded by only 10 percent of the storms) is reduced from about 2.9 M_R to 1.55 M_R.

3.6.1.6.3 In-Line Treatment Devices

In-line treatment devices reduce the concentration of stormwater runoff and overflows. Assuming the removal efficiency is sensitive to influent flow, as described in Equation 3-38, and assuming that the influent concentration is constant and equal for all storms, the effluent loading rate during a particular storm (i.e., pounds per hour) is a function only of the flow rate. Therefore, the effect of in-line treatment on the frequency distribution of storm loading rates may be approximated.

Larger storms (higher average flows) have lower percent removals, as shown in Figure 3-22 and Equation 3-38. Therefore, if a particular storm has a higher loading rate than another storm before treatment, it will also have a higher loading rate after treatment (though both are reduced). The *n*th percentile storm before treatment is thus the *n*th percentile storm after treatment, allowing a simple transformation from the untreated frequency distribution to the treated frequency distribution with Equation 3-38. This is demonstrated in Figure 3-27 for a device which gives 60 percent removal at the mean runoff flow (F = 60) and 80 percent removal at low flows (Z = 80). The untreated frequency distribution is drawn for $\nu_q = 1.15$, as in Figure 3-25. For example the 66th percentile (mean) storm which has a loading rate of W_R before treatment, has a loading rate of 0.4 W_R after treatment, corresponding to a 60 percent reduction. Note that for larger storms the untreated and treated curves become approximately parallel. This indicates that none of the additional runoff is receiving treatment, as is the case when the hydraulic capacity of the device is reached and the additional flows are merely by-passed.

3.6.1.6.4 Disinfection

Disinfection systems are assumed to operate in a similar fashion to in-line treatment devices. Given the basic assumptions presented in Section 5.5.2.7, the effluent coliform bacteria loading rate is a function of only the runoff flow. Larger storms (higher average flows) have lower percent removals due to the decreased contact time in the device and, in the case of a constant chlorine feed system, lower disinfectant concentrations.

Assuming the empirical disinfection device designed to achieve nearly plug flow (described in Section 5.5.2.7) is used, Figure 3-28 shows the frequency distribution of storm loading rates before and after disinfection. The untreated loading rate distribution is estimated by assuming that ν_q = 1.00. The treated loading rate distribution is calculated from Equations

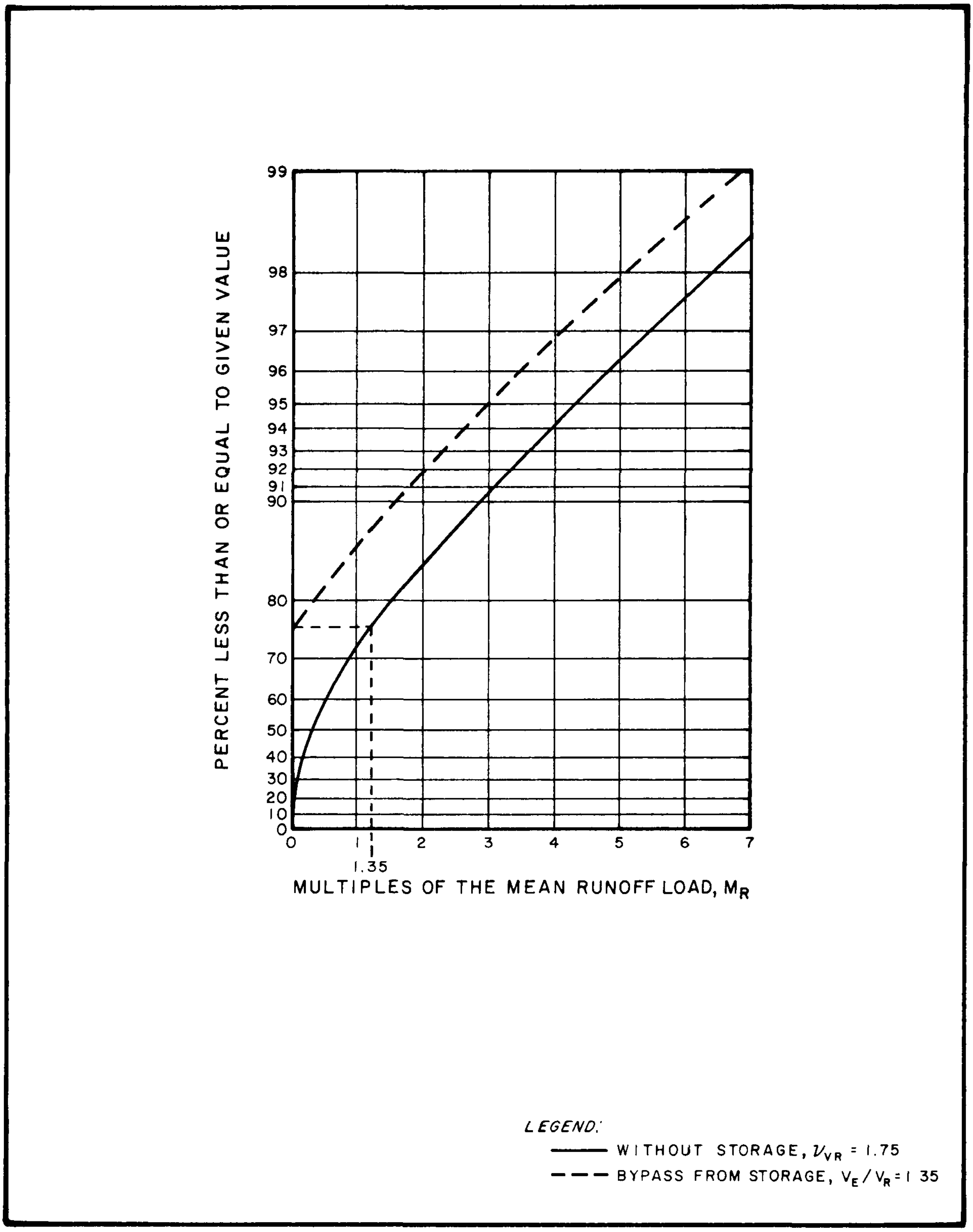


FIGURE 3-26

FREQUENCY DISTRIBUTION OF STORM LOADS
BEFORE AND AFTER STORAGE

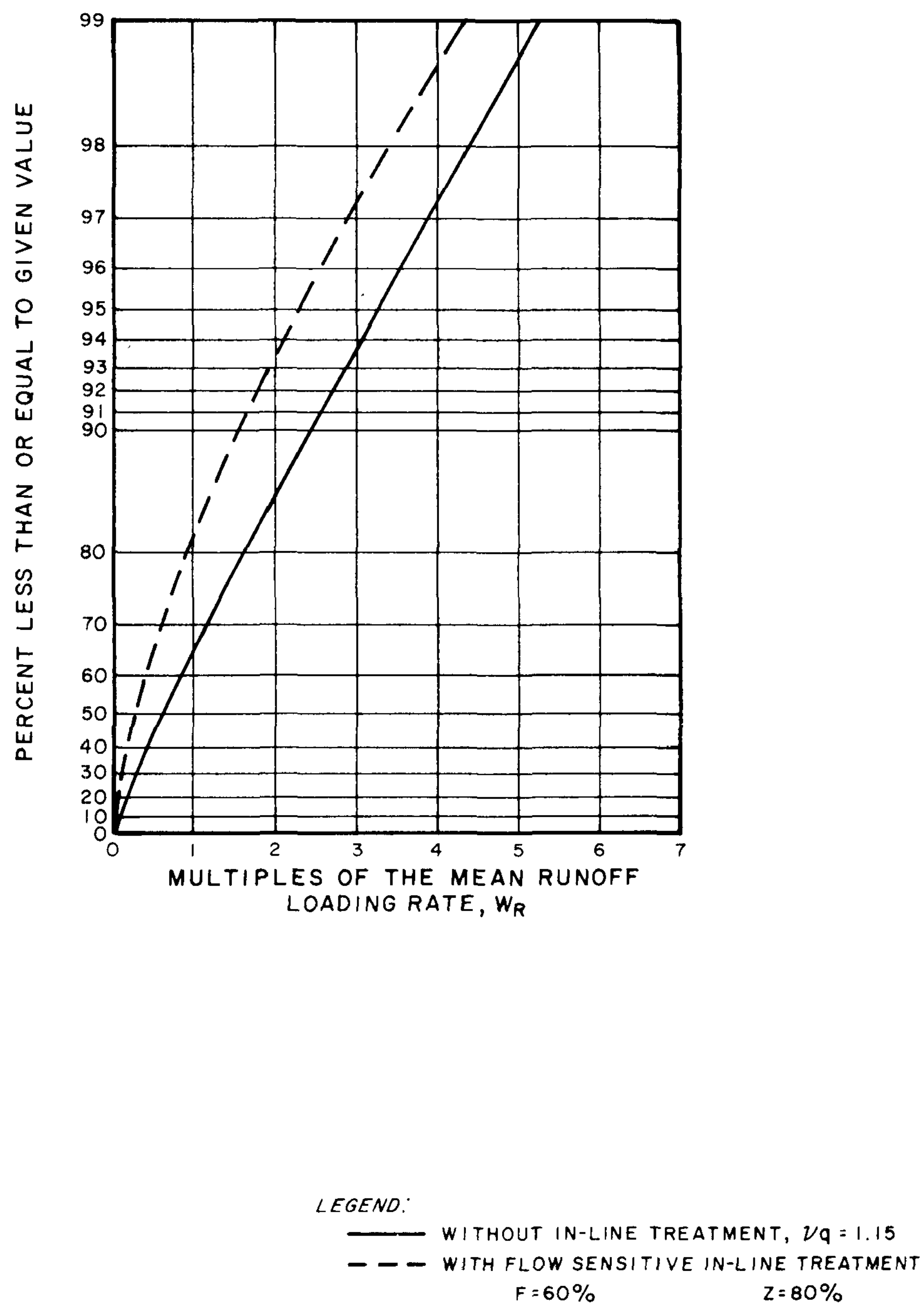


FIGURE 3-27

FREQUENCY DISTRIBUTION OF STORM LOADING RATES BEFORE AND AFTER IN-LINE TREATMENT

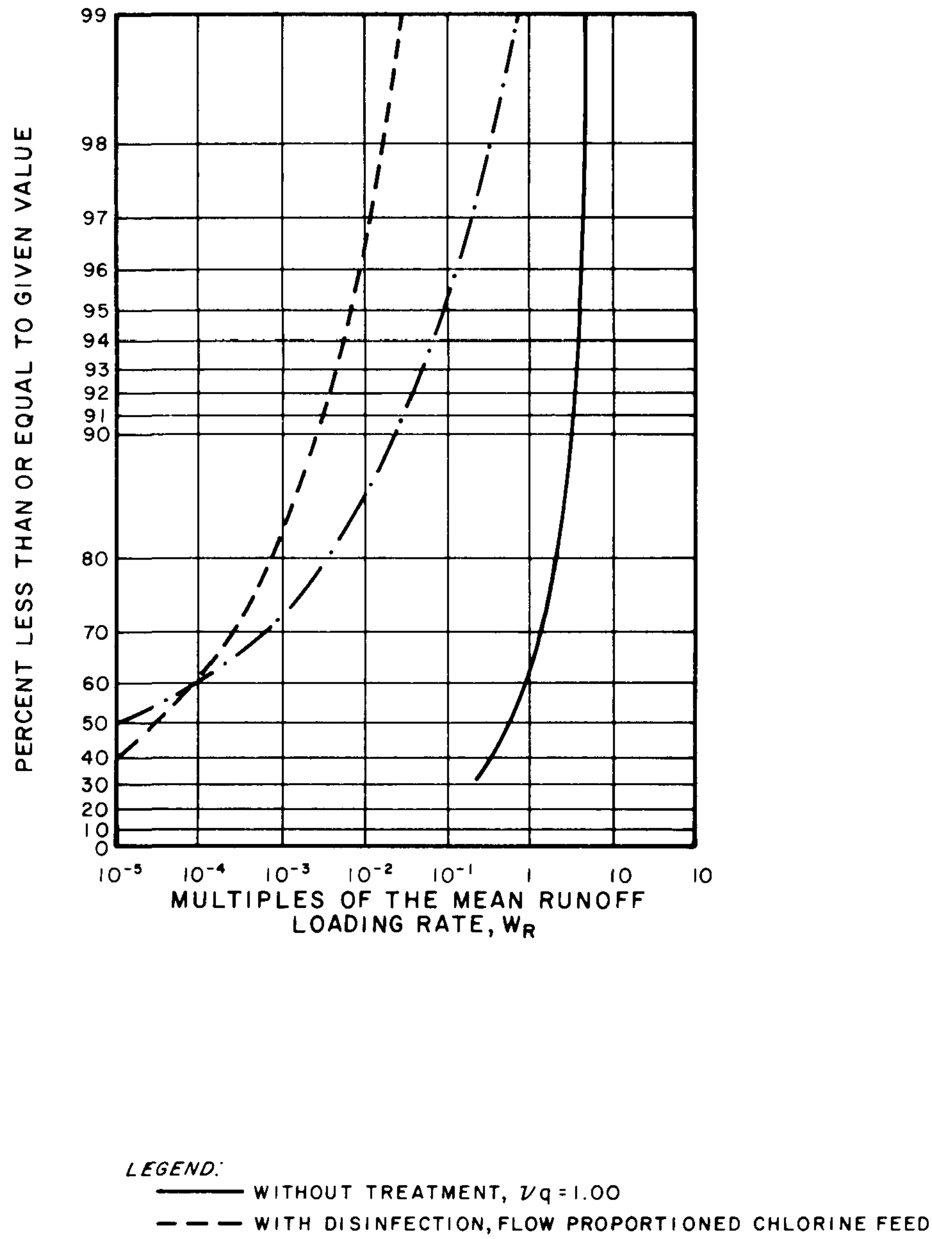


FIGURE 3-28

FREQUENCY DISTRIBUTION OF STORM LOADING RATES BEFORE AFTER DISINFECTION

5-25 and 5-26 in Numerical Estimates and shown for systems with a flow proportioned and a constant chlorine feed rate. Both systems are designed to give 99.99 percent removal (10^{-4} remaining) at the mean runoff flow. If it has been determined, for example, that coliform loading rates greater than 0.01 W_R result in unacceptable receiving water impacts, the frequency of occurrence of these impacts may now be estimated for each of the disinfection alternatives. The constant chlorine feed system results in about 13 percent of the storms having loading rates greater than 0.01 W_R while the flow proportioned feed system results in about 3.5 percent of the storms having loading rates greater than 0.01 W_R.

A few simple examples of frequency estimates have been presented. However, estimates of the frequency distribution of stormwater loads resulting from more complex combinations of treatment systems may require a more sophisticated analysis with simulation.

3.6.1.7 Comparison of Statistical Method to Simulations of Treatment

This section presents comparisons between the treatment performances predicted by the statistical method and results obtained with simulation studies. The comparisons support the validity of the theoretical curves and indicate that estimates based on these curves are likely to be similar to those obtained with simulation modeling techniques.

3.6.1.7.1 Effect of Previous Storms

To check the approximation for the effect of previous storms on the long term effective storage capacity given in Figure 3-17, the results of a simulation of an 8 square mile drainage basin in Dallas, Texas, are used. A simple rainfall/runoff ratio is used to convert the 1968 hourly rainfall record to runoff flows. A storage device is simulated and the amount of storage available at the beginning of each storm is recorded. Nine simulations are made with different size basins and different emptying rates. The results of these simulations are summarized in Figure 3-29. The curves for $\Delta\Omega/V_R$ = 0.5, 1.0, and 2.0 reasonably approximate the simulation results. However these curves are intended only for an initial assessment of storage device operation and not for actual design since they depend on an idealized representation of the basin operation.

3.6.1.7.2 Storage Device Performance

In a report on combined sewer overflow problems in the City of Trenton, New Jersey, Kaufman analyzes the potential impact of a detention basin at the storm by-pass of the City's sewage treatment plant (34, 35). The Trenton rainfall is characterized by ten years of United States Weather Bureau Records (1963-1973). Assumed relationships between the intensity of rainfall and the amount of runoff lost to infiltration and overland flow (not entering the combined sewer) are used to determine the volume of combined sewage overflowed during each storm at the treatment plant by-pass. These volumes are used to calculate the percent of the total overflow which would be captured by a detention basin of various sizes (35, Plate 14).

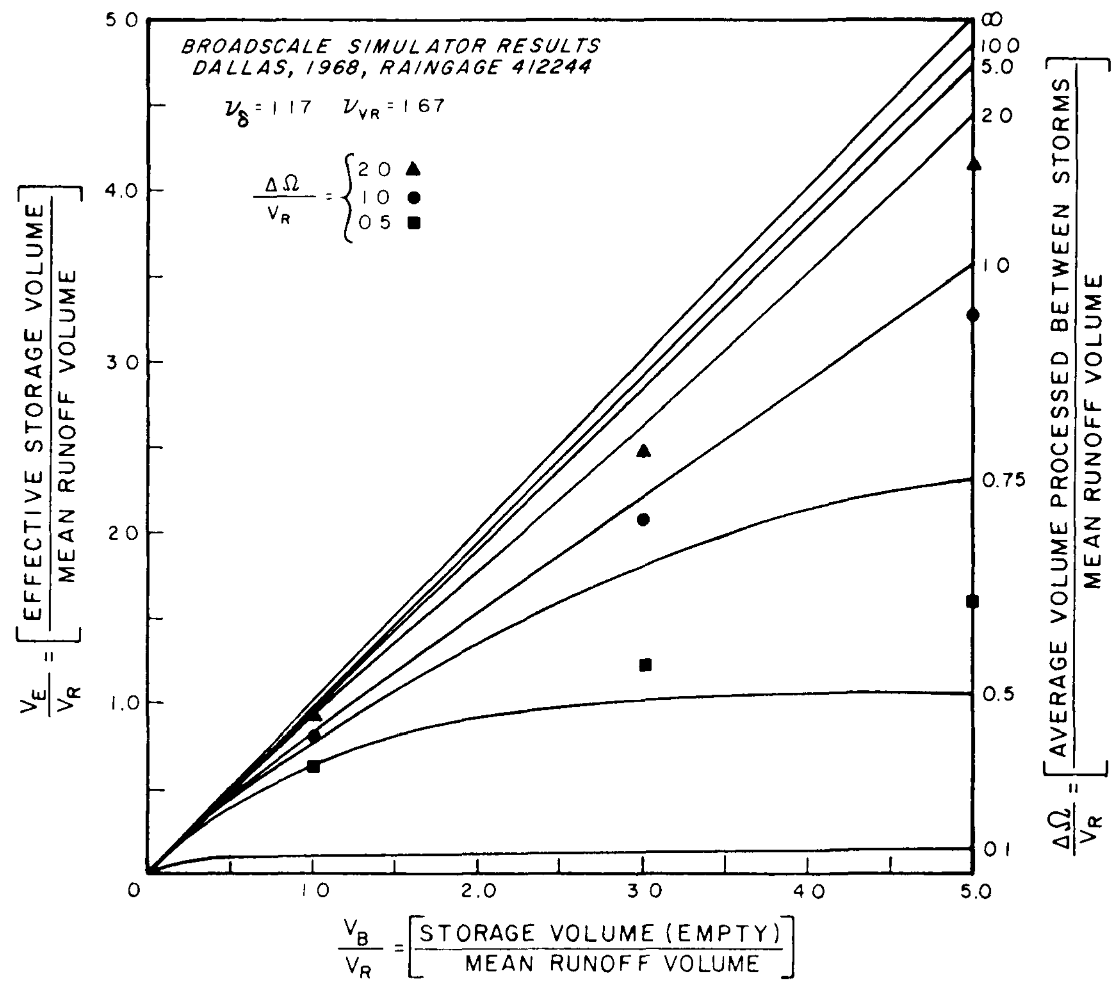


FIGURE 3-29
EFFECT OF PREVIOUS STORMS
ON LONG TERM EFFECTIVE STORAGE CAPACITY

To compare the Trenton results with the statistical method, the mean and the coefficient of variation of the overflow volumes were determined (V_R = 3.48 MG, ν_{VR} = 1.49). The results plotted in Figure 3-30 show a very close agreement between the theoretical and calculated long term performance. Note that storm overflow volumes are calculated in a more sophisticated manner than with a simple rainfall to runoff ratio. This demonstrates the flexibility of the general methodology, where more refined estimates may be used to determine the runoff statistics, depending upon the specific problem setting and the data available.

3.6.1.7.3 Interception and Storage

As part of a nationwide evaluation of combined sewer overflows and urban stormwater discharges, Heaney and Huber et al. use the STORM simulation model to develop storage-treatment isoquants for five cities in the United States (3, 36). A relatively simple transformation from rainfall to runoff is used to generate one year of hourly runoff flows and loads, and a storage-treatment system is employed to simulate the capture of these flows and loads. A number of these STORM simulations are executed with different combinations of storage and interception, and the percent of the yearly runoff load captured is noted for each simulation. Curves are fitted to storage-interception combinations with equal percentages of the runoff load captured to form the isoquants (3, Figures 12-16).

The curves and relationships of the statistical method presented in this Chapter can be used to generate similar isoquants for one of the cities, Denver, Colorado; and the results are compared to those generated by the STORM simulation. The STORM simulations were made with Raingage 052220 in Denver for the year 1960, and this record is therefore used to generate the appropriate runoff characteristics, summarized in Table 3-8, for use in the comparison. The mean runoff volume and flow are given per unit area, in inches per storm and inches per hour respectively. The effects of depression storage and evaporation are considered negligible and not included. Accumulation rates for the BOD and the effects of street sweeping which are used in the STORM simulations are also not incorporated. The storage-interception isoquants are generated using the statistics in Table 3-8, Figures 3-15, 3-17, 3-18, and 3-19, and Equation 3-36. The results are compared to the STORM simulations in Figure 3-31. The amounts of BOD captured as predicted using the curves and relationships of the statistical method are very similar to those simulated with the STORM model.

3.6.1.7.4 In-Line Treatment Device

The performance of an idealized flow sensitive treatment device is simulated using actual runoff quality (chemical oxygen demand, COD, and suspended solids, (SS) and flow data from Durham, North Carolina (37), Milwaukee, Wisconsin (38), Washington, D. C. (39), and Lubbock, Texas (40). A summary of the data is presented in Table 3-9. For each data set, the performance of a flow sensitive treatment device is simulated by assuming that Equation 3-38 applies exactly. The actual observed concentrations and flows are then subjected to a removal consistent with Equation 3-38. The within storm observations are processed sequentially for all storms.

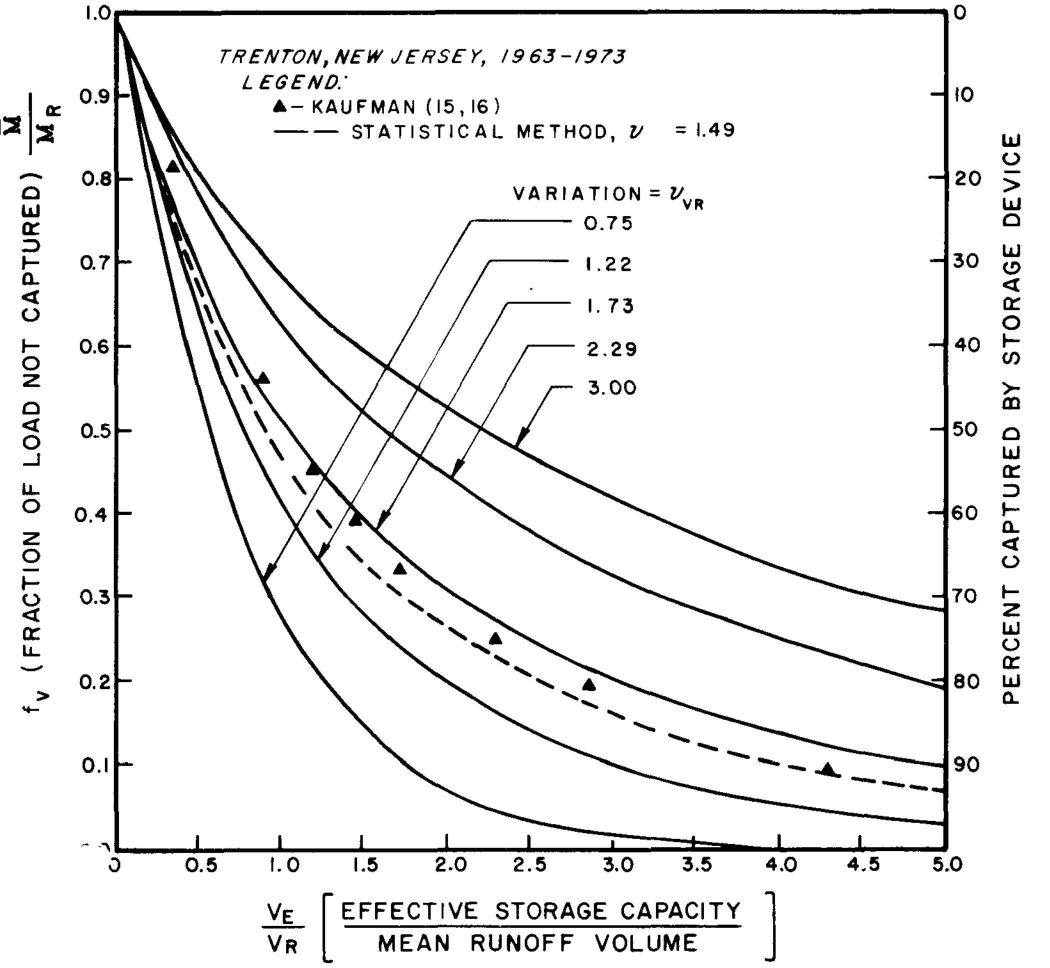


FIGURE 3-30
DETERMINATION OF LONG TERM STORAGE DEVICE PERFORMANCE

TABLE 3-8

SUMMARY OF RUNOFF STATISTICS
DENVER, COLORADO, RAINGAGE 052220, 1960

Runoff to Rainfall Ratio		=	0.39
Q_R = 0.0131 in/hr	ν_q	=	1.38
V_R = 0.078 in/storm	ν_{vR}	=	1.49
Δ = 119 hrs			

Assume Moderate First Flush

TABLE 3-9

SUMMARY OF RUNOFF DATA ANALYZED

City	Coefficient of Variation of Flow = ν_q (Between Storms)	Constituent Pollutants: COD No. of Storms	COD Total No. Samples	SS No. of Storms	SS Total No. Samples
Durham (37)	1.59	26	398	26	354
Milwaukee (38)	1.05	13	144	12	130
Washington, D.C. (39)	0.70	6	45	*	*
Lubbock (40)	0.68	11	93	11	93

*Insufficient data.

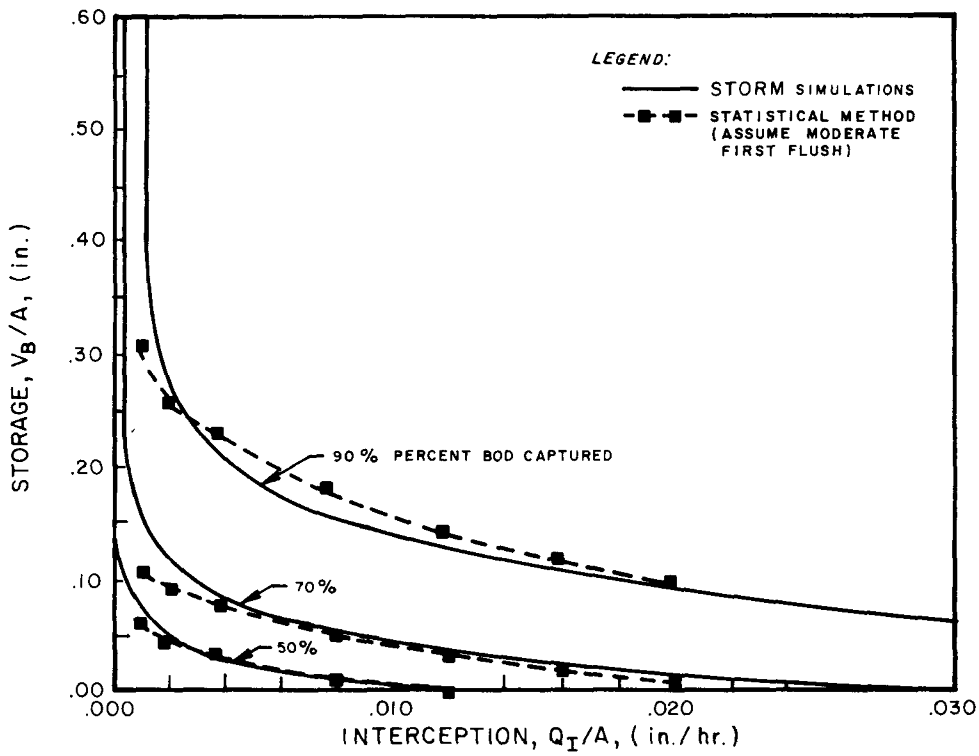


FIGURE 3-31
STORAGE / INTERCEPTION ISOQUANTS
PERCENT BOD CAPTURED WITH FIRST FLUSH
DENVER, 1960 RAINGAGE 052220

The results are analyzed for the average percent removal. The calculation is made with F/Z equal to 0.2, 0.4, 0.6, 0.8, and 0.9. The resulting removal over all the storms is compared to the theoretical reduction in Figure 3-32.

In general, the comparison between the theoretical and simulated reduction is quite good. Stipulation of the relative removal at the mean runoff flow (F/Z), and a knowledge of the coefficient of variation of the runoff flow (ν_q), allows a reasonable prediction of the long term performance. Furthermore, the differences between the theoretical and the simulated values are explainable in terms of two mechanisms previously mentioned: high within storm flow variation and flow-concentration correlation.

Lubbock, which has very high flow variability within storms, has simulated overall reductions smaller than those predicted by Equation 3-40. The relative magnitude of within storm flow variation is depicted in Table 3-10 as the average standard deviation of flow within storms ($\bar{\sigma}_{qw}$) divided by the standard deviation of flow between storms (σ_q). For all the cities except Lubbock, the within storm flow variation is small compared to the between storm variation.

The other effect, flow-concentration correlation, is depicted in Table 3-11 as the ratio of the flow-weighted average concentration ($\bar{c}_f$) to the standard, time average concentration ($\bar{c}$). If flow and concentration are independent, $\bar{c}_f/\bar{c}$ is nearly equal to one. If flow and concentration are positively correlated, $\bar{c}_f/\bar{c}$ is greater than one; and if the correlation is negative, $\bar{c}_f/\bar{c}$ is less than one. Note that the SS data tend to show a positive correlation (scour effect), particularly in Durham; while the COD data tend to show a negative correlation (dilution effect), with the exception of Durham. The ratios shown in Table 3-11 are consistent with the observation that:

1. The simulated COD reductions for Durham are very nearly equal to the theoretical reductions, while the SS reductions are less than predicted.

2. The simulated SS reductions for both Milwaukee and Lubbock are less than their respective COD reductions.

3. The simulated COD reductions for Washington, D. C., are greater than the theoretical reductions.

Despite the influence of within storm flow variation and flow-concentration correlation, Equation 3-40 provides a reasonable first estimate of the long term performance of flow sensitive stormwater treatment devices.

A number of other checks of the statistical method have been made using results from Atlanta, Georgia, Chippewa Falls, Wisconsin, and the four cities used in the comparison of in-line treatment devices. These comparisons together with the results presented in this section, demonstrate the utility of the statistical method for initial assessments of stormwater control systems.

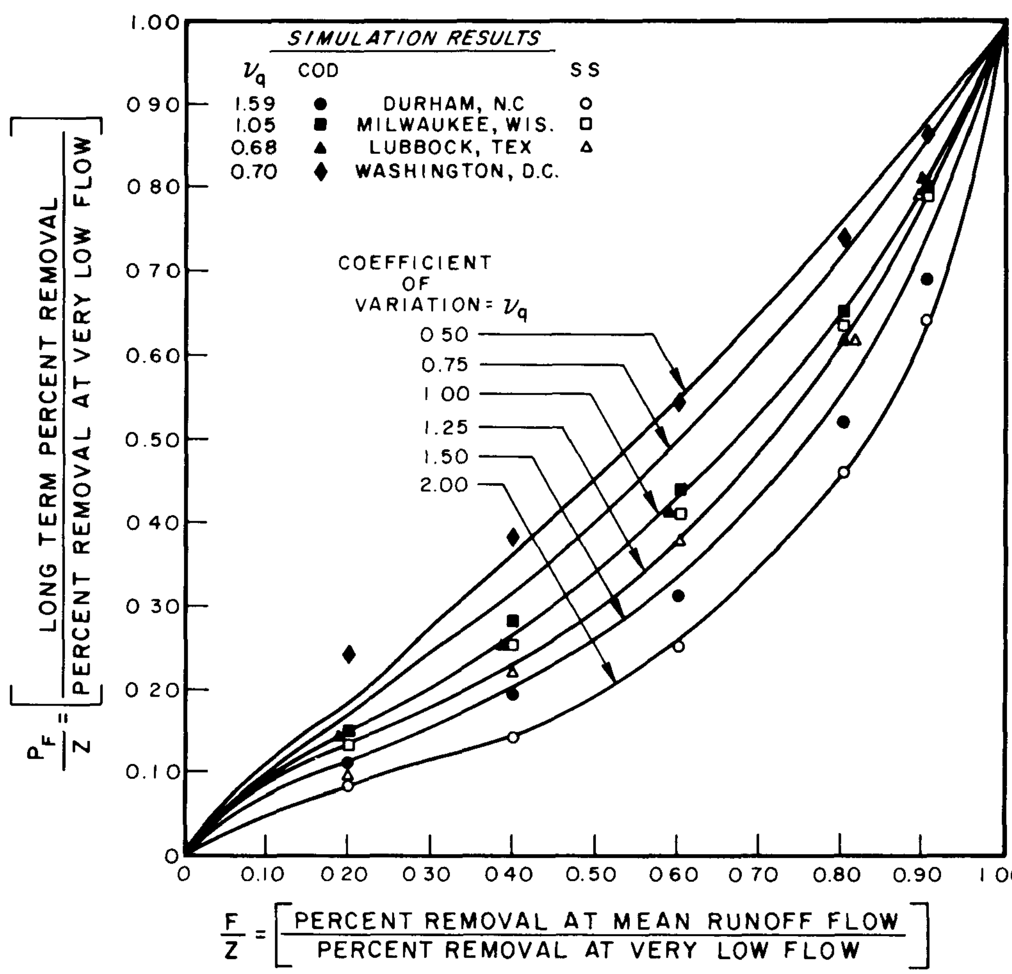


FIGURE 3-32

COMPARISON OF SIMULATED FLOW SENSITIVE DEVICE WITH THEORETICAL CURVES

TABLE 3-10

RATIO OF WITHIN STORM FLOW VARIABILITY
TO BETWEEN STORM FLOW VARIABILITY

City	$\bar{\sigma}_{qw}/\sigma_q$
Durham	0.48
Milwaukee	0.59
Washington, D.C.	0.64
Lubbock	1.43

- -

TABLE 3-11

FLOW CONCENTRATION CORRELATION

City	Constituent	$\bar{c}_f/\bar{c}$
Durham	COD	1.07
	SS	1.50
Milwaukee	COD	0.97
	SS	1.15
Washington, D.C.	COD	0.65
Lubbock	COD	0.82
	SS	1.17

3.6.2 Management Practices for Stormwater Control

Controls can be instituted at several stages of the stormwater pollution process. The previous section was directed towards controls instituted at the "end of the pipe," after the loads have been generated and conveyed. Because of the magnitude and intermittent nature of stormwater loads, the structural requirements to provide adequate end-of-pipe treatment may be significant. Management practices designed to reduce stormwater pollution before it is generated and conveyed provide an alternative to these large structural controls.

Management practices include source controls, such as street sweeping, catch basin cleaning programs, controls on the use and transport of harmful or hazardous materials, erosion control, control of surface flows, and for long range planning programs, land use control. Management techniques are also available for reducing pollutant concentrations and overflows from the sewer system. These include sewer separation, infiltration-inflow control, sewer flushing, polymer injection, and automated system controls. The effectiveness of each of these practices can be assessed by estimating their impact on the quality and quantity of stormwater runoff and overflows. A brief discussion of each of these management practices is presented, together with guidelines for quantitatively estimating their effectiveness.

3.6.2.1 Source Controls: Street Sweeping and Catch Basin Cleaning

Studies have been conducted to characterize the effect of street sweeping and catch basin cleanout on the removal of various contaminants (41,42,43,44). Important factors to consider are the type of sweeper used, the frequency of sweeping, the frequency of rain events, the rate at which contaminants accumulate in the drainage basin, and the street surface type and condition.

Current street sweeping practices in urban areas are estimated to be between 35 and 65 percent effective, averaging about 50 percent (42). Increased efficiencies can be achieved by reducing the speed of a sweeping pass to less than five m.p.h., and by increasing the frequency of passes (41). Enforced bans on parking along sweeping routes is necessary to insure effective removal. Utilization of more efficient machines and the adjustment of schedules to sweep more frequently near areas of high solids production can increase the total effectiveness of a street cleaning program (43).

Street sweeping approaches its maximum performance in terms of reducing the total yearly stormwater load when its frequency is much greater than the frequency of storm events. For example, street sweeping in areas with very long periods between storms, such as Phoenix, Arizona, would be quite effective at reducing the yearly stormwater load. One sweeping per week might be sufficient in these areas. However, in areas where it rains more frequently, such as Portland, Oregon, more frequent sweeping in the order of once per day may be necessary to significantly reduce the long term stormwater load.

To quantitatively assess the effect of street sweeping, an estimate may be made of the reduction in the average runoff concentration, $\bar{c}$. Experiments have demonstrated the accumulation of solids on street surfaces as a function of the elapsed time since the last cleaning by sweeping or rain (45), however, this is not the same as the concentration of contaminants in the runoff. Recently, numerous runs of the STORM simulation model were made by Heaney and Nix (46) for the City of Minneapolis, Minnesota, with varying street sweeping frequencies. From these runs, a relationship was developed for the long term fraction of street surface BOD removed as a function of the fraction of days streets are swept and the efficiency of the sweeper. This relationship is shown in Figure 3-33. Typical "pick-up" efficiency for a common brush-type sweeper is $\varepsilon = 0.50$ while more expensive vacuum sweepers may yield a higher efficiency of about $\varepsilon = 0.90$ (46). Note that Figure 3-33 was developed with one year's simulation in one location ($\Delta \simeq$ 3-3.5 days), and as previously discussed, may not be applicable to areas where the average time between storms (Δ) is significantly different. Ideally, a relationship similar to that shown in Figure 3-33 should be developed incorporating both the sweeping frequency and the average rainfall frequency. Until this analysis has been performed, however, Figure 3-33 may be used with caution for a first estimate. Also, note that Figure 3-33 describes the fraction of available street surface pollutants (BOD) removed, while nothing is said about pollutant runoff from other portions of the drainage area. Heany and Nix estimate that in a typical separate or unsewered area, 70 percent of the pollutant runoff is from the street surface, however, this number may vary (46). Area specific land use information may be used to refine the estimate of the fraction of the runoff load actually treatable by street sweeping. Finally, in combined sewer areas where overflow quality is largely influenced by the mix of runoff with sanitary sewage, street sweeping is considerably less effective at reducing stormwater loads.

3.6.2.2 Control of Harmful Materials

An effective method of decreasing the levels of toxic materials in stormwater runoff is to restrict the usage of those materials. Lead, zinc, antimony and asbestos are examples of toxic materials which are currently introduced to the environment through wear of automobile brakes and clutches and the breakdown of fuel and lubricants. In a long term program, these substances could be replaced by others which are less harmful. More immediate results could be expected through restrictions on the local use of pesticides and herbicides. The elimination of harmful compounds from deicing materials and a general improvement in the efficiency of deicing programs can reduce the pollution from this activity.

Very little is known about the relationship between the amount of a particular material introduced into the environment and its eventual concentration in stormwater runoff. Preliminary assessments may assume a direct relationship to estimate an order of magnitude concentration. Sophisticated simulation techniques are available for materials such as pesticides (2,47, 48), however, the projected improvement in runoff quality due to the restricted use of materials will probably only be a general estimate.

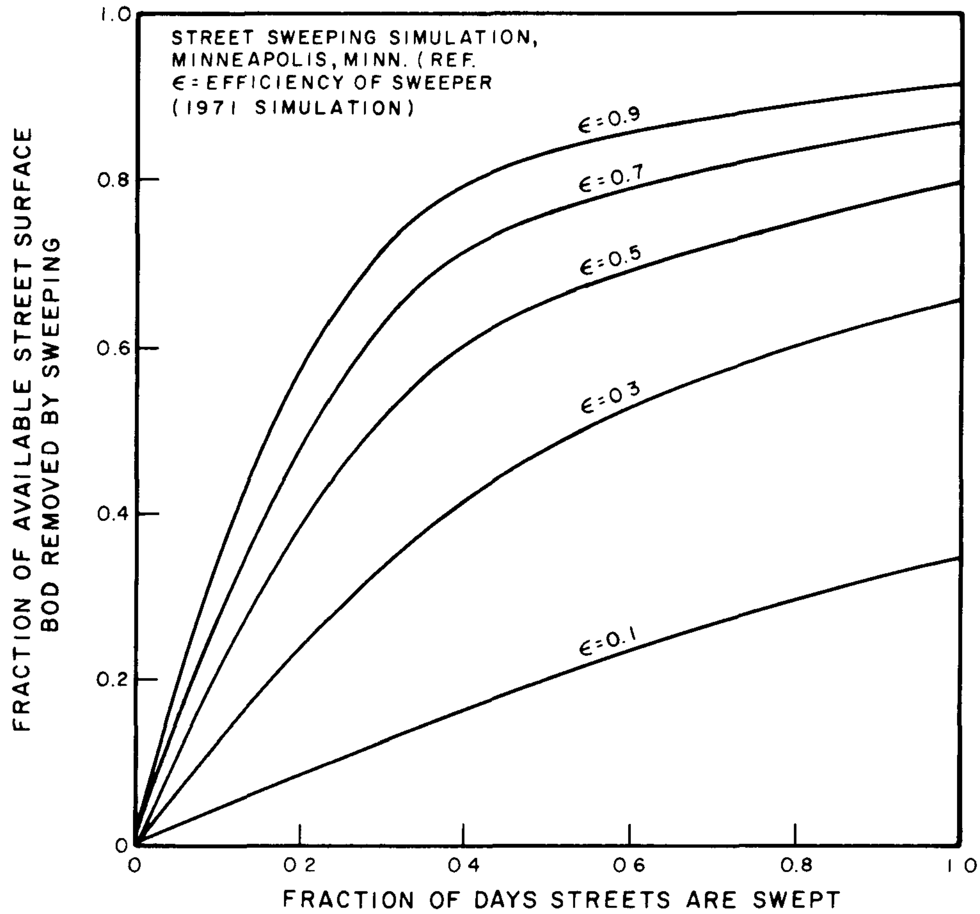


FIGURE 3-33
LONG-TERM EFFECTIVENESS OF STREET SWEEPING

3.6.2.3 Erosion Control

The following activities are suggested to improve control over erosion, which can introduce large amounts of nutrients and suspended solids to stormwater (42,44).

- proper selection of building and highway sites
- maintenance and protection of native vegetation
- use of mulches
- drainage channel protection modification
- careful backfilling after laying pipes
- protection of stockpiles for removed earth
- sediment retention basins
- scheduling of clearing and grading during season when erosion is less
- traffic control for construction and earth hauling equipment
- seeding areas with high erosion potential

Additional information for assessing the effect of erosion controls is presented in Chapter 4 of the Areawide Assessment Procedures Manual (2).

3.6.2.4 Control of Surface Flows

Several methods are available to reduce or delay stormwater runoff in urban areas. The increased attention these methods have received in recent years mark a new philosophy in the design of stormwater collection facilities which seek to retain stormwater within a drainage basin rather than transporting it as quickly as possible from the area.

A reduction of runoff can be brought about by increasing the period over which stormwater can percolate through permeable soil layers. Furthermore, stormwater which is not able to percolate into the soil can at least be delayed in order to reduce the surge or "slug" effects of the runoff. Methods for reducing or delaying runoff are listed in Table 3-12 (49). Although many of these methods actually involve structural modifications of the drainage basin, they are included in this section because they generally involve decentralized measures rather than larger, end-of-pipe facilities.

To assess the effectiveness of surface flow controls, estimates may be made of the increase in infiltration rates or the effective storage capacity provided throughout the watershed. Once these are determined, techniques developed for the analysis of long term interceptor and storage device performance can be applied. For example, the increased infiltration rate

TABLE 3-12

MEASURES FOR REDUCING AND DELAYING URBAN STORM RUNOFF

Area	Reducing Runoff	Delaying Runoff
Large flat roof	1. Cistern storage 2. Rooftop gardens 3. Pool storage or fountain storage 4. Sod roof cover	1. Ponding on roof by constricted downspouts 2. Increasing roof roughness a. Rippled roof b. Gravelled roof
Parking lots	1. Porous pavement a. Gravel parking lots b. Porous or punctured asphalt 2. Concrete vaults and cisterns beneath parking lots in high value areas 3. Vegetated ponding areas around parking lots 4. Gravel trenches	1. Grassy strips on parking lots 2. Grassed waterways draining parking lot 3. Ponding and detention measures for impervious areas a. Rippled pavement b. Depressions c. Basins
Residential	1. Cisterns for individual homes or groups of homes 2. Gravel driveways (porous) 3. Contoured landscape 4. Grandwater recharge a. Perforated pipe b. Gravel (sand) c. Trench d. Porous pipe e. Drywells 5. Vegetated depressions	1. Reservoir or detention basin 2. Planting a high delaying grass (high roughness) 3. Gravel driveways 4. Grassy gutters or channels 5. Increased length of travel of runoff by means of gutters, diversions, etc.
General	1. Gravel alleys 2. Porous sidewalks 3. Mulched planters	Gravel alleys

(ref. 49)

provided by porous pavement is equivalent to an available interceptor capacity, Q_I; with the captured runoff entering the groundwater regime. The amount of depression storage provided by rippled pavement or ponding on roofs is equivalent to an effective storage capacity, V_E, in the watershed.

3.6.2.5 Land Use Control

Increases in the degree of development and urbanization generally result in more severe stormwater impacts. The relationships between land use and stormwater loads are important for predicting future changes in stormwater loadings and for investigating the efficacy of land use control as a means of stormwater management. The Numerical Estimate sections of this manual discuss the impact of land use on both runoff quantity and quality. The relationships presented in these sections may be used to estimate the effectiveness of land use control.

A relationship between the percent impervious area and the average ratio of runoff to rainfall (R_V) is shown in Figure 5-20. Land use modifications which change the percent impervious area may be evaluated by calculating the new ratio of runoff to rainfall, the resulting change in stormwater loads, and the subsequent impact on receiving stream concentrations.

Urbanization often results in an even greater increase in runoff rates than in the total volume of runoff. This is due to a decrease in the attenuation time of the runoff event. The relationship between the attenuation of runoff events and urbanization is depicted in Figure 5-23 and Equations 3-13 and 5-9. Note that population density is used as a general indicator of land use conditions. Although the assessment of land use changes is thus somewhat indirect, estimates may still be made of the new mean runoff event duration (D_R), the resulting mean runoff flow (Q_R) and loading rate (W_R), and the subsequent change in stream concentrations.

Land use changes may also affect the quality of stormwater runoff. As discussed in Section 5.3.2 of Numerical Estimates, the ability to assess the effect of land use changes in runoff quality is dependent upon the establishment of a significant relationship for the particular study area. Assuming a satisfactory relationship exists, the new land use proposed for an area will result in a new estimate of the average pollutant runoff concentration ($\bar{c}$).

3.6.2.6 Collection System Management: Sewer Separation

It is recognized that routing stormwater through the same sewer system as sanitary sewage, i.e., a combined sewer system, can cause a reduction in municipal treatment plant efficiency during storm events. Furthermore, the surges in flow force the collection system to by-pass wastewater and discharge it directly to the receiving water body. Separate conveyance systems for the stormwater and the wastewater help to eliminate this problem, however, the cost of sewer separation in the older urban areas where combined sewers are prevalent may be prohibitive. Furthermore, if provisions are not made to treat the stormwater, direct stormwater discharges to a receiving water body from a separate system may contribute a greater pollution load than would be caused by occasional by-passes from a combined system (50).

To quantitatively estimate the effect of sewer separation on stormwater loads three steps are required. First, the stormwater concentration is changed in the now separately sewered area as indicated in Section 5.3.1 of Numerical Estimates. Secondly, the fraction of the runoff load formerly captured by the interceptor and treated at the municipal treatment plant now enters the receiving stream directly (though with a different pollutant concentration). Finally, the average sewage treatment efficiency at the municipal plant may improve, thereby decreasing the continuous municipal loading rate.

3.6.2.7 Infiltration and Inflow Control

"Infiltration" is the introduction of additional flow into a sewer through leaky joints and broken pipes. "Inflow" is the introduction of additional flow through deliberate or accidental sewer connections from water users. Both intrusions utilize a portion of the sewer capacity.

Infiltration can be prevented in new pipe systems through adequate design and testing; it can be eliminated in existing systems by survey and correction. Elimination systems can help reduce the necessity for by-passing during storm events. Roof leaders can either be reconnected to the storm sewer system, where the runoff will be treated or discharged to the receiving water body, or can be allowed to drain onto pervious areas. The reduction of the infiltration rates into a combined sewer system will increase the available interceptor capacity for stormwater capture, Q_I. This may be evaluated using Figure 3-15 or with the technique demonstrated in Figure 3-25. The control of infiltration and inflow into a separate sewer system should ideally eliminate wet weather overflows.

3.6.2.8 Sewer Flushing

The deposition of solids during dry weather in slow flowing portions of combined sewers and the trapping of sediments in portions of separate sewers provide a source of pollutants for resuspension during subsequent stormwater flows. Recent studies of this phenomena and the potential of flushing to reduce the resulting wet weather load provide some basis for evaluating the effectiveness of sewer flushing (51,52,46).

The study of deposition and flushing in the Boston area (52) presents general relationships for estimating the total mass of solids deposited in a combined sewer system as a function of the total collection system pipe length, drainage area, average pipe slope, and the average wastewater flow. Guidelines are also presented for estimating the extent of the sewer system over which significant deposition occurs. Finally, current research on sewer flushing operations is described. The relationships presented are appropriate for an initial assessment of the potential effectiveness of sewer flushing in a study area. Hendy and Nix (46) provide a further simplification of this approach which allows an estimate of the reduction in the deposition related stormwater load. This assessment may be used to modify the average overflow concentration ($\bar{c}$) to relate flushing programs to eventual changes in receiving water quality.

3.6.2.9 Polymer Injection

The flow capacity of certain pipes can be increased by adding polymers to the water flowing through them. Injection of a polymer-gelled slurry reduces wall friction and allows a temporary increase in line capacity. Automatic polymer-injecting units can be positioned along key sewer trunk lines and directed to inject the chemical at critical points in a storm event. The polymers selected for this use are similar to those developed for treatment plant clarifiers, and have been shown not to be disruptive to bacterial growth or to provide nutritive value to algae.

The overall combined sewer flow capacity is dictated by the critical points in the system, where the polymer injection should be directed. The long term effectivness of the polymer injection program is thus evaluated by estimating the resulting increase in the available interceptor capacity for storm runoff (Q_I).

3.6.2.10 Automated System Control

In combined sewer systems, interceptor lines which carry both dry weather sewage flows and storm runoff during rainfall events may provide a significant measure of control by virtue of their ability to retain a portion of the storm flow and route it to the sewage treatment plant. Consistent with the ability of the sewage treatment plant to handle increased flows, control may be improved by maximizing retention of storm flows in existing collection systems.

Both separate and combined sewer systems can be made more effective by the utilization of remote monitoring and control systems. These systems, which might include level sensors, tide gates, raingage networks, sewage and receiving water quality monitors, overflow detectors, and flow meters, can effectively regulate sewer flows and provide a controlled flow to the treatment facility. This approach can be useful in avoiding overflowing and by-passing by making a more effective use of the line capacities (53,54).

To assess the effectiveness of automated controls on a preliminary basis, an estimate may be made of the increase in the effective storage capacity (V_E) provided by the sewer system. A decrease in the average overflow concentration ($\bar{c}$) may also be appropriate for systems with selective, quality sensitive overflows. Simulation should be used for more detailed studies of automated control systems.

An overview of management practices available for the control of stormwater runoff and overflows has been presented. Guidelines for evaluating their effectiveness within the framework of the statistical assessment methodology have also been provided. The final step is to relate the improvements in receiving water quality due to the various treatment alternatives to the cost of each plan.

3.6.3 Benefit and Cost Evaluation

Stormwater controls in an urban area are implemented to protect or improve the quality of adjacent waterways. As discussed in Chapter 2, benefits are measured in terms of the value of beneficial uses of the water which are created or preserved. The costs are measured in terms of the resources required to develop, implement, and maintain the stormwater control strategy. The selection of a stormwater control plan requires a favorable balance between these benefits and costs.

Methods are available for quantitatively estimating the value of water uses, such a water supply, recreation, navigation, etc. If this can be done for a particular area, a rigorous cost-benefit analysis may be performed (55). Often, however, the benefits may be difficult to quantify, and the assessment framework becomes less direct. Receiving water benefits are measured in terms of compliance with water quality standards or criteria, which in turn are established by responsible planning agencies to protect or enhance specific beneficial uses. The problem then becomes one of meeting receiving water standards in a cost-effective manner. The following sections develop this approach and provide guidelines for implementing it in a planning study.

3.6.3.1 Improvements in Receiving Water Quality

Methods for estimating the impact of stormwater loads in the receiving water are presented in Section 3.5. These methods may again be used with the modified loads determined after treatment, and a new receiving water response is predicted. Different combinations of types of controls and sizes or amounts for each may be tested.

If a particular water quality standard is used as the basis for analusis, the load reductions necessary to meet the standard may be determined. For standards directed towards long term average water quality, reductions in the long term average loading rate (W_o) are determined. For transient impacts, load reductions necessary to violate a standard less frequently (i.e., only 3 times per summer rather than 15 times) are examined. In some areas, standards are now being written in a probabilistic manner, identifying the number or frequency of exceedances which are permitted (56). These types of standards are more appropriate for dealing with intermittent, stormwater related water quality problems. The primary problem in developing these standards is identifying the ecological effects of intermittent periods of high (or in the case of dissolved oxygen, low) instream concentrations. Research is needed to obtain a better understanding of the severity of these impacts in terms of water use limitations. Once these relationships are better understood, more meaningful probabilistic standards may be developed. Changes in the mean, variability, and frequency distribution of stormwater loads are then examined and translated into receiving water responses to test for compliance.

Analysis tools have been presented for quantitatively assessing pollutant concentrations in the receiving water. These tools are appropriate for addressing many of the standard water quality problems encountered in a planning study: dissolved oxygen, coliform bacteria, solids, toxicants, etc. Many of the stormwater problems found in an urban area, however, are not as readily addressed with these models. These problems include such things as floatables, and oils and grease in the vicinity of combined sewer overflows, particularly near residential or recreational areas, the deposition of organic matter near outfalls, and the erosion of natural stream channels. These problems are more difficult to analyze with generalized models, and a good understanding of local conditions and particular problems in the study area is required. The planner should visit the area during and following storms, and significant interaction with the public should be encouraged. In certain areas, these less easily modeled problems may be the most important for improving the quality of urban waterways (57).

3.6.3.2 Indirect Benefits of Stormwater Control

Although the most direct impact of stormwater control is the benefit on receiving water quality, there are also a number of other impacts which may result from the implementation of the plan. These indirect impacts are particularly important when considering stormwater control as one aspect of an overall, integrated plan for improving the quality of life in urban areas.

Flood control is one of the most obvious programs which may be integrated with a stormwater control plan. Storage ponds or tanks which reduce and attenuate pollutant loads may also serve to alleviate downstream flooding problems. Indeed, flood control may be the primary benefit of detention systems. This has important applications for the costing and funding of stormwater control plans. By integrating the controls with other aspects of urban planning and development, costs may be shared and an overall increase in the efficiency of community efforts may be obtained. Other examples of auxiliary benefits which may be integrated with stormwater control include land preservation through erosion control, improved drainage, cleaner streets due to street sweeping or catch basin cleaning, and improvements in street, utility, and conveyance system layouts for the purpose of maintenance and control. Finally, potentially harmful aspects of a stormwater control plan should also be identified and evaluated. These include factors such as relocation for construction, the contamination of surface or groundwaters from system failure or operational accidents such as chemical spills, and requirements for sludge or residual disposal after treatment.

3.6.3.3 Evaluating Cost of Controls

Planning assessments require an estimate of the cost of alternative stormwater control strategies. Curves for estimating the capital, operational and maintenance costs of stormwater treatment processes of various sizes are presented in Section 5.5.3 of the Numerical Estimates Chapter. Standard engineering costing procedures incorporating factors such as inflation and interest should be used.

Costs may be determined for a number of strategies, and a procedure developed for comparing alternatives. This procedure may involve a simple listing of alternative plans, their benefits and costs; or it may involve more sophisticated attempts to optimize with tools such as linear programming

(58). The planner is cautioned, however, not to let the decision making process become lost in a complex computer program or a routinized scheme. These may serve as useful aides, however, they are no substitute for sound, informed, and practical planning and engineering judgment for the solution of stormwater problems. The planning process should be flexible and developmental; cognizant of shortcomings in the analysis and data upon which it is based and seeking to improve the understanding of stormwater problems to develop effective and workable solutions.

3.7 References

1. Field, Richard, Anthony N. Tufuri, Hugh E. Masters, *Urban Runoff Pollution Control Technology Overview*, U. S. Environmental Protection Agency, EPA-600/2-77-047, Cincinnati, Ohio, March 1977.

2. *208 Areawide Assessment Procedures Manual*, Volume I, Chapter 2-5, Hydroscience, Inc. for U. S. Environmental Protection Agency, APA-600/9-76-014, Cincinnati, Ohio, July 1976.

3. Heaney, James P., Wayne C. Huber, Stephan J. Nix, *Storm Water Management Model, Level I, Preliminary Screening Procedures*, EPA-600/2-76-75, Cincinnati, Ohio, October, 1976.

4. Lager, John A., Theodor Didriksson, George B. Otte, *Development and Application of a Simplified Stormwater Management Model*, Metcalf and Eddy, Inc., for U. S. Environmental Protection Agency, EPA-600/2-76-218, Cincinnati, Ohio, 1976.

5. Brandstetter, Albin, *Assessment of Mathematical Models for Storm and Combined Sewer Management*, Battelle, Pacific Northwest Laboratories for U. S. Environmental Protection Agency, EPA-600/2-76-175a, Cincinnati, Ohio, 1976.

6. *Short Course Proceedings, Applications of Stormwater Management Models, 1976*, Edited by Francis A. Digiano, Donald A. Adrian, and Peter A. Mangarella, University of Massachusetts for Environmental Protection Agency, EPA-600/2-77-065, Cincinnati, Ohio, March 1977.

7. *Storm Water Management Model, Volume I - Final Report*, Metcalf and Eddy, Inc., University of Florida, and Water Resources Engineers for U. S. Environmental Protection Agency, Water Quality Office, Report No. 11024DOC07/71, September, 1971.

8. *Storm Water Management Model, Volume II - Verification and Testing*. Metcalf and Eddy, Inc., University of Florida, and Water Resources Engineers, Inc. for U. S. Environmental Protection Agency, Water Quality Office, report No. 11024DOC08/71, September, 1971.

9. *Storm Water Management Model, Volume III - User's Manual*, Metcalf and Eddy, Inc., University of Florida, and Water Resources Engineers, Inc., for U. S. Environmental Protection Agency, Water Quality Office, Report No. 11024DOC09/71, September, 1971.

10. *Storm Water Management Model, Volume IV - Program Listing,* Metcalf and Eddy, Inc., University of Florida, and Water Resources Engineers, Inc., for U. S. Environmental Protection Agency, Report No. 11024DOC10/71, September, 1971.

11. Smith, G. F., *Adaptation of the EPA Storm Water Management Model for Use in Preliminary Planning for Control of Urban Storm Runoff,* M.E. Thesis, Department of Environmental Engineering Sciences, University of Florida, Gainesville, May, 1975.

12. Hydrologic Engineering Center, Corps of Engineers, *Urban Storm Water Runoff: STORM,* Generalized Computer Program 723-58-L2520, Davis, California, January, 1975.

13. Howard, Charles D. D., *Theory of Storage and Treatment Plant Overflows,* Journal of the Environmental Engineering Division, ASCE, Vol. 102, No. EE4, August, 1976.

14. Di Toro, Dominic M., and Mitchell J. Small, Discussion of *Theory of Storage and Treatment Plant Overflows,* Journal of the Environmental Engineering Division, ASCE, Vol. 102, No. EE3, June, 1977.

15. *Water Quality Management Planning Methodology for Urban and Industrial Stormwater Needs,* Hydroscience, Inc., for Texas Water Quality Board, Arlington, Texas, 1976.

16. Chow, Ven Te, Ben Chie Yen, *Urban Stormwater Runoff: Determination of Volumes and Flowrates,* University of Illinois for U. S. Environmental Protection Agency, EPA-600/2-76-116, Cincinnati, Ohio, May, 1976.

17. Parzens, E., *Stochastic Processes,* Holden Day, San Francisco, 1964.

18. DiToro, Dominic M., *Statistical Analysis of Intermittent Runoff and Receiving Water Responses.* In press 1978.

19. *Snow Hydrology, Summary of Report of the Snow Investigations,* U. S. Army Corps of Engineers, North Pacific Division, Portland, Oregon, 1956.

20. Donigian, Anthony S., Jr., and Norman H. Crawford, *Modeling Nonpoint Pollution from the Land Surface,* Environmental Protection Agency, Research Grant No. R803315-01-0, February, 1976.

21. Dunne, Thomas, Anthony G. Price, Samuel C. Colbreck, *The Generation of Runoff from Subarctic Snowpacks,* Water Resources Research, Vol. 12, No. 4, August, 1976.

22. *Simplified Mathematical Modeling of Water Quality,* Hydroscience, Inc., and Mitre Corporation, Environmental Protection Agency Publication, March 1971.

A STATISTICAL METHOD FOR ASSESSMENT OF URBAN STORMWATER LOADS - IMPACTS - CONTROLS

FOR

EPA - NON POINT SOURCES BRANCH

WASHINGTON, D.C.

PROJECT OFFICER

DENNIS N. ATHAYDE
MANAGER
NATIONWIDE URBAN RUNOFF PROGRAM

JANUARY 1979

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY

DATE: MAR 19 1979

SUBJECT: Transmittal of Document Entitled "A Statistical Method for Assessment of Urban Runoff"

FROM: Swep T. Davis, Deputy assistant Administrator
Office of Water Planning and Standards ~~(WH-554)~~

TO: All Regional Water Division Directors
ATTN: All Regional 208 Coordinators
All Regional NPS Coordinators
All Nationwide Urban Runoff Prototype Projects
All State and Areawide Water Quality Management Agencies
Other Concerned Groups

TECHNICAL GUIDANCE MEMORANDUM-TECH- 49

Purpose

This document "A Statistical Method for Assessment of Urban Runoff" has been prepared to provide technical assistance to the Nationwide Urban Runoff prototype projects and other interested groups in assessing the impact of urban stormloads on the quality of receiving waters, and to evaluate the cost and effectiveness of control measures for reducing these pollutant loads.

Guidance

The enclosed report is provided in accordance with the Nationwide Urban Runoff Program established under Section 208 of the Clean Water Act of 1977. This methodology is appropriate for use at the planning level where preliminary assessments are made to define problems, establish the relative significance of contributing sources, assess feasibility of control, and determine the need for and focus on additional evaluations. It can also be used effectively in conjunction with detail studies, in evaluating the most cost-effective alternatives for controlling urban runoff.

Attachment

STORMWATER MANUAL

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23. *Mathematical Modeling of Natural Systems,* Summer Institute Notes, Environmental Engineering and Science Division, Manhattan College, Bronx, New York, May 1976.

24. Thomann, R. V., *System Analysis and Water Quality Management,* Environmental Research and Applications, Inc., New York, 1972.

25. *Water Quality Management Planning Methodology for Hydrographic Modification Activities,* Hydroscience, Inc. for the Texas Water Quality Board, 1976.

26. Vollenweider, R. A., *Scientific Fundamentals of the Eutrophication of Lakes and Flowing Waters--With Particular Reference to Nitrogen and Phosphorus as a Factor in Eutrophication,* Organization for Economic Cooperation and Development, DAS/CSI/68.27, Paris, France, 1968.

27. Dillon, P. J., and F. H. Rigler, *A test of a Simple Nutrient Budget Model Predicting the Phosphorus Concentrations in Lake Water,* Canada Fishery Research Board Journal, Vol. 31, 1974.

28. Thomann, R. V., D. M. Di Toro, R. P. Winfield, and D. J. O'Connor, *Mathematical Model of Phytoplankton in Lake Ontario,* U. S. Environmental Protection Agency, EPA-660/3-75-005, March 1975.

29. Hydroscience, Inc., *Effects of Dispersion Upon Storm Pulses in a Stream,* Internal Technical Memorandum, December 1976.

30. *Proceedings of Symposium on Diffusion in Oceans and Fresh Waters,* Columbia University, 1964.

31. *Handbook of Mathematical Functions with Formulas, Graphs, and Mathematical Tables,* U. S. Department of Commerce, National Bureau of Standards, Applied Math Series No. 55, 1964.

32. *Analytical Method: Further Analysis of Filters with Performance Dependent Upon Influent Concentration,* Internal Memorandum, Hydroscience, Inc., April 1976.

33. Di Toro, Dominic M., *Statistical Analysis of Urban Runoff Treatment Devices,* Presented at U. S. Environmental Protection Agency National Conference on 208 Planning and Implementation, Reston, Virginia, March 1977.

34. Kaufman, Herbert L., *Combined Sewage Overflows - Frequency, Environmental Effects, and a Corrective Program,* National Symposium on Urban Hydrology, Runoff, and Sediment Management, University of Kentucky, July 1974.

35. *Control of Combined Sewage Flooding and Overflows,* Clinton Bogert Associates for the City of Trenton, New Jersey, January, 1974.

36. Heaney, J. P., et al., *Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume II: Cost Assessment and Impacts,* University of Florida for U. S. Environmental Protection Agency, EPA-600/2-77-064, Cincinnati, Ohio, March 1977.

37. Colston, N. V., *Characterization and Treatment of Urban Land Runoff,* North Carolina State University for U. S. Environmental Protection Agency, EPA-670/2-74-096, Cincinnati, Ohio, December 1974.

38. Consoer, Townsend and Associates, *Humbolt Avenue Pollution Abatement Demonstration Project for the City of Milwaukee,* Engineering Report, September 1974.

39. Roy F. Weston, Inc., *Combined Sewer Overflow Abatement Alternatives, Washington, D. C.,* U. S. Environmental Protection Agency, EPA-11024, EXF 08/70, August 1970.

40. Texas Tech University, Water Resources Center, *Variation of Urban Runoff Quality with Duration and Intensity of Storms,* WRC-73-2, 1973.

41. Sartor, J. D. and G. B. Boyd, *Water Pollution Aspects of Street Surface Contaminants,* U. S. Environmental Protection Agency, EPA-R2-72-081, November 1972.

42. Lager, J. A. and W. G. Smith, *Urban Stormwater Management and Technology: An Assessment,* U. S. Environmental Protection Agency, EPA-670/2-74-040, December, 1974.

43. Shaheen, D. G., *Contributions of Urban Roadway Usage to Water Pollution,* U. S. Environmental Protection Agency, Washington, D. C., EPA-600/2-75-004, March 1975.

44. *Guidelines for Erosion and Sediment Control Planning and Implementation,* Maryland Department of Water Resources and Hittman Associates, Inc. for U. S. Environmental Protection Agency, 15030 FMZ, August 1972.

45. *Water Quality Management Planning for Urban Runoff,* URS Research Company, for U. S. Environmental Protection Agency, EPA-440/9-75-004, December 1974.

46. Heaney, James P., Stephen J. Nix, *Storm Water Management Model: Level I - Comparative Evaluation of Storage Treatment and Other Management Practices,* U. S. Environmental Protection Agency, EPA-600/2-77-083, April 1977.

47. Frere, M. H., C. A. Onstad, and H. N. Holtan, *ACTMO-An Agricultural Chemical Transport Model,* Agricultural Research Service, U. S. Department of Agriculture, Hyattsville, Maryland, ARS-H-3, June 1975.

48. Donigian, A. A., Jr., and N. H. Crawford, *Modeling Pesticides and Nutrients on Agricultural Lands,* U. S. Environmental Protection Agency, EPA-600/2-76-043, February 1976.

49. *Urban Hydrology for Small Watersheds*, U. S. Conservation Service, Tech. Release No. 55, NTIS PB-244 531, Washington, D. C., January, 1975.

50. Burm, R. J., D. F. Krawczyk, and G. L. Harlow, *Chemical and Physical Comparison of Combined and Separate Sewer Discharges*, Journal of the Water Pollution Control Federation, Vol. 40, No. 1, January 1968.

51. *A Flushing System for Combined Sewer Cleaning*, FMC Corporation for U. S. Environmental Protection Agency, 11020 DNO, March 1972.

52. Pisano, William C., Celso S. Queiroy, *Procedures for Estimating Dry Weather Pollutant Deposition in Sewerage Systems*, U. S. Environmental Protection Agency, EPA-600/2-77-120, July 1977.

53. Leiser, E.P., *Computer Management of a Combined Sewer System*, U. S. Environmental Protection Agency, EPA 670/2-74-022, Cincinnati, Ohio, July 1974.

54. Labadie, J. W., N. S. Grigg, B. H. Bradford, *Automatic Control of Large-Scale Combined Sewer Systems*, Journal of the Environmental Engineering Division, ASCE, Vol. 101, No. EE1, February 1975.

55. Dorfman, Robert, and Nancy S. Dorfman, *Economics of the Environment, Selected Readings*, W. W. Norton and Company, Inc., New York, 1972.

56. *Evaluation of Discharge Alternatives for South (San Francisco) Bay Dischargers Authority*, Hydroscience, Inc., December 1975.

57. *Girtys Run, A Study in Urban Watershed Management*, Carnegie-Mellon University, Pittsburgh, Pennsylvania, May 1974.

58. Kirshen, Paul H., David H. Marks, and John C. Schaake, *Mathematical Model for Screening Stormwater Control Alternatives*, Department of Civil Engineering, Massachusetts Institute of Technology, Report No. 157, October 1972.

CHAPTER 4

SIMULATION OF STORMWATER IMPACTS

In addition to the statistical methods presented in Chapter 3, storm loads and their impacts may be estimated by the use of computer simulation techniques. Simulators provide a representation of the actual temporal sequence of storm events, their loads, and the time sequence of the receiving water response. In this sense, simulation is a "brute-force" method for evaluating wet weather impacts and their potential control. Like the statistical method which develops direct, analytical solutions for pertinent problems, simulators require a degree of simplification of the real rainfall-runoff-receiving water process. However, computer simulators often allow a more detailed representation of the spatial and temporal complexities of the actual urban drainage system, and are thus particularly useful for the later, more refined levels of stormwater planning. If simulators are used in the initial assessment stage of runoff analysis, simpler versions, able to process relatively long periods of rainfall records, at the sacrifice of detail for individual events, are preferable.

4.1 Available Models

Models for evaluating wet weather impacts are generally divided into two classes: (1) land-side drainage simulators, and (2) receiving water models; though some packages include both aspects. The land-side simulators utilize rainfall records and drainage area characteristics to develop storm flows and loads. They incorporate factors such as the catchment hydrology, sewer hydraulics, runoff quality, and control alternatives (storage, treatment, etc.). Receiving water models may be used generally for a range of problems, or developed specifically for analyzing wet weather quality impacts. They include factors such as the physical characteristics of the water body, pollutant transport, and the chemical and biological reaction rates of pollutants. A wide range of complexity, spatial and temporal detail are found in available models.

Three recent reports provide useful summaries of computer simulation models currently available to planners. These reports are:

1. "Areawide Assessment Procedures Manual, Appendix A" (1),

2. "Evaluation of Water Quality Models: A Management Guide for Planners" (2), and

3. "Assessment of Mathematical Models for Storm and Combined Sewer Management" (3).

Appendix A of the Areawide Assessment Procedures Manual discusses models which are generally available, well-documented, and have been tested and applied. The models covered are listed in Table 4-1. The three reports listed above, as well as the specific model documentations, should be referenced for details on the program capabilities, limitations, and use.

It is noteworthy that while a wide range of land-side simulators are available specifically for stormwater assessments, including simplified models appropriate for the preliminary evaluation procedures presented in this manual (4,5), less effort has been made to make receiving water models particularly applicable to broad scale stormwater assessments. A relatively simple receiving water model useful for initial wet weather evaluations has been developed for this manual. This receiving water model, described in the following sections, is designed to accept as input, the time variable storm loads generated by the available land-side simulators.

4.2 Broadscale Receiving Water Simulator

The Broadscale Receiving Water Simulator (BRWS) provides the engineer or planner with an analytical tool to determine the response of a river (stream) or estuary to coliform bacteria, BOD, and dissolved oxygen deficit stormwater discharges and combined sewer overflows. The model is geared to use with available land side simulators for preliminary assessments. It can provide the user with an added level of temporal detail of loads and impacts, and greater spatial definition. Like the land side simulators which are used in such assessments, BRWS adopts a simplified approach.

A one-dimensional representation of the receiving water stream or estuary, and simple input parameters are employed. A single storm load discharge location is used, with input flows and loads which vary with time according to rainfall records and the output of a land side simulator, with or without reductions due to applied control measures.

The model calculates the time history of concentrations which result in the receiving water due to such intermittent loads, at up to six different locations. Results are displayed in a matrix showing concentration as a function of location and time, and also in a plot of concentration vs. time. A separate run is required for each contaminant analyzed. The model can accomodate conservative contaminants (Dissolved Solids), simple reactive contaminants (coliform bacteria), or coupled reactions (BOD-D.O.).

At the completion of each run, BRWS calculates and outputs the frequency distribution, the mean concentration and the standard deviation of the contaminant analyzed at each of the six locations in the receiving water. Concentration history is compared with a water quality standard, and the percentage of time this criterion is violated at a particular location within the river or estuary, is calculated and output. The model is thus designed to address directly the significant planning questions relevant to wet weather impact evaluations. The procedure is outlined schematically in Figure 4-1.

TABLE 4-1

AVAILABLE COMPUTER MODELS
(FROM AAPM, APPENDIX A (1))

Land-Side Simulators:

Acronym	Model Origin
STORM	Corps of Engineers
SWMM	EPA
HSP	Hydrocomp
MITCAT	Mass. Inst. of Technology
HVM-QQS	Dorsch Consult
(Simplified Simulator)	Metcalf & Eddy
AGRUN	Water Resources Eng.
ILLUDAS	Ill. State Water Survey

Receiving Water Quality Models:

DOSAG-I	EPA
QUAL-II	EPA
RECEIV	EPA
RECEIV-II	Raytheon (EPA)
SRMSCI	Systems Control Inc.
WRECEV	Water Resources Eng. (EPA)
HWQM	Hydrocomp, Inc.
LAKECO	Office of Water Resources Research
LEVEL III	EPA (by Medina, in press 1979)

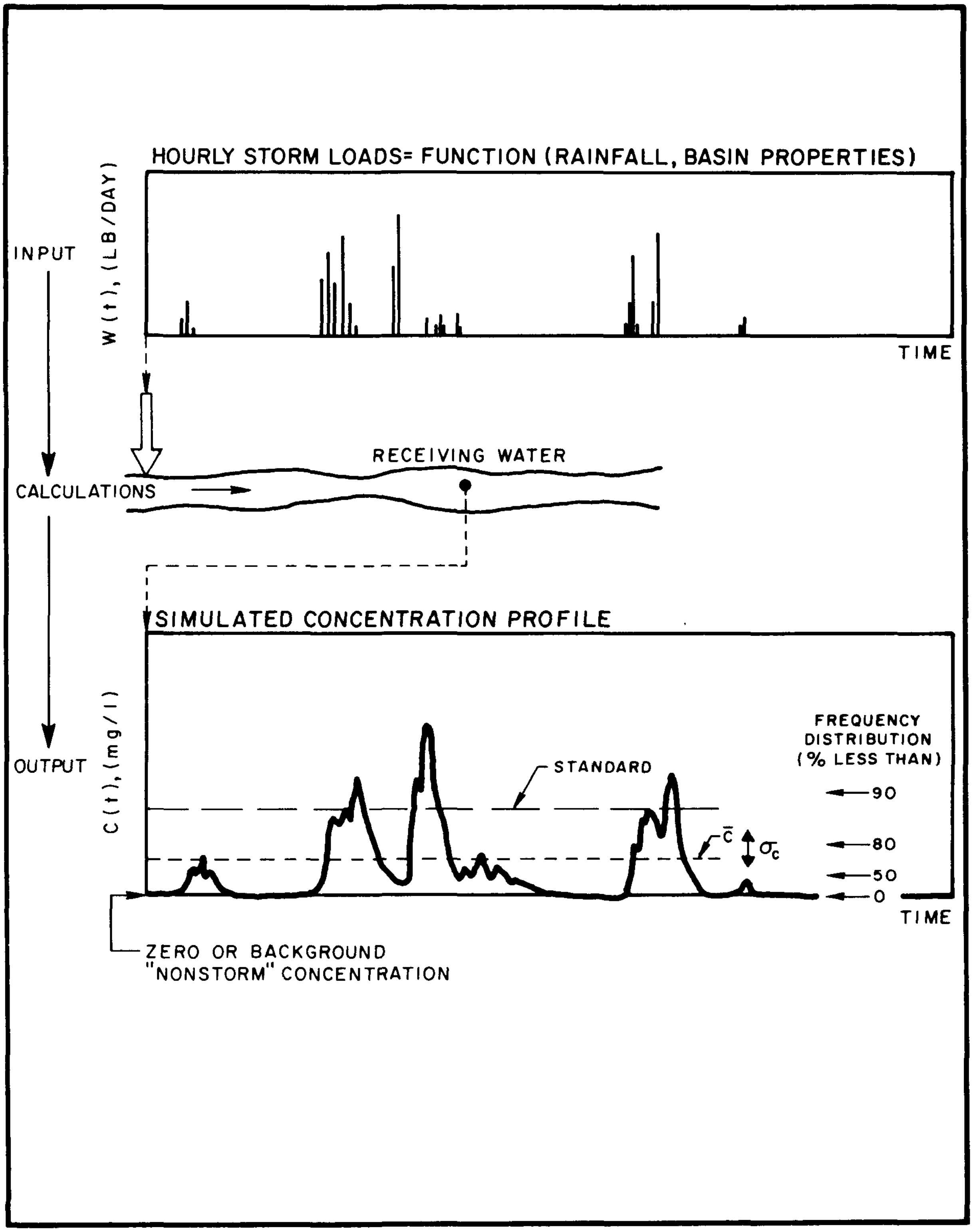


FIGURE 4-1

SCHEMATIC OF BROADSCALE
RECEIVING WATER SIMULATOR OPERATION

4.2.1 Internal Calculations

The BRWS model treats each recorded time interval of rainfall, flow, and load as a separate, discrete event. The response for each is calculated from the appropriate analytical solution for an impulse function (of specified duration δ, e.g. 1 hour, 24 hours) shown in Table 4-2. The water quality response for the entire rainfall-loading sequence is generated via superposition of the discrete response. This is based on the assumption that the receiving water concentration responds linearly to pollutant loadings. The program is written in Fortran IV, and a complete listing is provided in Appendix B.

4.2.2 Program Capabilities and Limitations

The basic capabilities and limitations of the BRWS model are outlined as follows:

1. Capabilities:

   a. Receiving water bodies - rivers (streams) and estuaries;
   b. Water quality constituents - coliform bacteria, BOD, and dissolved oxygen; Note: conservative constituents can be handled using the BOD input routine, by setting reaction rate (K) = 0.
   c. Maximum length of rainfall sequence - approximately 1,500 records (two months) if hourly intervals are selected; four years if daily interval is selected;
   d. Number of discharge locations (per run) - one (1);
   e. Maximum number of locations within river or estuary for evaluating water quality response - six (6);
   f. Ability to include stormwater flow in calculation of concentration if significant;
   g. Ability to include background conditions in rivers; and
   h. Input either from disk files (generated by a drainage basin simulator) or from punched cards.

2. Limitations:

   a. Geophysical parameters (cross sectional area, temperature, reaction rates) are assumed to be constant throughout the river or estuary;
   b. BOD removal will be calculated even if river or estuary becomes anaerobic;
   c. Inability to specify background (boundary) conditions for estuaries;
   d. Inability to correct velocity (time-of-travel) if stormwater flow is significant in relation to base stream flow;
   e. Only net dispersion is modeled in estuaries, not flow reversals within tidal cycles.

4.2.3 Model Inputs

The following inputs are required for the BRWS model:

TABLE 4-2

ANALYTICAL SOLUTIONS
USED IN BRWS

Stream BOD Response:

$$c(x,t) = \delta(t - \frac{x}{u}) \frac{M}{Q} e^{-\frac{Kx}{u}}$$

Stream Dissolved Oxygen Deficit Response:

$$D(x,t) = \delta(t - \frac{x}{u}) \frac{M_{BOD}}{Q} (\frac{K_d}{K_a - K_r})(e^{-\frac{K_r x}{u}} - e^{-\frac{K_a x}{u}}) + \delta(t - \frac{x}{u}) \frac{M_{DOD}}{Q} e^{-\frac{K_a x}{u}}$$
$$+ D_o e^{-\frac{K_a x}{u}}$$

Dissolved Oxygen Response:

$$c(x,t) - C_s - D(x,t)$$

Estuary BOD Response[a]:

$$c(x,t) = \frac{M}{A\sqrt{4\pi Et}} e^{-\frac{(x - ut)^2}{4Et} - K_r t}$$

Estuary Dissolved Oxygen Deficit Response[a]:

$$D(x,t) = \frac{K_d}{K_a - K_r} \frac{M_{BOD}}{A\sqrt{4\pi Et}} \{ e^{-\frac{(x-ut)^2}{4Et} - K_r t} - e^{-\frac{(x-ut)^2}{4Et} - K_a t} \}$$
$$+ \frac{M_{DOD}}{A\sqrt{4\pi Et}} e^{-\frac{(x-ut)^2}{4Et} - K_a t}$$

[a]Note that there is no finite solution for t = 0, so an arbitrarily small time was selected for the first calculation at the discharge point. This causes some error in the calculation, and the exact discharge location, x = 0, should be avoided (i.e. chose a location a litte upstream or downstream of the discharge for simulation).

1. Specification of water body type (river or estuary);
2. Water quality constituent type;
3. Location of evaluation points relative to the discharge;
4. Geophysical parameters including flow, dispersion, cross sectional area, reaction rates, background concentrations, and dissolved oxygen saturation value;
5. Water quality standards to be checked; and
6. Rainfall and discharge record (storm flows and loads).

The input structure for the BRWS model is presented in Appendix A. An example input deck is shown in Table 7-8 of the Kingston, New York application of the simulation model (Section 7.2.5).

4.2.4 Model Output

The following output is produced from a BRWS model run:

1. Listing of input parameters used;
2. Time history plot of the rainfall, load, and concentration calculated at each location specified;
3. Time history listing of the concentrations calculated at each location specified; and
4. Summary for each location including the cumulative distribution function, the number of occurances and the duration of standard violations, and the average, standard deviation, and maximum concentration simulated.

A complete application of the receiving water simulator for the Kingston, New York example, is presented in Section 7.2.5. This includes portions of the simulator output (Figures 7-17 (a)-(e)).

4.3 References

1. *Areawide Assessment Procedures Manual, Volume II, Appendix A*, U.S. Environmental Protection Agency, EPA-600/9-76-014, July 1976.

2. Grimsrud, G. Paul, E.J. Finnemore, and H.J. Oen, *Evaluation of Water Quality Models: A Management Guide for Planners*, Systems Control Inc. for U.S. Environmental Protection Agency, EPA 600/5-76-004, July 1976.

3. Brandstetter, Albin, *Assessment of Mathematical Models for Storm and Combined Sewer Management*, Battelle, Pacific Northwest Laboratories for U.S. Environmental Protection Agency, EPA-600/2-76-175a, July 1976.

4. Lager, John A., T. Didriksson, G.B. Otte, *Development and Application of a Simplified Stormwater Management Model*, Metcalf and Eddy, Inc., for U.S. Environmental Protection Agency, EPA-600/2-76-218, August 1976.

5. Hydroscience, Inc., and Consoer, Townsend and Associates, *City of Milwaukee, Wisconsin, Humboldt Avenue Pollution Abatement Demonstration Project, Appendix I - Description of Detention Tank Model*, September 1974.

CHAPTER 5

NUMERICAL ESTIMATES FOR STORMWATER ASSESSMENT METHODOLOGIES

A variety of parameters are required for the calculation of stormwater loads and impacts, by either the statistical method described in Chapter 3 or by simulators of the type discussed in Chapter 4 of this manual. These include site specific characterizations of rainfall, runoff quantity and quality, receiving water properties, and the performance and cost of treatment alternatives. Guidelines for estimating these factors are presented in this Chapter. While these guidelines are useful for initial assessments and as a means of comparison, they are no substitute for site specific information. Methods for obtaining site specific information through a local monitoring program are described in Chapter 6 of this manual.

5.1 Rainfall

A method for statistically characterizing rainfall is discussed in Section 3.3. The following sections describe considerations for the use and application of this method. Examples of precipitation characteristics in different parts of the United States are presented. Various procedures for defining the end of a storm event are examined. This is necessary for both the statistical method and simulators. An examination of the applicability of the gamma distribution for storm properties is included and methods are presented for addressing the areal distribution of rainfall.

5.1.1 Precipitation Characteristics in Different Parts of the United States

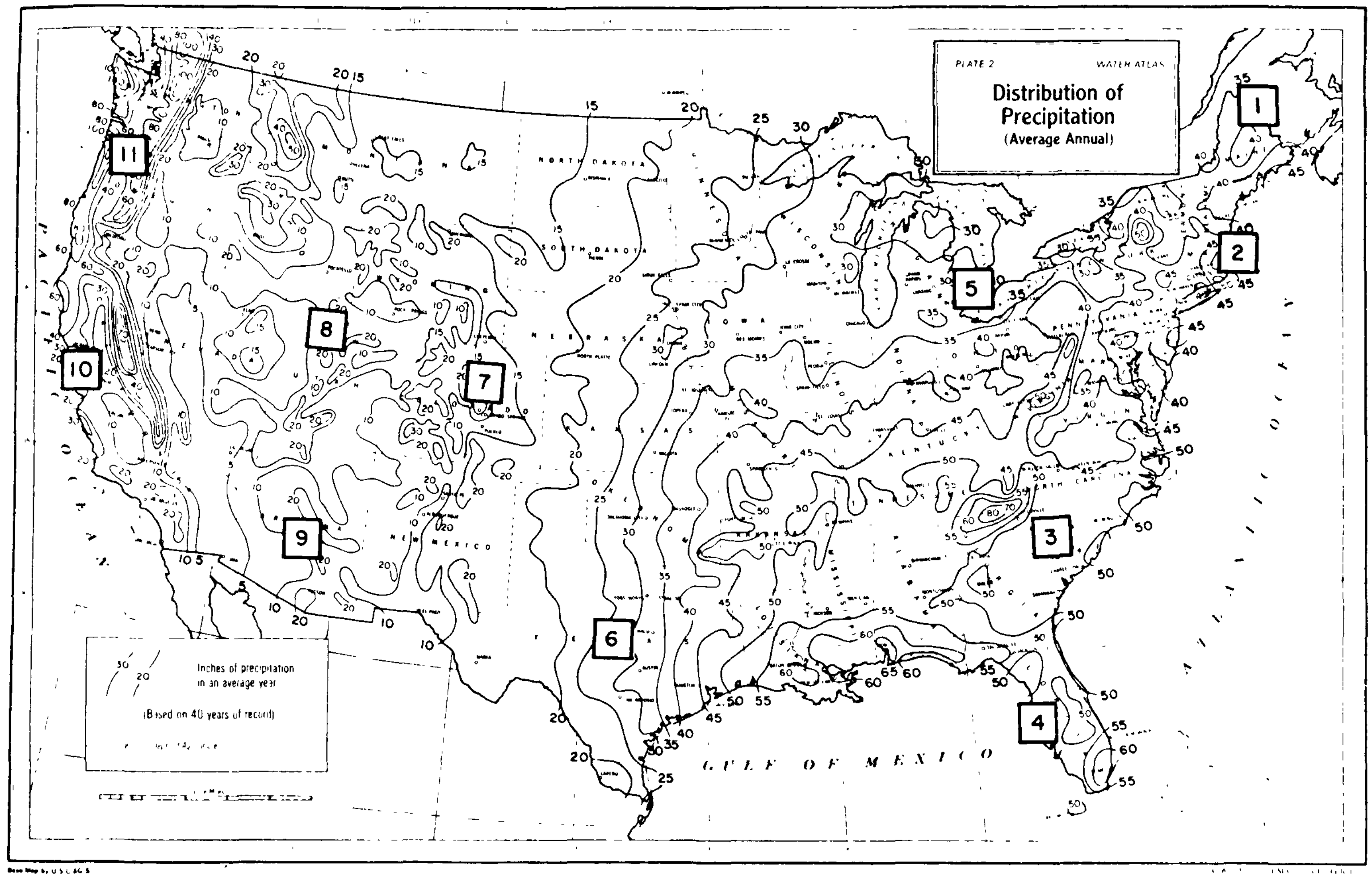
Rainfall properties vary markedly from one part of the United States to another. This section presents the results of analyses of raingage data from eleven cities in the coterminous United States. The purpose of this section is simply to indicate the general magnitude and range of storm characteristics which occur in different areas. It does not replace the need to obtain and analyze raingage data in a specific study location.

The eleven cities selected are located in major geographical subdivisions of the country, as indicated in Table 5-1 and in Figure 5-1, which also shows the distribution of average annual rainfall. The study cities can be ordered in terms of increasing average annual precipitation: Pheonix (10 in), Salt Lake City (15 in), Denver (15 in), Oakland (20 in), Detroit (30 in), Dallas (35 in), Caribou (35 in), Portland (40 in), Boston (45 in), Columbia (50 in), and Tampa (50 in). For a given year, however, the total precipitation may vary considerably from the long term average. This is demonstrated in Figures 5-2 (a, b, c, d).

TABLE 5-1

CITIES SELECTED FOR RAINGAGE ANALYSIS

1.	Caribou, Maine	- Northeast
2.	Boston, Massachusetts	- Northeast
3.	Columbia, South Carolina	- Southeast
4.	Tampa, Florida	- Southeast
5.	Detroit, Michigan	- Midwest
6.	Dallas, Texas	- South
7.	Denver, Colorado	- Rocky Mountain
8.	Salt Lake City, Utah	- Rocky Mountain
9.	Pheonix, Arizona	- Southwest
10.	Oakland, California	- Far West
11.	Portland, Oregon	- Northwest



(LOCATION NUMBERS FROM TABLE 5-1)

FIGURE 5-1

DISTRIBUTION OF PRECIPITATION AND LOCATION STUDY CITIES

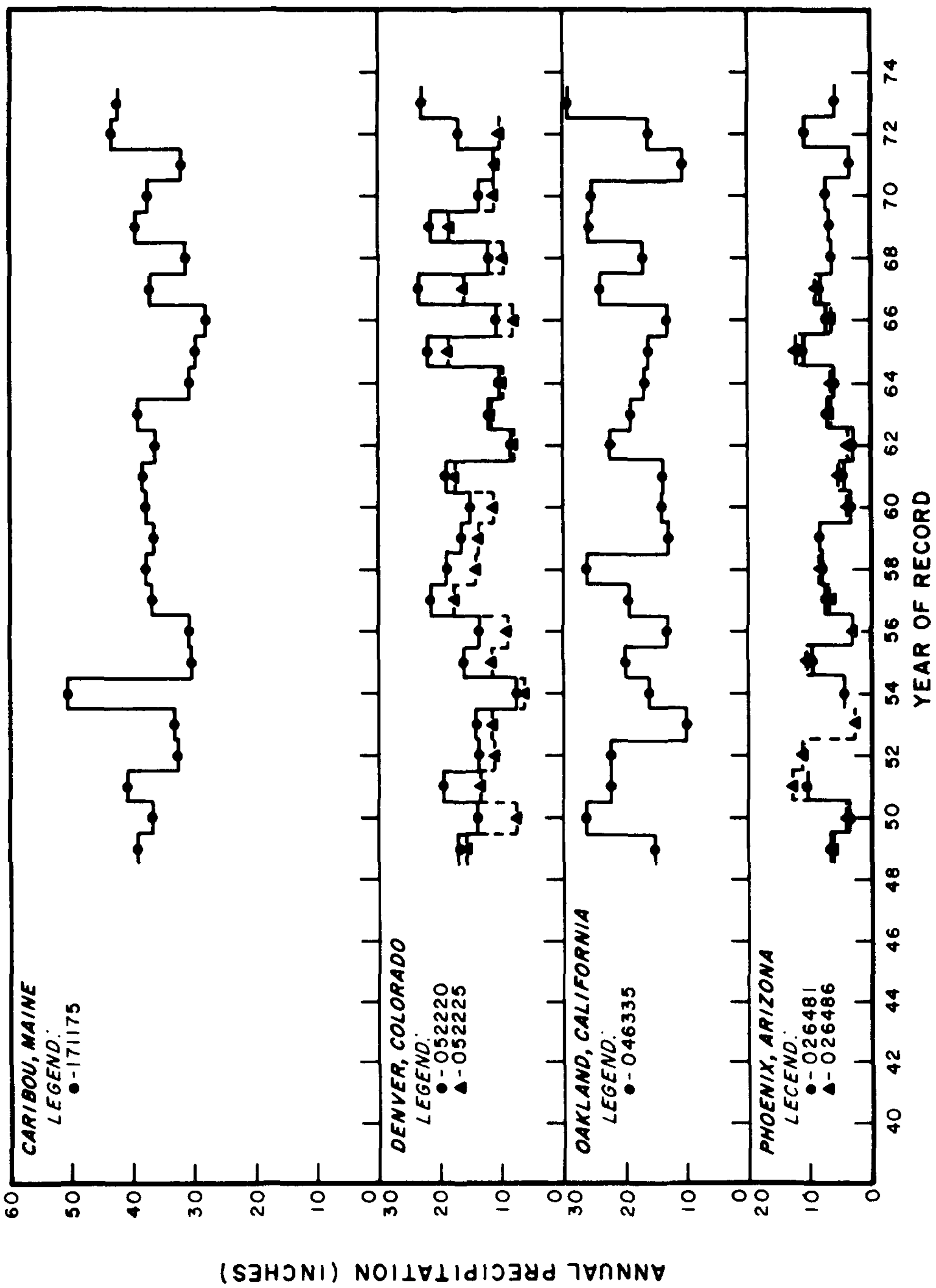


FIGURE 5-2 (a)
ANNUAL PRECIPITATION

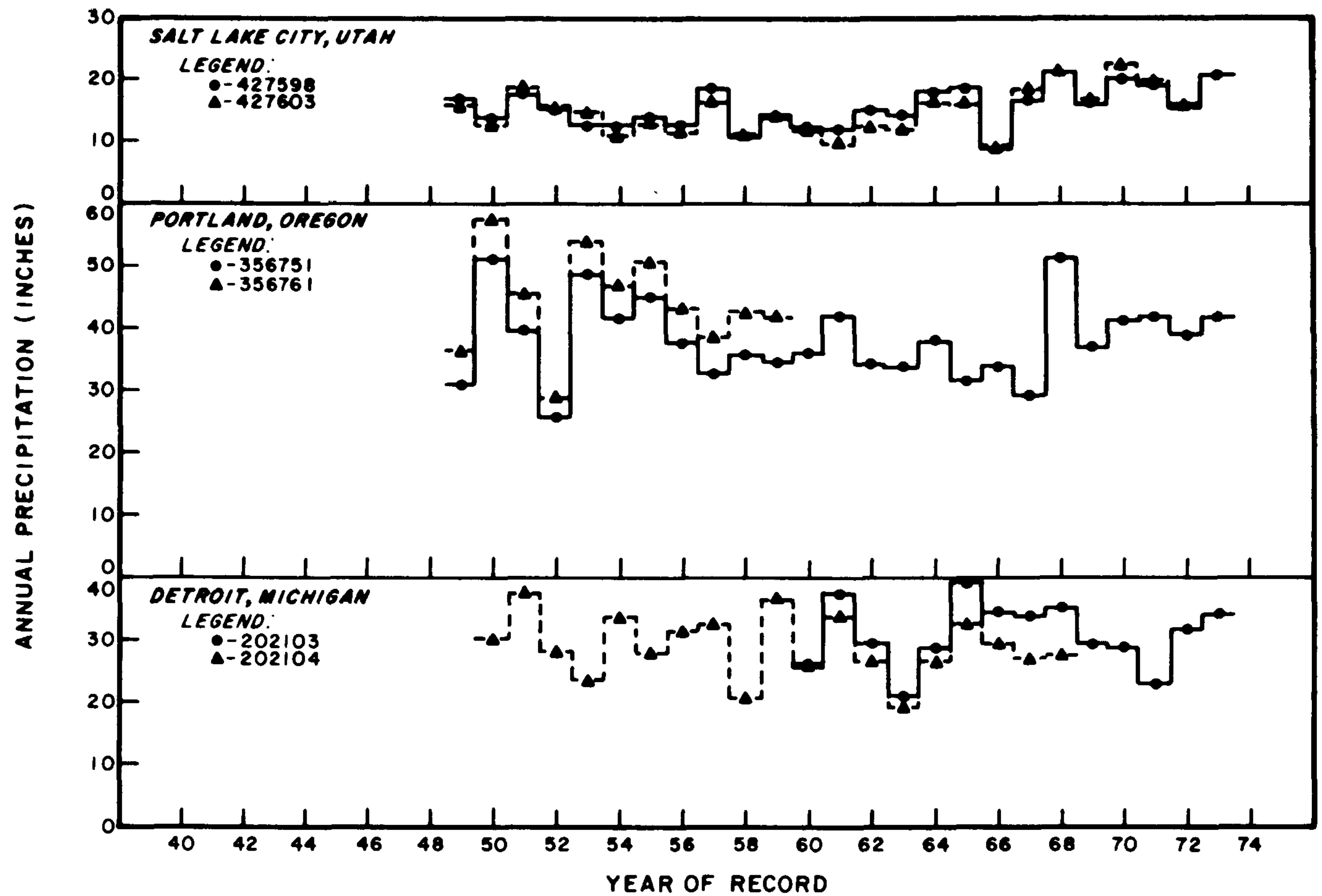


FIGURE 5-2 (b)
ANNUAL PRECIPITATION

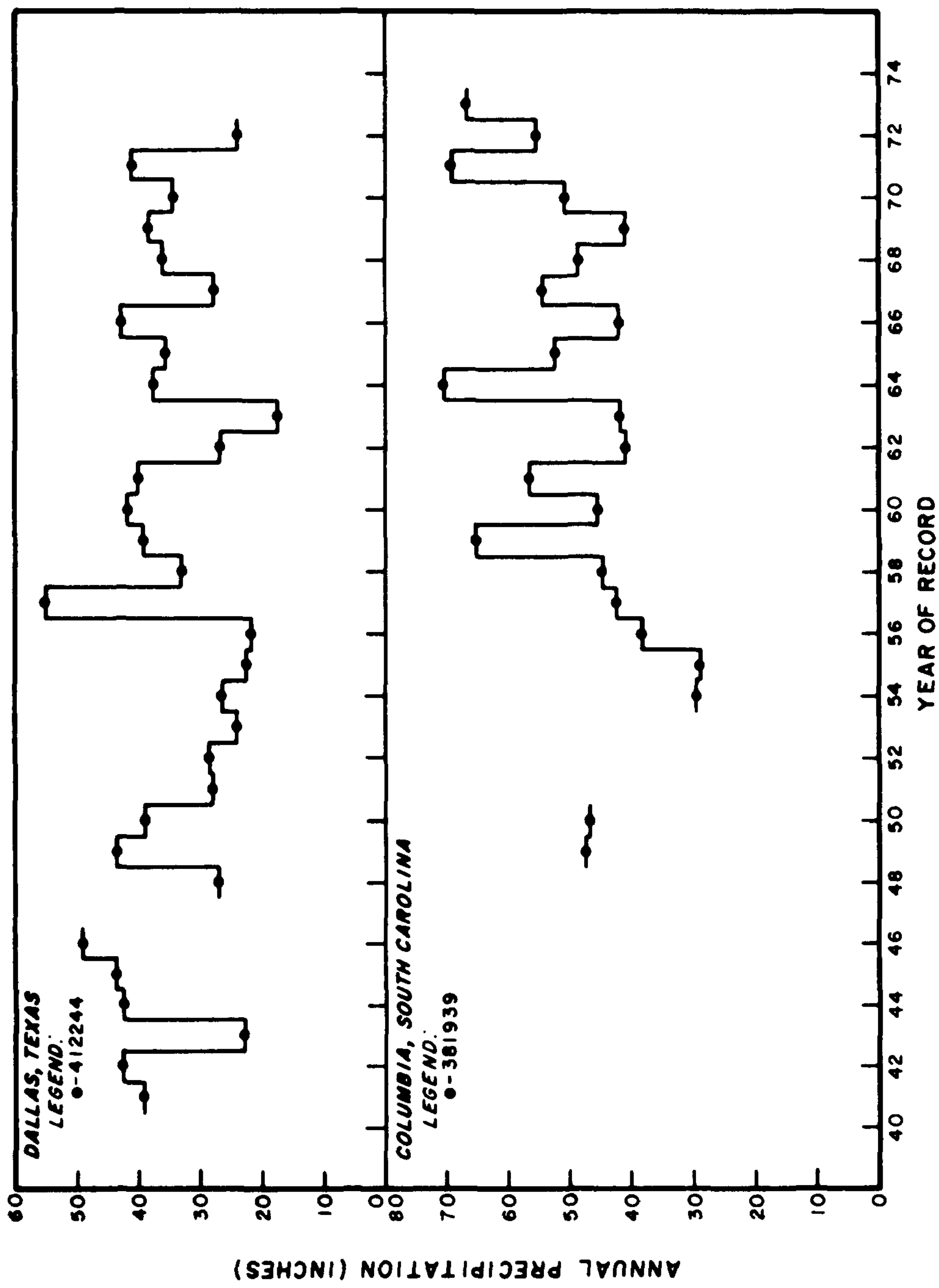


FIGURE 5-2 (c)
ANNUAL PRECIPITATION

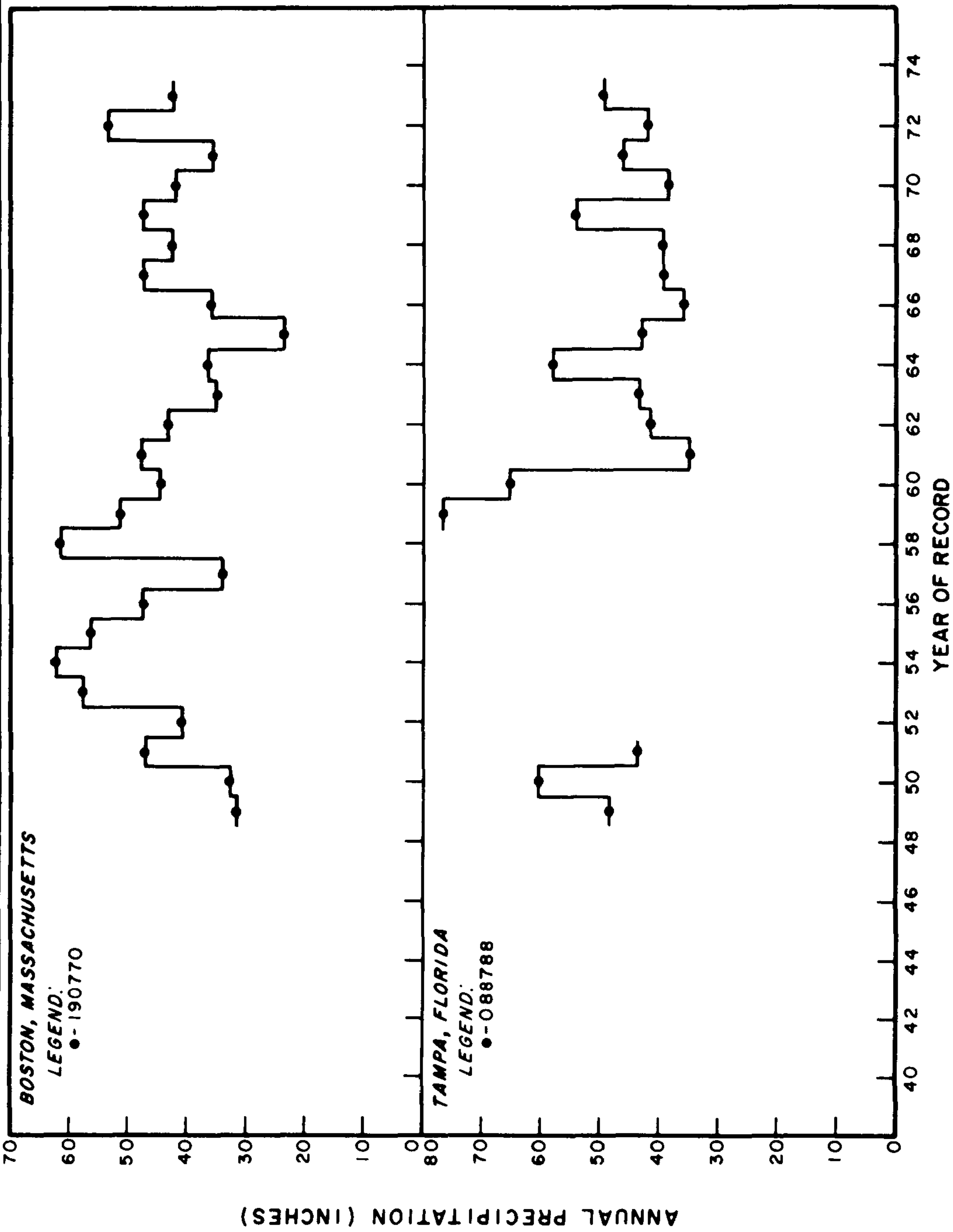


FIGURE 5-2 (d)
ANNUAL PRECIPITATION

For a few of the cities, the yearly rainfall is shown for two weather stations, and differences are evident. For example, at Denver more rain falls at the airport than at the city station. At Portland, this phenomenon is reversed. Differences may be due to orographic effects or locations relative to predominant storm paths in the area.

The synoptic analysis which separates the rainfall record into a series of discrete storm events is described in Section 3.3. The statistical properties of the four event characteristics (duration, volume (depth), average intensity, and storms interval) are determined. Seasonal patterns are identified by grouping storms according to month.

The data from the eleven cities presented below are all analyzed by requiring three consecutive dry hours to end a storm event. As discussed in the Section 5.1.2, various criteria may indicate that a different storm definition is more appropriate for certain cities. The statistical properties will change somewhat given a different storm definition, however, the basic trends and seasonal patterns will remain basically unchanged.

The results of the monthly rainfall characterization are shown in Figures 5-3 (a-k). A number of patterns are evident from the plots. The mean event duration decreases during the summer months at each of the eleven cities while the average intensity increases everywhere during the summer except Oakland and Portland. The highest summer storm intensities occur in Columbia and Tampa, followed by Dallas, Detroit, and Pheonix.

Longer storms tend to occur in the Northeastern and Northwestern portions of the United States with mean durations of from 4 to 8 hours in Boston, Caribou and Portland. The average duration in the rest of the country generally varies from 2 to 6 hours. The cities with the highest mean precipitation depth per storm are Columbia, Tampa, Dallas and Boston where the average volume is generally greater than 0.25 inches throughout the year.

Considerable geographic differences are indicated in the seasonal patterns of the mean time between storms. The most frequent storms occur in Portland during the non-summer months where the mean time between storms (Δ) ranges from 30 to 40 hours. The most consistently frequent events occur in Caribou where Δ ranges from 40 to 60 hours throughout the year. The least frequent storms occur in Oakland and Phoenix during the summer, where Δ is very high. Note the different plotting scale used for Δ in these cities. Events occur much more frequently during the winter in Oakland, Phoenix, Salt Lake City, and Portland; while more events occur during the summer in Tampa and Denver.

The coefficient of variation of the time between storms (ν_ε) is generally near one. For those cities where ν_δ is significantly greater than one, in particular, in the western portion of the country, the rainfall analysis should also be examined with a minimum dry interval that defines a storm of greater than 3 hours. The coefficient of variation of intensities (ν_i) generally ranges from 0.8 to 1.6, though ν_i is somewhat higher in Columbia, and lower in Oakland during the summer. The coefficient of

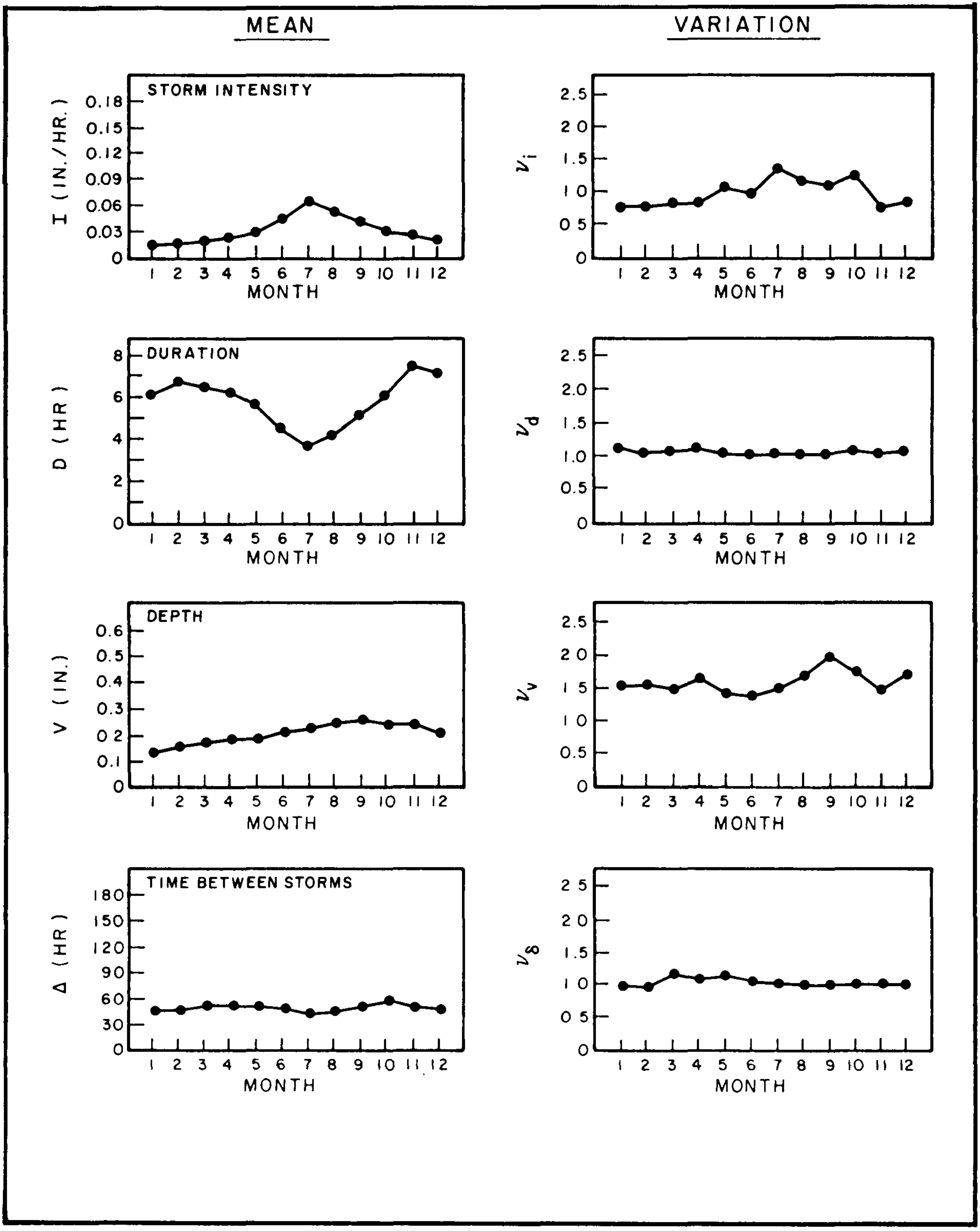


FIGURE 5-3 (a)

MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
CARIBOU, MAINE
STATION 171175, (1948-1973)

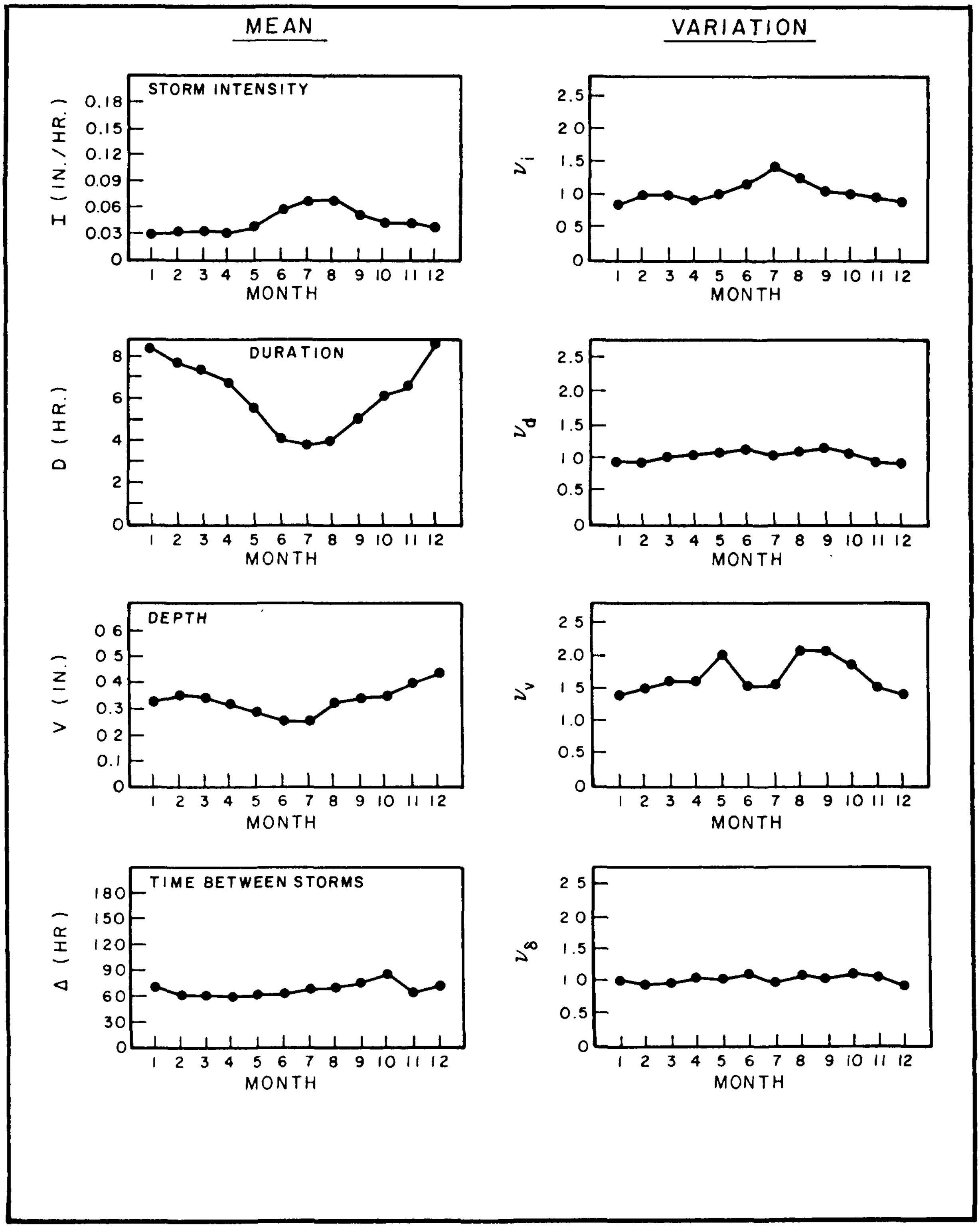


FIGURE 5-3 (b)

MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
BOSTON, MASSACHUSETTS
STATION 190770, (1948-1973)

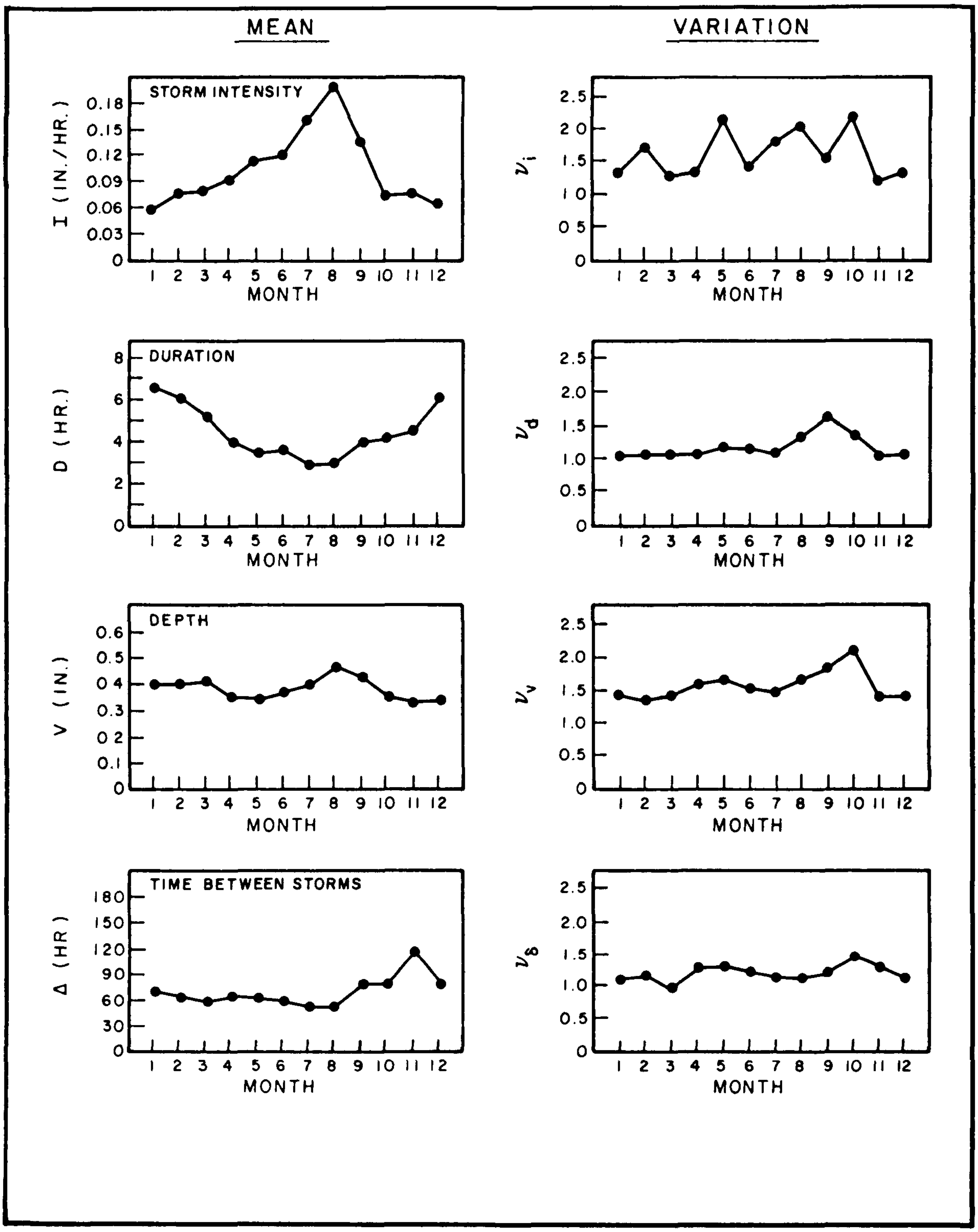


FIGURE 5-3 (c)
MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
COLUMBIA, SOUTH CAROLINA
STATION 381939, (1948-50, 54-73)

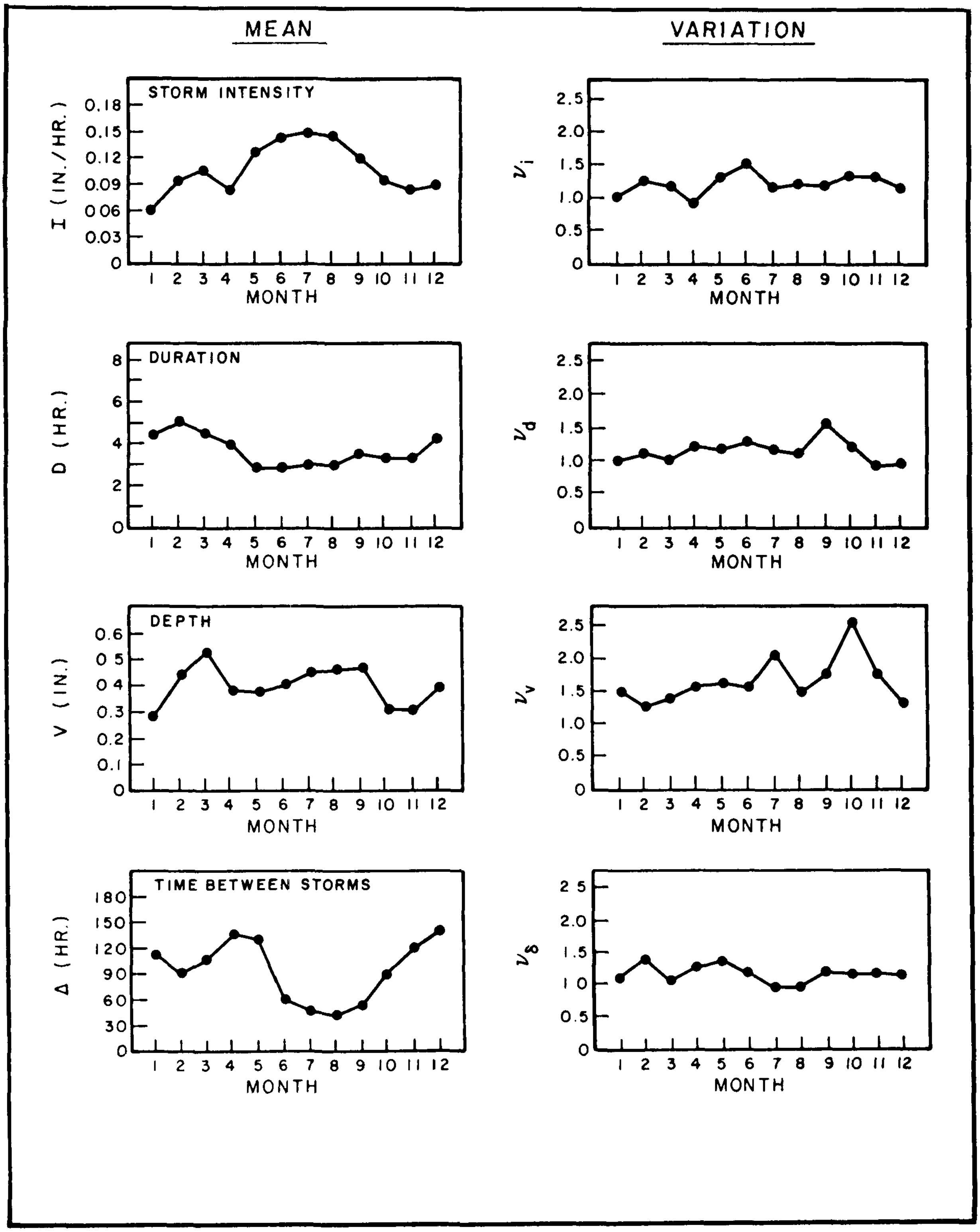


FIGURE 5-3 (d)
MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
TAMPA, FLORIDA
STATION 088788, (1948-51, 59-73)

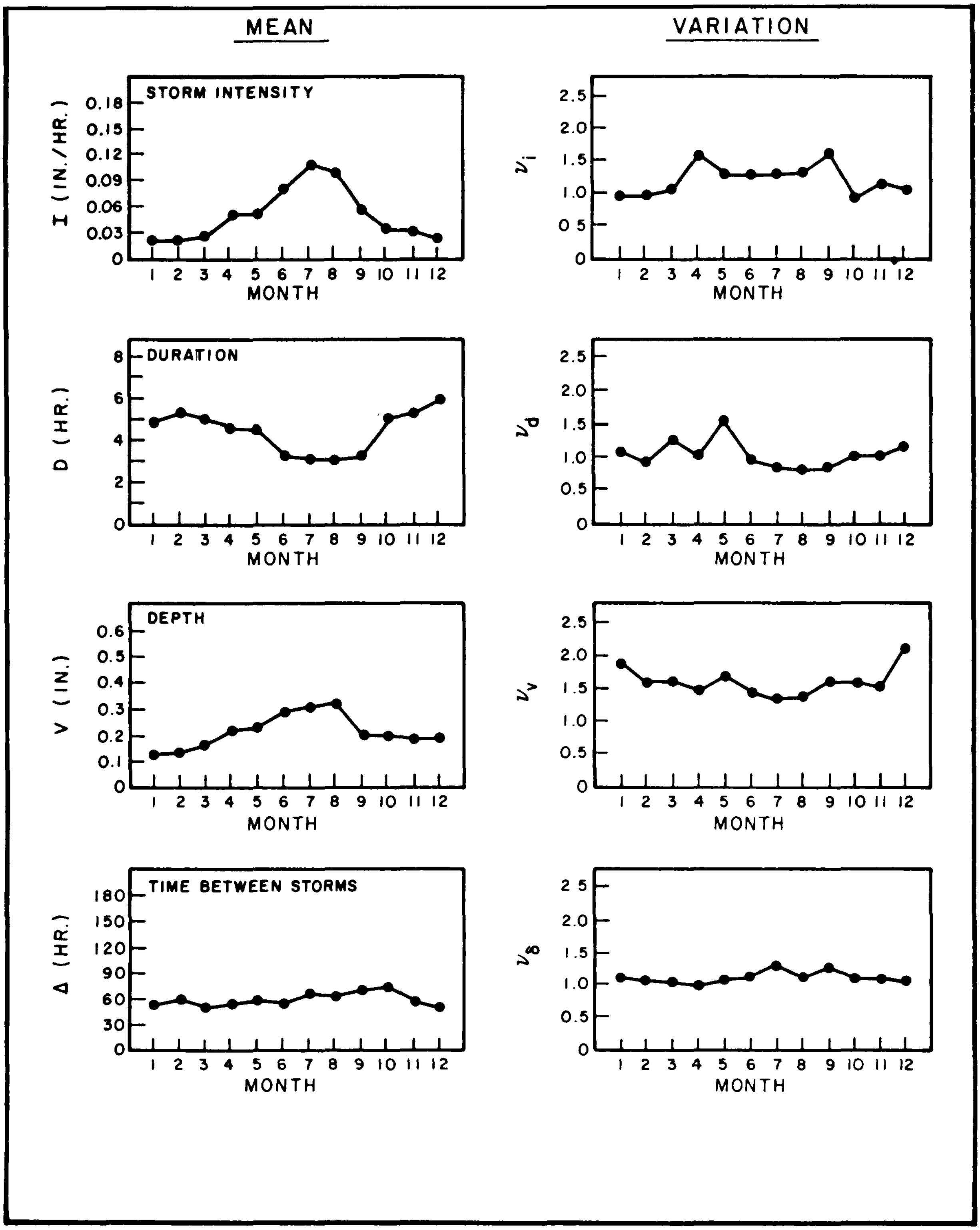


FIGURE 5-3 (e)
MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
DETROIT, MICHIGAN
STATION 202103, (1960-1973)

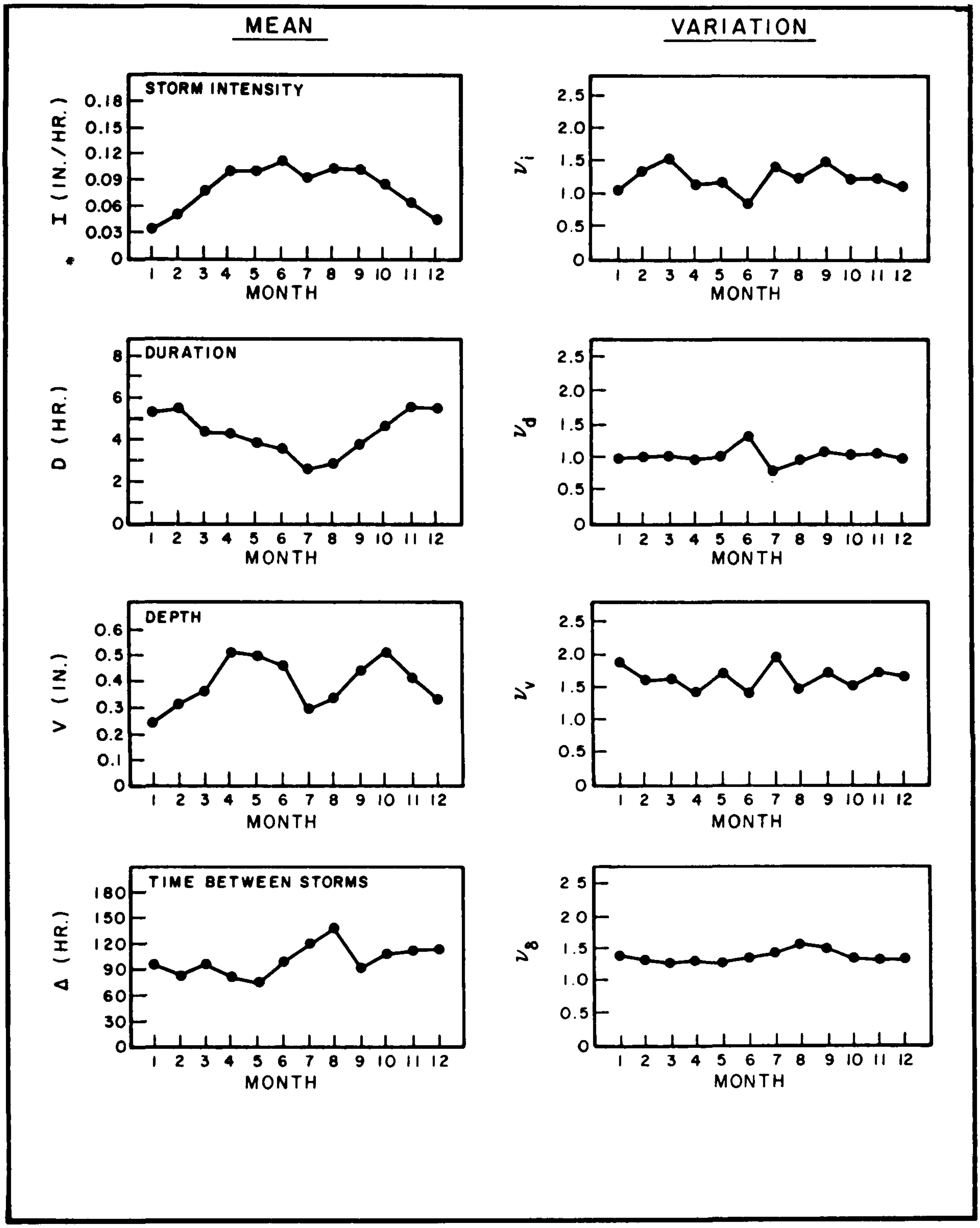


FIGURE 5-3 (f)
MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
DALLAS, TEXAS
STATION 412244, (1941-46, 48-73)

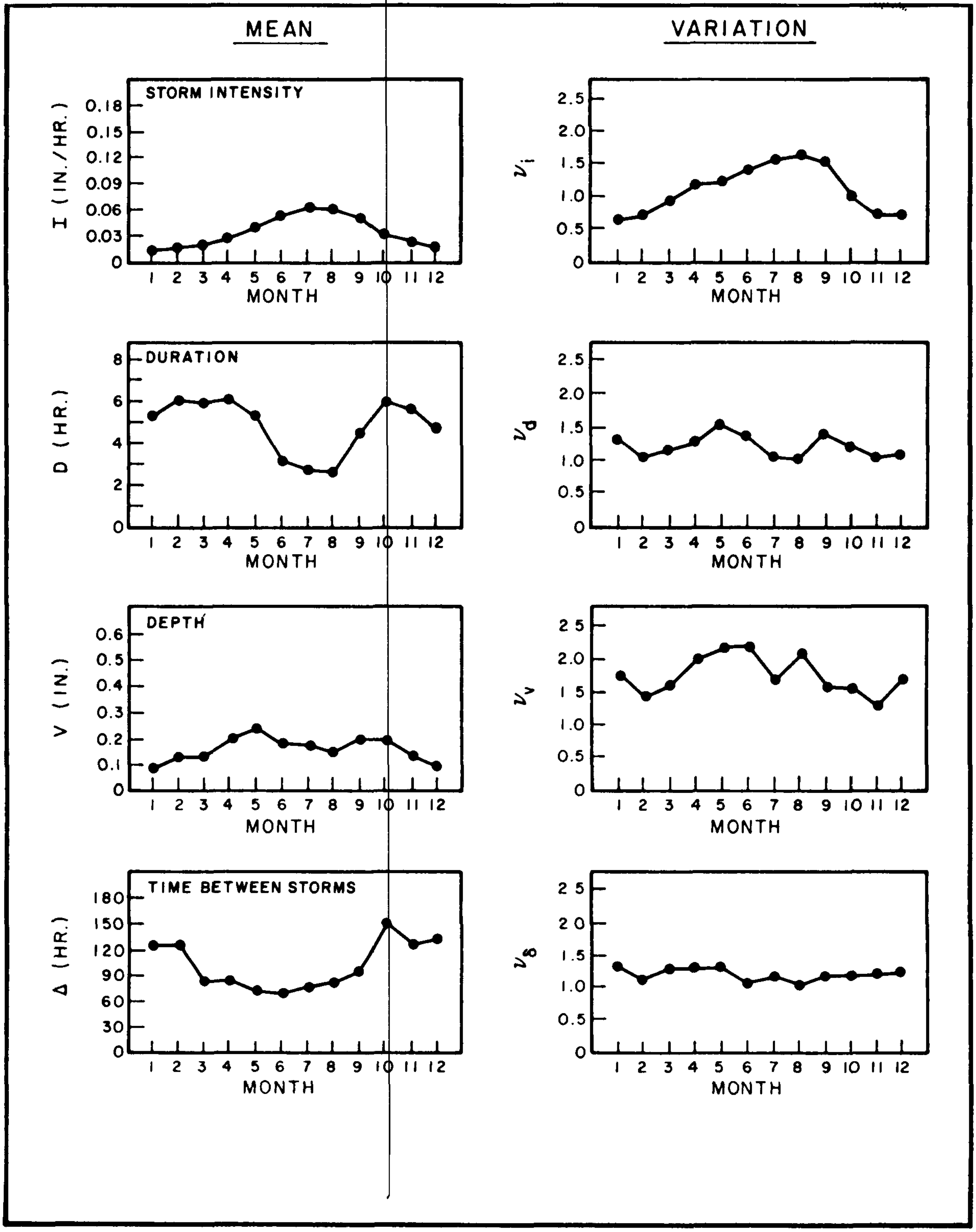


FIGURE 5-3 (g)

MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
DENVER, COLORADO
STATION 052220, (1948-1973)

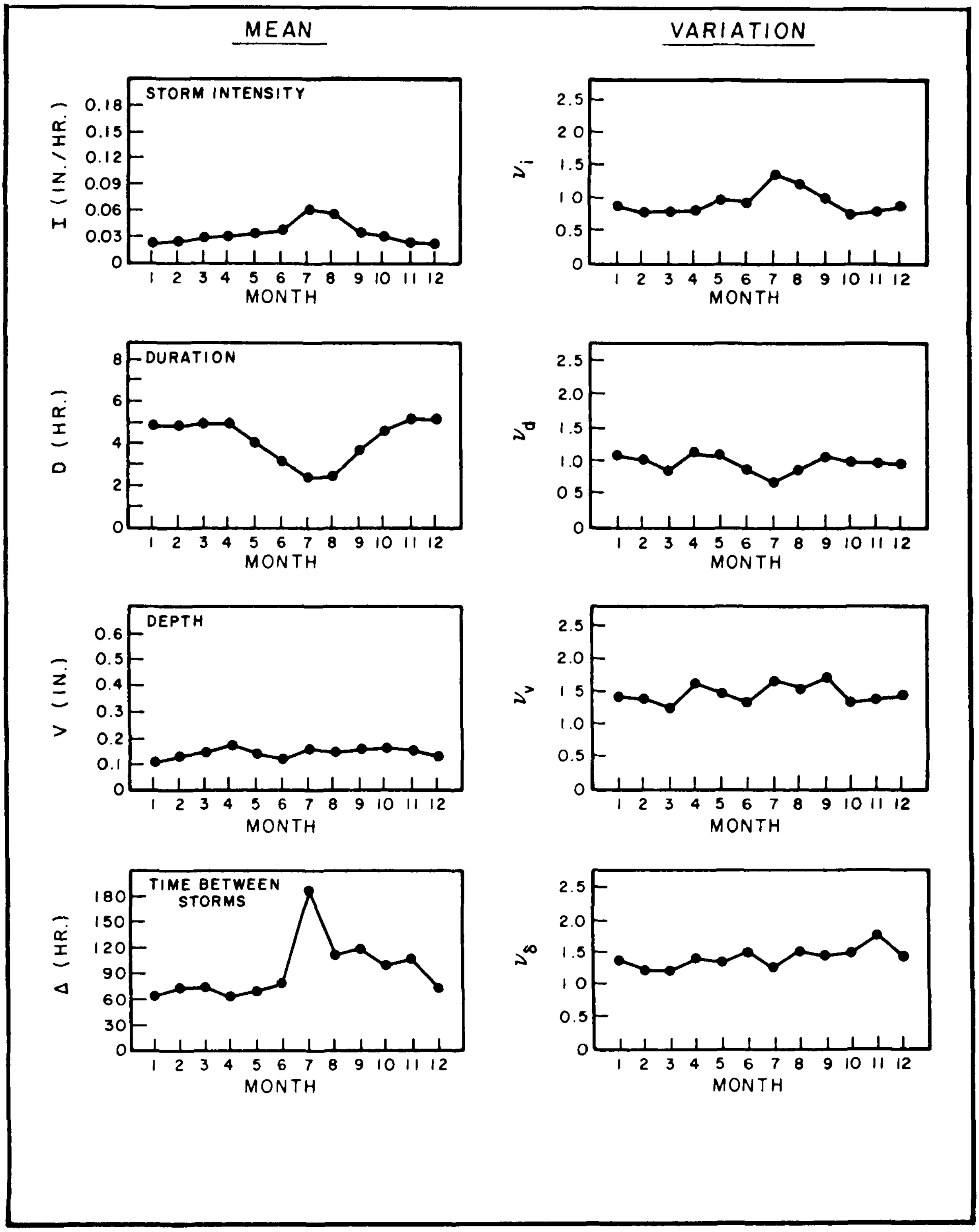


FIGURE 5-3 (h)
MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
SALT LAKE CITY, UTAH
STATION 427598, (1948-1973)

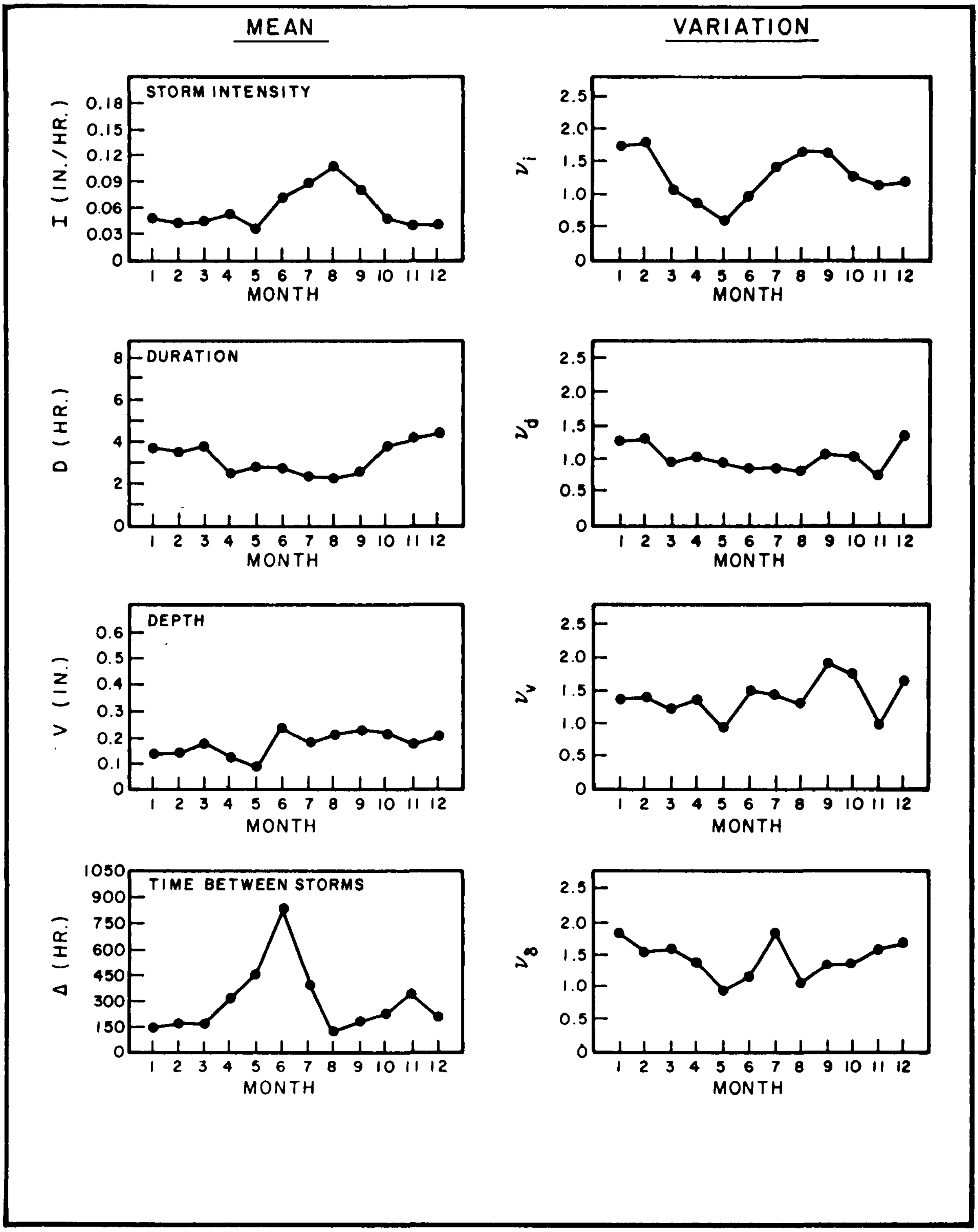


FIGURE 5-3 (i)
MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
PHEONIX, ARIZONA
STATION 026481, (1948-1973)

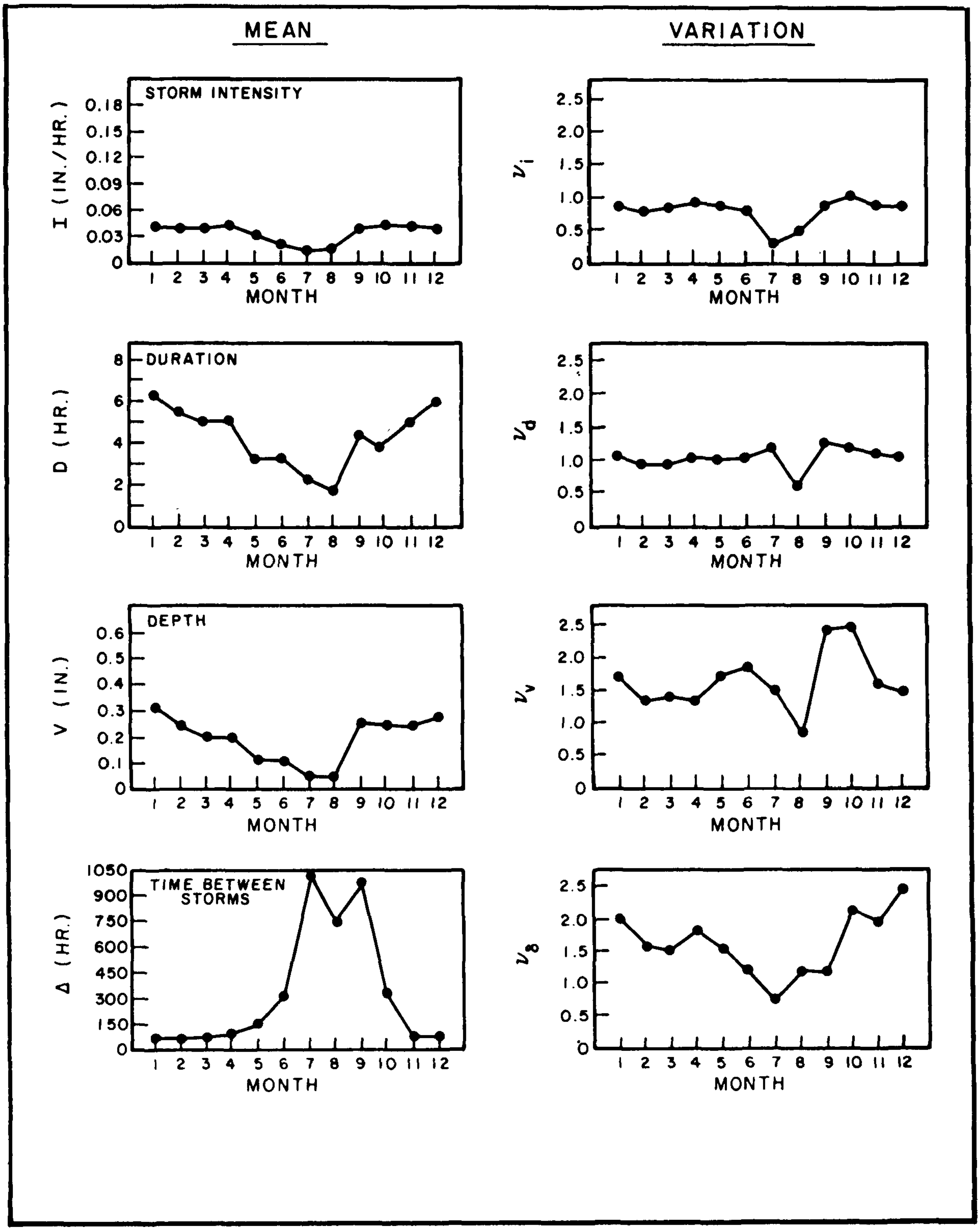


FIGURE 5-3 (j)

MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
OAKLAND, CALIFORNIA
STATION 046335, (1948-1973)

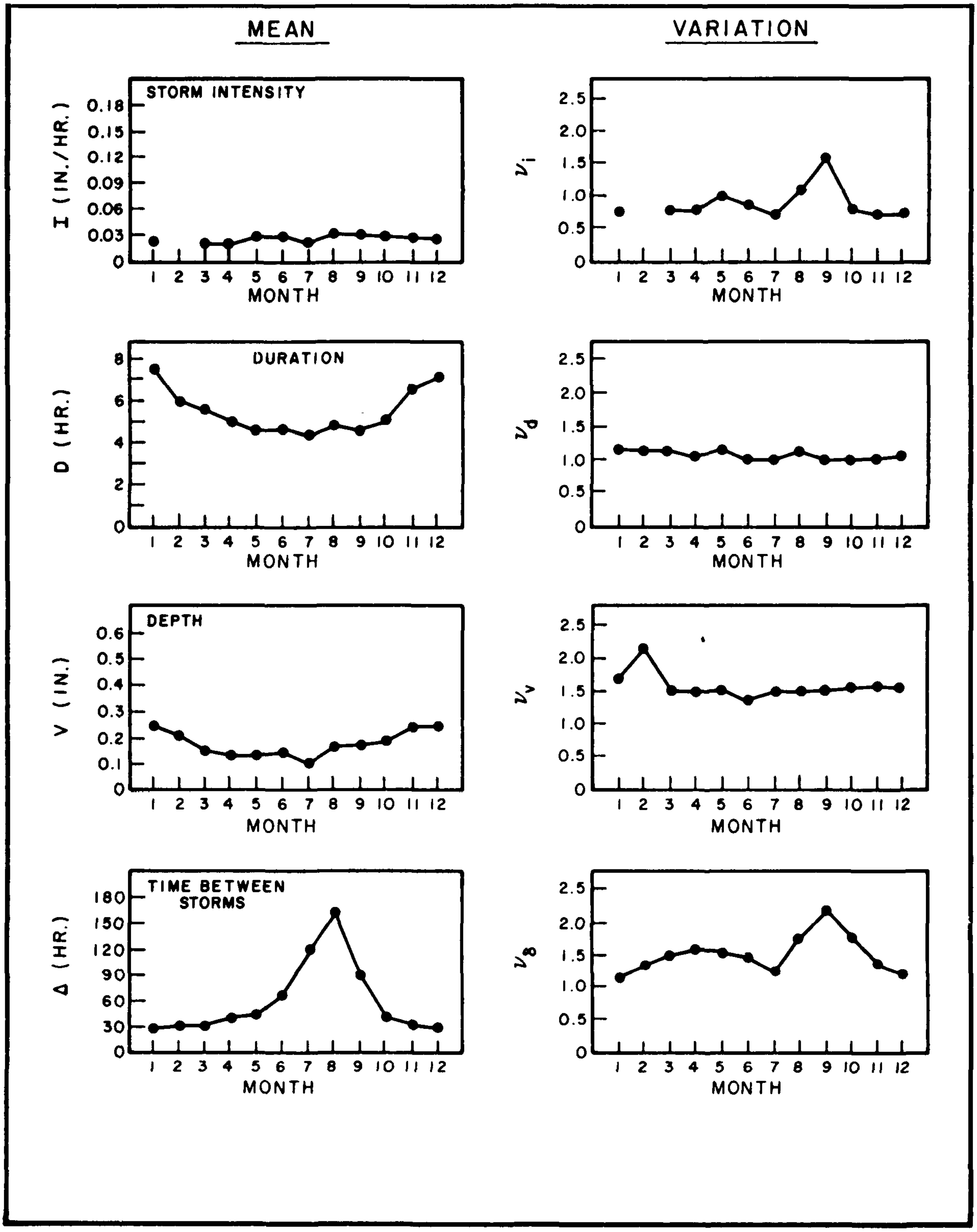


FIGURE 5-3 (k)

MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
PORTLAND, OREGON
STATION 356751, (1948-1973)

variation of durations (ν_d) is generally near one while the coefficient of variation of unit volumes (ν_v) ranges from 1.0 to 2.4.

5.1.2 Definition of Storm Event

The grouping of hourly rainfall records into discrete events requires a definition of the end of a storm. This is conveniently specified by some minimum number of consecutive dry hours that marks the end of a storm event. A number of procedures for selecting the minimum inter-event time have been proposed.

Heaney, et al (1) and Howard (2) suggest an autocorrelation analysis of the hourly rainfall records to determine the time lag at which the hourly rainfall records become uncorrelated. An example of this type of analysis is presented in Figure 5-4. The autocorrelation coefficient of hourly rainfall is shown as a function of the lag time. At a lag time of zero the autocorrelation equals one, as each rainfall hour is completely correlated with itself. The other points on the curve indicate the correlation of data from a given hour with data of another hour at the indicated interval. The 95 percent tolerance limits for significant nonzero correlation are also shown. The first minimum in the autocorrelation function occurs at 10 hours, although insignificant correlation (at the 95 percent probability level) is reached at about 7 hours. This is then used to indicate the appropriate interevent time to separate the rainfall record into independent events. Heaney et al (1) and Howard (2) suggest using the first minimum (in this case, 10 hours), although the point where the autocorrelation is no longer statistically significant may also be considered.

Although this method is properly directed towards defining when storm events may be considered independent, there are a number of difficulties in the approach. For example, the appropriate tolerance limits for considering records uncorrelated must be chosen. More fundamentally, however, the correlation procedure does not limit itself to the correlation of rainfall records which are separated by a given number of consecutive dry hours. Since the analysis is made for the entire rainfall record, the autocorrelation coefficient includes correlation within storms as well as between storms. For example, during a nine hour period of continuous rainfall, there are five occurrences of rainfall at a given hour and its four hour lag. These occurrences inflate the autocorrelation coefficient for short lag times, while indicating nothing about the independence of data after a given number of consecutive dry hours. A more appropriate analysis would isolate those periods separated by a given number of dry hours, and check for correlation between these.

Another approach is to plot the number of events per year versus the minimum inter-event time. The point at which an increase in the minimum inter-event time does not result in a correspondingly significant reduction in the number of storm events is considered adequate. As indicated by Heaney, et al (1), however, this graphical approach may not always indicate a well-defined transition point.

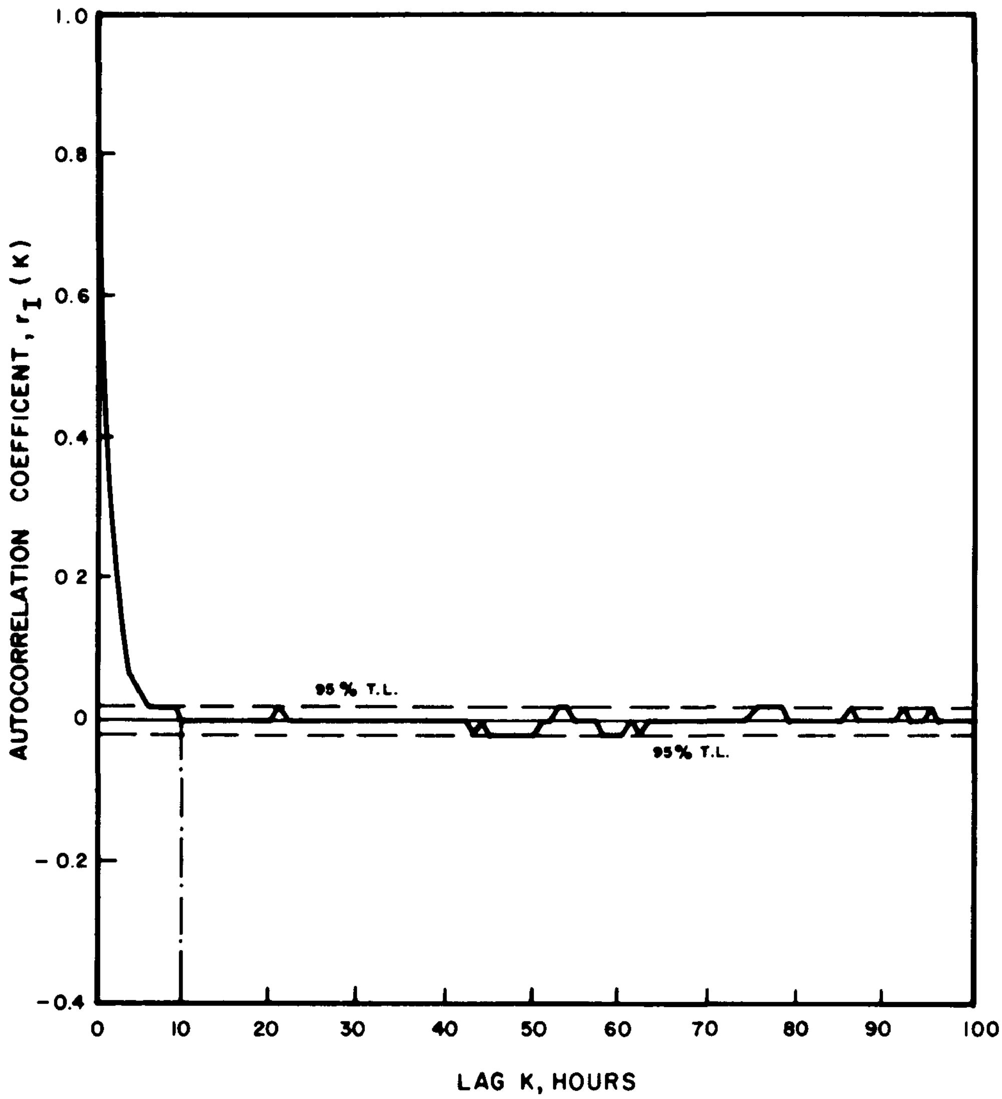


FIGURE 5-4

LAG-K AUTOCORRELATION FUNCTION OF DES MOINES, IOWA HOURLY RAINFALL, 1968

A third approach has been examined in the development of this manual. The approach is based upon the characterization of the rainfall process as a random, Poisson process, as described in Section 3.1. If storm events occur in this fashion, there are a number of implications. One is that the time between events (δ) is an exponentially distributed random variable (equivalent to a gamma distributed variable with coefficient of variation (ν_δ) equal to one). Grouping the storm events (i.e. selecting the number of dry hours) such that this property holds, provides a useful criteria for selecting the minimum inter-event time.

It is observed that when the statistical rainfall analysis is performed with a short minimum inter-event time, the coefficient of variation (ν_δ) is greater than one. As the minimum inter-event time is increased, ν_δ decreases and eventually becomes less than one. This is demonstrated in Table 5-2 using rainfall records from Newark, New Jersey. Note that greater inter-event times also result in fewer storms, greater average durations (D), greater average depths (V), and a greater average time between storms (Δ). The important point, however, is that at an inter-event time of about 4 hours, ν_δ is equal to one.

To demonstrate that δ is exponentially distributed with the minimum 4 dry hour storm definition, the cumulative density function for δ is plotted in Figures 5-5 (a, b, and c) with a 3, 4 and 6 dry hour storm definition respectively. The gamma distribution fits the observed data quite well, and in particular, the case when $\nu_\delta = 1.02$ (minimum inter-event period of 4 hours) closely corresponds to the special case of the gamma distribution when $\nu = 1$, which is the exponential distribution. A minimum inter-event time of 4 hours is chosen for the storm definition. A similar analysis was conducted for the Minneapolis-St. Paul raingage and a minimum inter-event time of 6 hours was found to yield ν_δ nearly equal to one. The cumulative density function for δ given this definition is shown in Figure 5-6.

It is important to note that while the guideline suggested for choosing the storm definition is useful, it is by no means absolute. While a random storm process will have δ exponentially distributed, this does not guarantee that the events occur randomly. It is felt, however, that barring marked seasonal changes in δ, the time between two given storms should be independent of the time between any other two storms. Seasonal patterns should be noted and the storm definition may be chosen to have ν_δ nearly equal to one during a particular critical or design period. Finally, the effects of alternative storm definition should be included as part of the sensitivity analysis in the overall assessment. For example, the differences in mean unit storm volume (V) indicated in Table 5-2 may change the predicted performance of different size storage units in the Newark area.

5.1.3 Applicability of the Gamma Distribution for Storm Event Characteristics

The two parameter gamma distribution has been used in this manual for the probabilistic analysis of rainfall events and their impacts, with little discussion or background as to why it was selected. This section presents a more detailed presentation of the applicability of the gamma distribution

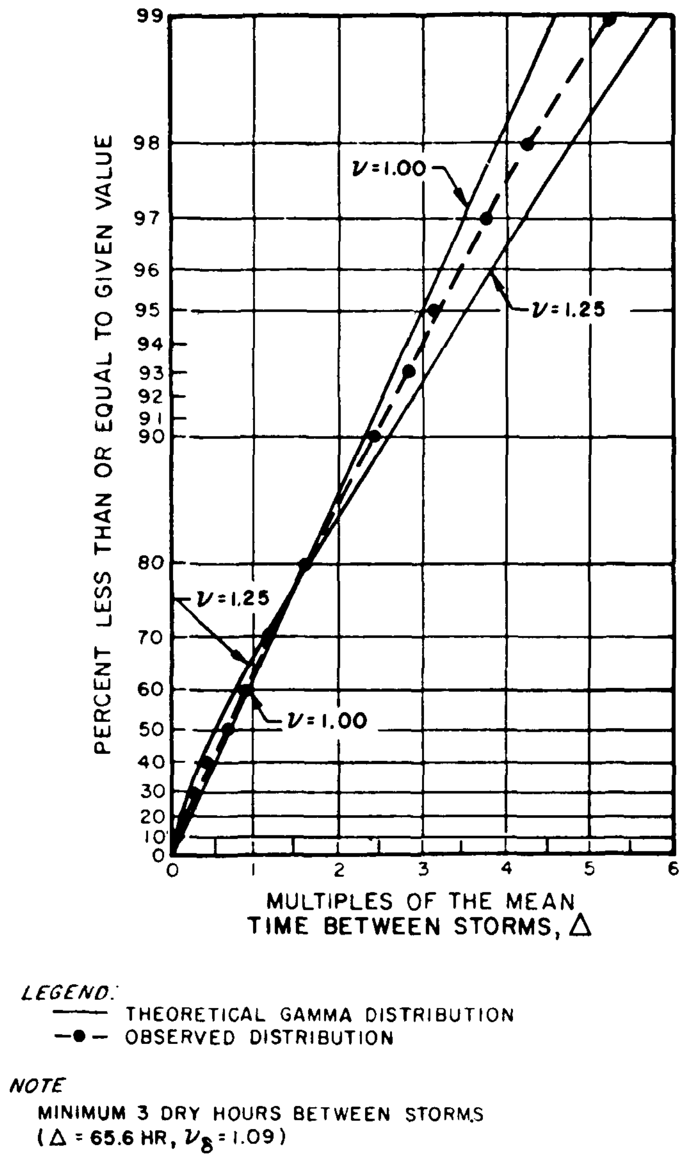


FIGURE 5-5 (a)
COMPARISON OF OBSERVED AND THEORETICAL
CUMULATIVE DISTRIBUTION OF TIME BETWEEN STORMS
NEWARK AIRPORT, RAINGAGE 286026, 1948-1975

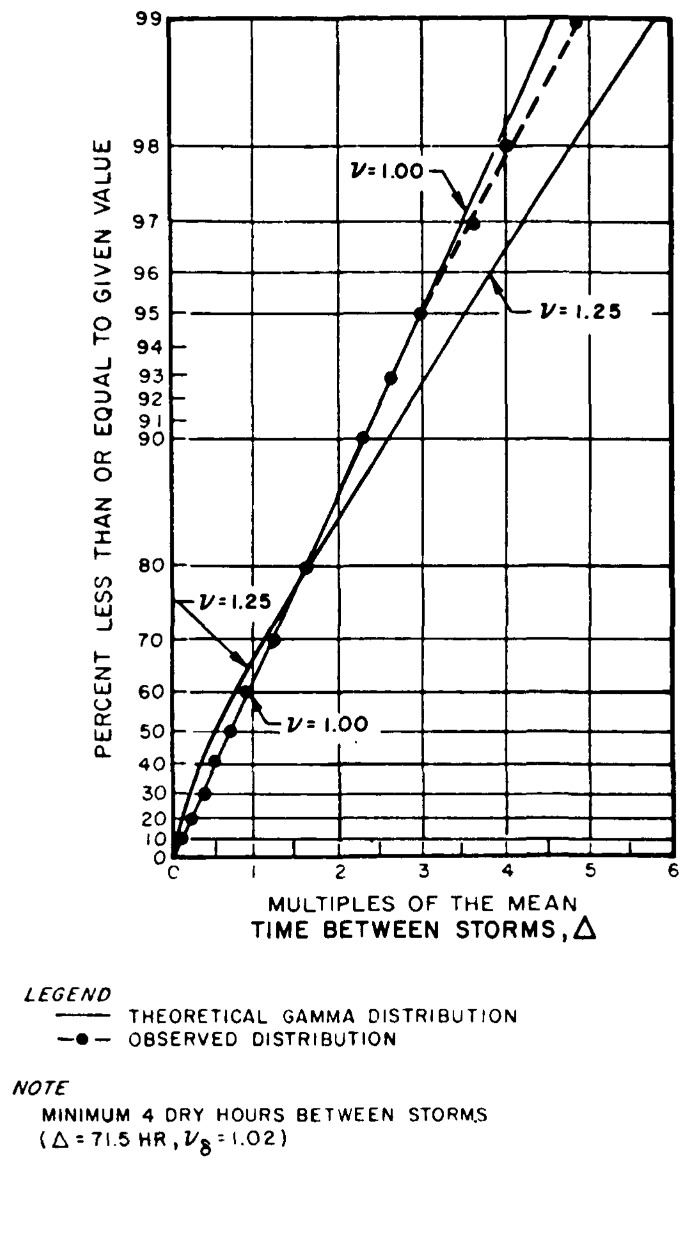
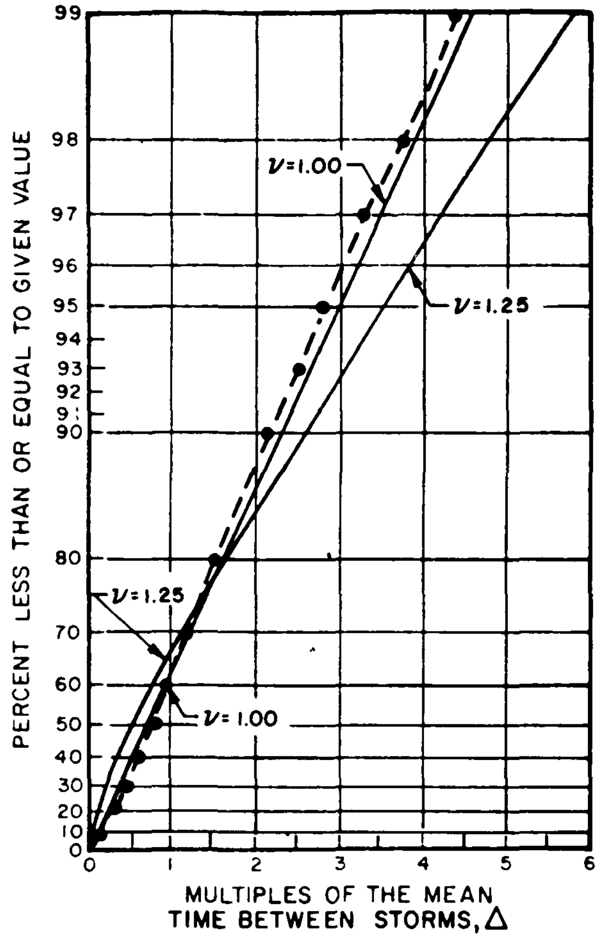


FIGURE 5-5 (b)

COMPARISON OF OBSERVED AND THEORETICAL CUMULATIVE DISTRIBUTION OF TIME BETWEEN STORMS NEWARK AIRPORT, RAINGAGE 286026, 1948-1975



LEGEND.
—— THEORETICAL GAMMA DISTRIBUTION
—•— OBSERVED DISTRIBUTION

NOTE
MINIMUM 6 DRY HOURS BETWEEN STORMS
(Δ = 80.8 HR, ν_δ = 0.91)

FIGURE 5-5 (c)
COMPARISON OF OBSERVED AND THEORETICAL CUMULATIVE DISTRIBUTION OF TIME BETWEEN STORMS NEWARK AIRPORT, RAINGAGE 286026, 1948 – 1975

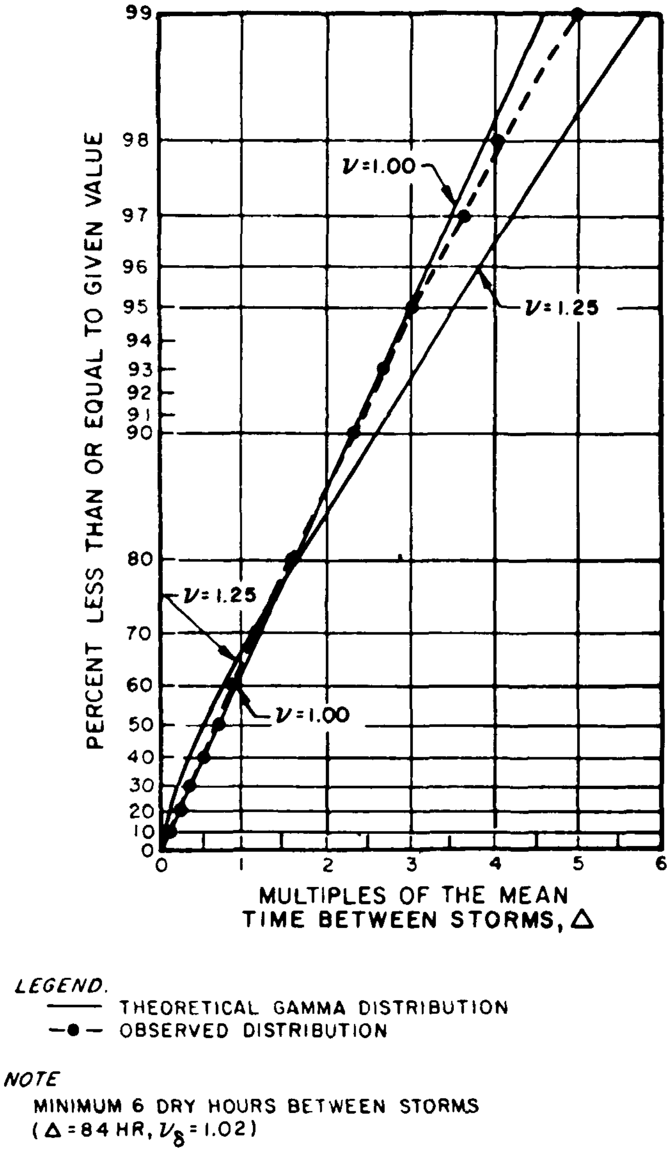


FIGURE 5-6

COMPARISON OF OBSERVED AND THEORETICAL
CUMULATIVE DISTRIBUTION OF TIME BETWEEN STORMS
MINNEAPOLIS/ST. PAUL AIRPORT, RAINGAGE 215435, 1948-1973

TABLE 5-2

EFFECT OF ALTERNATIVE STORM DEFINITION
RAINGAGE 286026, NEWARK AIRPORT, MAY 1948-DECEMBER 1975 (27.58 yr)

Minimum Dry Interval Between Storms (hr)	No. Storms	No. Storms Per Year	Duration D (hr)	ν_d	Intensity I (in/hr)	ν_i	Volume V (in)	ν_v	Time Between Storms Δ (hr)	ν_δ
3	3680	133	5.50	1.09	0.051	1.37	0.313	1.62	65.6	1.09
4	3378	122	6.26	1.09	0.051	1.32	0.341	1.59	71.5	1.02
5	3157	114	6.98	1.07	0.051	1.33	0.364	1.53	76.5	0.96
6	2991	108	7.65	1.07	0.051	1.33	0.385	1.50	80.8	0.91
7	2886	105	8.14	1.05	0.051	1.34	0.399	1.47	83.7	0.88

for storm event characterization and its use in the statistical method for the assessment of runoff and treatment.

The distributions of rainfall event characteristics (duration, unit volume, average intensity, and time between storms) are quite skewed. There are a few very large observations, with many relatively smaller ones. Distributions which are commonly used to represent such highly skewed random variables include the lognormal, Weibull, exponential, and gamma probability density functions.

The gamma distribution has traditionally found application in the analysis of precipitation data (3,4). Howard bases his analysis of stormwater storage and treatment on the assumption that event durations, intensities, and the time between storms are exponentially distributed, that is, a special case of the gamma distribution with the coefficient of variation equal to one (2). Chow and Yen use the gamma distribution to describe the basic rainfall event characteristics (5,6).

The Weibull distribution is put forth by Eagleson for the time between independent storm events and event durations; and the gamma distribution is suggested for storm depths (unit volumes) (7). Note that the Weibull distribution for the special case when the coefficient of variation (ν) equals one, is also equivalent to the exponential distribution and that the gamma and Weibull distributions are very similar when ν is nearly equal to one, as is usually the case for the time between storms and event durations. Smith has recently indicated that the lognormal distribution may also be appropriate for rainfall analysis in some cases (8).

To illustrate comparisons of observed cumulative density functions and the gamma distribution chosen for use in this manual, the hourly rainfall data from Central Park, New York, and the Minneapolis-St. Paul Airport are analyzed. The resulting empirical distributions are shown in Figures 5-7 through 5-14. Note that the observed distributions are well represented by the theoretical gamma curves in all cases except for the intensity at the Minneapolis-St. Paul raingage, where a lognormal distribution results in a better fit. This type of comparison should be conducted for at least one gage in a particular study area, to check the applicability of the basic probabilistic assumptions. Note also that the comparisons shown in Figures 5-7 through 5-14 accentuate the less frequent end of the distribution function, as most of the stormwater analyses presented in this manual are predominantly influenced by this range. Comparisons in the more frequent range may not be as favorable, particularly for the time between storms which has a practical minimum determined by the inter-event time selected for storm definition. A small shift in the distribution to represent this minimum may be appropriate.

The gamma distribution is selected for general application in this manual because it is sufficiently accurate in most cases, while simple enough for determining analytical solutions for loading, receiving water, and treatment device evaluations. There is nothing absolute about the gamma distribution, however, and other functions may fit observed distributions more closely in selected areas. Given the level of uncertainty in

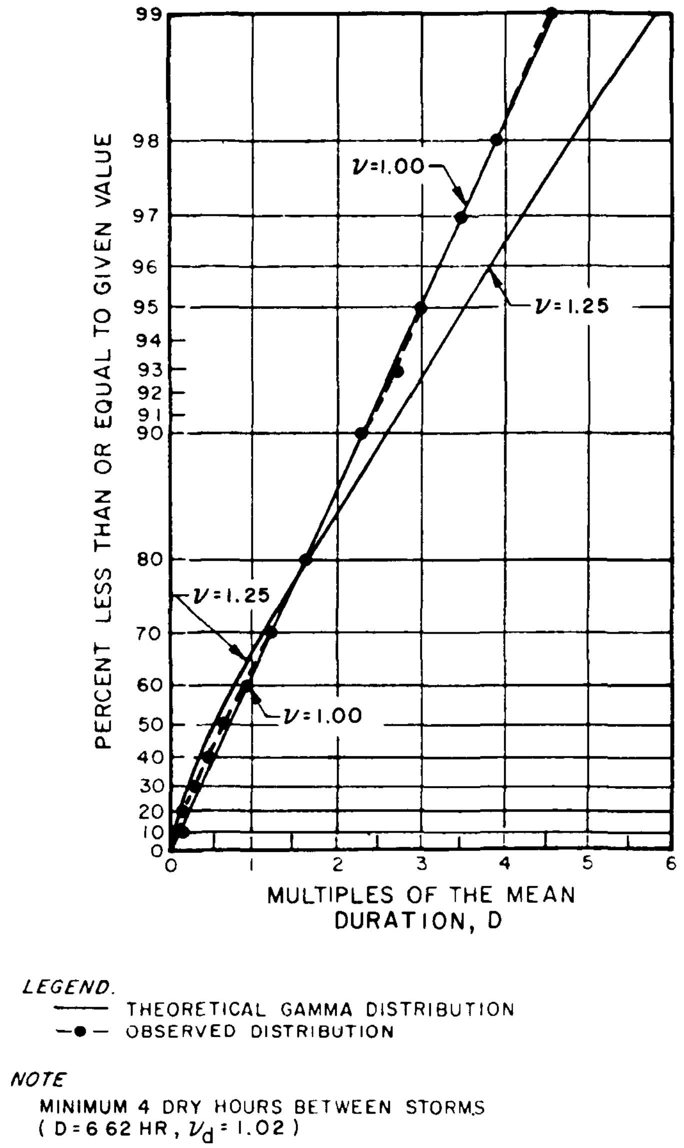


FIGURE 5-7

COMPARISON OF OBSERVED AND THEORETICAL CUMULATIVE DISTRIBUTION OF DURATION CENTRAL PARK, NEW YORK, RAINGAGE 305801, 1948-1973

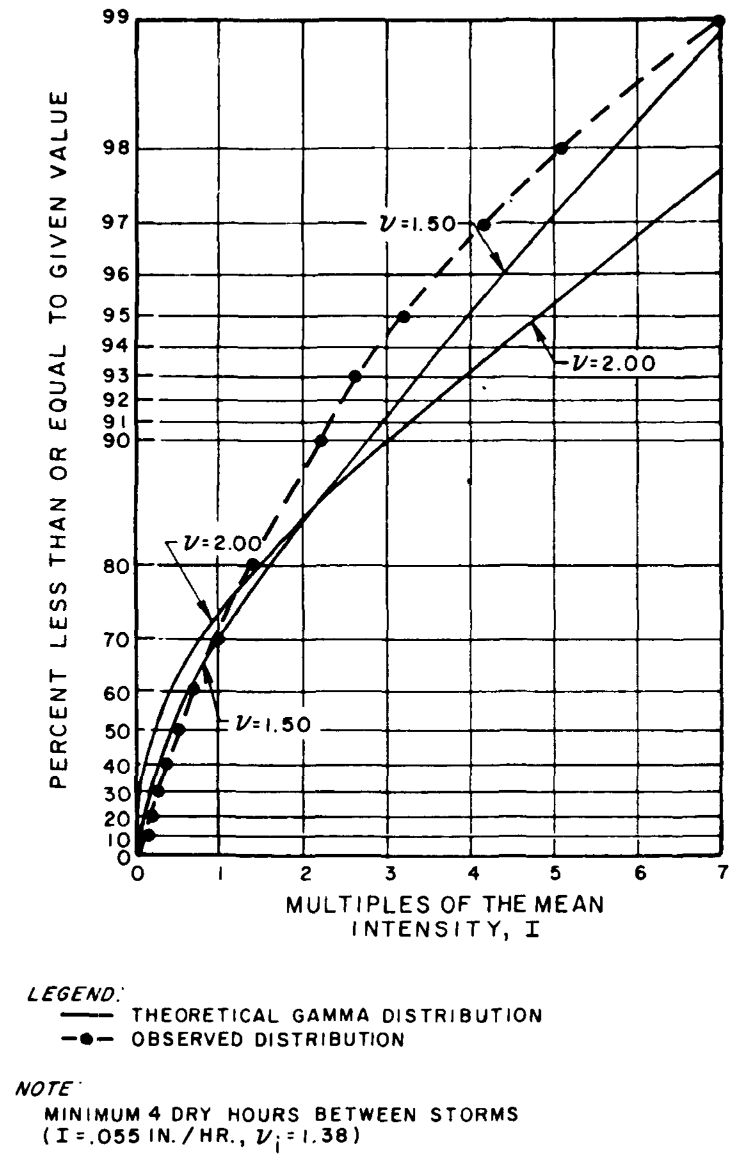


FIGURE 5-8
COMPARISON OF OBSERVED AND THEORETICAL
CUMULATIVE DISTRIBUTION OF INTENSITY
CENTRAL PARK, NEW YORK, RAINGAGE 305801, 1948-1975

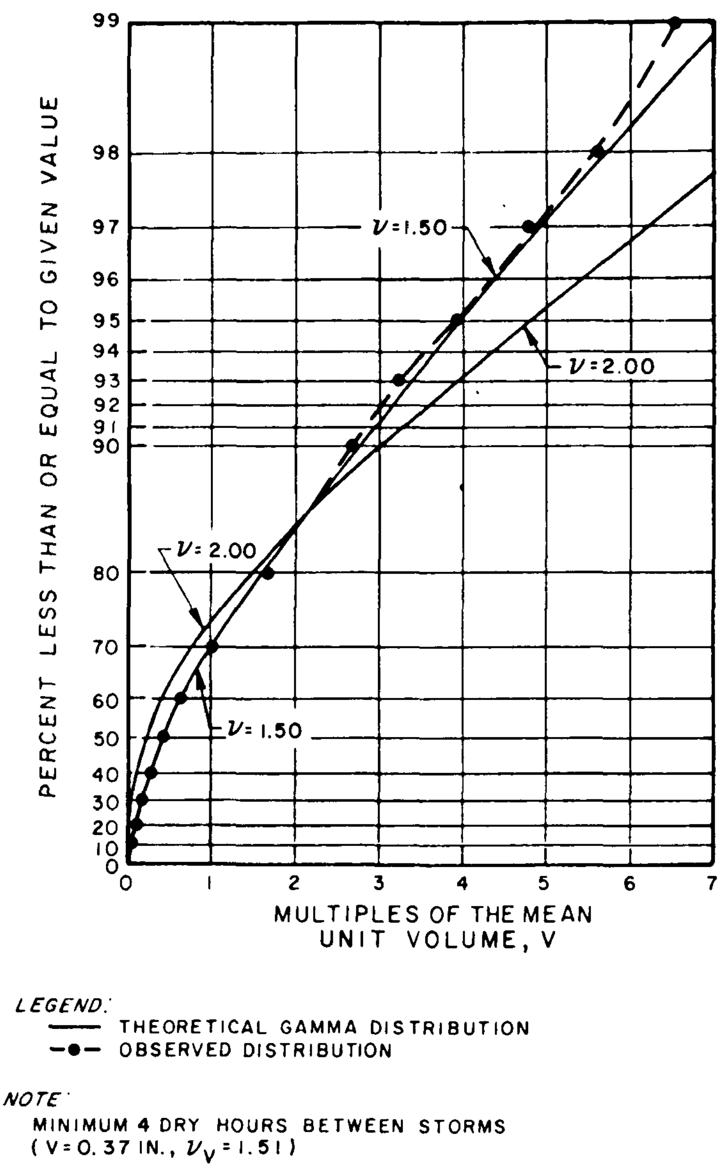


FIGURE 5-9
COMPARISON OF OBSERVED AND THEORETICAL CUMULATIVE DISTRIBUTION OF UNIT VOLUME CENTRAL PARK, NEW YORK, RAINGAGE 305801, 1948-1975

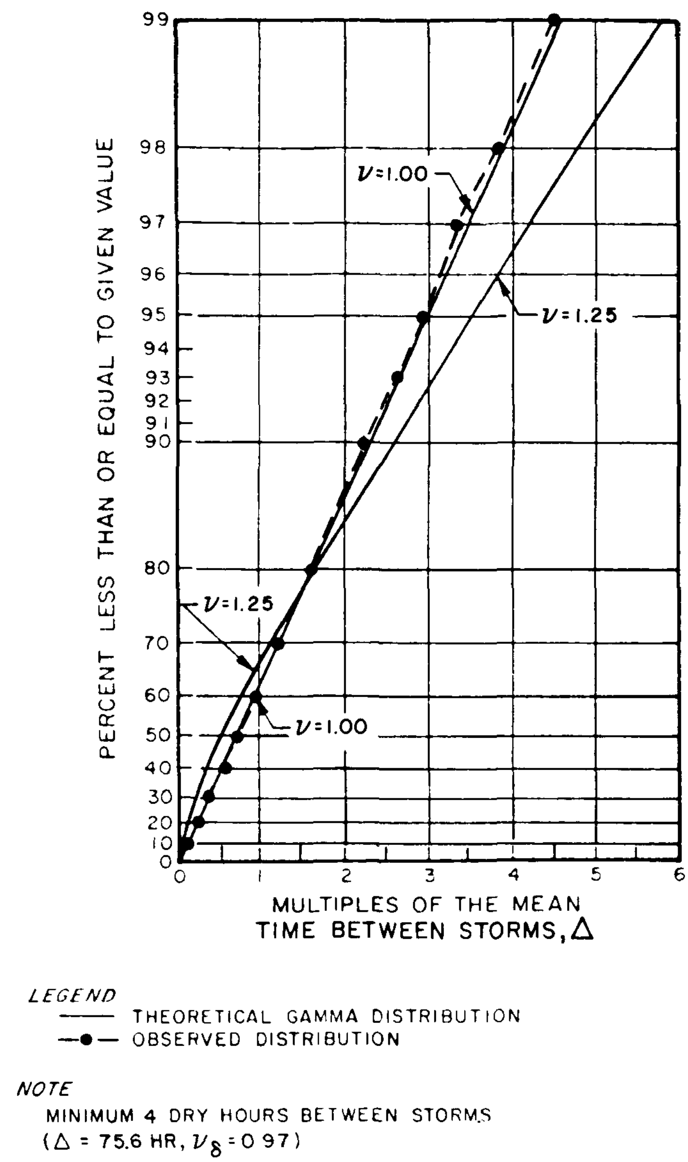
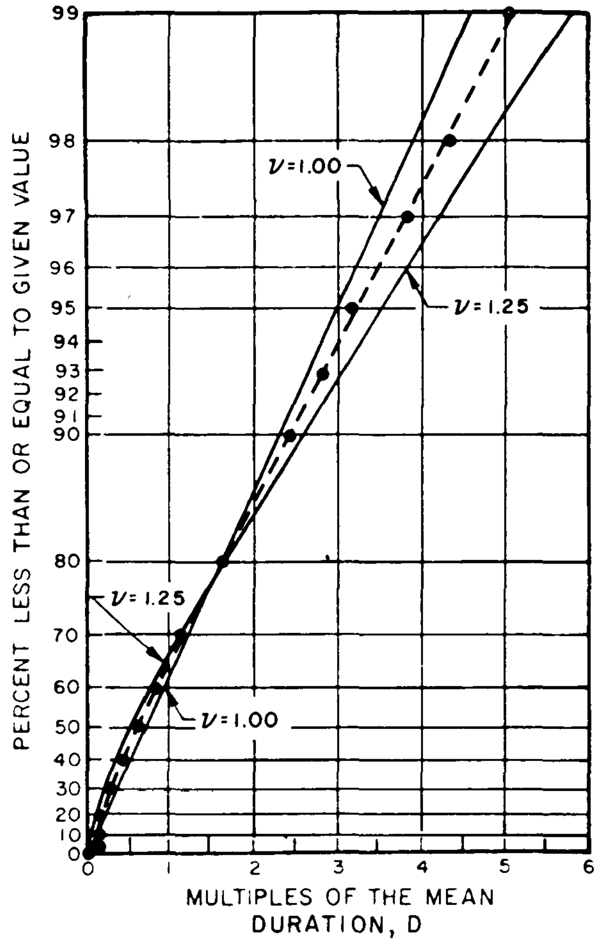


FIGURE 5-10

COMPARISON OF OBSERVED AND THEORETICAL CUMULATIVE DISTRIBUTION OF TIME BETWEEN STORMS CENTRAL PARK, NEW YORK, RAINGAGE 305801, 1948-1975



LEGEND

——— THEORETICAL GAMMA DISTRIBUTION

—●— OBSERVED DISTRIBUTION

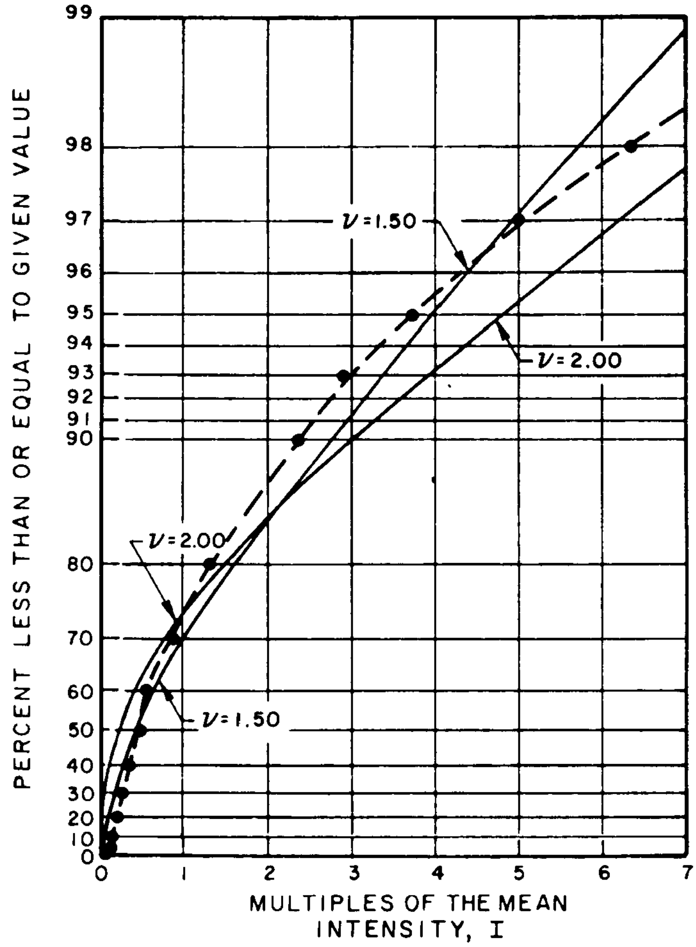
NOTE

MINIMUM 6 DRY HOURS BETWEEN STORMS

(D = 6 30 HR, ν_d = 1.14)

FIGURE 5-11

COMPARISON OF OBSERVED AND THEORETICAL CUMULATIVE DISTRIBUTION OF DURATION MINNEAPOLIS/ST. PAUL AIRPORT, RAINGAGE 215435, 1948-1975



LEGEND:
—— THEORETICAL GAMMA DISTRIBUTION
–●– OBSERVED DISTRIBUTION

NOTE.
MINIMUM 6 DRY HOURS BETWEEN STORMS
(I = .047 IN /HR., ν_i = 1.73)

FIGURE 5-12
COMPARISON OF OBSERVED AND THEORETICAL
CUMULATIVE DISTRIBUTION OF INTENSITY
MINNEAPOLIS/ST. PAUL AIRPORT, RAINGAGE 215435, 1948-1975

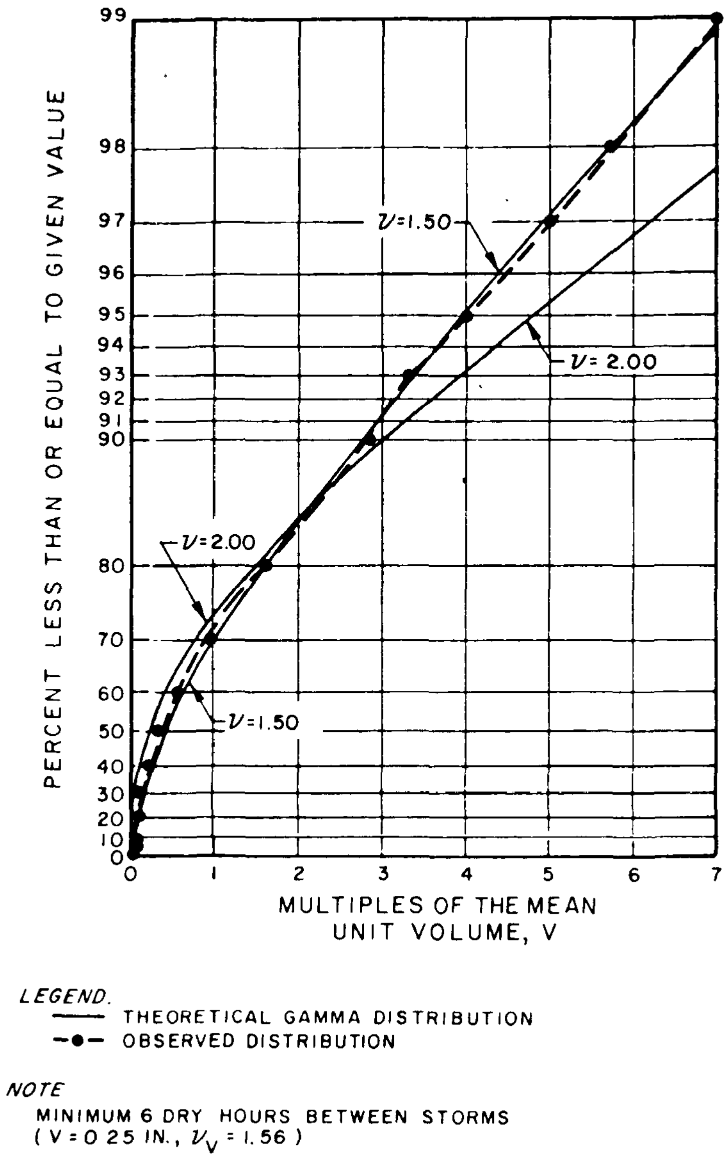


FIGURE 5-13
COMPARISON OF OBSERVED AND THEORETICAL
CUMULATIVE DISTRIBUTION OF UNIT VOLUME
MINNEAPOLIS/ST. PAUL AIRPORT, RAINGAGE 215435, 1948-1975

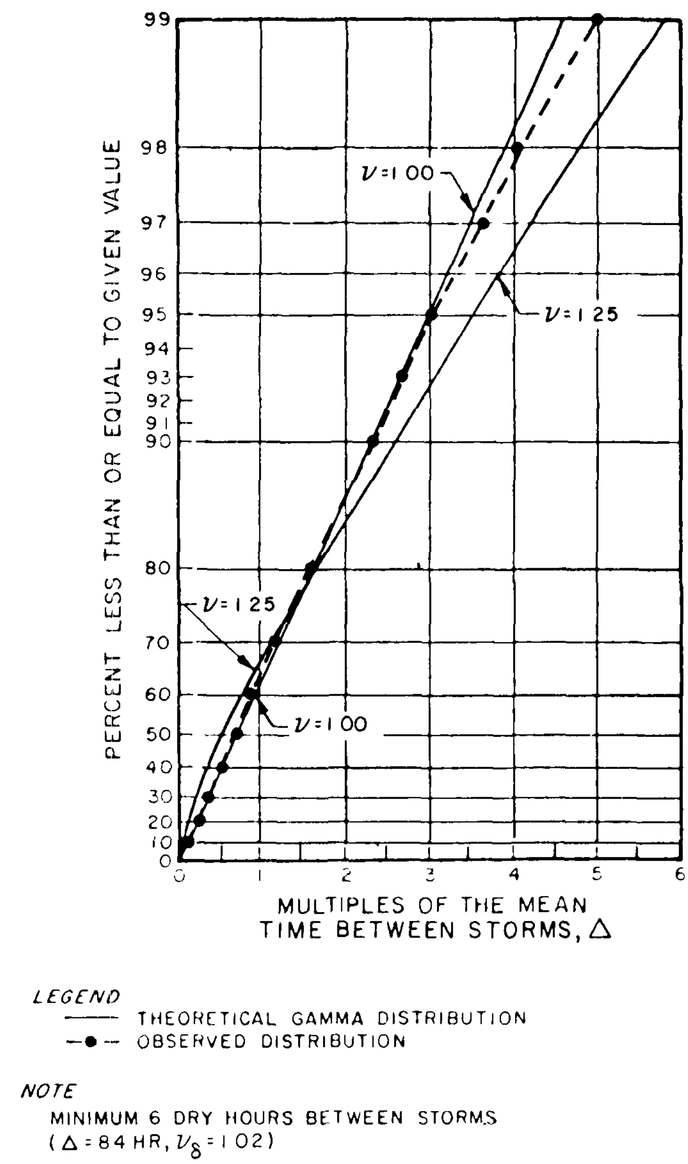


FIGURE 5-14

COMPARISON OF OBSERVED AND THEORETICAL CUMULATIVE DISTRIBUTION OF TIME BETWEEN STORMS MINNEAPOLIS/ST PAUL AIRPORT, RAINGAGE 215435, 1948-1973 stormwater analyses, the error introduced by the misrepresentation of the distribution function is not expected to be of major importance in the overall assessment.

Another consideration that is appropriate for more detailed probabilistic analyses is the relationship between the different event characteristics. Eagleson describes joint probability density functions for event durations and storm depth (7). Chow and Yen use conditional probability relationships in their analysis, and demonstrate an inverse relationship between average intensities and durations in Urbana, Illinois. A portion of this relationship may be due to the seasonal effects discussed previously, and the seasonal segregation of records may result in a reasonable level of independence between i and d. Their approach is useful, however, particularly when more than one storm characteristic must be specified, such as in the analysis of instream concentrations presented in Section 3.5.2.1, where both runoff flows and event durations are required to evaluate the dispersion of the storm pulse.

5.1.4 Areal Distribution of Rainfall

The methods presented for the statistical characterization of storm properties are thus far limited to point rainfall records. Although the rainfall occurring at any location may have relatively similar long-term properties throughout a study area, any method of calculating runoff and pollutant loads caused by storm events require rainfall depths over portions (and for some of the analyses, all) of the study area. Because precipitation does not occur simultaneously over the entire study area, but varies both in time and space, areal rainfall properties differ somewhat from those measured at a single location.

Rodriguez-Iturbe and Mejia provide a good review of studies dealing with the transformation of point to areal rainfall (9). These studies concentrate primarily on estimating the average rainfall occurring over an area during a specified interval of time such as a month, a day, an event, an hour, etc. The rainfall volume recorded at a single gage is reduced as a function of the size of the area to estimate the average areal rainfall. The estimating procedures have developed from empirically based curves, to theoretical estimates based on the spatial and temporal correlation properties of precipitation in a particular region.

An example of the type of correlation analysis performed is demonstrated in Figure 5-15 which shows the correlation coefficient of monthly rainfall totals between raingages in the New York City Metropolitan area. Four years (48 observations) of data from 12 raingages are analyzed. The 12 gages result in 66 possible combinations (without duplication) for which correlations are performed. The results indicate the expected general decrease in the correlation coefficient as a function of the distance between gages. Note, however, that even at the most distant gages, the correlation coefficients are very high. This shows the areal uniformity of monthly rainfall totals in this region. Correlation coefficients for shorter time intervals, however, such as daily or hourly rainfall, can be expected to fall off much

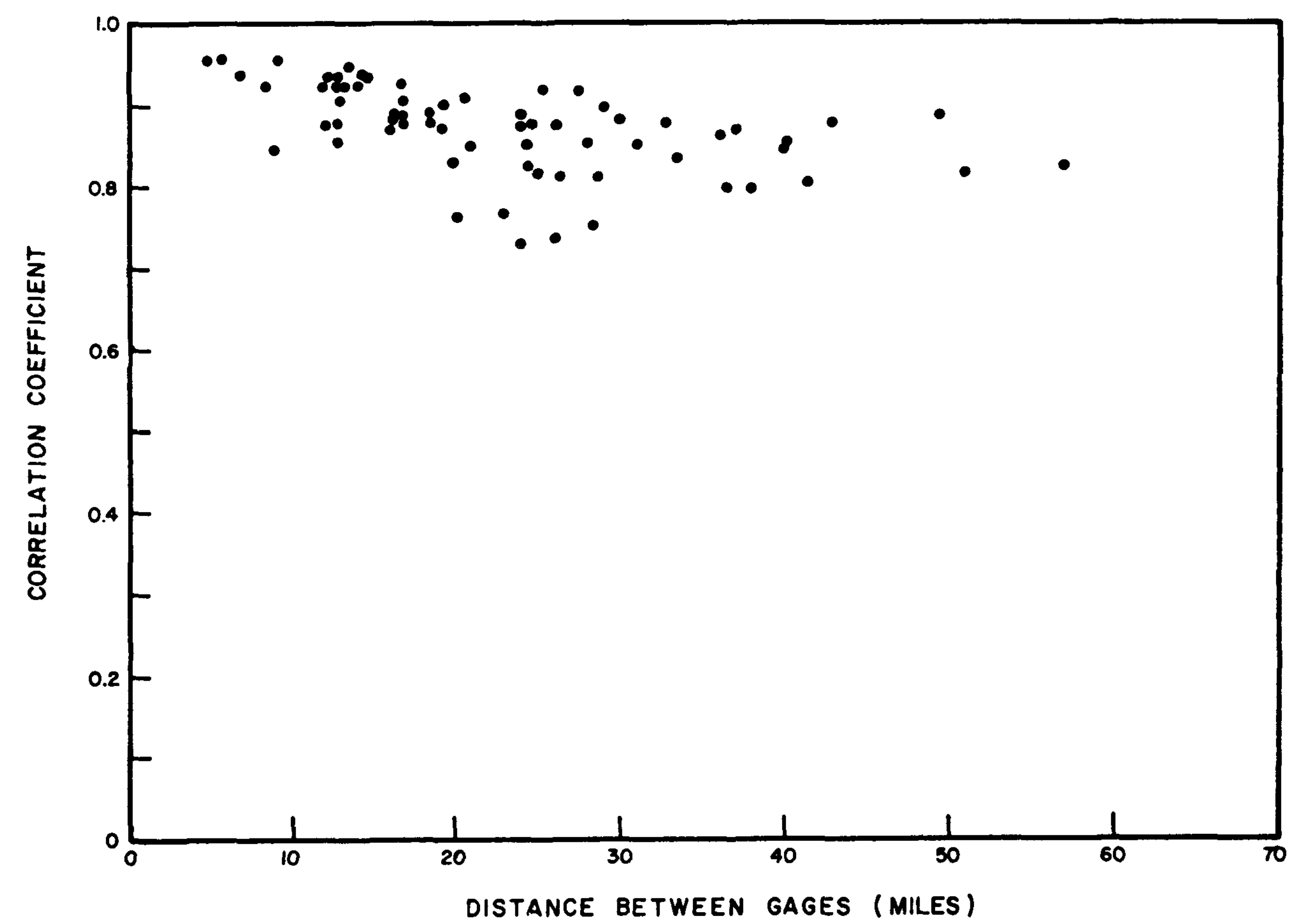


FIGURE 5-15
CORRELATION OF MONTHLY RAINFALL IN THE NEW YORK CITY AREA

ore rapidly with distance, although lag correlation computations may be ppropriate if storms tend to move across the area in a favored direction.

While the principal application of methods to transform point to areal ·ainfall has been to correct the average depth of particular events, the ıethods applied in this manual are based primarily on long-term rainfall ;tatistics; in particular, the mean and coefficient of variation of storm ıntensities, durations, depths, and the time between events. A raingage aggregation method for estimating these statistical event properties over an area, rather than at a single point, is developed and applied in the following section.

5.1.4.1 Raingage Aggregation

Rainfall records from different gages in a study area may be aggregated to form a single record representative of the area. The procedure is outlined in Figure 5-16. The hourly rainfall of the aggregated record is calculated as a weighted average (in this case, with equal weights) of the three individual records. A computer program which combines raingage records over a selected time period, with selected weighting factors for each gage, is used. The synoptic analysis program which computes event statistics then analyzes the combined record.

The raingage aggregation analysis is performed using records from January 1, 1950 - December 31, 1960 (11 years) at nine gages in the New York Metropolitan area, shown in Figure 5-17. Six pairwise gage combinations are examined as well as a combination of all nine gages. Equal weights are used in all the analyses. The results of the synoptic event analysis for each of the nine original records, and the seven combined records, are shown in Table 5-3(a). Table 5-3(b) lists the ratio of the event statistics of the combined record (D', ν_d', etc.) to the event statistics of the individual, point records. The denominator used to represent the point records (D, ν_d, etc.) is the average of the statistics of the individual gages in the combined record (Table 5-3). The ratios are plotted in Figure 5-18 as a function of the distance, x, between the gages. The combination of all nine gages is shown with a triangular symbol at the edge of the plot, since distance between gages is not appropriate for a nine gage aggregation.

Clear patterns are evident in the modification of event statistics as the distance between gages increases. The average storm intensity decreases with greater gage separation, indicating the smoothing effect of the areal processing. There are more storms, as indicated by the drop in Δ'/Δ, but the average unit volume (depth) per storm is smaller as indicated by the decrease in V'/V. The consistency of the drop in time between storms (Δ) and the drop in unit volume (V) is due to the uniformity of total rainfall quantities when considering a long-term period. The average duration is shown to decrease by a factor of about 0.94 and is relatively independent of the distance between gages. This decrease in average duration seems to be due to the fact that most of the additional storms added to the record (storms which occur at one gage but not at the other), are of short duration.

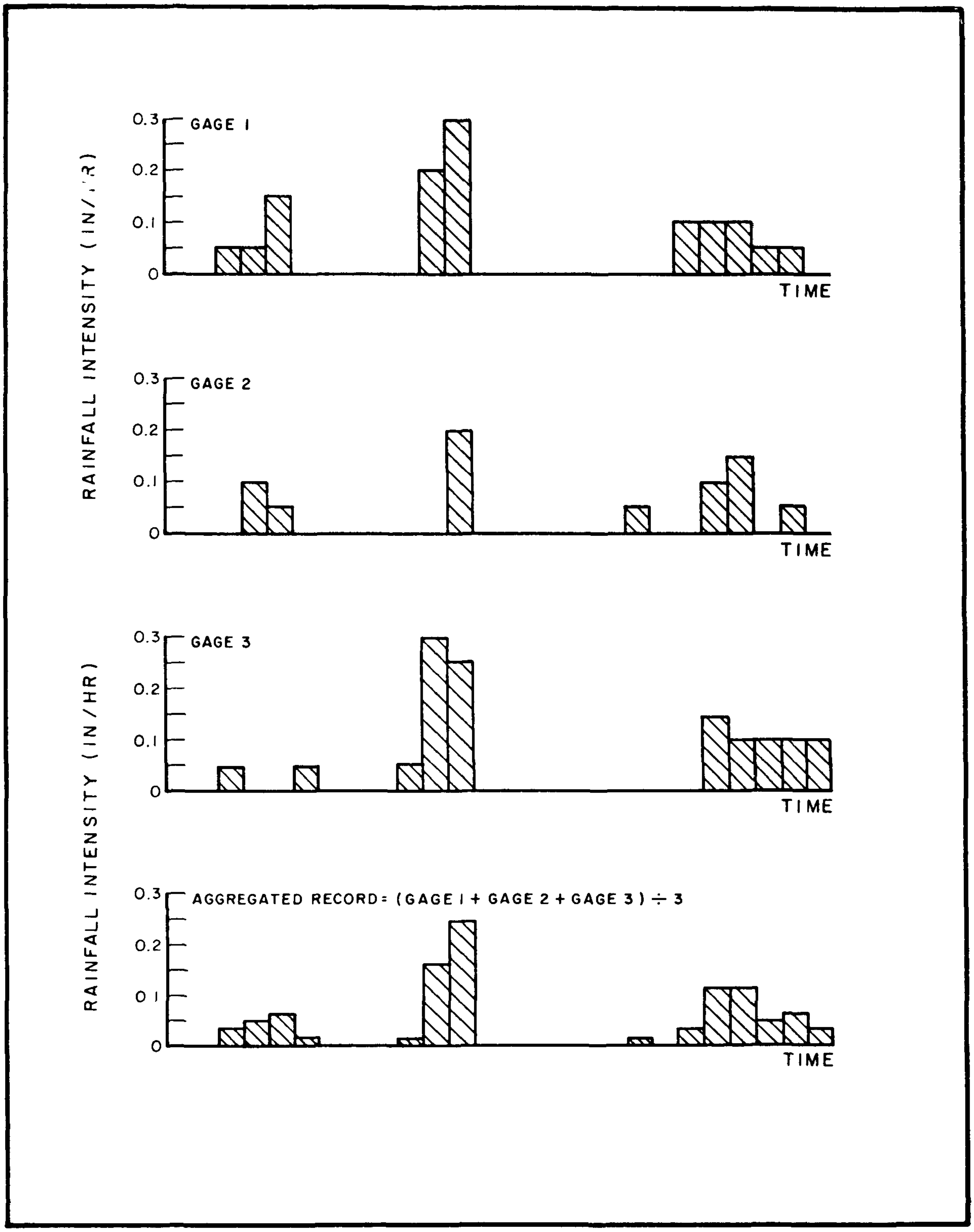


FIGURE 5-16

METHOD FOR RAINGAGE RECORD AGGREGATION

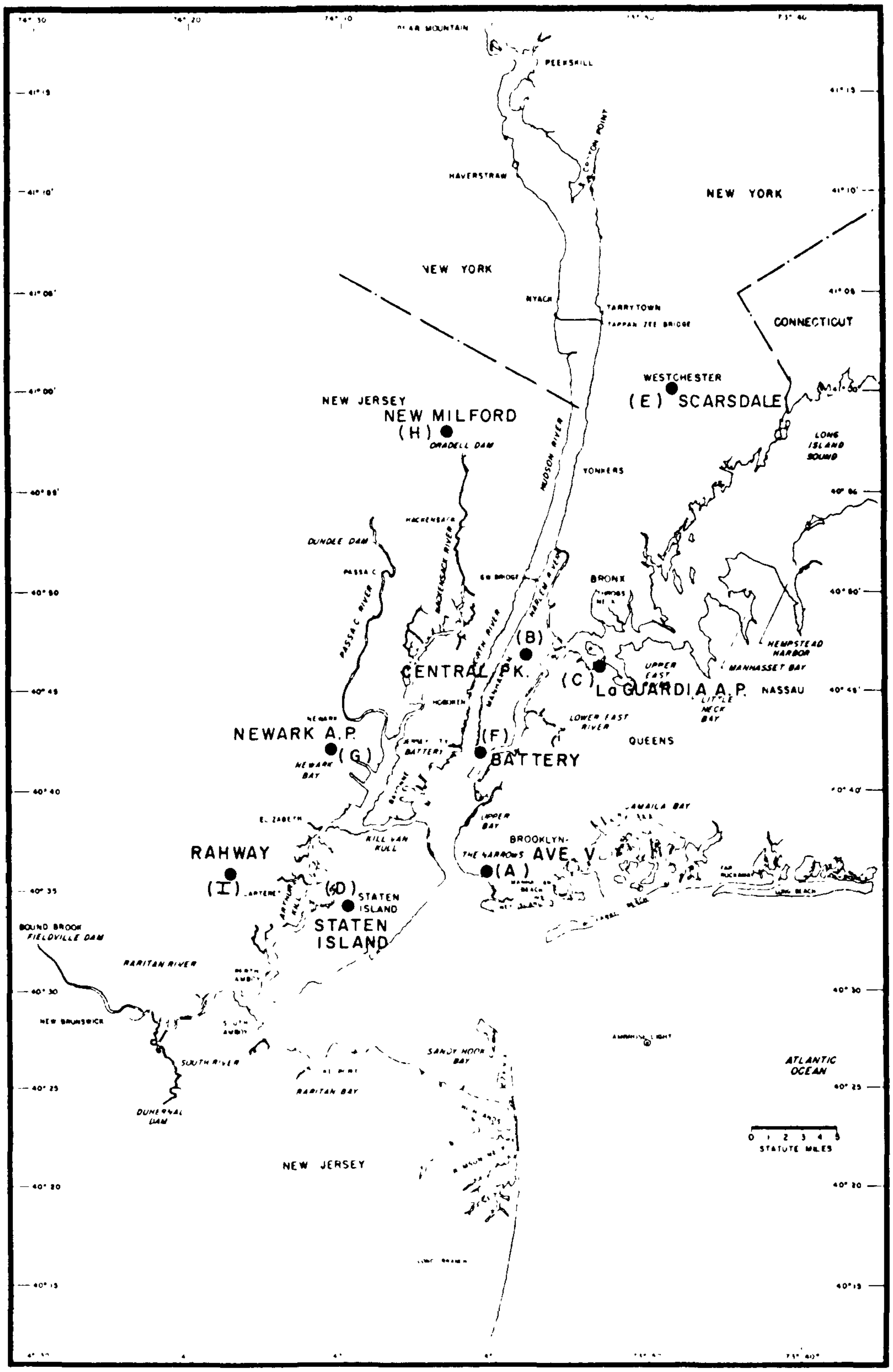


FIGURE 5-17
RAINGAGE AGGREGATION LOCATIONS

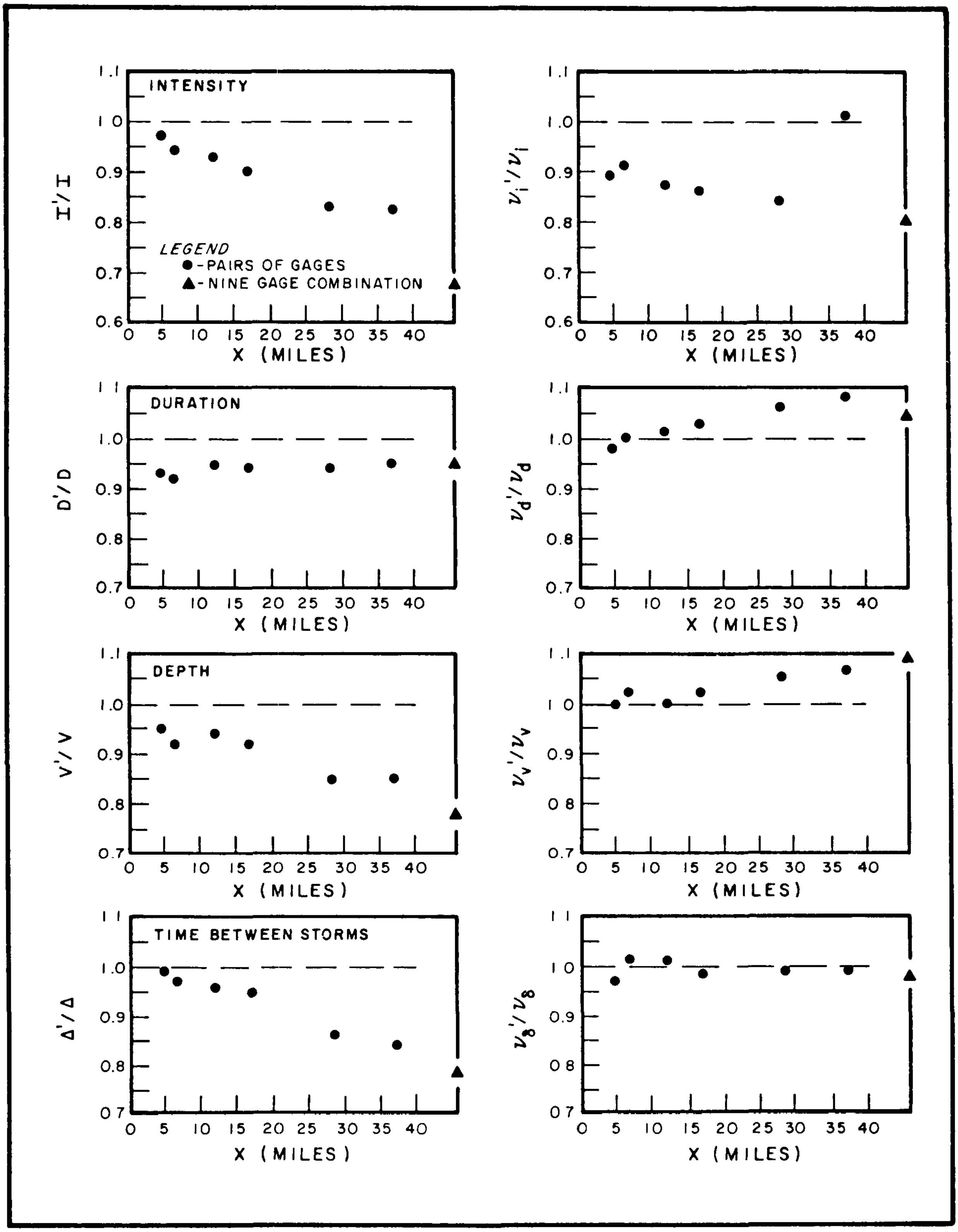


FIGURE 5-18

EFFECT OF RAINGAGE AGGREGATION
ON STATISTICAL EVENT PROPERTIES

TABLE 5-3 (a)

SUMMARY OF NEW YORK CITY RAINGAGE AGGREGATION ANALYSIS
PERIOD OF RECORD: 1/1/50 - 12/31/60
(Minimum 4 Dry Hours Between Storms)

Gage	Duration D (hr)	ν_d	Intensity I (in/hr)	ν_i	Depth V (in)	ν_v	Time Between Storms Δ (hr)	ν_δ	Number Storms
A. Ave. V, Brooklyn	6.58	1.01	0.059	1.57	0.384	1.52	76.0	0.98	1234
B. Central Park	6.62	1.05	0.052	1.32	0.344	1.46	74.9	0.92	1277
C. La Guardia Airport	6.52	1.09	0.051	1.23	0.364	1.73	69.3	0.95	1231
D. Staten Island	6.69	1.09	0.052	1.39	0.379	1.61	71.1	0.92	1336
E. Scarsdale	7.64	0.99	0.056	1.42	0.436	1.62	76.5	0.94	1213
F. Battery Park	6.72	1.09	0.052	1.22	0.373	1.63	76.0	0.93	1259
G. Newark Airport	6.62	1.06	0.051	1.22	0.348	1.50	70.9	0.99	1348
H. New Milford	7.39	0.99	0.058	1.21	0.420	1.47	82.9	0.92	1154
I. Rahway	7.07	0.96	0.060	1.45	0.415	1.46	85.4	1.00	1105
a. Combination of B and C	6.11	1.05	0.050	1.13	0.335	1.59	71.4	0.91	1350
b. Combination of B and F	6.13	1.07	0.049	1.15	0.330	1.58	72.9	0.93	1322
c. Combination of A and G	6.25	1.05	0.051	1.22	0.345	1.51	70.8	0.99	1362
d. Combination of B and D	6.27	1.10	0.047	1.17	0.331	1.57	69.7	0.90	1384
e. Combination of H and I	6.77	1.03	0.049	1.12	0.355	1.54	72.7	0.95	1325
f. Combination of E and I	6.97	1.05	0.048	1.45	0.360	1.63	67.7	0.96	1423
g. Combination of A through I	6.54	1.09	0.037	1.07	0.299	1.70	60.0	0.93	1606

TABLE 5-3(b)

RATIO OF AGGREGATED RAINGAGE STATISTICS
TO POINT RAINGAGE STATISTICS

Combination of Gages	Distance Between Gages x (miles)	Duration		Intensity		Depth		Time Between Storms	
		D'/D	$\nu_{d'}/\nu_d$	I'/I	$\nu_{i'}/\nu_i$	V'/V	$\nu_{v'}/\nu_v$	Δ'/Δ	$\nu_{\delta'}/\nu_\delta$
a. B and C	4.8	0.93	0.98	0.97	0.89	0.95	1.00	0.99	0.97
b. B and F	6.5	0.92	1.00	0.94	0.91	0.92	1.02	0.97	1.01
c. A and G	12.0	0.95	1.01	0.93	0.87	0.94	1.00	0.96	1.01
d. B and D	16.8	0.94	1.03	0.90	0.86	0.92	1.02	0.95	0.98
e. H and I	28.2	0.94	1.06	0.83	0.84	0.85	1.05	0.86	0.99
f. E and I	37.0	0.95	1.08	0.83	1.01	0.85	1.06	0.84	0.99
g. A thru I	-	0.95	1.05	0.68	0.80	0.78	1.09	0.79	0.98

Note: Point raingage statistics (D, ν_d, I, etc.) calculated as average of contributing gages from Table 5-3.

This apparently overrides the increased duration of storms which are lagged from one gage to the other, an effect illustrated in Figure 5-16.

The coefficient of variation of intensity (ν_i) decreases with distance between gages, again demonstrating a smoothing effect. An exception to this is noted for one of the combinations (c) where missing records from one gage correspond to an intense storm at the other. The aggregation program bases the created record on only the available records, so an intense storm is included with the other smoothed storms, resulting in an overestimation of ν_i. The coefficient of variation of durations (ν_d) and depths (ν_v) increase slightly with the distance between gages, while the coefficient of variation of the time between storms (ν_δ) remains relatively unchanged.

To use the results shown in Figure 5-18 to modify the event statistics for drainage areas in the New York area, a transformation is needed from the distance between two gages to the size of the area they represent. Methods suggested for gage correlation analysis (9) based on the average distance between randomly selected pairs of points in an area, do not appear appropriate for this approach, as the aggregated record clearly represents only the area between the gages. Estimates could be based on the assumption that the gages are on opposite points of a regular geometric figure: sides of a square ($A = x^2$); of the diagonal of a square ($A = 0.5\ x^2$); the diameter of a circle ($A = 0.79\ x^2$); the bisecting segment of an equilateral triangle ($A = 0.5\ x^2$); etc. Some guidance is gained by examining the results of the nine gage combination, which may be roughly outlined to cover an area of about 1,000 square miles. Based on the shape of the curves generated by the two gage combinations in Figure 5-18, the results generated by the nine gages (triangular symbol) correspond to a distance of about 45 miles between gages. An estimate of $A = 0.5\ x^2$ would then yield $A = 0.5\ (45)^2 \simeq 1{,}000$ square miles. A factor of one half therefore seems reasonable for converting x^2 to A in the New York study area.

To illustrate the application of the areal rainfall modification, consider a 30,000 acre (47 square mile) drainage area in the New York study region. The appropriate distance between gages is determined:

$$\begin{aligned} A &= 0.5\ x^2 \\ x &= \sqrt{2A} \\ &= \sqrt{2 \times 47} \\ &= 9.7 \simeq 10 \text{ miles} \end{aligned}$$

Figure 5-18 indicates the following approximate transformations for $x = 10$ miles: $I'/I = 0.94$, $\nu_i'/\nu_i = 0.89$, $D'/D = 0.94$, $\nu_d'/\nu_d = 1.01$, $V'/V = 0.94$, $\nu_v'/\nu_v = 1.01$, $\Delta'/\Delta = 0.96$, $\nu_\delta'/\nu_\delta = 1.00$. For example, if the mean storm volume is estimated as $V = 0.37$ in. at a point, the areal estimate is $V' = 0.94(0.37) = 0.35$ in. The difference is negligible, particularly when considering the overall uncertainties in any rainfall-runoff model. While greater modifications may be expected during summer periods due to the occurrence of more localized thunderstorms (rather than larger frontal storms), the use of long-term event statistics to characterize the New York area precipitation appears to reduce some of the need for point to areal rainfall transformations, except when very large drainage areas are considered.

The method presented for point to areal rainfall transformations is empirically based. Curves such as those shown in Figure 5-18 will vary from location to location, depending upon the rainfall correlation properties of each study area. The advantage of the method is that it directly addresses long-term areal rainfall properties, rather than individual events. The raingage aggregation is useful not only for statistical analyses but when developing input to simulation models which accept records from only one gage, even though a large basin is being modeled.

5.2 Runoff Quantity

Runoff flows are generated when rainfall exceeds the storage and infiltration capacity of the drainage basin. Surface runoff begins as thin sheetflows which are consequently gathered into natural stream channels or manmade conveyance systems. At the beginning of the storm event, little or no runoff is generated as the initial precipitation is captured by the available depression storage or intercepted by the vegetation throughout the basin. This capture is often referred to as initial abstraction. As the storm continues and the available depression storage is filled, runoff occurs as the difference between the rainfall rate and the infiltration rate. While depression storage is available in both pervious and impervious areas, infiltration is confined to pervious land surfaces (although water may runoff from impervious to pervious areas). The rate of runoff at various times during the event is affected by time lags of contributing flow from different portions of the catchment and the means by which the flow is routed through the conveyance system. These processes are the basic mechanisms that determine modeling the quantity of stormwater runoff.

While a variety of detailed techniques are available for modeling the journey from raindrop to stormwater runoff or overflow, the methods presented in this manual rely on simple transformations from rainfall to runoff. The basic parameters required include the average ratio of runoff to rainfall (R_v) for use in the statistical method and broadscale drainage basin simulators, and the average duration of runoff events (D_R) for use in the statistical method. Methods for estimating these factors based on drainage basin characteristics are presented in this section. As discussed previously, this simplified approach for evaluating runoff quantity is appropriate only for the assessment of a long term series of rainfall events and their impacts, and not for the accurate representation of individual storms.

5.2.1. Determination of Average Runoff to Rainfall Ratio

The average ratio of runoff to rainfall (R_v) is used to convert rainfall volumes to runoff volumes, as described in Equation 3-12. It is a composite coefficient incorporating the effects of depression storage and infiltration into a single ratio. It should not be confused with the runoff coefficient traditionally employed in the Rational Method to calculate the peak rate of runoff after the initial abstraction has occurred and all portions of the drainage catchment are contributing (10). Rather it provides an estimate of the fraction of rainfall which becomes runoff over a long term period. In this sense R_V is a volume weighted average ratio, accounting for correlations between storm depth and the runoff fraction of individual storms, when they exist. (There is often a positive correlation between storm depth and the runoff fraction, as larger storms are relatively less affected by depression storage, and may also result in saturated soil conditions).

Drainage basin characteristics which tend to increase the average ratio of runoff to rainfall include a high percent of impervious surfaces (streets, sidewalks, rooftops, parking lots, etc.), tight, clay-type soils, and steep land slopes. Impervious surfaces are much less effective at generating runoff when they drain onto pervious areas (such as roof gutters onto lawns), than when they are directly linked to the conveyance system. During wet years or seasons, R_V is also higher because of the more saturated state of the soils and the more frequent reduction of available depression storage due to previous storms.

The primary factor used to predict R_V in urban runoff studies is the percent impervious area. Miller and Viessman investigate the effect of imperviousness on the rainfall-runoff relationship in four urban drainage catchments (11). A linear relationship is found between the rainfall which remains after the initial abstraction is subtracted, and the amount of runoff:

$$R = 1.165\ (I - 0.17)\ (P - I_a) \qquad (5\text{-}1)$$

where R is the runoff in inches, I is the fraction impervious area, P is the rainfall in inches, and I_a is the initial abstraction in inches. The value of I_a is estimated to be between 0.10 and 0.15 inches, and the equation is applicable to individual storms in areas where I ranges from 0.35 to 0.80. To apply these results to the simpler representation where the initial abstraction is lumped together with other factors into a single ratio (R_V), a correction must be made for the effect of I_a on R_V. Assuming the initial abstraction is equivalent to a storage device (see Sections 3.6.1.2 and 3.6.2.4), the long term effect of an I_a = 0.10 inches may be estimated with Figure 3-18 (reproduced as Figure 5-19). If V = 0.40 inches and ν_V = 1.5 (typical values), a depression storage of about 0.10 inches reduces the long-term runoff about 20 percent. Equation (5-1) is then modified to estimate R_V as follows:

$$\begin{aligned} R_V &= (1-0.20)(1.165)(I - 0.17) \\ &= 0.93\ (I - 0.17) \end{aligned} \qquad (5\text{-}2)$$

where I is the fraction impervious area. The equation is applicable for $0.35 \leq I \leq 0.80$. This relationship is plotted in Figure 5-20 and referred to as the extension of Miller and Viessman.

The STORM simulation model computes a runoff coefficient equal to 0.15 + 0.75 I, where I is the fraction impervious area. Again this value represents the runoff to rainfall ratio after depression storage has been removed. Heaney, et al found however, that the long term runoff is overpredicted by

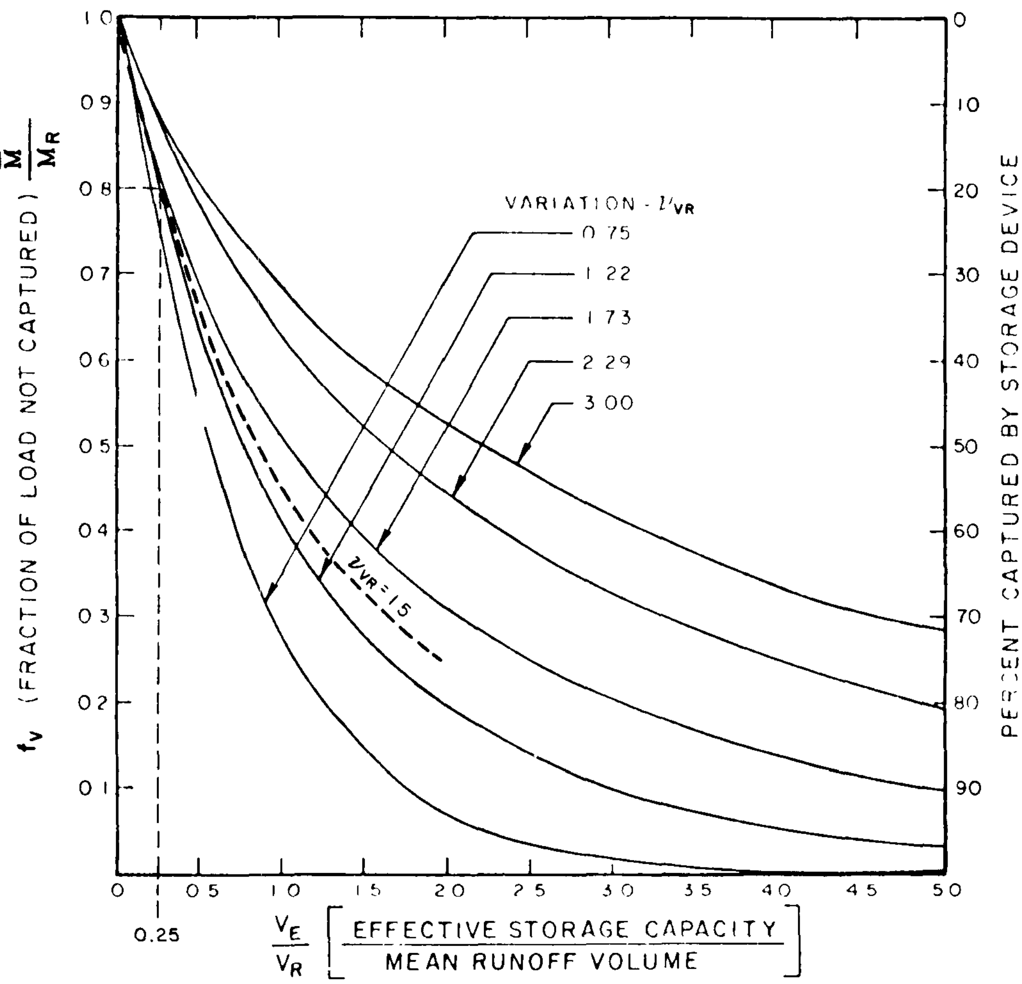
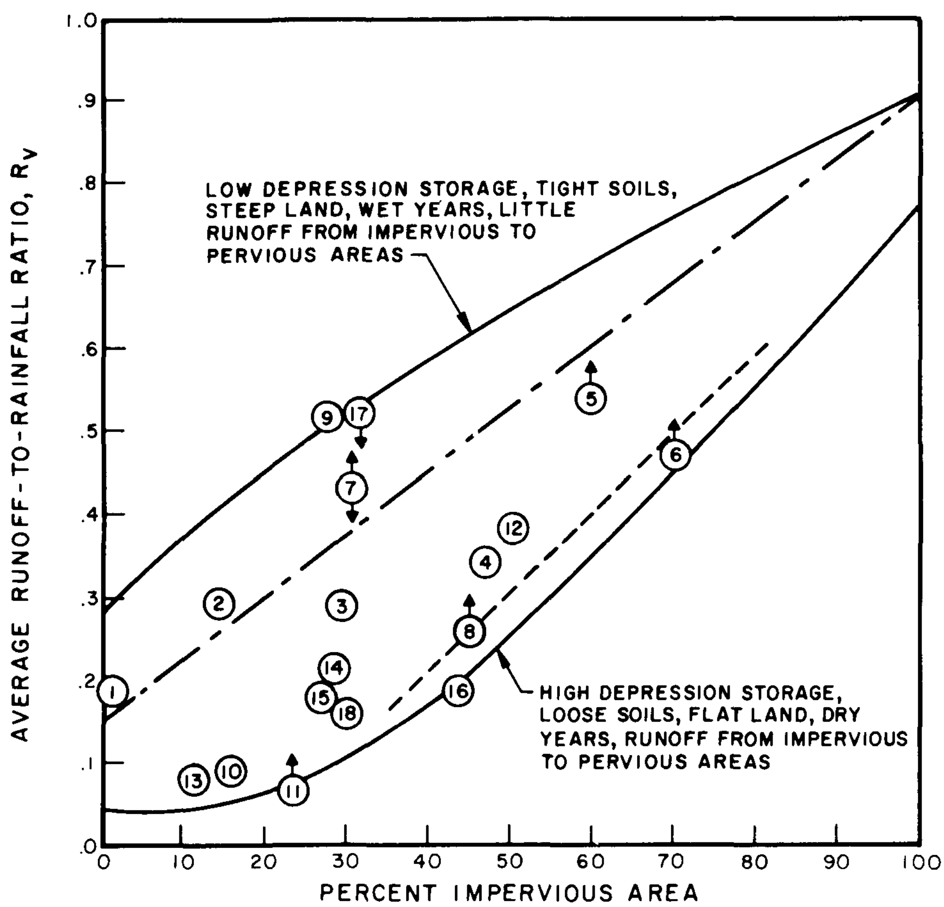


FIGURE 5-19
DETERMINATION OF LONG TERM STORAGE DEVICE PERFORMANCE



STUDY LOCATION CODE:

1 UPPER WHITE ROCK, DALLAS	10 FIRST CREEK, KNOXVILLE
2 LOWER WHITE ROCK, DALLAS	11 PLANTATION HILLS, KNOXVILLE
3 BACHMAN BRANCH, DALLAS	12 NORTHAMPTON, ENGLAND
4 TURTLE CREEK, DALLAS	13 BRIGHOUSE, ENGLAND
5 HENDRIX CREEK, NEW YORK CITY	14 BRADFORD, ENGLAND
6 SPRING CREEK EAST, NEW YORK CITY	15 MADISON, WISCONSIN
7 THURSTON BASIN, NEW YORK CITY	16 TULSA, OKLAHOMA
8 FOURTH CREEK, KNOXVILLE	17 DURHAM, NORTH CAROLINA
9 THIRD CREEK, KNOXVILLE	18 TROUT RUN, ROANOKE, VIRGINIA

LEGEND:

—·— "STORM" EQUATION ESTIMATE (12)
- - - - EXTENSION OF MILLER AND VIESSMAN (11)
- SUSPECT ERROR, DIRECTION OF PROBABLE CORRECTION

FIGURE 5-20
RELATIONSHIP BETWEEN
IMPERVIOUS AREA AND RUNOFF-TO-RAINFALL RATIO

only about 0.3 equivalent inches per year by ignoring the depression storage correction in the STORM calculation (1). The assumption that this correction is minimal is also made implicitly by Metcalf and Eddy in their simplified stormwater simulation model (12). Rainfall is converted to runoff with a runoff coefficient (K), which is equivalent to the R_V used in this manual, and the ratio of runoff to rainfall is referenced by Metcalf and Eddy from the STORM model as:

$$K = R_V = 0.15 + 0.75\ I \qquad (5\text{-}3)$$

This relationship is shown in Figure 5-20, and identified as the STORM equation estimate.

To supplement the Miller and Viessman study and the STORM equation, data from 8 studies on 18 different catchments are analyzed. The results of these studies are summarized in Table 5-4, and the average ratio of runoff to rainfall (R_V) is plotted for each as a function of the percent impervious area in Figure 5-20. A number of the studies include information which indicate possible errors, and these are noted in the comments. For example, considerable transmission losses are observed in the Plantation Hills and possibly the Fourth Creek stream channel in the Knoxville, Tennessee study.

While Figure 5-20 indicates the expected positive relationship between the percent impervious area and R_V, a great deal of variability is observed, and other factors are clearly important in determining the rainfall-runoff relationship. In choosing an R_V for a particular study area, engineering and planning judgement should be used to estimate the effect of the other factors indicated in Figure 5-20. If no other information is available, the STORM equation appears to provide a reasonably conservative estimate and may be used until local information is collected. Local data and monitoring are clearly needed to better refine the estimate of R_V, and the effects of drainage basin characteristics on this estimate.

In order to use the percent impervious area to estimate R_V, the percentage of impervious area in a drainage basin may be determined by examining aerial photographs or detailed land use plans. This may be a long and tedious task, however, particularly for large drainage areas. Satelite photographs (LANDSAT) may be used to help evaluate the land cover of larger basins (23). Estimates may also be made on the basis of the general fraction of land in different land use categories. Specific land use classifications are assigned an average percent impervious area as shown below. The indicated values or other handbook estimates (24,25) may be modified or refined on the basis of local information, past experience or field inspection surveys.

Land Use Category	Percent Impervious Area
Residential	
Low Density	20
Medium Density	40
High Density	60
Commercial	80

TABLE 5-4

RUNOFF TO RAINFALL RATIOS FROM VARIOUS STUDIES

Study Location	Drainage Area (acres)	Percent Impervious (%)	Average Ratio of Runoff to Rainfall, R_V	Extent of Study	Comments	Reference
1. Upper White Rock, Dallas	18,800	1	0.18	1962-70	Moderately tight soils; 2-15% slopes	(13)
2. Lower White Rock, Dallas	24,500	14	0.29	1962-70	Moderately tight soils; 2-15% slopes	
3. Bachman Branch, Dallas	6,400	30	0.28	1964-70	Moderately tight soils; 2-5% slopes	
4. Turtle Creek, Dallas	5,100	47	0.34	1962-70	Moderately tight soils; 2-7% slopes	
5. Hendrix Creek, N.Y.C.	492	60	0.54	22 storms	R_V represents ratio of overflow (rather than runoff) to rainfall, though interceptors capture little. Suspected flow into Thurston from adjacent area.	(14,15)
6. Spring Creek, N.Y.C.	1,382	70	0.47	16 storms		
7. Thurston Basin, N.Y.C.	1,980	30	0.44	13 storms		
8. Fourth Creek, Knoxville	525	45	0.27	1971-73	Suspected transmission losses in permeable portions of stream channels at Plantation Hills and possibly Fourth Creek	(16)
9. Third Creek, Knoxville	1,025	28	0.52	1971-73		
10. First Creek, Knoxville	320	16	0.09	1971-73		
11. Plantation Hills, Knoxville	153	23	0.06	1971-73		
12. Northampton, England	229	50	0.38	1960-61	0.013 median sewer slope	(17)
13. Brighouse, England	594	11	0.08	1958-61	0.044 median sewer slope	
14. Bradford, England	167	28	0.21	1961-64	0.021 median sewer slope	
15. Madison, Wisconsin	123	27	0.18	34 storms		(18)
16. Tulsa, Oklahoma	197	44	0.19	12 storms	Most roofs drain to pervious areas	(19)
17. Durham, North Carolina	1,070	17	0.52	33 storms	Rainfall possibly underestimated by 40% (21)	(20)
18. Trout Run, Roanoke, Va.	997	30	0.16	14 storms		(22)

Industrial	70
Institutional, Public	30
Open, Undeveloped	0

The percent impervious area for the entire drainage area is calculated by taking a weighted average of the individual components of the area. For example, assume an area has the following land use characteristics:

Low Density Residential	30%
Medium Density Residential	20%
Commercial	10%
Industrial	10%
Institutional, Public	5%
Open, Undeveloped	25%
Total	100%

The overall percent impervious area is:

$$\begin{aligned}\text{Percent Impervious Area} &= (0.30)(20) + (0.20)(40) \\ &+ (0.10)(80) + (0.10)(70) \\ &+ (0.05)(30) + (0.25)(0) \\ &= 30.5\%\end{aligned}$$

This value is used in Figure 5-20 to determine the average ratio of runoff to rainfall, R_V.

If information on the land use categories cannot be obtained, the portion of developed areas which is impervious may be estimated from the population density. Graham et al. (Washington, DC) (26), the American Public Works Association (27) and Stankowski (New Jersey) (24) have developed equations to predict imperviousness as a function of population density. The imperviousness is estimated for the developed portion of the urbanized area only. The weighted average imperviousness and population density have been calculated for nine Ontario cities (28). These results are plotted on Figure 5-21 together with the three estimating curves (29). If the New Jersey data, which is based on 567 municipalities, is selected as a reasonable guideline, the equation used to estimate imperviousness is:

$$\text{Percent Impervious Area} = 9.6\ PD_d^{\ (0.573-0.0391\ \log_{10} PD_d)} \tag{5-4}$$

where: PD_d = population density in developed portion of the urbanized area (persons/acre).

This provides a convenient estimate for an initial estimate of the runoff coefficient.

5.2.2 Determination of Average Duration of Runoff Event

In large, unurbanized drainage basins the runoff flow may continue well beyond the end of the rainfall event. If the average storm runoff flow is calculated directly from the average intensity and the rainfall to runoff

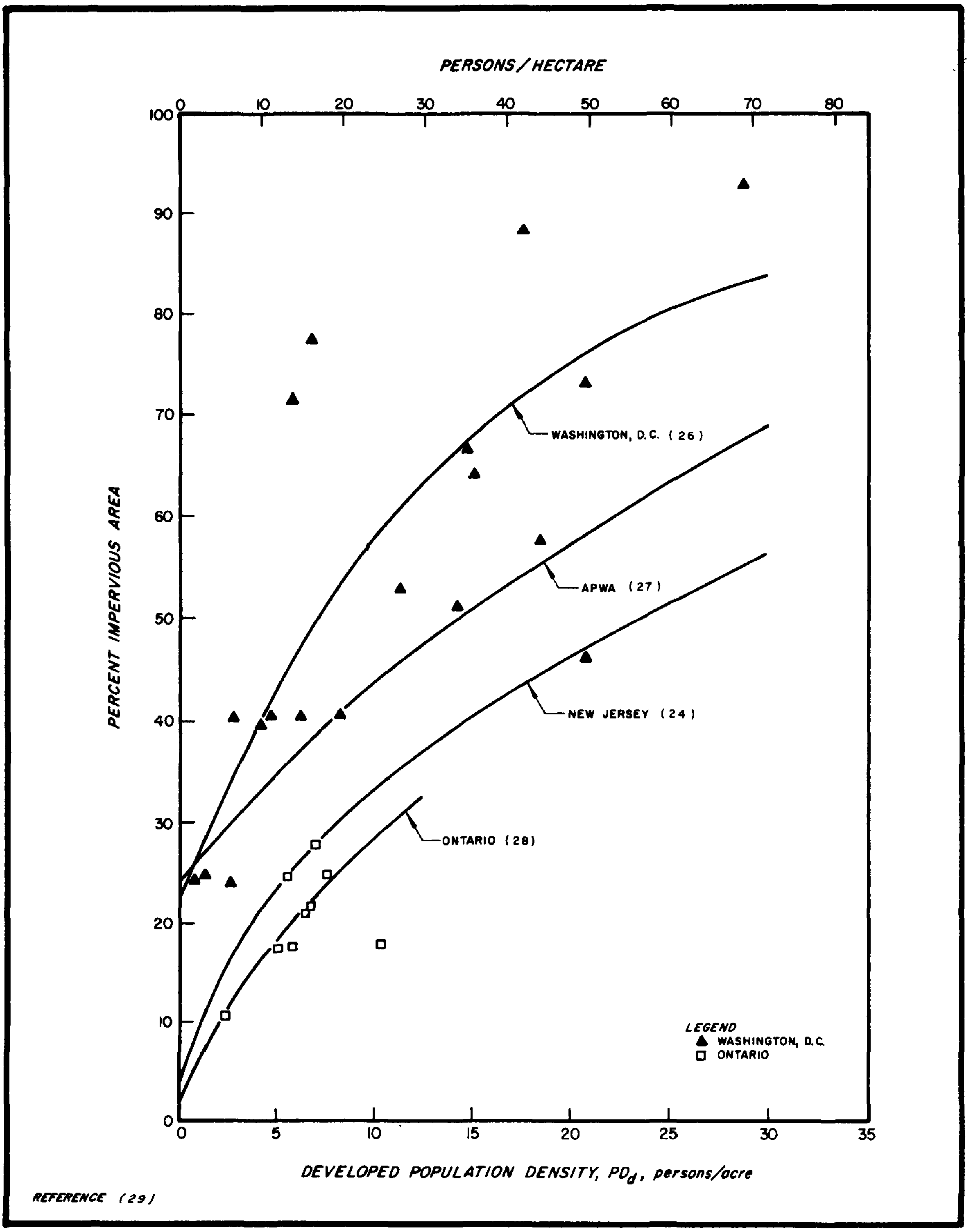


FIGURE 5-21
IMPERVIOUSNESS AS A FUNCTION OF
DEVELOPED POPULATION DENSITY

ratio ($Q_R = R_V IA$), it is assumed that the duration of runoff events is essentially equal to the duration of rainfall events. When the runoff duration is significantly modified by drainage basin characteristics, however, the average runoff flow can be considerably overestimated. To account for this, the correction factor (Equation 13 of Chapter 3) is suggested:

$$Q_R = R_V IA \; (D/D_R) \tag{5-5}$$

where D/D_R is the ratio of the average rainfall event duration to the average runoff event duration. This section provides a method for estimating the average runoff event duration (D_R) based on the hydrologic properties of the drainage basin.

The approach for estimating D_R is based on the analysis of unit hydrographs. The basic unit hydrograph and its shape parameters are shown in Figure 5-22. A profile of discharge versus time at the outlet of the catchment or basin is determined from a unit volume of precipitation excess over some time interval (t_o), which should be no larger than one fourth (10) to one half (30) the lag time to peak (t_p). To estimate the average duration of runoff events, two steps are suggested: (1) the average duration of precipitation excess (D_e) is calculated; (2) the width of the hydrograph is determined.

The early portions of the storm generate negligible runoff until the initial abstraction is removed. The duration of precipitation excess is thus shorter than the rainfall duration. The average duration of precipitation excess (D_e) is approximated by assuming that the initial abstraction occurs as storage, as shown in Figure 3-14(d). The duration of precipitation excess is equal to the widths of the unshaded portions of Figure 3-14(d). The average duration of precipitation excess is equal to the expected value of the unshaded durations:

$$D_e = \int_{i=0}^{\infty} \int_{d=V_d/i}^{\infty} (d - V_d/i) \; p_d(d) \; p_i(i) \; dd \; di \tag{5-6}$$

where V_d is the storage due to the initial abstraction. Equation (5-6) is solved for the special case where storm durations and intensities are independent and exponentially distributed and, dividing by the mean rainfall duration (D), the solution is:

$$D_e/D = 2\sqrt{\frac{V_d}{DI}} \; K_1 \left[2\sqrt{\frac{V_d}{DI}}\right] \tag{5-7}$$

where K_1 is a modified Bessel function, D and I are the average intensity and duration. Equation (5-7) is plotted in Figure 5-23. The initial abstraction (V_d) is estimated by Miller and Viessman to be generally between 0.1 and 0.15 inches in urbanized catchments. V_d may also be estimated as equal to the depression storage (though all the depression storage may not be available at the beginning of every storm). The STORM model estimates the depression storage for pervious and impervious areas as follows:

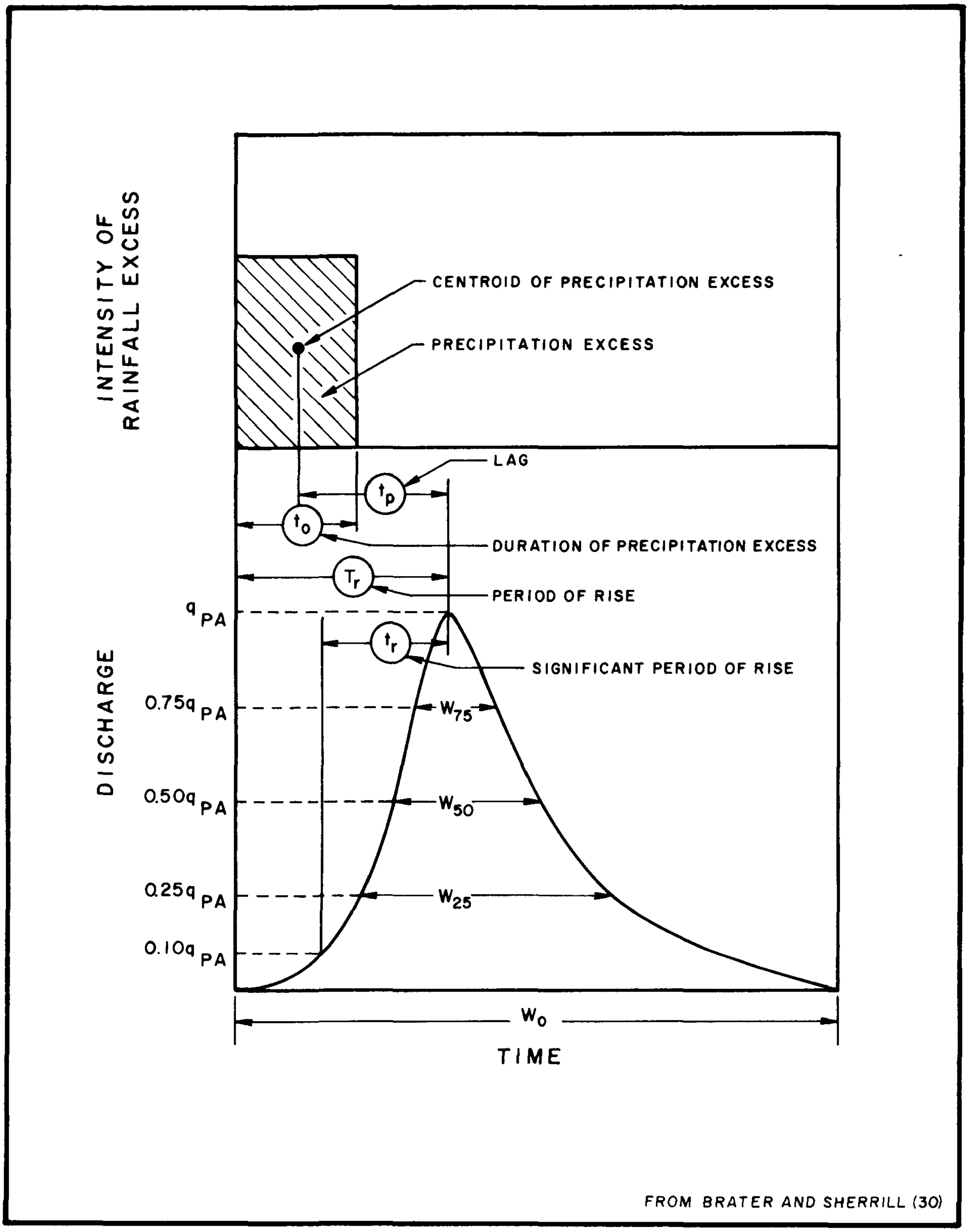


FIGURE 5-22

UNIT HYDROGRAPH DEFINITION SKETCH

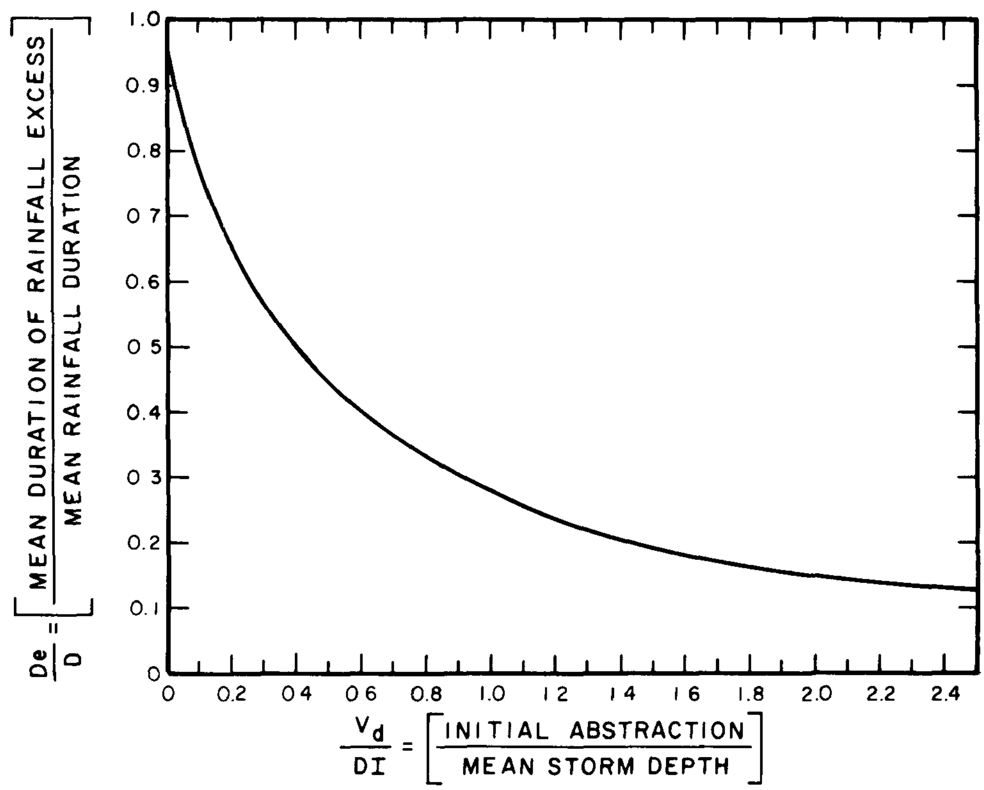


FIGURE 5-23
ESTIMATE OF MEAN DURATION OF RAINFALL EXCESS

Land Use	Depression Storage (in)
Impervious	0.0625
Pervious	0.25

For a given land use, the area weighted depression storage, (DS), in inches, is:

$$DS = 0.25 - 0.1875\ (\text{Percent Impervious Area}/100) \tag{5-8}$$

Using these guidelines, the estimate of V_d is generally in the range found by Miller and Viessman. For an area with a mean storm depth of 0.3 inches and an estimated V_d of 0.1 inches, Figure 5-23 indicates that D_e/D is about 0.54.

Once the mean duration of precipitation excess (D_e) is estimated, the period of runoff attenuation must be added. Brater and Sherrill (30) present relationships between unit hydrograph parameters and drainage area characteristics:

> The unit hydrograph peaks as well as their time characteristics such as their periods of rise and widths at various fractions of the peak discharge can be correlated with watershed areas and population density to provide statistically significant relations which enable hydrologists to estimate the runoff characteristics of ungaged areas. These relationships were derived from the analysis of hundreds of flood hydrographs from 53 drainage basins from five states. The areas of these basins vary from 0.02 to 743 sq. mi. and the population densities cover a range from less than 100 to more than 14,000 persons/sq. mi. (30, p.1.).

Note that population density is used as a general indicator of the percent impervious and channel condition effects. Modifications for intensive commercial or industrial areas can be made by using a larger-than-actual population density to reflect the level of urbanization in the basin.

Assuming that the time period (t_o) used to generate each unit hydrograph is small (each of the resulting hydrographs are superimposed to generate the total runoff), the runoff duration may be estimated as the duration of the rainfall excess plus the time necessary for the runoff generated by the final rainfall excess to reach some arbitrarily small value. Using the time of the base (W_o) assumes that the runoff flow must return to zero for the runoff event to be considered complete. This will yield a very large estimate of D_R, which will lead to a very small estimate of Q_R. A more reasonable estimate of the period of effective runoff, W_{25}, is suggested since a large majority of the runoff volume has been accounted for by then, and D_R will not be biased by the long tail of the hydrograph. These considerations lead to an estimate of the runoff event duration as:

$$D_R = D_e + W_{25} \tag{5-9}$$

Based on the work of Brater and Sherrill, estimates of W_{25} may be made from Figure 5-24, using the drainage area (sq. miles) and the population density (people/sq. mile). As indicated in Figure 5-24, W_{25} increases with increasing drainage area and decreasing population density. Note that in small catchments the reduction in the period of runoff due to the initial abstraction may be larger than the attenuation reflected in W_{25}, and D/D_R is estimated to be greater than one.

As an example consider a basin which has the following characteristics:

$A = 20{,}000 \text{ acres} = 31.25 \text{ sq. miles}$

$R_V = 0.50$

$PD = 8000 \text{ people/sq. mile}$

$V_d = 0.10 \text{ inches}$

$I = 0.06 \text{ in/hr}$

$D = 6 \text{ hr.}$

The uncorrected mean runoff is calculated as:

$$Q_r = R_V IA$$

$$= 0.50\ (0.06)\ 20{,}000$$

$$= 600 \text{ cfs}$$

Using the correction, however:

$$V_d/DI = 0.10/(6 \times 0.06)$$

$$= 0.28$$

$$D_e/D = 0.58 \text{ (from Figure 5-23)}$$

$$D_e = 0.58\ (6)$$

$$= 3.5 \text{ hr}$$

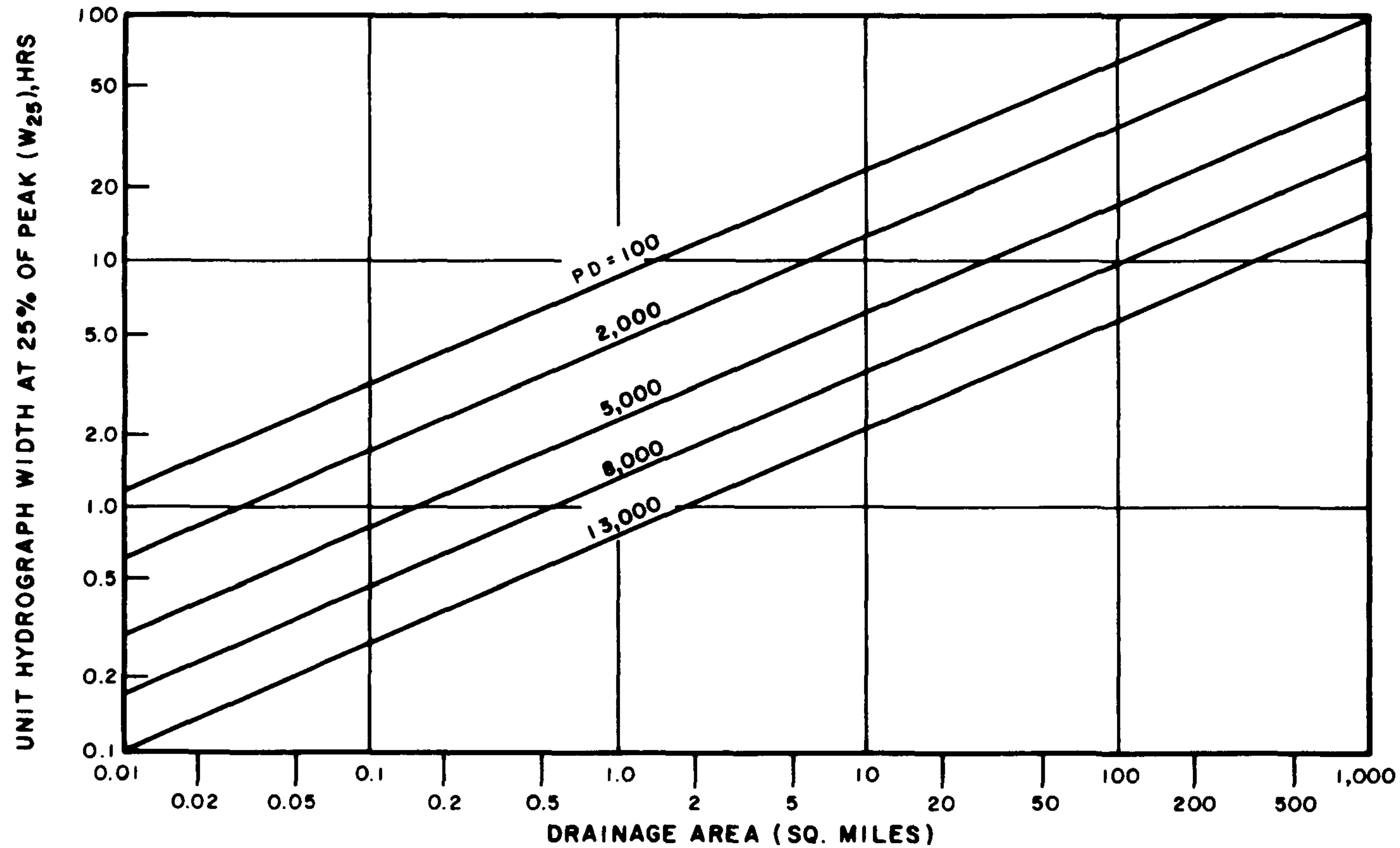
$$W_{25} = 5 \text{ hr (from Figure 5-24)}$$

$$D_R = De + W_{25}$$

$$= 3.5 + 5$$

$$= 8.5 \text{ hr}$$

$$Q_R = R_V IA\ (D/D_R)$$



FROM BRATER AND SHERRILL (30)

FIGURE 5-24
WIDTH AT 25 PERCENT OF PEAK VERSUS AREA-DESIGN CURVES

$$= 600\ (6/8.5)$$

$$= 425 \text{ cfs}$$

This estimate is presumably more representative of the mean runoff flow and more appropriate for initial load estimates and treatment device screenings. Of course if local runoff monitoring data are available, they are preferable to the estimating technique outlined in this section.

5.3 Runoff Quality

A variety of factors influence the quality of stormwater runoff and overflows. Precipitation falling through the atmosphere washes out both natural and man-made contaminants. While the contaminant concentration of rainfall may be important for specific pollutants in some situations (31,32, 33,34), the most significant portion of stormwater loadings are generally collected from the land surface and the conveyance system. The amount of pollutant washoff from the land surface is affected by the land use, vegetation cover, population density, activities which generate pollutants such as automobile traffic, litter, animal wastes, construction, transportation of materials, and fertilization. Pollutants discharged will also be influenced by activities which reduce pollutants such as street cleaning, road maintenance, erosion control, and general upkeep. The conveyance system may contribute a significant mass, particularly in combined sewer systems where the contaminants in the sanitary sewage mix with the runoff. The type of conveyance system (natural, separate, or combined) and its characteristics (slope, state of repair, tendency for deposition or stratification, channel stability, etc.) are important factors in influencing the quality of wet weather flows. Finally, storm properties may also affect the pollutant concentration of stormwater flows. Long intervals between storms allow greater dirt and dust accumulation on the land surface and larger depositions in the sewerage system. High intensity storms may dislodge and transport large quantities of solids and cause significant scour in the sewerage system while high volume storms may demonstrate a dilution effect if the source of contaminants is limited. Combinations of effects may occur due to variations in storm properties.

With the myriad of complex factors which influence the quality of stormwater runoff and overflows, it is very difficult to establish useful deterministic relationships for predicting appropriate pollutant concentrations. Furthermore, the great variability in observed concentrations, both within and between storms, and from one location to another, make any estimate highly uncertain. This section attempts to provide some guidance for evaluating stormwater pollutant concentrations in a study area. Reported values for average concentrations, its variability, and attempts to correlate runoff and overflow quality with land use are discussed.

5.3.1 Determination of Average Pollutant Concentrations

The most recent reviews of stormwater quality surveys throughout the country are presented in the Urban Stormwater Management and Technology reports prepared by Metcalf and Eddy, Inc. for the U.S. Environmental

Protection Agency (35,36). The results of the 1973 assessment (35) are shown in Table 5-5 and 5-6 for combined sewer overflows and separate stormwater runoff respectively. The results from the 1977 update (36) are shown in Table 5-7 and 5-8, for combined and separate runoff respectively. The studies are from diverse locations with different land uses and rainfall patterns. The results from both studies are summarized in Figure 5-25. For some of the pollutants, only a few study locations are represented. Figure 5-25 indicates the wide range of possible concentrations which may be encountered. Typical or conservative values for the average concentration, $\bar{c}$, may be chosen, but local monitoring and data are clearly needed to better refine estimates of stormwater quality in a particular area. The Urban Rainfall-Runoff Data Base compiled for the U.S. Environmental Protection Agency by the University of Florida (37) may be referred to for a detailed source of stormwater quality data from a number of studies.

5.3.1.1 Variability of Pollutant Concentrations

Figure 5-25 demonstrates the wide variation in stormwater quality from one location to another. Pollutant concentrations also vary considerably from storm to storm and within storms at a given site. To illustrate the typical level of variability between storms, data from five sites are analyzed for chemical oxygen demand (COD) and suspended solids (SS) concentrations. The average concentration for each storm is calculated and these are used to determine the overall average concentration, the standard deviation (reflecting between storm variability), and the coefficient of variation between storms (ν_c) at each site. The results are shown in Table 5-9. The coefficient of variation of stormwater quality (ν_c) is seen to range from about 0.50 to 1.00. For measurements of bacteria organisms, somewhat higher between storm variability is expected, although a portion of this may be due to the sampling error.

Runoff and overflow concentrations also vary markedly within a storm. The primary descriptive tool used to represent this variation is the first flush profile, although this describes only a portion of the within storm variability. To demonstrate the general applicability of the first flush profile, the results of the quality data normalization from the 1977 update of the Urban Stormwater Management and Technology report are reproduced in Figure 5-26. On the average, overflow and runoff concentrations of BOD are about three times higher in the first half hour of storms as compared to the later portions of events ($c_p/c_o = 3$) and the ratio for SS is about two to one ($c_p/c_o = 2$).

5.3.2 Effect of Land Use On Stormwater Quality

A number of conflicting claims have been made concerning the relationship between the land use of a drainage area and the concentration of pollutants in its stormwater runoff and overflows. Some maintain that strong correlations exist, while others conclude that little information about runoff quality may be inferred from land use. While most seem to feel that different land uses yield different runoff quality, the primary problems appear to be:

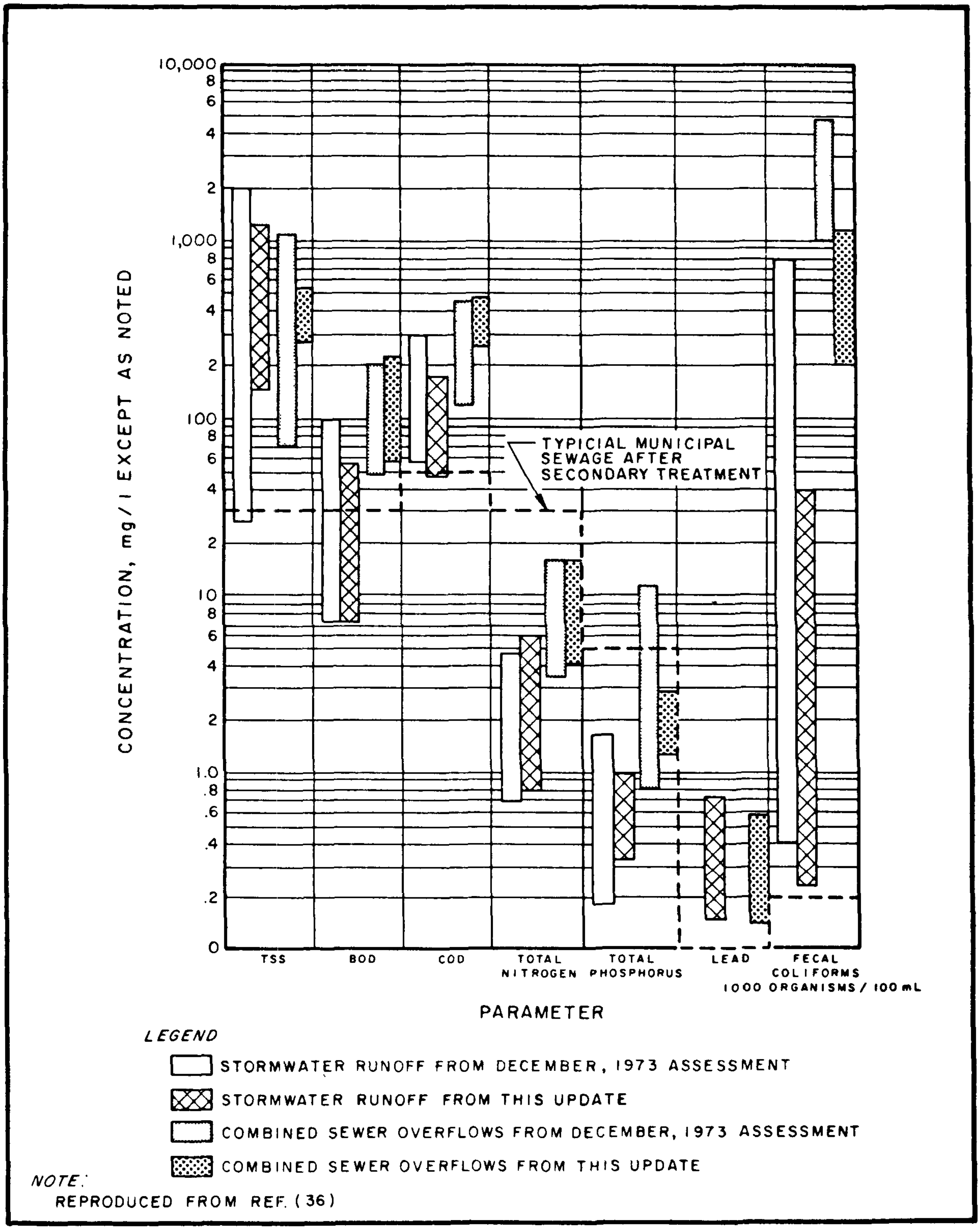
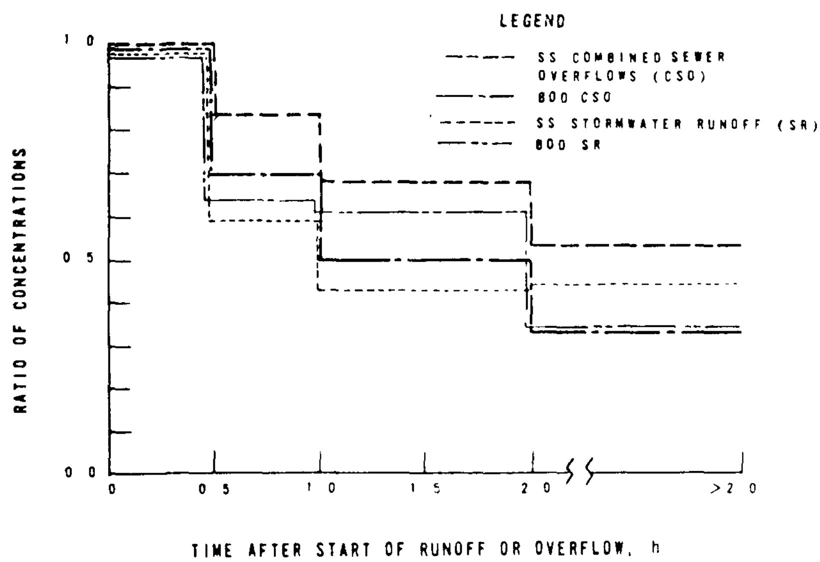


FIGURE 5-25

REPRESENTATIVE STORMWATER DISCHARGE QUALITY [2]



NOTE
REPRODUCED FROM FIGURE 4, REF. 36

FIGURE 5-26

TIME WEIGHTED NORMALIZATION

Type of wastewater, location, year, Ref. No.	BOD_5, mg/l		COD, mg/l		DO, mg/l	SS, mg/l		Total coliforms, MPN/100 ml		Total nitrogen, mg/l as N	Total phosphorus, mg/l as P
	Avg	Range	Avg	Range	Avg	Avg	Range	Avg	Range	Avg	Avg
Typical untreated municipal	200	100-300	500	250-750	--	200	100-350	5×10^7	1×10^7-1×10^9	40	10
Typical treated municipal											
Primary effluent	135	70-200	330	165-500	--	80	40-120	2×10^7	5×10^6-5×10^8	35	7.5
Secondary effluent	25	15-45	55	25-80	--	15	10-30	1×10^3	1×10^2-1×10^4	30	5.0
Selected combined											
Atlanta, Ga., 1969	100	48-540	--	--	8.5	--	--	1×10^7	--	--	1.2[b]
Berkeley, Calif.[c] 1968-69	60	18-300	200	20-600	--	100	40-150	--	--	--	--
Brooklyn, N.Y., 1972	180	86-428	--	--	--	1,051	132-8,759	--	--	--	1.2[b]
Bucyrus, Ohio 1968-69	120	11-560	400	13-920	--	470	20-2,440	1×10^7	2×10^5-5×10^7	13	3.5
Cincinnati, Ohio, 1970	200	80-380	250	190-410	--	1,100	500-1,800	--	--	--	--
Des Moines, Iowa, 1968-69	115	29-158	--	--	--	295	155-1,166	--	--	12.7	11.6
Detroit, Mich., 1965	153	74-685	115	--	--	274	120-804	--	--	16.3[d]	4.9
Kenosha, Wis., 1970	129	--	464	--	--	458	--	2×10^6	--	10.4[d]	5.9
Milwaukee, Wis., 1969	55	26-182	177	118-765	--	244	113-848	--	2×10^5-3×10^7	3-24	0.8[b]
Northampton, U.K., 1960-62	150	80-350	--	--	--	400	200-800	--	--	10[e]	--
Racine, Wis., 1971	119	--	--	--	--	439	--	--	--	--	--
Roanoke, Va., 1969	115	--	--	--	--	78	--	7×10^7	--	--	--
Sacramento, Calif., 1968-69	165	70-328	238	59-513	--	126	56-502	5×10^6	7×10^5-9×10^7[f]	--	--
San Francisco, Calif., 1969-70	49	1.5-202	155	17-626	--	68	4-426	3×10^6	2×10^4-2×10^7	--	--
Washington, D.C., 1969	71	10-470	382	80-1,760	--	622	35-2,000	3×10^6	4×10^5-6×10^6	3.5	1.0

a. Data presented here are for general comparisons only. Since different sampling methods, number of samples, and other procedures were used, the reader should consult the references before using the data for specific planning purposes.
b. Only orthophosphate.
c. Infiltrated sanitary sewer overflow.
d. Only ammonia plus organic nitrogen (total Kjeldahl).
e. Only ammonia.
f. Only fecal

NOTE:
REPRODUCED FROM TABLE II, REF.35, 1973 ASSESSMENT.

TABLE 5-5

COMPARISON OF QUALITY OF COMBINED SEWAGE FOR VARIOUS CITIES[a]

Type of wastewater, location, year, Ref No.	BOD5, mg/l		COD, mg/l		DO, mg/l	SS, mg/l		Total coliforms, MPN/100 ml		Total nitrogen, mg/l as N	Total phosphorus, mg/l as P
	Avg	Range	Avg	Range	Avg	Avg	Range	Avg	Range	Avg	Avg
Typical untreated municipal	200	100-300	500	250-750	--	200	100-350	5×10^7	1×10^7-1×10^9	40	10
Typical treated municipal											
Primary effluent	135	70-200	330	165-500	--	80	40-120	2×10^7	5×10^6-5×10^8	35	7.5
Secondary effluent	25	15-45	55	25-80	--	15	10-30	1×10^3	1×10^2-1×10^4	30	5.0
Storm sewer discharges											
Ann Arbor, Mich., 1965	28	11-62	-	--	--	2,080	650-11,900	--	--	3.5	1.7
Castro Valley, Calif., 1971-72	14	4-37	--	--	8.4	--	--	2×10^4	4×10^3-6×10^4	1.9[b]	--
Des Moines, Iowa, 1969	36	12-100	--	--	--	505	95-1,053	--	--	2.2	0.87
Durham, N.C., 1968	31	2-232	224	40-660	--	--	--	3×10^5	3×10^3-2×10^6[c]	--	0.18
Los Angeles, Calif., 1967-68	9.4	--	--	--	6.9	[illegible]	--	--	3×10^3-2×10^6	--	--
Madison, Wis., 1970-71	--	--	--	--	--	81	10-1,000	--	--	4.8	1.1
New Orleans, La., 1967-69[d]	12	--	--	--	4.5	26	--	1×10^6	7×10^3-7×10^8	--	--
Roanoke, Va., 1969	7	--	--	--	--	30	--	--	--	--	--
Sacramento, Calif., 1968-69	106	24-283	18	21-176	--	71	3-211	8×10^5	2×10^4-1×10^7[c]	--	--
Tulsa, Okla., 1968-69	11	1-39	85	12-405	--	247	84-2,052	1×10^5	1×10^3-5×10^8	0.3-1.5[e]	0.2-1.2[f]
Washington, D.C., 1969	19	3-90	335	29-1,514	--	1,697	130-11,280	6×10^5	1×10^5-3×10^6	2.1	0.4

a Data presented here are for general comparisons only. Since different sampling methods, number of samples, and other procedures were used, the reader should consult the references before using the data for specific planning purposes.
b Only ammonia plus nitrate.
c Only fecal.
d Median values from 1 sampling station.
e Only organic (Kjeldahl) nitrogen.
f. Only soluble orthophosphate.

NOTE:
REPRODUCED FROM TABLE 12, REF. 35, 1973 ASSESSMENT.

TABLE 5-6

COMPARISON OF QUALITY OF STORM SEWER DISCHARGES FOR VARIOUS CITIES[a]

	Average pollutant concentration, mg/L									
	TSS	VSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	PO_4-P	OPO_4-P	Lead	Fecal coliforms[a]
Des Moines, Iowa	413	117	64	. .		4.3	1 86	1 31	...	...
Milwaukee, Wisconsin	321	109	59	264	4 9	6 3	1 23	0 86	...	...
New York City, New York										
Newtown Creek	306	182	222	481	...	....	....	...	0 60	....
Spring Creek	347	...	111	358	.	16.6	4 5[b]		....	....
Poissy, France[c]	751	387	279	1005	..	43	17[b]	....	....	....
Racine, Wisconsin	551	154	158	...	...	....	2.78	0 92	...	201
Rochester, New York	273	.	65	...	2.6	...	.....	0.88	0 14	1140
Average (not weighted)	370	140	115	367	3.8	9.1	1 95	1 00	0 37	670
Range	273-551	109-182	59-222	264-481	2.6-4 9	4 3-16.6	1.23-2 78	0 86-1 31	0.14-0.60	201-1140

a. 1000 organisms/100 mL.

b. Total P (not included in average).

c. Not included in average because of high strength of municipal sewage when compared to the United States.

NOTE:

REPRODUCED FROM TABLE 33, REF. 36 , 1977 ASSESSMENT.

TABLE 5-7

POLLUTANT CONCENTRATIONS IN COMBINED SEWER OVERFLOWS

City	TSS	VSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	Phos-phorus	OPO_4-P	Lead	Fecal coliforms[a]
	Average pollutant concentrations, mg/L									
Atlanta, Georgia	287	..	9	48	0 57	0.82	0 33	....	0 15	6 300
Des Moines, Iowa	419	104	56	. .	2 09	3 19	0 56	0 15	....	......
Durham, North Carolina	1 223	122	..	170	0 96	. . .	0 82	....	0 46	230
Knoxville, Tennessee	440	..	7	98	1 9	2 5	0 63	0.30	0 17	20 300
Oklahoma City, Oklahoma	147	..	22	116	2 08	3 22	1 00	1 00	0 24	40 000
Tulsa, Oklahoma	367		12	86	0 85	..	.	0 38	....	420
Santa Clara, California	284	70	20	147	..	5 8	0 23	. .	0 75	......
Pullach, Germany	158	53	11	125	. .	...	. ..	..	....	......
Average (not weighted)	415	88	20	113	1 41	3 11	0 62	0 46	0.35	13 500
Range	147-1 223	53-122	7-56	48-170	0 57-2 09	0 82-5 8	0 33-1 00	0 15-1 00	0 15-0 75	230-40 000

a Organisms/100 mL

NOTE:
REPRODUCED FROM TABLE 21, REF.36 , 1977 ASSESSMENT.

TABLE 5-8
POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF

TABLE 5-9

VARIABILITY OF RUNOFF AND OVERFLOW CONCENTRATIONS
(Between Storms)

Location	Conveyance Type	Number of Storms	COD Mean (mg/l)	COD Std. Dev. (mg/l)	COD ν_c	SS Mean (mg/l)	SS Std. Dev. (mg/l)	SS ν_c	Ref.
Durham, N.C.	Separate	26	182	89	0.49	1,198	902	0.75	(20)
Lubbock, Texas	Separate	11	365	283	0.78	1,056	630	0.60	(38)
Milwaukee, Wis.	Combined	13	339	184	0.54	338	225	0.67	(39)
Newark, N.J. (1)	Combined	11	395	338	0.86	251	275	1.10	(40)
Kearny, N.J. (2)	Combined	11	269	205	0.76	137	63	0.46	(40)

(1) City Dock, Newark

(2) Ivy Street, Kearny

1. Isolating the effects of land use from other sources of variability in the runoff concentration.

2. Establishing a statistically significant relationship.

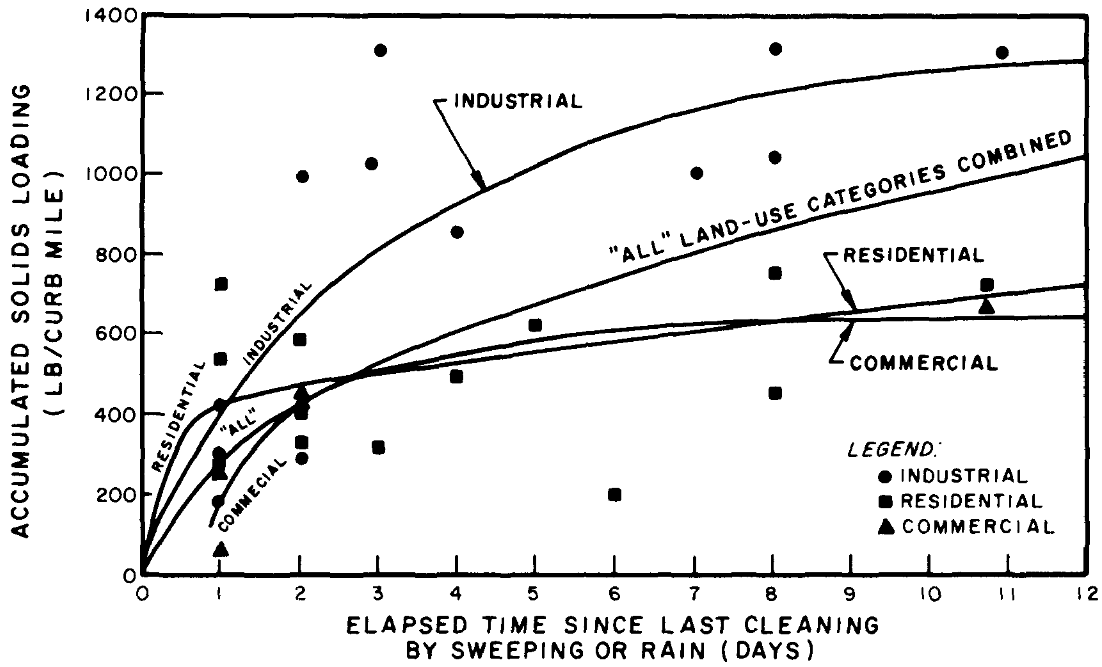
This section presents a review of some of the work that has appeared in the literature relative to these problems.

Study of Street Surface Contaminants:

Sartor and Boyd present the results of an extensive runoff surveying program in "Water Pollution Aspects of Street Surface Contaminants" (41). Devices were used to simulate rainfall and samples were collected and analyzed at a large number of test sites. Their results indicate differences between land use types:

> Contaminant loading intensities were found to vary with respect to land-use patterns in the surrounding locale. In general, industrial areas have substantially heavier than average loadings. All industrial test sites (20 of them) taken together have an average loading of some 2800 lb/curb mile; twice the mean for cities on the whole. This is probably because industrial areas tend to be swept less often and because generation rates of dust and dirt tend to be high (e.g., "fall-out", spillage from vehicles, unpaved dirt areas, streets in poor conditions, etc.). Of these, heavy industrial areas showed the heaviest loadings, medium industrial the lightest. The loadings varied so widely between individual sites that it would be speculative to state why one type of industrial area is dirtier than another.
>
> Commercial areas have substantially lighter loading intensities than the mean for cities on the whole (290 lb/curb mile average vs. 1400). This is probably because they are swept so often; typically several times weekly, daily in prime areas.
>
> Residential areas were found to have an average loading intensity comparable to the average for all land uses of all cities taken together: 1200 lb/curb mile. Here again, the loadings varied widely from site to site, and it would be speculative to state why one city is more heavily loaded than another or why one type of residential neighborhood is cleaner than another. The data implies, however, that there is some tendency for newer, more affluent neighborhoods to be cleaner; possibly because they are better maintained by residents and/or are further from sources of contamination (41, p. 6).

Since the effects of street sweeping frequency are hypothesized to be a factor in the differences in loading rates between the land use types an attempt was made to separate the land use and street sweeping effects by calculating the accumulation rate since the last rain or sweeping at each site. The results summarized in Figure 5-27, show there are still some differences due to land use. The writers conclude that, "In general, industrial land-use areas tend to accumulate contaminants faster than commercial



REPRODUCED FROM REF (41)

FIGURE 5-27

TIME SINCE LAST CLEANING VERSUS SOLIDS LOADING or residential areas." (41, p. 4). However, the large scatter of the data upon which the curves in Figure 5-27 are based weakens the conclusion.

URS Research Company Study:

In studies stemming from the previous report, the URS Research Company analyzed storm runoff data from fifteen cities in the United States in their report, "Water Quality Management Planning for Urban Runoff" (42). The data were gathered from published reports and analyzed to investigate the effects of climate (as determined by the geographic sector in which the city is located), land use, type of street surface, traffic density, and the type of surrounding landscape.

The writers analyzed the data by grouping the records according to the independent parameters (climate, land use, etc.), calculating the mean and other basic statistics of the dependent variables (runoff quality parameters) and testing the differences between these means for statistical significance using the Student's t test. The results are summarized in Table A-24 of Appendix A of the URS report. Loading rates are given in terms of the lbs/curb mile/day of total solids. Loading rates for particular pollutants are then determined from the ratio of pollutant mass to mass of dry solid. Examples of the writers' conclusions concerning the relationship between land use and the solids loading rate include

> ...(1) commercial and residential area means are different at the 97 percent confidence level, (2) commercial and light industrial area means are different at only the 73 percent confidence level, (3) commercial and heavy industrial area means are different at the 95 percent confidence level, and (4) the commercial mean differs from the mean of the set of all data at the 98 percent confidence level (42, P. A-32).

As found by Sartor and Boyd, industrial land uses have the highest solids loading rate, followed by residential, commercial, and open space, in that order.

Although the URS report does find that the grouping of the data indicates differences in the street surface runoff from different land uses, the writers temper their conclusions about the nature of this relationship in their summary:

> The URS staff has assembled all presently available data on the rates of accumulation of solids and the concentrations of various constituents in those solids that collect on street surfaces. The range and scatter in the available data are extreme. Both the sampling variability and the complexity of natural systems contributed to this extreme variance. Attempts to test conceptual models with the available data have met with absolutely no success because the models are much too simple to adequately describe the conditions in the real world (42, p. A-43).

University of Florida Study:

A study team from the University of Florida presents a general procedure for predicting runoff quality from land use and population density in "Storm Water Management Model, Level I, Preliminary Screening Procedures" (29). The writers comment on the variability of the data encountered in formulating their procedure and present the methodology as follows:

> It is unfortunate that perhaps the only consistent remark about urban runoff quality analysis in general is that data and results of previous studies are so remarkably inconsistent. Few studies have been made of characteristics of street liter, and they offer a wide range of values of concentrations and loads. Effluent data show a similar scatter. However, it is necessary that a decision be made regarding actual values for use in the analysis. Table 8 (shown as Table 5-10) presents a predictive equation developed after a review of available stormwater pollutant loading and effluent concentration data (1). The equation permits one to estimate BOD_5, SS, VS, PO_4 and N loads as a function of land use, type of sewer system, precipitation, population density, and street sweeping frequency. Loading in combined sewer areas are assumed to be 4.12 times as large as loadings in separate areas. They are assumed to vary as a function of developed population density as shown in Figure 4 (shown as Figure 5-28). The intercept (0.142) was determined based on data for open space. The exponent (0.54) is based on the exponent of the imperviousness equation at a population density of 8 persons per acre (20 persons per ha) such that pollutant concentration increases as a function of population density. Lastly, the coefficient (0.218) is based on an average of data points with a PD_d ranging from 5 to 15 persons per acre (12 to 37 persons per ha) to yield a value of f_2 (PD_d) of 0.895 at 10 persons per acre (25 persons per ha). (Note the large spread in the observed data). The street sweeping relationship was derived by making numerous runs of STORM with varying street sweeping frequencies. The results are shown in Figure 5 (shown as Figure 5-29) (29, p. 16).

The results and relationships presented in the University of Florida study are directed towards the effect of land use on runoff quality. Due to the wide variability in observed data, however, the results are only applicable for preliminary estimates when local data are not available.

Atlanta Study:

In stormwater studies conducted in Atlanta, "Storm and Combined Sewer Pollution Sources and Abatement, Atlanta, Georgia" (43), attempts are made to correlate both combined and separate sewer runoff to local land use and population indicators. Initially, six land use classifications are employed with a linear regression analysis to predict the quality from six drainage areas. To achieve significant results, the dependent variables are simplified to include only population density and the percent undeveloped land. Their results are summarized as follows:

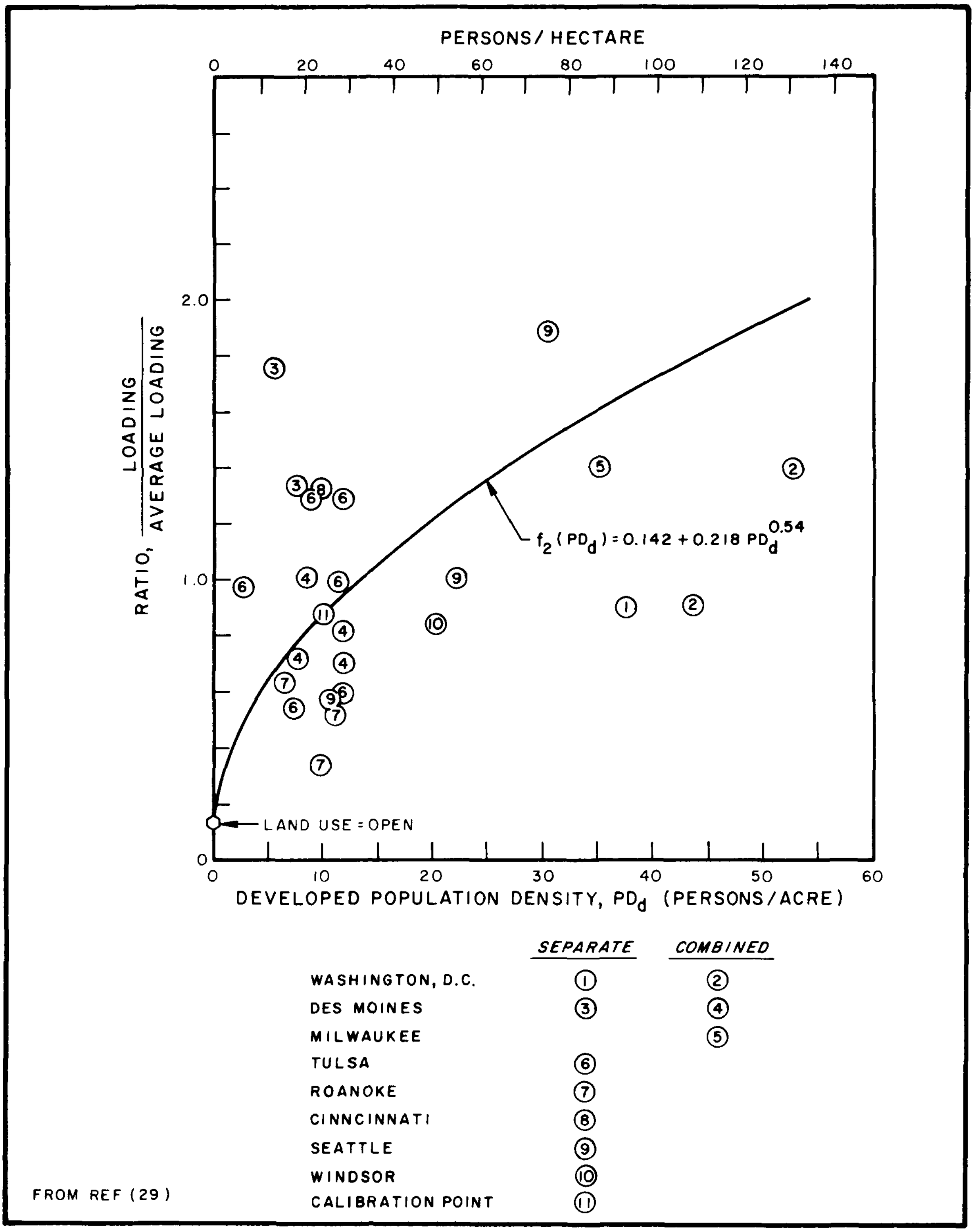
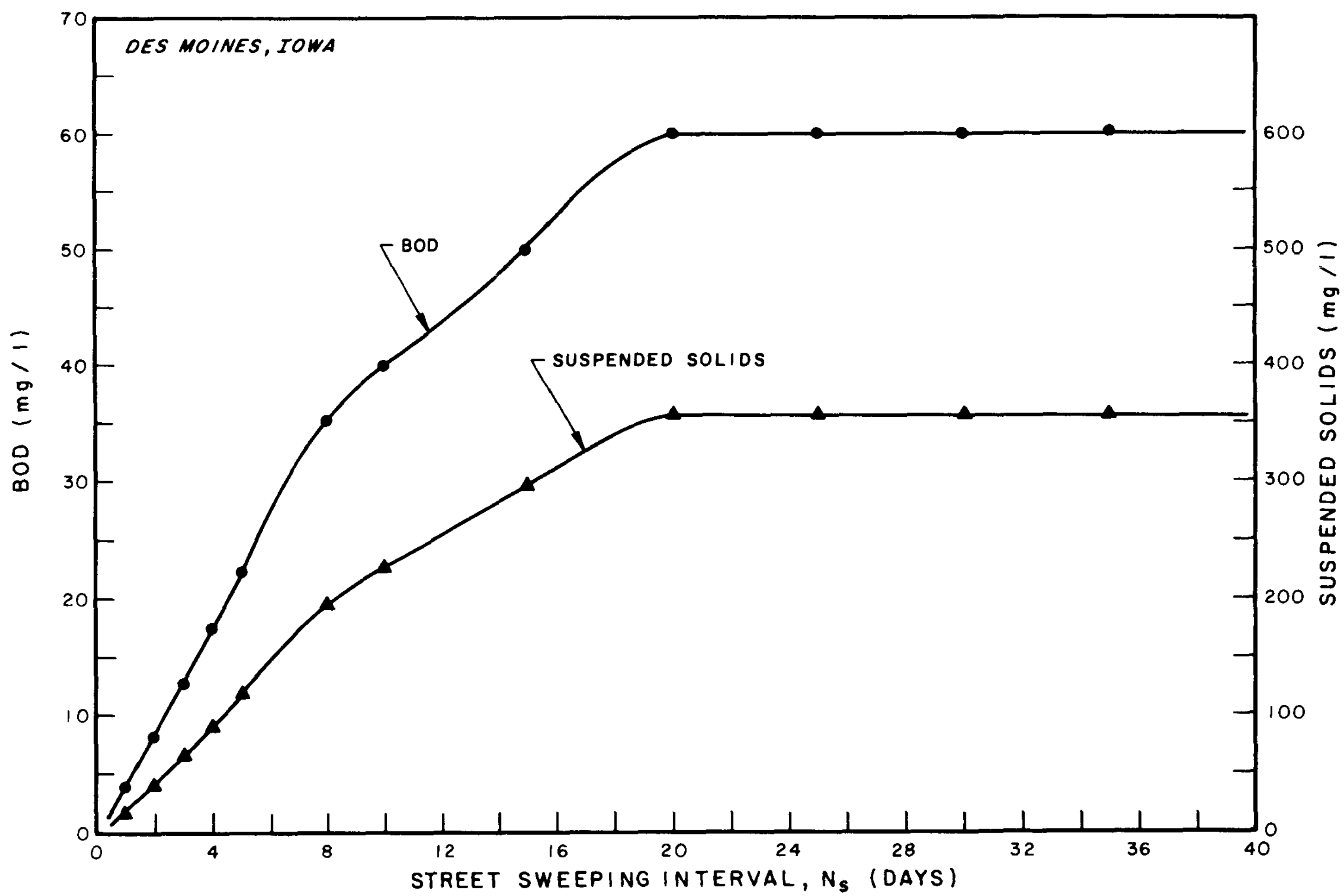


FIGURE 5-28

NORMALIZED BOD LOADINGS
VERSUS DEVELOPED POPULATION DENSITY



FROM REF. (29)

FIGURE 5-29

EFFECT OF STREET SWEEPING FREQUENCY
ON ANNUAL BOD CONTRATION IN URBAN STORMWATER RUNOFF

The following equations may be used to predict annual average loading rates as a function of land use, precipitation and population density.

Separate Areas: $M_s = \alpha(i,j) \cdot P \cdot f_2(PD_d) \cdot \gamma \frac{lb}{acre\text{-}yr}$

Combined Areas: $M_c = \beta(i,j) \cdot P \cdot f_2(PD_d) \cdot \gamma \frac{lb}{acre\text{-}yr}$

where
- M = pounds of pollutant j generated per acre of land use i per year,
- P = annual precipitation, inches per year,
- PD_d = developed population density, persons per acre,
- α, β = factors given in table below,
- γ = street sweeping effectiveness factor, and
- $f_2(PD_d)$ = 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:
- i = 1 $f_2(PD_d) = 0.142 + 0.218 \cdot PD_d^{0.54}$
- i = 2,3 $f_2(PD_d) = 1.0$
- i = 4 $f_2(PD_d) = 0.142$

Factors α and β for Equations: Separate factors, α, and combined factors, β, have units lb/acre-in. To convert to kg/ha-cm, multiply by 0.442.

	Land Use, i	Pollutant, j: 1. BOD_5	2. SS	3. VS	4. PO_4	5. N
Separate Areas, α	1. Residential	0.799	16.3	9.45	0.0336	0.131
	2. Commercial	3.20	22.2	14.0	0.0757	0.296
	3. Industrial	1.21	29.1	14.3	0.0705	0.277
	4. Other	0.113	2.70	2.6	0.00994	0.0605
Combined Areas, β	1. Residential	3.29	67.2	38.9	0.139	0.540
	2. Commercial	13.2	91.8	57.9	0.312	1.22
	3. Industrial	5.00	120.0	59.2	0.291	1.14
	4. Other	0.467	11.1	10.8	0.0411	0.250

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

$$\gamma = \begin{cases} N_s/20 & \text{if } 0 \le N_s \le 20 \text{ days} \\ 1.0 & \text{if } N_s > 20 \text{ days} \end{cases}$$

NOTE:
REPRODUCED FROM TABLE 8, REF. (29)

TABLE 5-10
POLLUTANT LOADING FACTORS FOR DESKTOP ASSESSMENT

> For combined sewer areas, land use may not be the appropriate primary variable upon which to base pollution load estimates. For all but initial scouring flows and high flow rates, pollution characteristics depend primarily upon dilution of domestic and industrial waste entering the system. The strength of this waste determines the BOD concentration in the overflow. The pertinent parameter for correlation would therefore be the population equivalent of the domestic and industrial waste, which would entail a detailed analysis of industrial waste discharges within each combined area. Storm sewer areas and combined sewer areas at high flow rates yield BOD concentrations that may well depend upon land use more than upon population; however, further data will be required to determine the nature of the relationship (43, p. 110).

The differences in approach that is necessary for combined and separate sewer areas are demonstrated by these findings.

Lubbock Study:

In a study in Lubbock, Texas, "Variation of Urban Runoff Quality and Quantity with Duration and Intensity of Storms -- Phase III" (44), researchers from Texas Tech University examined the effect of storm characteristics and antecedent conditions on runoff quality in a single drainage area using multiple regression and lag regression techniques. Although they did not examine land use effects, the difficulties they encountered shed light on this problem as well:

> ... the extreme range in concentration of constituents contained in runoff makes it unlikely that anyone will ever be able to predict urban runoff quality with a high degree of accuracy. Variations in concentrations of one to two orders of magnitude are common, and analysis of all data that was available in August, 1971 from the watershed in Lubbock showed standard deviations in concentrations of all constituents normally being in the range of two-thirds to three-quarters of the mean concentration (41, p. 12).

Rochester Study:

Runoff quality from combined sewer overflows in Rochester, New York, is examined by Metcalf and Eddy, Inc. in "Development and Application of a Simplified Stormwater Management Model" (12). Logarithmic regression analysis is first used to relate storm and antecedent conditions to runoff quality during 29 storms with two to seven subareas averaged for each storm. The results are probably not statistically significant with a correlation coefficient of 0.257 for the equation predicting the chemical oxygen demand concentration of the overflows, 0.181 for the equation predicting total suspended solids, and 0.487 for the equation predicting nitrogenous oxygen demand (where the NOD concentration was found to be negatively correlated with the number of days since the last rain). The F statistic and the resulting level of significance are not presented. The writers state that, "The correlation coefficients are not high because stormwater quality is highly variable, and the data that were developed from the sampling program had some major irregularities" (12, p. 65).

The Rochester data are also analyzed to examine the effects of land use:

> Attempts were also made to develop equations including population density to reflect the impact of land use patterns on stormwater quality. These equations were correlated with Data Set 2. The correlation coefficient indicated that there was essentially no correlation between the measured values and the predicted values from these equations. This lack of correlation may be because of irregularities in the data and because of the particular blend of land uses in the City of Rochester (12, p. 66).

The difficulties encountered in obtaining significant correlations lead the writers to suggest a conceptually simpler approach of analyzing the average quality of subareas:

> These subarea averages can be ranked to indicate trends. The significant land use or surface characteristics can also be ranked. If these rankings are indicated on a simplified map of the study area, areal trends in overflow quality can be noted (12, p. 66).

The complexity and variability of the quality-land use relationships, particularly in a combined sewer area such as Rochester appear to justify these simpler attempts to gain some level of insight, rather than sophisticated efforts to predict actual concentrations.

Urban Stormwater Management and Technology Update:

A number of studies are cited in the recent update of the Urban Stormwater Management and Technology report (36) which are directed towards identifying correlations between runoff quality and land use or storm properties. These include analyses of separate runoff in Atlanta, Georgia, Tulsa and Oklahoma City, Oklahoma, and Santa Clara County, California. Some water quality parameters are found to be correlated with land use or storm properties.

Land Use - Receiving Water Studies:

The concern for the water quality effects of land use has also led to studies relating average, instream quality to land use and watershed activity indicators. Haith found, "... significant correlations between land uses and average nitrogen, phosphorus, and suspended solids concentrations..." (45, p. 9) in his study of New York State streams.

A Carnegie-Mellon University study of 52 streams in the Pittsburgh, Pennsylvania area (46) employed discriminant analysis to predict whether a stream would be likely to violate applicable water quality standards based on indirect indicators of watershed activities such as land use, the fraction of the developed land unsewered, population, municipal and industrial activity, inactive mines, etc. The writers conclude that, "... watershed activity indicators not only provide a measure of overall water quality, but also a reliable mechanism for predicting which water quality parameters are likely to be problematic" (46, p. 249).

Although these studies deal with receiving stream quality, rather than runoff or overflow concentrations, the techniques employed and their results are of interest.

Conclusions and Implications:

A survey of the available information and a range of perspectives on the relationship between land use and stormwater runoff quality has been presented. A number of conclusions emerge from this review:

1. It would be desirable to establish some relationship between land use ard stormwater quality to permit evaluation of the effect of future land use changes and certain management practices on runoff quality.

2. Although nationwide studies have found significant differences in runoff quality from different land uses, the variability of the data precludes the establishment of a good general characterization of quality as a function of land use, and makes the application of the results to any particular area quite tenuous. Perhaps the clearest implication is that local data is necessary.

3. When investigating land use effects in a particular study area, conceptually simple approaches, such as looking for grouped averages or discriminant analysis of groupings, may yield more insight than attempts to predict actual runoff concentrations. Land use effects in combined sewer areas should be particularly difficult to identify because of the predominant influence of sanitary sewage on the quality of overflows. Finally, the analyst should anticipate that due to the complexity and uncertainty involved in the stormwater runoff process, that even for separate sewers and with a large local database, significant correlations between land use and stormwater runoff quality may not be evident.

5.4 Receiving Water

The methodologies suggested for initial assessments of receiving water quality require a number of estimates for parameters, coefficients, and rates. These include transport properties such as flow and dispersion, coefficients to describe the reaction of pollutants, and estimates of the background quality of the receiving water. Guidelines for estimating these parameters are presented in the following sections.

5.4.1 Transport Properties

To assess the impact of stormwater loads, an estimate of the average flow upstream of the loading area is required. In any drainage basin, stream flow will vary over the year depending on the rainfall and/orzsnowmelt. Figure 5-30 shows the average monthly distribution of runoff as a percent of the total runoff over the year for 16 river basins in the United States. It is interesting to note that in the Kissimmee River Basin in Florida, the stream flow is relatively constant over the year. However, in the Yellowstone River Basin, Montana, the maximum monthly stream flows are about 10

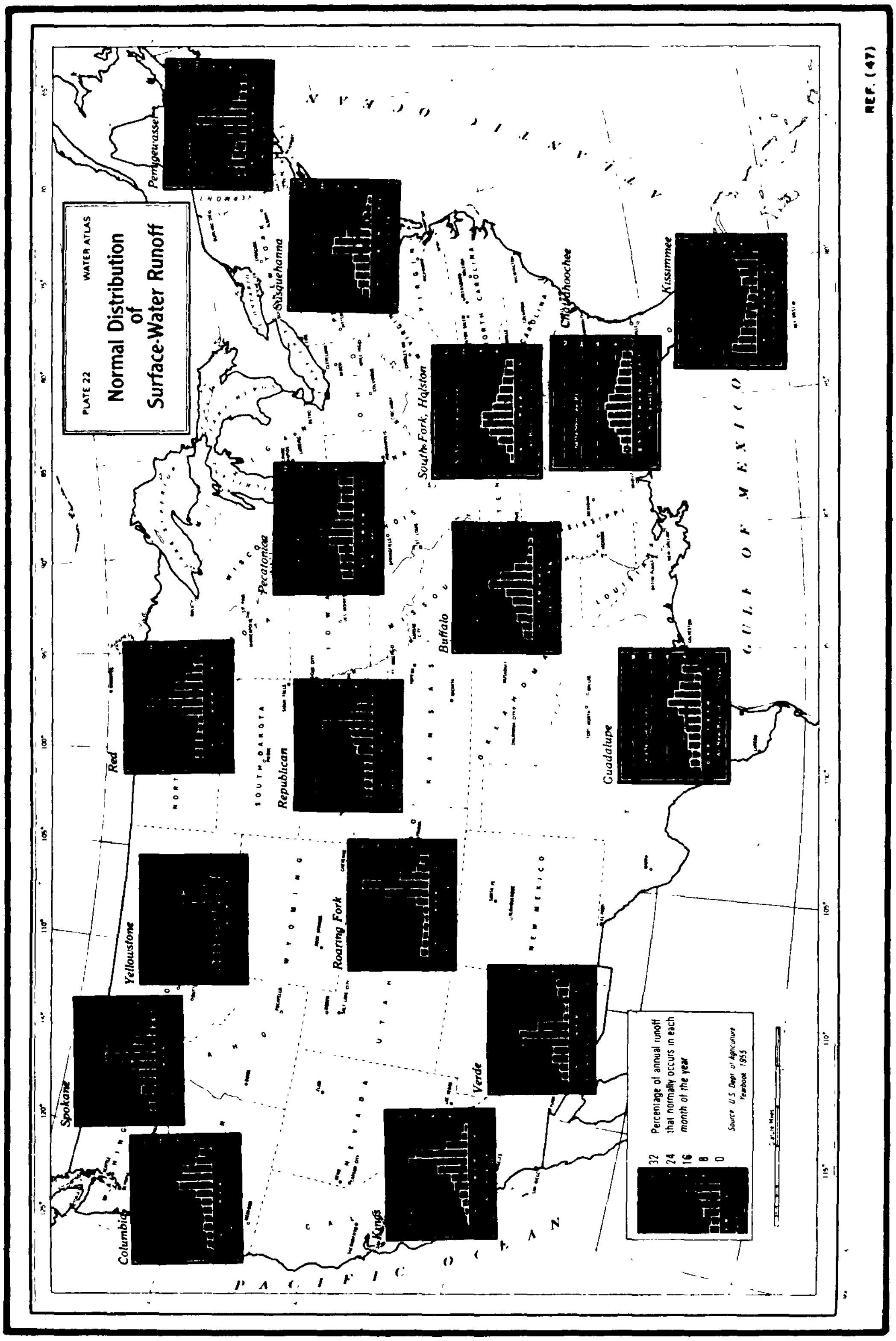


PLATE 22
WATER ATLAS
Normal Distribution of Surface-Water Runoff
Percentage of annual runoff that normally occurs in each month of the year
32
24
16
8
0
Source U.S. Dept. of Agriculture Yearbook 1955
Pemigewasset
Susquehanna
Kissimmee
Chattahoochee
South Fork, Holston
Pecatonica
Buffalo
Red
Republican
Guadalupe
Yellowstone
Roaring Fork
Spokane
Columbia
Kings
Verde
ATLANTIC OCEAN
GULF OF MEXICO
PACIFIC OCEAN
REF. (47)

FIGURE 5-30

times as great as the minimum monthly stream flows. In addition, the Yellowstone River has a minimum stream flow which occurs in the month of February by contrast to the majority of streams in the country, in which low flow occurs in the late summer or early fall.

Because of the variation in runoff across the country, it is usually necessary to collect site specific stream flow data. In general, there will be one or more flow gaging stations within the 208 planning boundaries. These records are published as annual surface water reports by the U.S.G.S. Table 5-11 is a sample data sheet from the U.S.G.S. surface water records. The data sheet summarizes the drainage area, daily flows for the year, monthly average flows, and monthly maximum and minimum flows at the gaging station.

For ungaged streams, an estimate of the average stream flow can be made using Figure 5-31, which shows the distribution of average annual runoff yield. Preliminary high and low flow runoff estimates can be made with the use of the monthly runoff distributions presented in Figure 5-30.

In tidal rivers and estuaries, the freshwater flow is estimated by adding the flows orginating from tributaries, municipal and industrial point sources, and surface and groundwater inflow. While the freshwater flow in tidal rivers and estuaries results in a net seaward flow the freshwater velocity in tidal waters is generally small compared to the average tidal velocity. Tidal velocity information is available from the National Ocean Survey, which is a branch of the National Oceanic and Atmospheric Administration. Tidal velocity information is used to calculate the atmospheric reaeration coefficient and the average tidal translation which are required for the water quality impact analysis.

5.4.1.1 Channel Geometry

Stream channel depth and cross-sectional area data are also required for the main receiving water body. This information may be available from the U.S.G.S., state agencies, EPA Coast and Geodetic Surveys, or from previous studies. If no depth or cross-sectional area data are available, it is usually necessary to make in-stream measurements.

In some study areas, time of passage (travel time) information will be available from the U.S.G.S. or from previous studies. Time of passage data along with stream cross-sectional area and depth data are used for computing the average velocity. The time of passage data and the channel cross-sectional area data should be related to the stream flow. If insufficient data are available to establish these relationships, then Equations 5-10 to 5-13 can be used to estimate the changes in river characteristics under different flow regimes.

$$\frac{a_2}{a_1} = \left(\frac{Q_2}{Q_1}\right)^{0.5} \qquad (5\text{-}10)$$

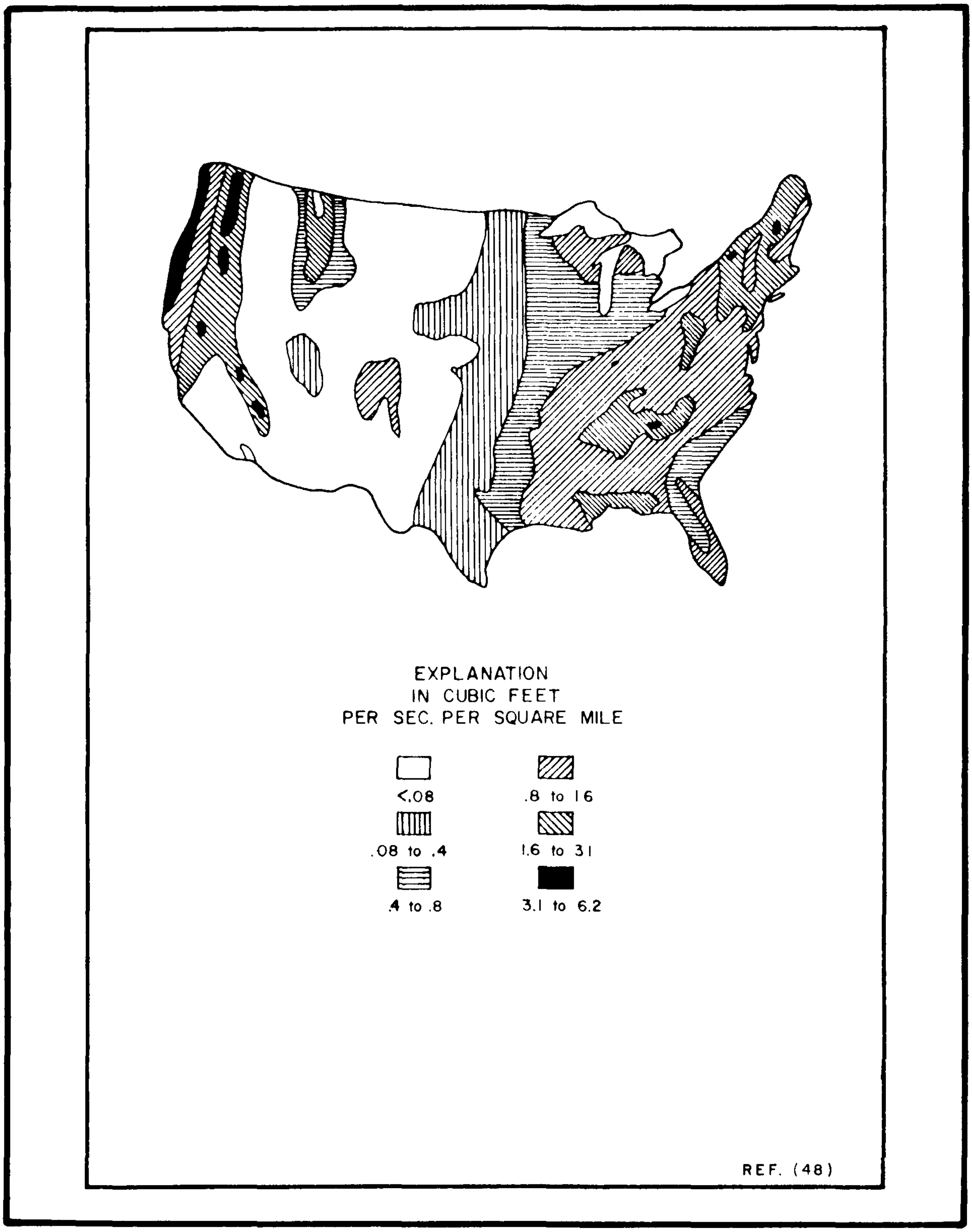


FIGURE 5-31

AVERAGE RUNOFF YIELDS IN THE COTERMINOUS UNITED STATES

STREAMS TRIBUTARY TO LAKE ONTARIO

04260500 BLACK RIVER AT WATERTOWN, N.Y.

LOCATION.--Lat 43°59'08", long 75°55'30", Jefferson County, on downstream side of right abutment of Vanduzee Street Bridge at Watertown, 3.5 mi (5.6 km) upstream from Philomel Creek.

DRAINAGE AREA.--1,876 mi² (4,859 km²).

PERIOD OF RECORD.--July 1920 to current year.

GAGE.--Water-stage recorder. Datum of gage is 374.88 ft (114.263 m) above mean sea level. Prior to Sept. 3, 1921, nonrecording gage at same site and datum.

AVERAGE DISCHARGE.--54 years, 3,891 ft³/s (110.2 m³/s).

EXTREMES.--Current year: Maximum discharge, 19,400 ft³/s (549 m³/s) Mar. 8, Apr. 7, maximum gage height, 8.13 ft (2.478 m) Mar. 8, minimum, 116 ft³/s (3.28 m³/s) July 21 (gage height, 0.27 ft or 0.082 m), minimum daily, 1,080 ft³/s (30.6 m³/s) Aug. 25.
Period of record: Maximum discharge, 36,700 ft³/s (1,040 m³/s) Apr. 5, 1963 (gage height, 11.57 ft or 3.527 m), minimum, 10 ft³/s (0.28 m³/s) Sept. 2, 1934 (gage height, -0.19 ft or -0.058 m), minimum daily, 137 ft³/s (3.88 m³/s) Sept. 4, 1939.
Maximum discharge known, about 39,700 ft³/s (1,120 m³/s) Apr. 23, 1869 (from New York State Museum Bulletin 85).

REMARKS.--Records fair. Flow regulated by Stillwater Reservoir (see station 04256500), Fulton Chain of Lakes (see station 04253500), and other reservoirs. Extensive diurnal fluctuation at low and medium flow caused by mills and powerplants in and above Watertown. During canal season, water is diverted out of basin through Forestport feeder and Black River Canal (flowing south), see station 04252000. Water-quality records for the current year are published in Part 2 of this report.

REVISIONS.--WSP 759: Drainage area.

DISCHARGE, IN CUBIC FEET PER SECOND, WATER YEAR OCTOBER 1973 TO SEPTEMBER 1974

DAY	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	1,410	2,020	6,870	11,900	8,890	6,130	4,090	6,230	3,600	2,080	4,350	1,870
2	1,520	3,890	6,470	9,840	8,290	5,720	4,620	6,600	3,540	2,620	3,880	1,420
3	1,620	5,310	5,530	8,160	6,840	5,100	7,520	6,600	3,500	2,420	2,960	1,780
4	2,200	5,490	4,600	6,930	5,790	5,720	16,700	6,400	3,500	3,940	2,560	1,890
5	2,320	5,170	4,600	6,200	4,780	9,910	15,700	6,180	3,280	4,260	3,960	2,710
6	2,470	4,470	5,400	5,580	4,490	11,300	18,900	5,860	3,020	4,420	5,570	2,610
7	2,710	3,760	6,730	4,780	3,740	16,700	20,400	6,150	2,800	4,130	5,760	2,190
8	2,750	3,300	7,930	4,470	3,580	18,700	15,600	6,650	2,270	3,280	5,240	2,130
9	2,680	3,010	8,790	4,490	3,520	15,400	12,600	7,040	2,140	2,660	4,070	2,220
10	2,180	2,850	10,800	4,450	3,340	12,800	11,200	8,670	2,080	2,930	2,890	1,360
11	1,730	2,560	11,000	4,510	3,120	10,400	9,030	10,200	3,040	2,690	2,450	1,220
12	1,850	2,480	11,000	4,620	3,070	8,750	8,250	12,200	4,530	3,000	1,630	1,130
13	1,850	2,530	10,200	4,340	3,070	7,020	8,890	13,300	4,400	2,530	1,920	1,100
14	1,930	2,580	8,650	3,740	3,100	5,680	8,440	14,400	3,720	1,900	1,710	1,160
15	1,990	2,530	6,990	3,170	3,250	5,220	11,700	15,300	3,160	1,860	1,450	1,730
16	1,880	4,360	5,510	3,420	2,990	4,570	13,700	13,700	3,100	1,860	1,320	1,710
17	1,950	5,770	3,890	3,740	2,710	4,440	17,200	12,000	3,800	1,370	1,330	1,510
18	1,860	6,150	2,780	3,760	2,700	4,220	13,900	10,400	4,310	1,330	1,280	1,360
19	1,860	6,100	3,580	3,480	2,870	4,370	12,000	9,140	3,760	1,440	1,710	1,680
20	2,070	5,560	3,820	3,320	2,760	4,240	10,400	8,390	3,260	1,380	2,130	1,920
21	1,990	4,490	4,820	2,890	3,300	4,900	9,470	7,520	2,960	1,260	1,810	1,700
22	2,050	3,560	5,890	3,120	3,500	3,660	8,530	6,620	2,690	1,760	1,440	2,050
23	2,050	3,480	6,580	4,450	6,130	3,780	9,380	5,760	2,500	1,660	1,250	3,060
24	1,910	3,500	8,290	5,600	6,250	3,660	8,640	5,270	2,480	1,630	1,180	3,240
25	1,780	3,440	8,990	6,050	6,670	4,150	9,800	5,100	2,580	1,650	1,080	2,540
26	1,520	4,360	10,600	6,250	6,990	3,460	9,440	5,080	2,530	1,440	1,170	2,240
27	1,600	4,640	13,500	7,050	7,140	3,720	8,610	4,880	2,540	1,520	1,360	2,180
28	1,600	4,800	15,300	9,380	6,730	3,200	7,580	4,570	2,800	1,920	1,210	2,080
29	1,600	5,790	17,600	9,380	------	3,480	6,400	4,130	2,530	2,580	1,300	1,890
30	1,700	6,580	17,600	9,700	------	3,440	6,180	3,880	2,160	3,560	1,570	2,080
31	1,820	------	14,700	9,770	------	4,000	------	3,780	------	4,000	1,840	------
TOTAL	60,650	124,530	259,010	178,540	129,610	207,840	324,870	242,000	92,580	75,080	73,380	57,760
MEAN	1,956	4,151	8,355	5,759	4,629	6,705	10,830	7,806	3,086	2,422	2,367	1,925
MAX	2,750	6,580	17,600	11,900	8,890	18,700	20,400	15,300	4,530	4,420	5,760	3,240
MIN	1,410	2,020	2,780	2,890	2,700	3,200	4,090	3,780	2,080	1,260	1,080	1,100

CAL YR 1973 TOTAL 1,865,359 MEAN 5,111 MAX 18,600 MIN 859
WTR YR 1974 TOTAL 1,825,850 MEAN 5,002 MAX 20,400 MIN 1,080

PEAK DISCHARGE (BASE, 17,000 CFS)

DATE	TIME	G. H.	DISCHARGE	DATE	TIME	G. H.	DISCHARGE
12-29	2200	8.03	19,000	4-07	0400	8.12	19,400
3-08	0430	8.13	19,400				

SOURCE: USGS, SURFACE WATER RECORD, NEW YORK STATE, 1974 VOL I

TABLE 5-11
SAMPLE U.S.G.S. SURFACE WATER RECORD DATA SHEET

$$\frac{H_2}{H_1} = \left(\frac{Q_2}{Q_1}\right)^{0.4} \qquad (5\text{-}11)$$

$$\frac{U_2}{U_1} = \left(\frac{Q_2}{Q_1}\right)^{0.5} \qquad (5\text{-}12)$$

$$\frac{\text{Time of Travel}_2}{\text{Time of Travel}_1} = \left(\frac{Q_2}{Q_1}\right)^{-0.5} \qquad (5\text{-}13)$$

where Q is the flow rate, a is the cross-sectional area, H is the mean stream depth, and U is the average velocity.

It should be recognized that these relationships are true only for free flowing rivers and that the exponents may actually vary by 50% for any river. Therefore, if possible, site specific exponents should be established from available data.

5.4.1.2 Dispersion Coefficient Estimates

Estimates of dispersion coefficients are required to predict the attenuation of storm pulses in primarily advective streams (Section 3.5.2.1) and to describe mixing in tidal rivers and estuaries (Section 3.5.1.2). The longitudinal dispersion which attenuates concentrations in natural streams (appropriate for the first analysis) depends on factors such as flow, shear velocity, and channel characteristics. Considerable research has been conducted into methods for estimating the longitudinal dispersion coefficient (49,50,51,52). A recent study by Liu (52) indicates that the relationship:

$$E = \frac{\beta \; Q^2}{U_* \; R^3} \qquad (5\text{-}14)$$

is particularly successful for estimating applicable longitudinal dispersion coefficients. In the above formulation,

Q = stream flow

U_* = shear velocity = $\sqrt{gRs}$
where g = gravitational acceleration
and s = channel slope

R = hydraulic radius, or mean depth in large, wide rivers

and β = dimensionless coefficient

Typical values of β range from 0.001 to 0.1. To define a likely value for β, the relationship

$$\beta = 0.18 \, (U_*/U)^{1.5} \qquad (5\text{-}15)$$

where U = mean velocity is suggested (52). Equations (5-14) and (5-15) are appropriate for an order of magnitude estimate of E, with errors expected to be no greater than a factor of six. Sensitivity analyses should be conducted to identify the effect on predicted maximum concentrations.

The dispersive properties of tidal rivers and estuaries are primarily influenced by the density-induced enculations and the tidal velocities. The effect in many estuaries is sufficient to reduce significantly the effect of freshwater flow on the transport of pollutants, although the mixing effect is a function of the density difference between saline water from the ocean and the freshwater from the rivers. Thus, the magnitude of the dispersion coefficient is relatively large in the vicinity of the mouth of the estuary where both salinity and tidal effects are great. It decreases in the upstream direction with decreasing salinity and tides. It is further reduced in the non-saline but tidal section of the estuary but is still of sufficient magnitude to be taken into account in conjunction with the freshwater flow. Typical values of E in tidal rivers and estuaries are shown in Figure 5-32 (48).

5.4.2 Reaction of Pollutants

Reactive constituents are subject to change within the receiving water due to physical, chemical and biological reactions. Examples of pollutants that fall into this category are BOD, coliform bacteria and nutrients. Although total nitrogen and phosphrous can often by treated as conservative on an annual average basis, they can be reactive during the summer low flow period due to algal uptake of the nutrients and subsequent removal by settling.

Decay mechanisms occur for each of these constituents and first order kinetics are assumed to be applicable. Representative reaction rates for these constituents are indicated in Table 5-12. The coliform death rate tends to increase with increasing salinity and exposure to light (53). The BOD reaction rates are particularly applicable to the carbonaceous fraction, but in a preliminary analysis, the rate is also considered appropriate for the nitrogenous oxygen demand, though analyses to confirm the occurrence of nitrification should be performed. The nutrient removal rates are generally applicable to conversion to other nutrient forms, but the lower range also applies to an estimate of the first order removal coefficient due to algal settling.

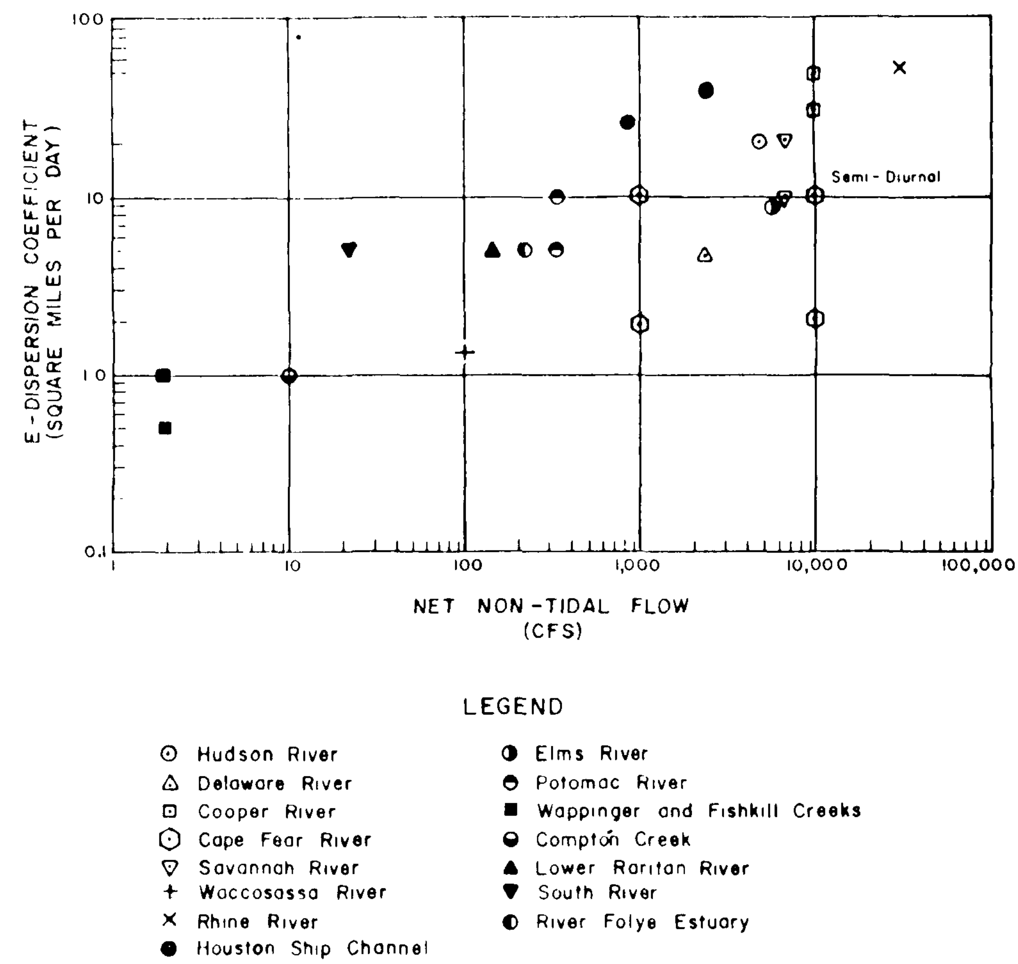


FIGURE 5-32
DISPERSION COEFFICIENT FOR DIFFERENT
TIDAL RIVERS AND ESTUARIES

TABLE 5-12

RANGE OF VALUES OF REACTION COEFFICIENTS IN NATURAL WATERS (48)

Substance	K_r (per day)[a] Freshwater Streams	Tidal Rivers and Estuaries
Coliform Bacteria	1 - 3	2 - 4
BOD_5	0.2 - 2.0	0.2 - 0.5
Nutrients	0.1 - 1.0	0.1 - 0.25

[a]Base e, 20°C

The coefficients in Table 5-12 are for water temperatures near 20°C. For preliminary estimates, temperature corrections can generally be ignored. Where appropriate, conversion to other temperatures can be made by:

$$K_r(T) = K_r(20)(\theta)^{T - 20} \tag{5-16}$$

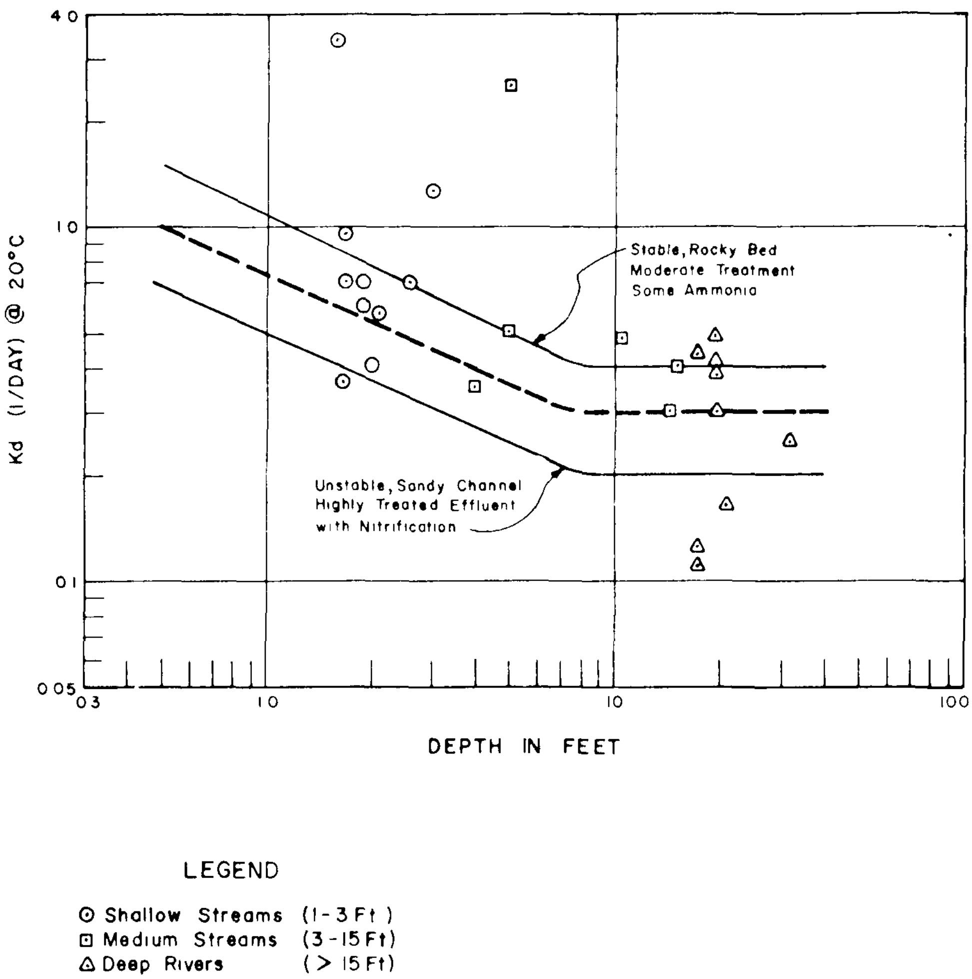
where $K_r(T)$ is the reaction coefficient at temperature T(°C), $K_r(20)$ is the reaction coefficient at 20°C, and θ ranges from 1.02 to 1.1.

5.4.2.1 Sequential Reactions

Sequential reactions occur if the growth and removal of the initial consitutent causes changes in a second constituent. For a preliminary dissolved oxygen analysis, the initial constituent is ultimate oxygen demand (UOD) and the dissolved oxygen deficit is the second constituent.

The loadings are expressed as UOD by scaling the carbonaceous BOD_5 by a factor (an estimate of the ratio of ultimate carbonaceous BOD to BOD_5) which ranges from 1.3 to 2.5. This ratio varies depending on the components in the wastewater. Therefore, if a measured value is available for the different waste loads in the system, this ratio should be used in the input analysis. The nitrogenous oxygen demand can be approximated by multiplying the reduced nitrogenous constituents (organic nitrogen and ammonia) by 4.57, which is the mass of oxygen in pounds required to completely oxidize one pound of ammonia. Total kjeldahl nitrogen measures both ammonia-N and organic-N. The organic-N fraction is assumed to oxidize as ammonia does although sequential reactions can be important.

The removal of UOD causes an uptake of oxygen and an increase in the DO deficit of the stream. In a preliminary analysis, the UOD oxidation rate (K_d) is assumed equal to the UOD removal rate (K_r). This assumes there is no settling of the UOD. Where significant settling occurs, such as in the vicinity of combined sewer overflows, K_r is somewhat higher than K_d. (Note that the settling of organic material often leads to bottom deposits which exert a subsequent uptake of oxygen. An estimate of this "bottom demand" should be included in more detailed modeling studies (54). An estimate of K_d can be made using the information in Figure 5-33. This



SOURCE: (48)

FIGURE 5-33

DEOXYGENATION COEFFICIENT (Kd) AS A FUNCTION OF DEPTH

Figure relates the oxidation rate to the stream depth and is based on data collected during many stream studies (48).

The deficit caused by the oxidation is itself reduced through reaeration. Reaeration coefficients (K_a) may be calculated from a number of formulae (55,56,57) such as the O'Connor-Dobbins equation (55):

$$K_a = 12.96\ U^{1/2} / H^{3/2} \qquad @\ 20^oC \tag{5-17}$$

where K_a is per day, U is the average stream velocity in ft/sec and H is the water depth in feet. When necessary, temperature corrections can be made using:

$$K_a(T) = K_a(20)\ (\theta)^{T-20} \tag{5-18}$$

Figure 5-34 is an alternate method useful for estimating the reaeration coefficient, where $K_a = K_L/H$ and K_L is the surface transfer rate (ft/day). This Figure shows the range of measured values together with the curves representing the theoretical formula.

Stream dissolved oxygen concentrations are calculated by deducting the DO deficits from the temperature-dependent saturation concentration. Curves of dissolved oxygen saturation concentrations versus temperature are shown in Figure 5-35.

5.4.3 Background Receiving Water Conditions

The primary emphasis of this manual is the analysis of urban runoff and overflows. Depending, however, on the characteristics of the study region, such as the relative size of the upstream drainage area, upstream activities, and areal storm patterns, the water quality of the receiving stream may be dominated by the background conditions. Because the background quality of the receiving stream entering the urban area is usually taken as a boundary condition in the analysis, some guidance is needed to estimate pollutant concentrations in the incoming flows.

Background water quality concentrations are usually available for most drainage basins, and data sources should be reviewed. In general, background water quality is affected by land uses and water use practices upstream of the urban area, and concentrations are therefore in excess of natural levels. A number of approaches have been suggested for estimating average background concentrations based on land use, geographic location, drainage basin size, precipitation patterns and other factors (58,59,60,61), however the variability of receiving water conditions, even from adjacent and similar basins, indicates the need for site specific measurements of water quality. If no data are available, background water originating from relatively undisturbed drainage areas can be estimated using the values in Table 5-13 (62).

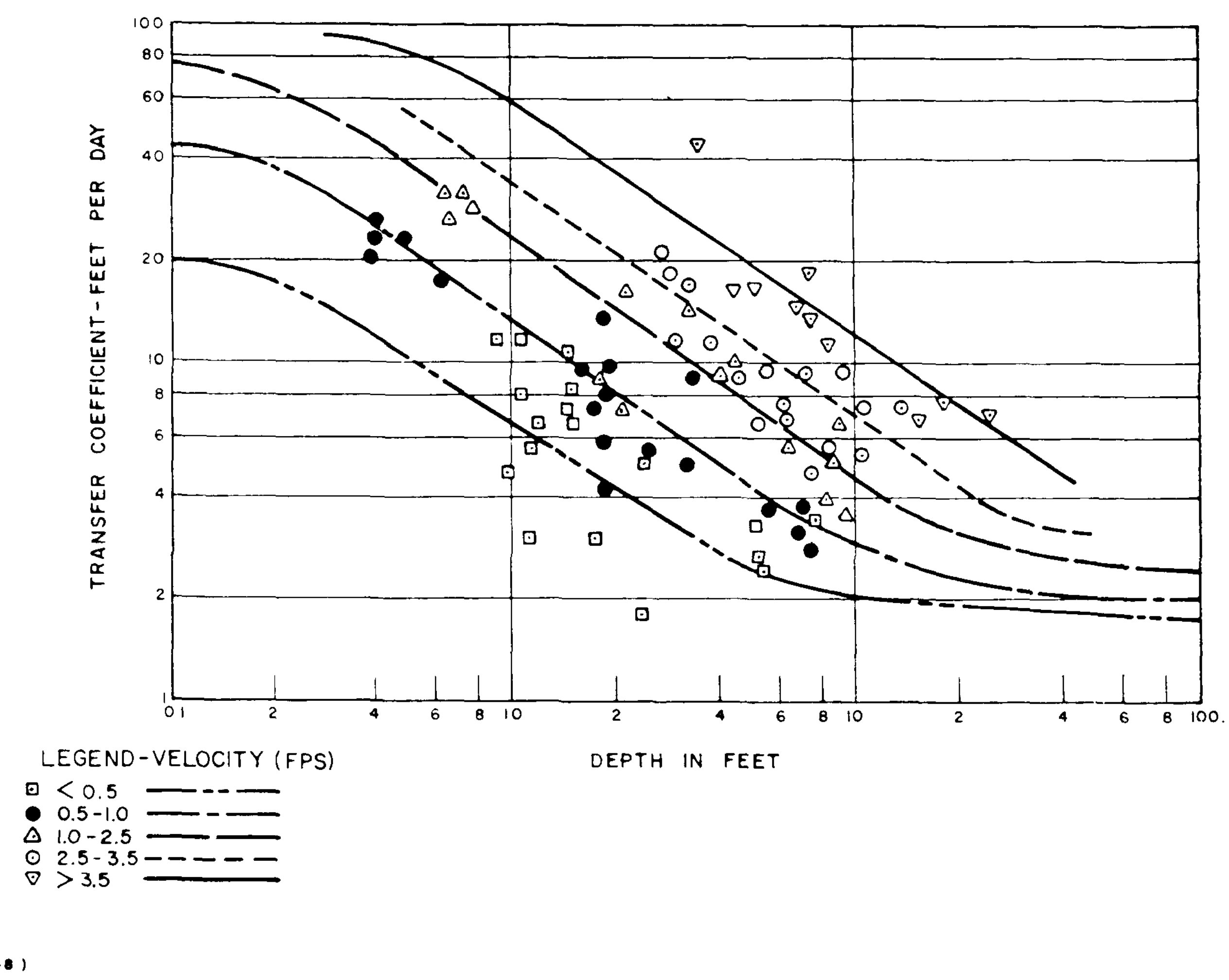


FIGURE 5-34
TRANSFER COEFFICIENT (K_L) AS A FUNCTION OF DEPTH

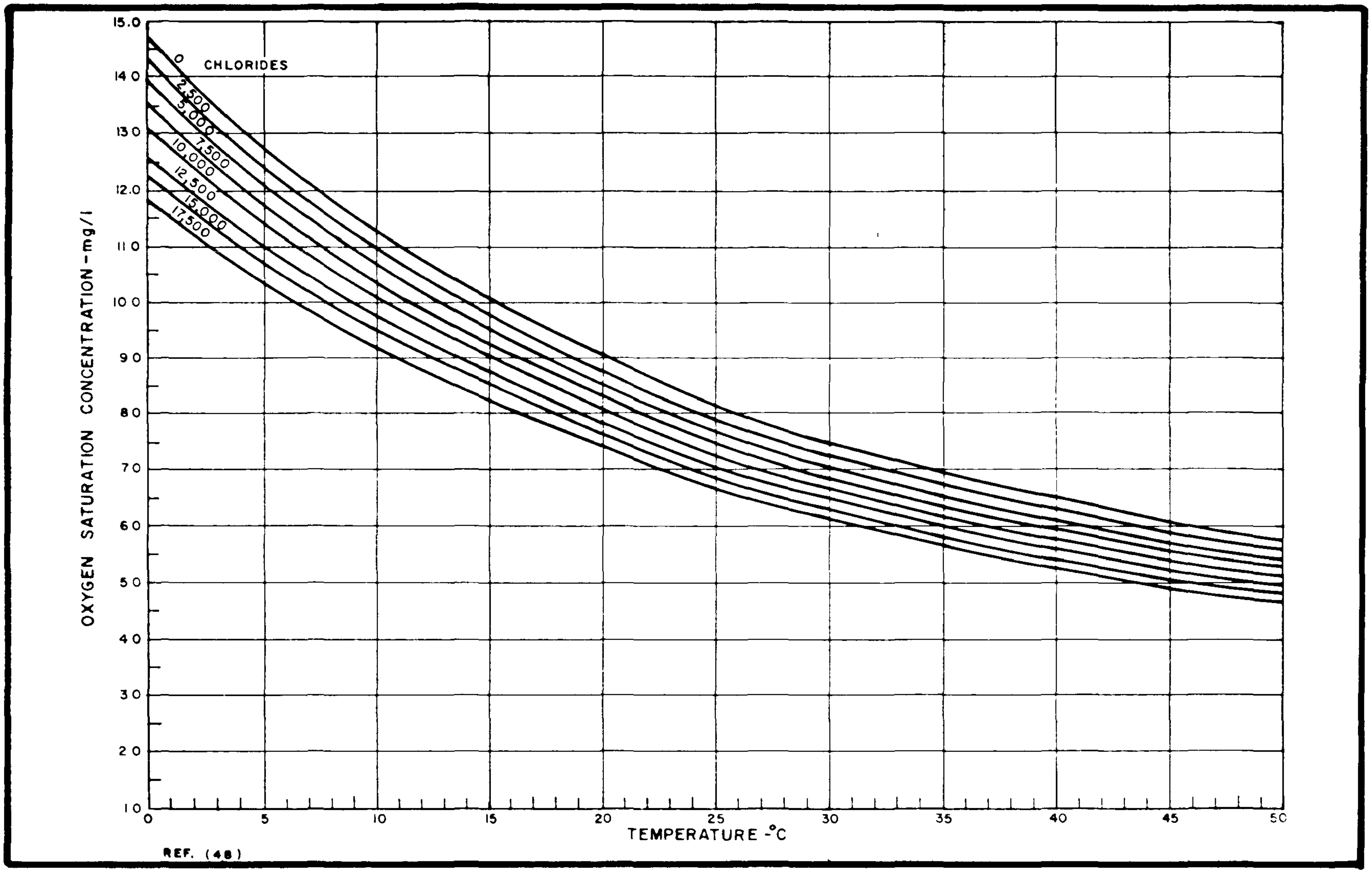


FIGURE 5-35

SATURATION-TEMPERATURE-CHLORIDE RELATIONSHIPS

TABLE 5-13

SUMMARY OF BACKGROUND CONCENTRATIONS[a] FROM VIRGIN LAND

Parameter	Concentration Range (mg/l)	Comments
Nitrogen (inorganic)	0.05 - 0.50	highest concentrations: Iowa, Illinois, Indiana lowest concentrations: South, East,West coasts
Phosphorus (total)	0.0 - 0.20	highest concentrations: Iowa, Nebraska, Dakotas lowest concentrations: South, East, West coasts
BOD_5	0.50 - 3.0	highest concentrations: Iowa, Illinois lowest concentrations: South, East West coasts
Coliform (total)[b]	100 - 2,000	highest concentrations: west of Mississippi River lowest concentrations: Northeast, Southwest
Sediment (TSS)	2 - 100	highest concentrations: Montana, South Dakota, Nebraska lowest concentrations: East, West coasts

(a) See Midwest Research Institute (59) for iso-concentration maps of virgin land runoff concentrations.

(b) Number/100 ml.

5.4.3.1 Variability of Background Conditions

For any upstream location in the receiving water the concentration of pollutants may vary considerably with time. The amount of variability is a function of waste load variations and drainage basin characteristics. Concentrations in larger rivers should tend to be somewhat less variable because of dilution and the temporal smoothing of contributing runoff. In small watersheds the receiving water variability is comparable to that of the storm related input, because it is largely a direct result of runoff from individual, localized storm events. Analyses of data from a number of Northeastern streams and rivers has indicated a trend towards greater variability in smaller basins, though the results are not entirely conclusive (63).

Another useful approach to investigate the variability of background quality is through flow-concentration correlations. If concentration is plotted as a function of flow a number of possibilities exist. A concentration-flow rating curve may be used to describe the relationship:

$$c = a \left(\frac{Q}{A}\right)^b \tag{5-19}$$

where:
- c = constituent concentration
- Q = streamflow
- A = drainage area
- a,b = coefficients

When concentrations tend to decrease at higher flow rates, this is commonly referred to as a dilution effect. This occurs when lower concentration runoff mixes with relatively constant flow sources of higher concentration such as municipal or industrial point sources. If concentrations tend to increase at higher flow rates, this is referred to as a runoff effect, where storm runoff is a major source of pollutants in the stream. An alternative cause of the runoff effect is the scouring and resuspension (during high flow) of particles which have settled during the lower flow periods. Other factors may also contribute, such as relationships between flow and reaction rates, and seasonal flow patterns. Finally, concentrations may be observed to be independent of flow. This occurs where a variety of factors affect the quality-flow relationship, resulting in no net correlation.

A number of studies have investigated the relationship between flow and concentration (64,65,66,67). In addition to the studies referenced, Hem found that specific conductance (an indicator of dissolved solids) decreased with flow in the San Francisco River at Clifton, Arizona, and was basically independent of flow in the Gila River at Bylas, Arizona, though very high flows did exhibit a lower specific conductance (68). A study of the Sacramento River found suspended solids concentrations positively correlated with flow (69).

Background sources are usually a relatively significant factor in nutrient water quality, and the relationship between flow and nutrient concentrations has received considerable attention in the literature. Wang and Evans found that orthophosphate concentrations demonstrate a dilution effect in the Illinois River, a tendency to decrease with flow, while nitrogen forms

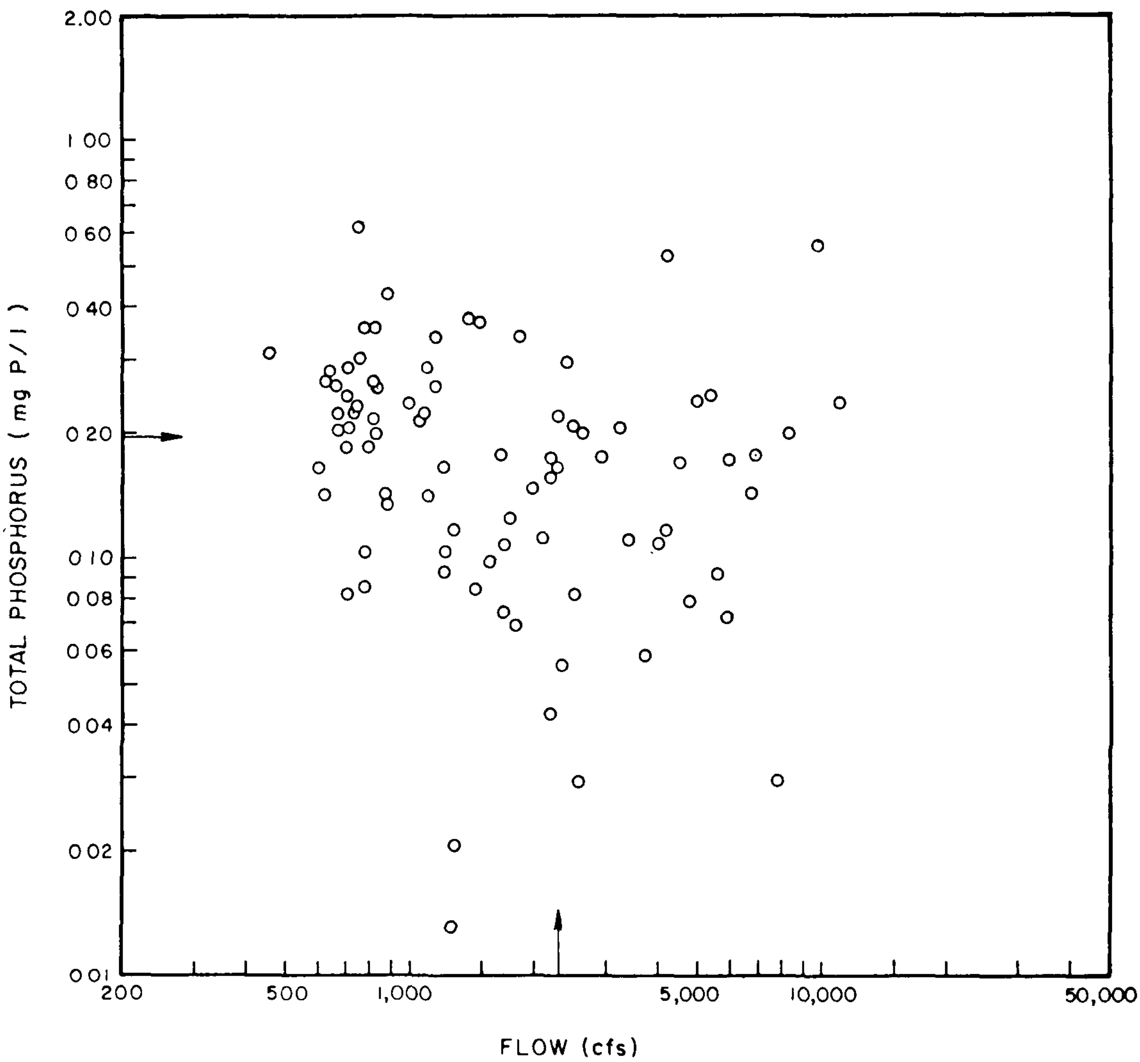


FIGURE 5-36
RELATIONSHIP BETWEEN TOTAL PHOSPHORUS
AND FLOW FOR GENESEE RIVER
(1968-1974)

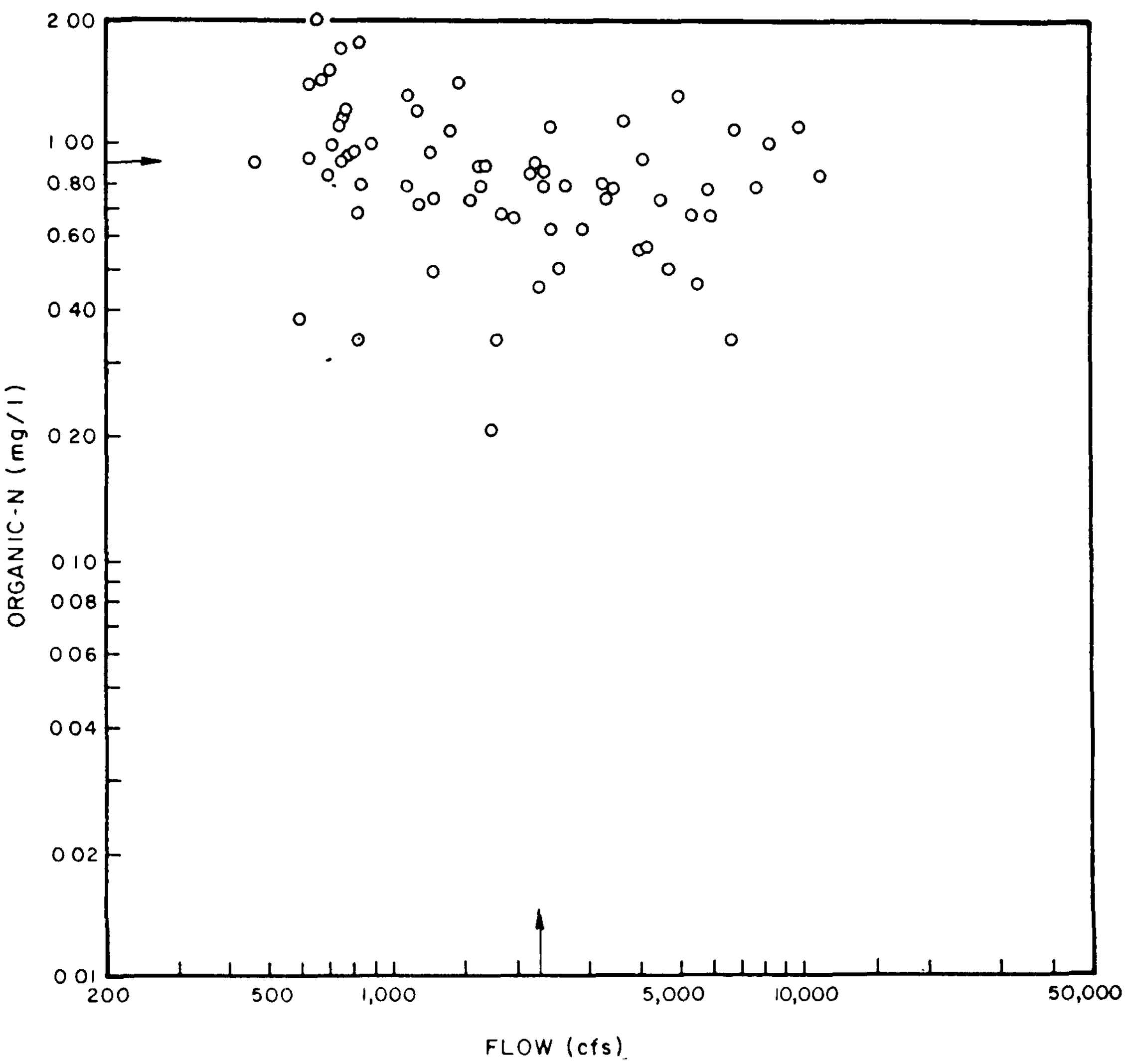


FIGURE 5-37
RELATIONSHIP BETWEEN ORGANIC NITROGEN
AND FLOW FOR GENESEE RIVER
(1968 – 1974)

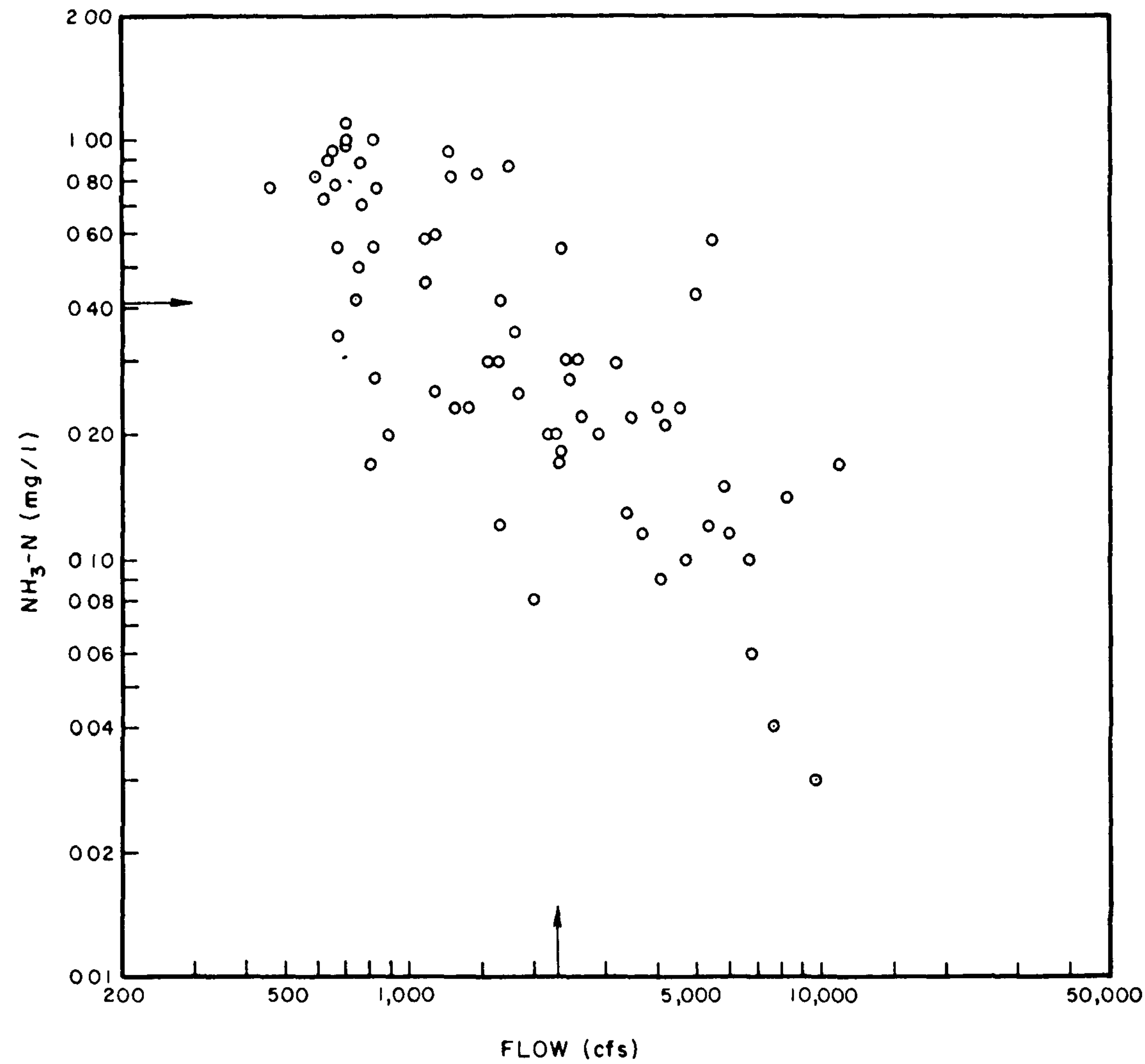


FIGURE 5-38
RELATIONSHIP BETWEEN AMMONIA AND FLOW FOR GENESEE RIVER (1968-1974)

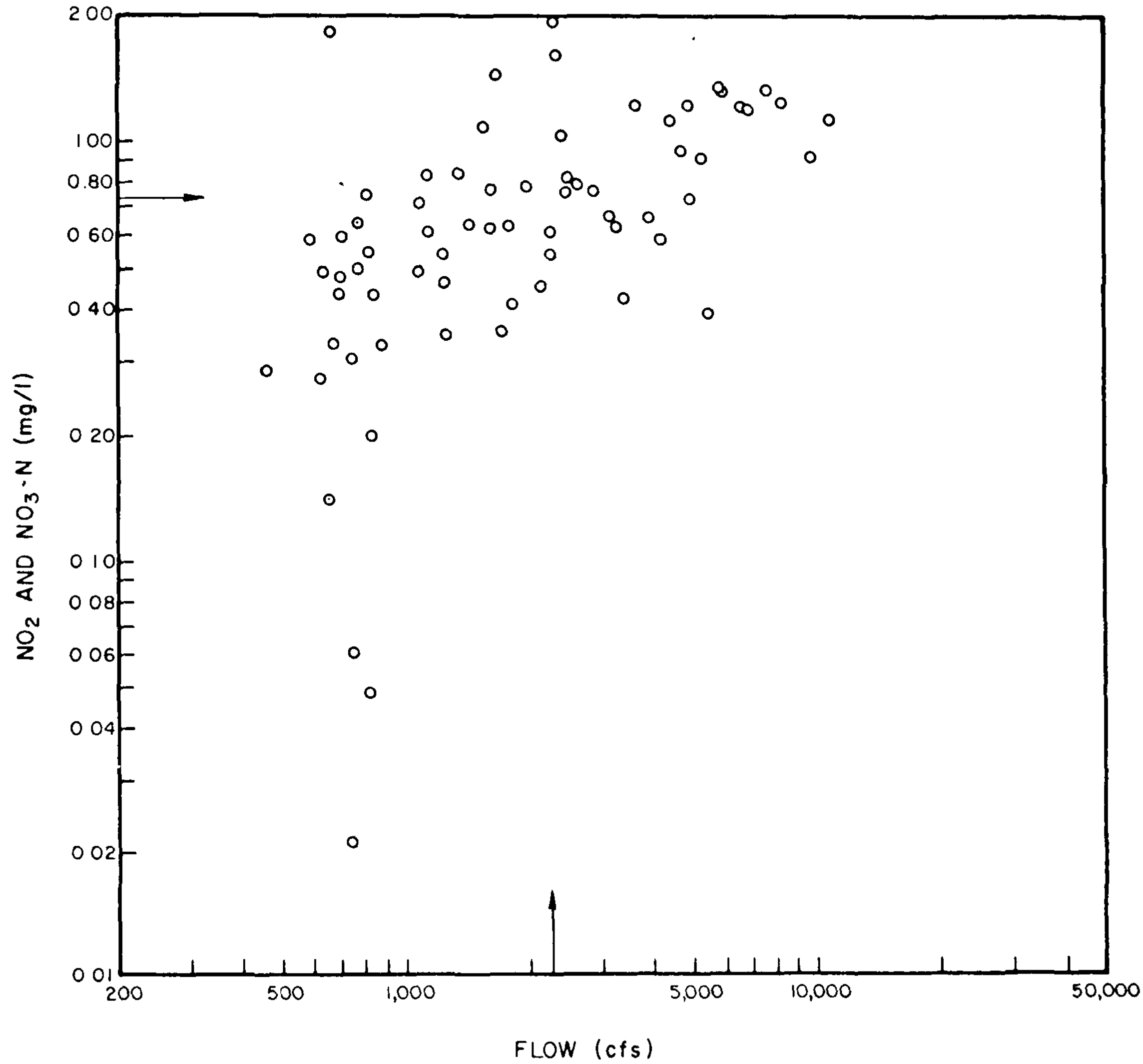


FIGURE 5-39
RELATIONSHIP BETWEEN NITRATE AND NITRITE
AND FLOW FOR GENESEE RIVER
(1968-1974)

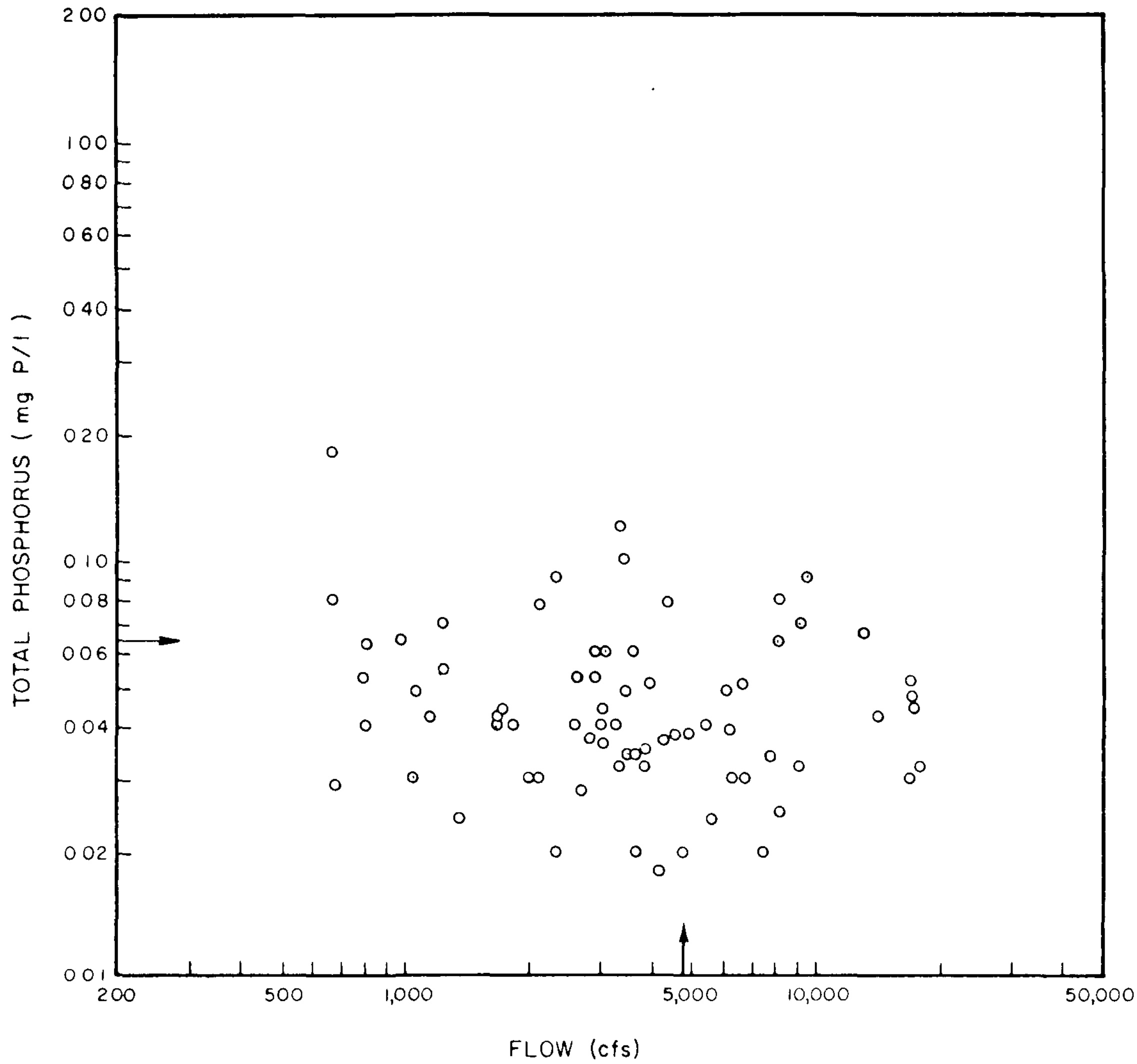


FIGURE 5-40
RELATIONSHIP BETWEEN TOTAL PHOSPHORUS
AND FLOW FOR TRENT RIVER
(1967-1973)

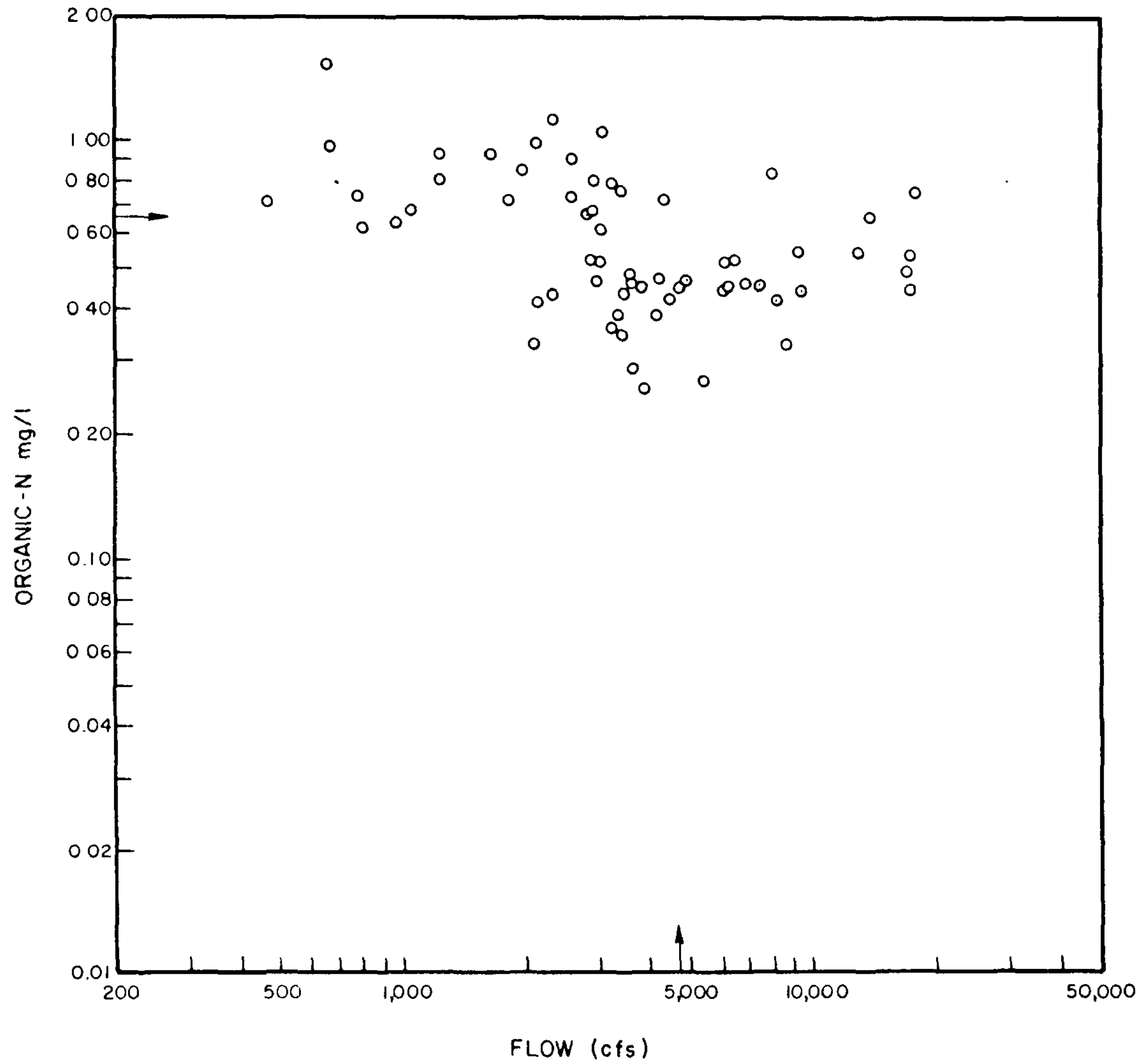


FIGURE 5-41

RELATIONSHIP BETWEEN ORGANIC NITROGEN CONCENTRATION AND FLOW FOR TRENT RIVER (1967-1972)

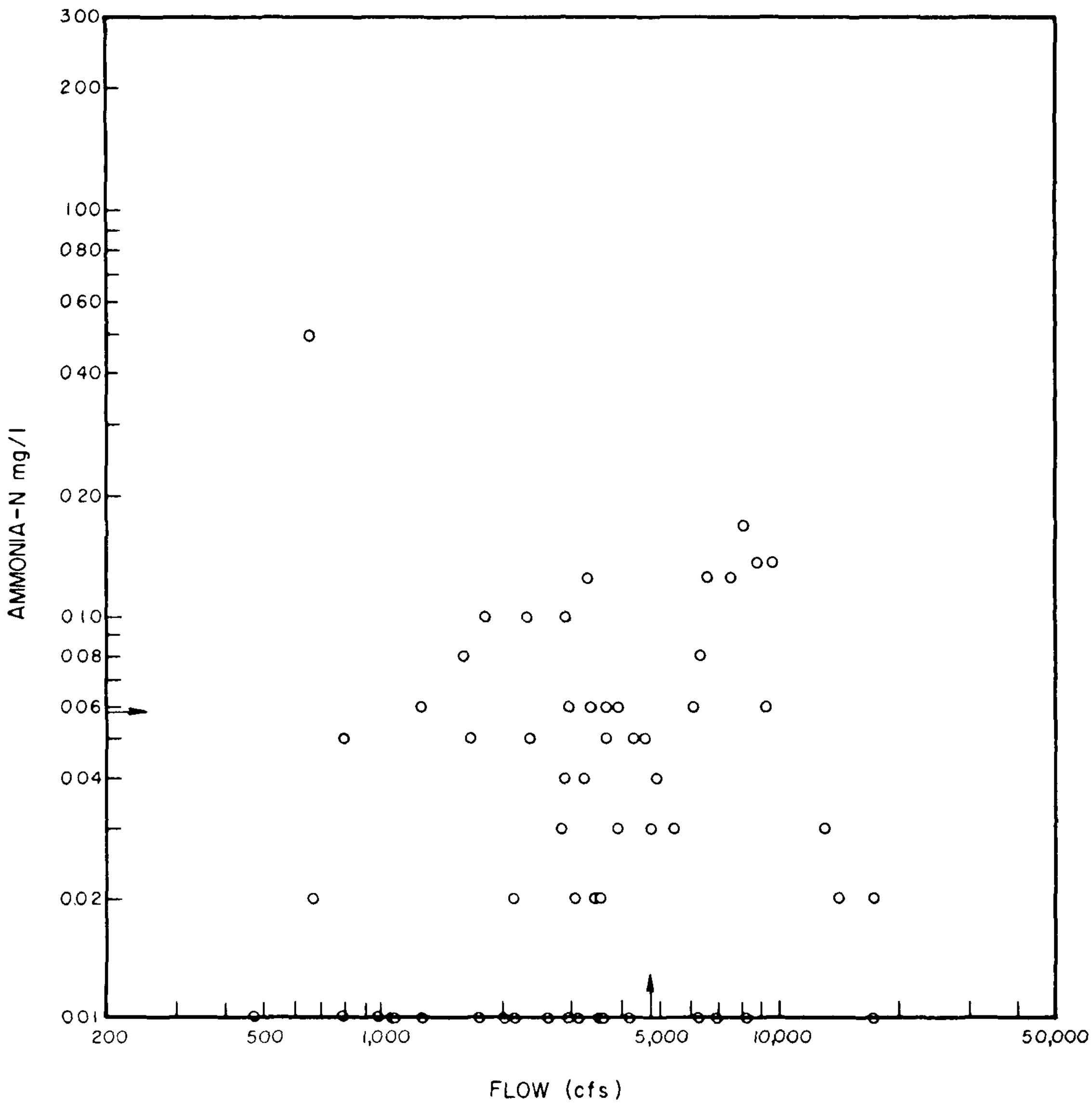


FIGURE 5-42
RELATIONSHIP BETWEEN AMMONIA
AND FLOW FOR TRENT RIVER
(1967-1972)

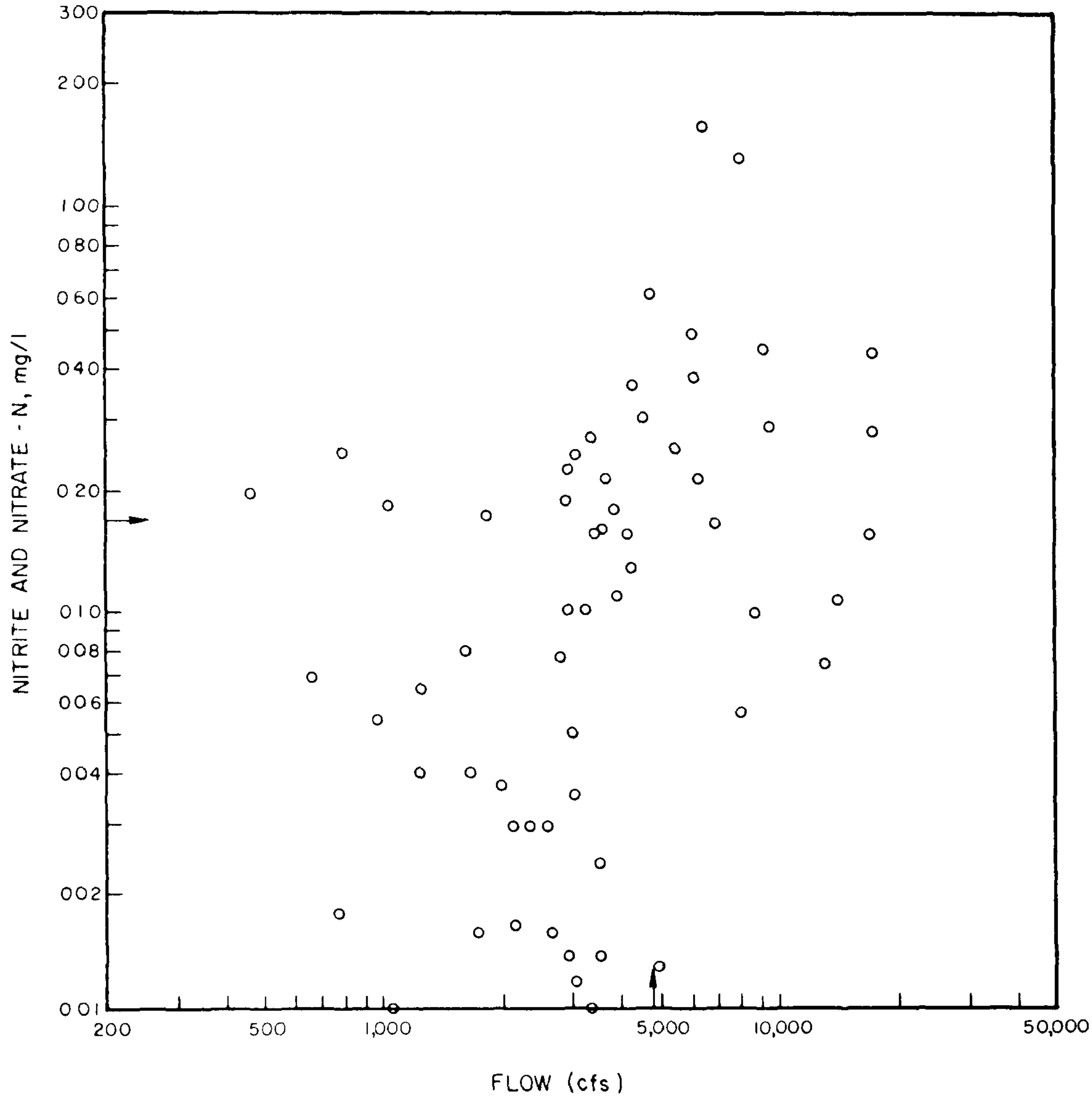


FIGURE 5-43
RELATIONSHIP BETWEEN NITRATE AND NITRITE AND FLOW FOR TRENT RIVER (1967-1972)

tend to demonstrate more of a runoff effect (70). An Enviro Control report on many drainage basins found that a runoff effect is more common for total phosphate than for orthophosphate (71). Cahill, Imperato, and Verhoff found that phosphate tends to be lower at higher flows in the Brandywine River when considering steady state conditions, however, during unsteady stage flow, phosphate tends to increase with increasing water flow rate (72). Two possible explanations are the increased scouring of sediments high in adsorbed phosphate during increasing flow periods and the contribution of phosphate in runoff from only a limited area near the waterway. A study of tributaries to Lake Ontario found that total phosphorus and nitrogen components are generally independent of flow in the Genessee and Trent Rivers (73). Figures 5-36 through 5-43 illustrate the relationships observed in these waterways. Mean concentration and mean flow are indicated by arrows on the plots. The exceptions to the independence finding are that nitrate-nitrite concentrations increase with increased flow in the Genessee River, while ammonia concentrations decrease with the flow. Analyses of other Northeastern watersheds indicates a general independence of flow rates and pollutant concentrations. Local analyses are clearly needed to establish flow concentration relationships for a particular study area.

5.5 Treatment Device Performance

A number of wastewater treatment devices have been developed or modified for application to combined sewer overflows. Section 3.6.1.4. presents the general methodology for estimating the long term efficiency of treatment devices which operate on varying runoff flows. To apply this methodology to a particular device of a given size, information is needed on the removal efficiency of the device at the mean runoff flow and the removal efficiency at very low flows. This information may be determined by examining the removal efficiency curve for each device. Performance curves developed in this study are based on available information from laboratory, pilot scale, and prototype scale treatment studies reported in the literature. Numerous reports are available which summarize the results of pilot scale and prototype studies on the treatment of combined sewer overflows. Two reports (35, 36) have been completed by Metcalf and Eddy on the State of the Art of combined sewer overflow treatment technology. The performance equations for the various treatment alternatives in the EPA Stormwater Management Model (SWMM) (74) were also reviewed and incorporated in the development of this section. The information used to develop the performance curves is based primarily on experience with dry weather flow and combined sewage. Some of the treatment methods may be applicable to separate storm runoff, although experience in this area is very limited, and care should be taken when applying results obtained from combined sewage studies to separate runoff analyses. This will be discussed in more detail in following sections.

As additional performance data becomes available thru literature sources, or from local pilot studies, new performance curves can be prepared and utilized in the methodology described in this manual.

5.5.1 Considerations for Various Pollutants

Most of the treatment devices examined for stormwater control operate by removing a portion of the suspended solids from the waste stream. These devices separate the solids physically and may achieve additional separation through chemical coagulation. The removal efficiency curves are thus presented in terms of the percent removal of suspended solids. The reduction in the concentration of other contaminants is determined by the portion of the pollutant which is associated with the suspended solids and the portion which is in the soluble form. This breakdown depends upon the composition of the stormwater and will vary in different locations. Separate runoff will tend to have a very small portion of the BOD present in the soluble form, while the soluble fraction may be somewhat higher in combined sewage. General guidelines are suggested in the following section, and may be modified for particular study areas where sampling results indicate a different relationship between a given pollutant and suspended solids.

The relationships for determining BOD, nutrients, and heavy metal removal are expressed so that the percent removal of the substance equals the percent removal of suspended solids times a factor. The range of reported values in the literature reviewed for the ratio of BOD removal to SS removal is 35 to 80 percent without chemical treatment, and 50 to 100 percent with chemical treatment. As discussed, these ratios will be affected by variation in the composition of combined sewage or runoff in different locations and also the relative association of BOD with different size particles and the treatment process's capability to remove each particle size. To approximate the BOD removal for each physical treatment device using the performance curves for suspended solids removal, the following general relationships can be used.

$\% \ BOD_r = .50 \ (\% \ SS_r)$ - no chemicals
$\% \ BOD_r = .60 \ (\% \ SS_r)$ - with chemicals

Relationships developed for nitrogen, phosphorus and heavy metals removals as a function of suspended solids removal are based on a number of studies dealing with the characterization and treatment of sanitary and combined sewage (35,41,90,94,97,103,104). The factors developed for nitrogen removal as a function of suspended solids removal for combined sewage are:

$\% \ TN_r = .30 \ (\% \ SS_r)$ - no chemicals
$\% \ TN_r = .40 \ (\% \ SS_r)$ - with chemicals

The relationships developed for phosphorus removal are:

$\% \ TP_r = .15 \ (\% \ SS_r)$ - no chemicals
$\% \ TP_r = .60 \ (\% \ SS_r)$ - filtration with polymer
$\% \ TP_r = 1.0 \ (\% \ SS_r)$ - lime coagulation

Since one study (41) shows that street surface heavy metals were almost completely insoluble in water after 25 days, they are assumed to be completely associated with the suspended solids and therefore removed at the same rate as suspended solids:

$\%$ Heavy metals$_r$ = 1.0 ($\%$ SS_r)

5.5.2 Treatment Device Performance Efficiencies

The stormwater treatment devices analyzed, which rely primarily upon the physical separation of solids, include sedimentation basins, air flotation units, swirl concentrators, high rate filtration units, and screens and microscreens. For each of these devices, a brief description of the process operation is presented. A removal efficiency curve is presented for each device, together with the documentation of the studies, tests, and analyses used to develop the curve. When chemica addition may be used to enhance the performance of the device, this is discussed and demonstrated. Typical pollutant removal for BOD and suspended solids, hydraulic loading rates, and detention times for these unit operations are summarized in Table 5-14.

Sections are also included on the applicability and analysis of biological treatment methods and disinfection for stormwater control. Finally, the suitability of stormwater treatment at dry weather municipal treatment plants is discussed.

5.5.2.1 Sedimentation Performance

In sedimentation, suspended solids are removed by gravity settling and the removal efficiency is related to the settling velocity of the particles contained in the wastewater and the design overflow rate of the settling tank. If the settling velocity of a particle is greater than the overflow rate or upflow velocity in the settling tank, the solids particle is removed. Chemical addition can be used to increase particle agglomeration and settling rates. Higher influent suspended solids concentration has also been shown to increase removal efficiency.

The removal efficiency curve developed for sedimentation tanks, which relates the percent removal of suspended solids as a function of the hydraulic overflow rate (gpd/ft^2) for various levels of influent suspended solids, is developed by examining the original EPA SWMM model performance curves (74), the revised SWMM curve based on work by the University of Florida (75), performance data from a small treatment plant treating combined sewage in New Providence, New Jersey (98), data on sedimenation tank performance presented by Eckenfelder and O'Connor (79) and the constant hydraulic loading performance of settling basins in Rochester, New York treating combined sewage (106). The Rochester study utilized a floculation basin with 6 to 14 minutes detention time for polymer addition and floc development prior to the sedimentation basin. The sedimentation performance was evaluated for 19 storms. Polymer addition of 1 mg/l was added during several tests and three constant hydraulic loading conditions of 800, 1500, and 2000 gpd/ft^2 were evaluated. These Rochester data provided the basis for the performance curve presented in Figure 5-44.

The removal curve is based on suspended solids removal efficiency at constant hydraulic loading conditions. Fluctuation of the hydraulic loading rate of settling basins will result in reduced removal efficiencies by creating flow surges and turbulence in the unit. Performance data for

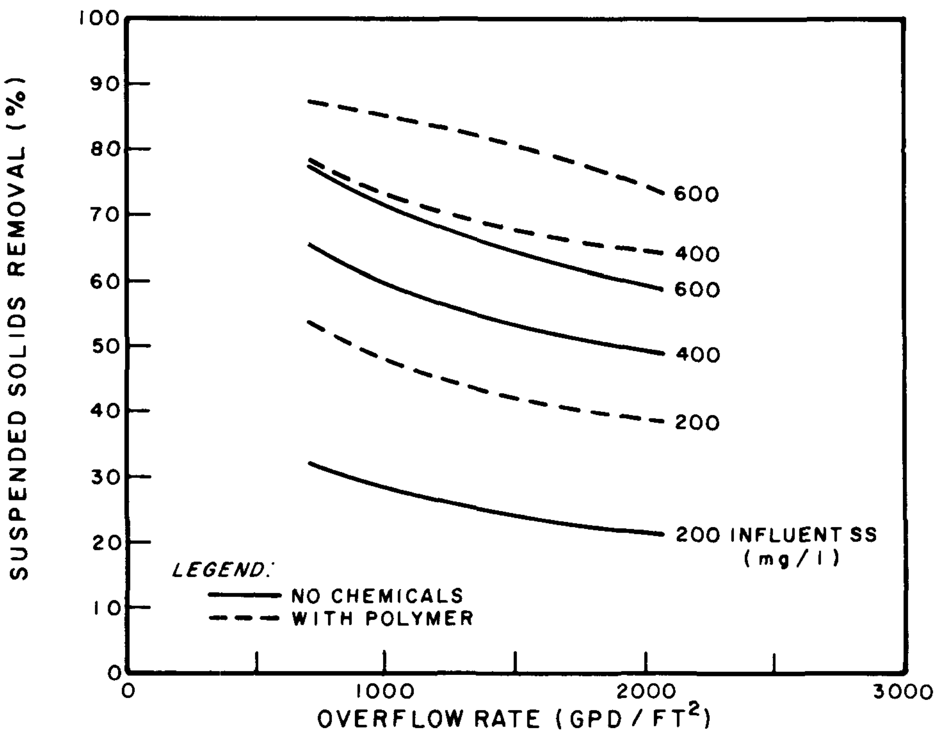


FIGURE 5-44

SEDIMENTATION TANK PERFORMANCE

TABLE 5-14

COMPARISON OF TREATMENT ALTERNATIVES

	Typical Pollutant Removals (%)[a]				Hydraulic Loading Rate	Detention Time
	BOD		SS		$(1{,}000\ gpd/ft^2)$	(min.)
	No Chemicals	With Chemicals	No Chemicals	With Chemicals	Range	Range
Physical Treatment						
1. Storage						
2. Screens						
Static	4-8	-	5-10	-	70-260	-
Rotary	8-28	-	10-35	-	20-70	-
Microscreen	16-72	-	20-90	-	20-70	-
3. Sedimentation	20-40	40-50	30-70	40-75	1-3	40-120
4. Air Flotation	30-50	48-80	40-70	58-90	2-4.5	15-20
5. Swirl Concentrator	0-35	-	20-70	-	10-250	0.4-10
6. High-rate Filtration	15-40	40-60	40-80	80-96	10-60	1-8
Biological Treatment						
1. Contact Stabilization	80	-	90	-	40-170[b]	15-120[d]
2. High-rate Trickling Filtration	65-80	-	65-85	-	0.9-2.7	40-120[d]
3. Lagoons	50-90	-	50-95	-	0.1-2.3[b]	2-20[c,d]
4. Rotating Biological Contactors	50-85	-	75-85	-	.002-.036	15-20[d]

[a] Minimum removal is at maximum hydraulic loading rates

[b] Lbs BOD_5/1,000 cubic feet volume of contact with biological mass/day

[c] Expressed in days

[d] Time of flow in contact with biological mass, does not include sedimentation.

Rochester showed results similar to those from a full scale system in Toronto treating combined sewer overflow, based on performance reported by O'Brien and Gere (106). A study by Beak Consultants Limited (105) reported that settling velocities of solids in stormwater runoff are an order of magnitude lower than those in similar settling studies conducted on sanitary sewage. Limited settling test data show that median settling rates are 1100 and 385 gpd/ft^2 for sanitary sewage and urban stormwater respectively. No tests were conducted on combined sewage. The relatively low settling velocities found for the stormwater tests are attributed to coarse clays and silt in the samples. The study demonstrates that testing should be conducted for specific areas and soil types. It is also observed that storage prior to sedimentation may increase the settling rates of particles due to agglomeration of small particles during storage.

The removal efficiency for sedimentation can be increased with the use of chemicals such as alum, polymer and ferric chloride. Pilot scale performance data from Rochester, New York, utilizing polymer, are used as the basis for the performance curves for polymer addition shown in Figure 5-44.

5.5.2.2 Dissolved Air Flotation Performance

Dissolved air flotation (DAF) removes suspended solids by the release of fine air bubbles from a pressurized, saturated recycle stream of wastewater. Particles rise to the surface and are removed by skimming. A typical DAF installation consists of pre-screening, a saturation or retention tank to dissolve air into the recycle flow, an air compressor, a small mixing chamber, a flotation cell, recycle pumps, a solids handling system to remove floated solids, and chemical and polymer feed equipment.

The principal factors that affect the removal efficiency are the hydraulic overflow rate, the pressurized recycle flowrate, and chemical addition. The most critical design parameter is the rise velocity of the combined air-solid particle expressed as a surface overflow rate. Metcalf and Eddy (35) report maximum recommended air flotation design overflow rates for combined sewage of 4,500 gpd/ft2.

Figure 5-45 shows the percent removal of suspended solids, both with and without chemicals, as a function of the surface loading rate. The DAF performance is calculated from the SWMM model equations using an average solids concentration of 410 mg/l and an average BOD concentration of 120 mg/l, based on data for combined overflows for several cities (35). The calculated performance curve compared fairly well with the limited real data that were available from pilot scale and prototype studies in Milwaukee, Wisconsin (78), Fort Smith, Arkansas (84), San Francisco, California (83) and Racine, Wisconsin (36).

5.5.2.3 Swirl Concentrator Performance

This device consists of a circular chamber in which the kinetic energy of the combined sewage is used to impart a rotary motion. The principle of the device is based on centrifugal force, created by the rotary motion of combined sewage which follows a spiral path through the circular chamber.

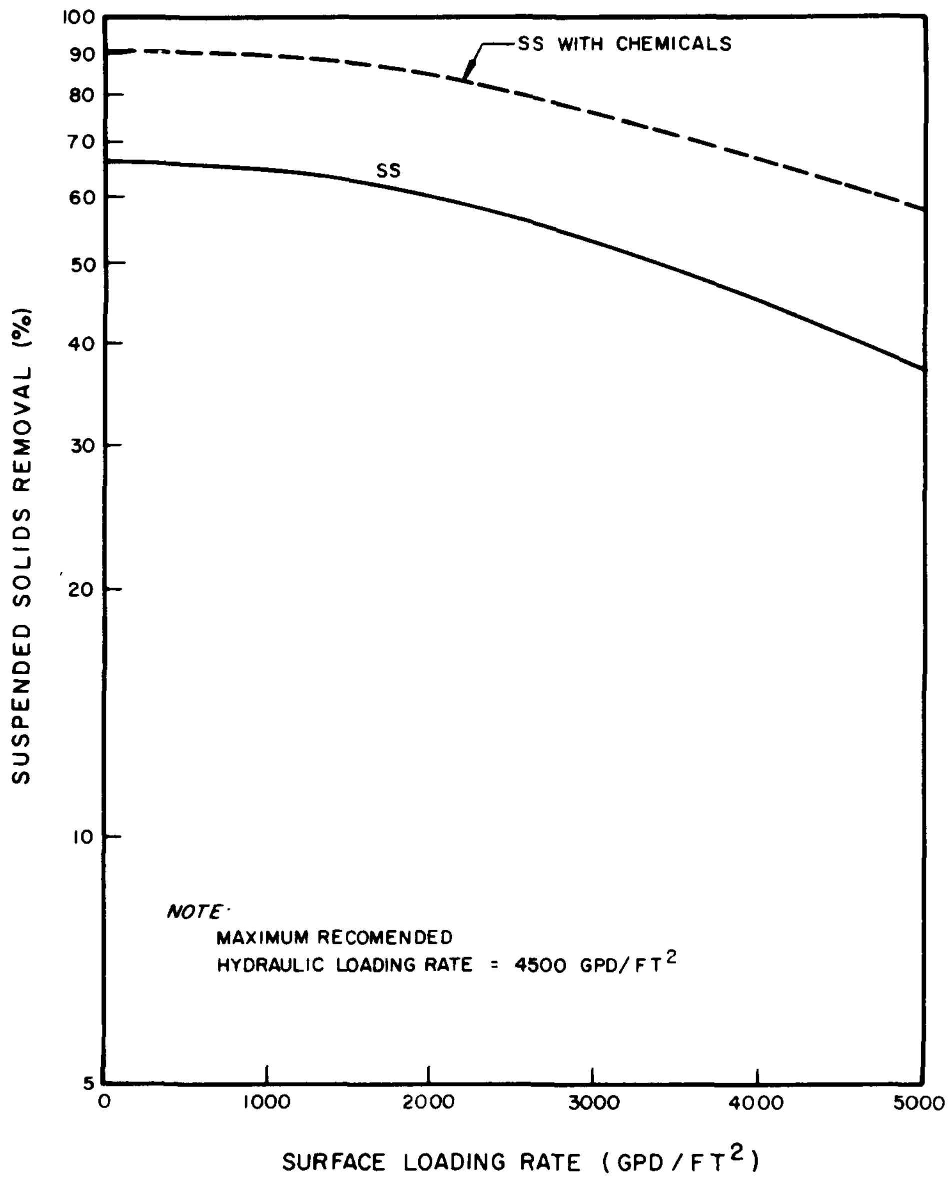


FIGURE 5-45

AIR FLOTATION PERFORMANCE

This centrifugal force increases the removal of suspended solids by providing an additional force on suspended particles causing them to settle at higher rates than in conventional quiescent settling tanks. Solids tend to concentrate along the bottom and are discharged from the swirl concentrator through a foul sewer underflow. This flow can be directed to the dry weather sewer flow and subsequently treated. Effluent flow discharges over a circular weir and can be conveyed to further treatment, and/or the receiving water.

Suspended solids removal in a swirl concentrator is due to the reduction in suspended solids concentration in the overflow and a reduction in the flowrate due to the foul sewer underflow fraction captured. The higher the ratio of the influent flowrate to the foul sewer underflow, the less significant is the additional mass removal caused by the foul sewer underflow. As the hydraulic loading increases through a swirl concentrator, suspended solids removal decreases. As with conventional sedimentation, higher influent suspended solids concentrations have been shown (89,106) to result in increased removals at similar hydraulic loading rates.

The American Public Works Association (APWA) has completed laboratory simulation and mathematical modeling studies of the swirl concentrator (85). Combined sewer overflow characteristics such as the particle distribution and specific gravity were simulated using synthetic materials. The body of the report details the basis of the assumptions used to establish the character and amount of flow to be treated, and the design of the swirl concentrator based upon the hydraulic and mathematical studies.

The APWA recommended (85) that three flow rates be considered in the design of a swirl concentrator:

1. the peak dry weather flow;
2. the design flow, i.e., the flow for which the optimum treatment is desired; and
3. the maximum flow likely to pass through the chamber.

The peak dry weather flow should pass through the unit to the foul sewer underflow and thence to the dry weather interceptor. The recommended design overflow rate for the swirl concentration based on these studies (85) was 13,300 gallons per day per foot diameter to the 5/2 power (gpd/ft$^{5/2}$). This power function of 5/2 is used to scale up the laboratory unit performance to full scale based on Froude's Law.

The swirl concentrator treatment option has been added recently to the original EPA SWMM model, based on work by Florida University (75). The unit is modeled in such a manner that given the flow, the size of the swirl concentrator, the particle sizes and specific gravities, and the fraction of particles by weight of each size, the performance can be computed based on particle settling velocities and Stoke's Law, for each particle size and corresponding specific gravity.

Two swirl concentrators have been evaluated (106) in Rochester, New York for 19 storm events. A 3 foot diameter unit operating at a constant hydraulic loading per storm event which ranged from 4200 to 19,500 gpd/ft$^{5/2}$ was sed to remove heavier particles or grit. Suspended solids removals were 0-80 percent at the low loading rates and 20-30 percent at the higher loadng rates. Removals increased with higher influent suspended solids concenrations at similar hydraulic loading conditions. A second swirl concentraor of 6 foot diameter received the effluent from the first swirl concentraor. Loading rates were also kept constant for each storm and varied from 500 to 7000 gpd/ft$^{5/2}$. Removals ranged from 60-80 percent at the lower oading rate to 35 to 45 percent at the higher loading rate.

Storm runoff data have been reviewed for a twelve foot diameter swirl concentrator in Syracuse, New York (109). The design flow was 6.8 mgd. Total mass loading removals for suspended solids ranged from 44% to 65% with concentration reductions ranging from 18% to 55%. The foul sewer discharge was approximately 10%. The higher mass loading reduction through the swirl concentrator is due to both flow and concentration reduction.

The data performance from the Syracuse prototype unit indicates good removals at the beginning of the storm, when concentrations were high, and at the end of the storm when flow rates were low such that a high percent of the flow discharged to the foul sewers. The data based on 11 storm events showed average storm hydraulic loading rates as high as 6600 gpd/ft$^{5/2}$.

Figure 5-46 shows the performance of a swirl concentrator for mass suspended solids removal as a function of flow/ diameter$^{5/2}$. In defining the performance of different size swirl concentrators as a function of the flow rate, the loading rate of gpd/ft$^{5/2}$ is used, based on the scale-up relationship developed in the initial study. Performance curves are shown for three, ten, and twenty percent foul sewer underflows. These underflows are expressed as percent of the design flow or peak storm flow (13,300 gpd/ft$^{5/2}$) as used in the APWA report (85). A comparison of this performance curve developed from the APWA study results with actual operating data from the Rochester and Syracuse units, based on average per storm hydraulic loading conditions, showed good agreement.

5.5.2.4 High Rate, Deep Bed Media Filtration Performance

In the high rate filtration process, wastewater is passed through a dual media filter bed typically composed of anthracite and sand. Loading rates are significantly higher than those commonly used in water treatment. The filter bed is usually deeper and coarser to permit greater penetration of solids into the media at high rates. Some screening must be provided before the filters to remove coarser solids, to extend filter runs, reduce head losses, and provide a more efficient operation.

The most important factors which affect the suspended solids removal efficiency of high rate filters are the flux or filtration rate, the type or size of the particlates, and the media size. Removal efficiency is inversely proportional to the flux rate, and media size. Chemical treatment using polyelectrolytes and/or alum can be used to increase suspended solids removal.

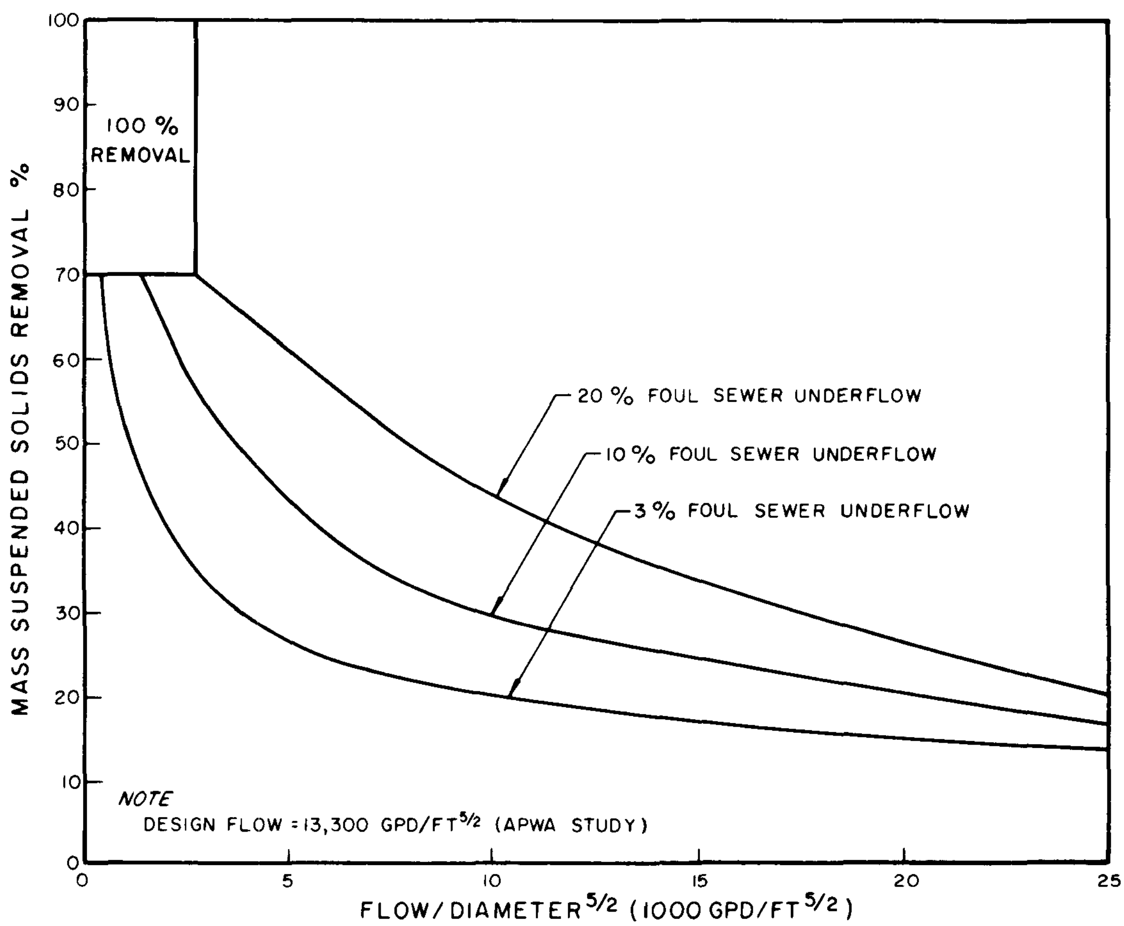


FIGURE 5-46
SWIRL CONCENTRATOR PERFORMANCE

Pilot scale filtration studies were conducted by Hydrotechnic, Inc., (90) to investigate high rate filtration techniques for combined sewer overflows in Cleveland, Ohio. The most successful results were obtained using a dual media filter, consisting of anthracite coal over sand, with a fine 420 micron screen as a pretreatment device. The use of the screen reduced the solids loading on the filter to yield acceptable filter run lengths. The screen was operated at a hydraulic loading of 100 gallons per minute per square foot, for the duration of the pilot testing and the filters were operated at constant hydraulic loading rates from 10 to 40 gpm/ft^2. The dual media consisting of five feet of No. 3 anthracite (effective size 4.0 mm) over three feet of No. 12 sand (effective size 2.0 mm) was determined to achieve the optimum results of the several alternatives evaluated. Polyelectrolyte feed was found to be an essential part of the system to achieve maximum solids removal efficiency. A maximum design loading rate of 24 gpm/ft^2 was recommended based on a deterioration in filtration performance at higher loadings.

Pilot scale high rate filter studies were conducted in Rochester, New York by O'Brien and Gere Engineers (106). Performance of the filters was evaluated at hydraulic loading rates of 10 to 25 gpm/ft^2 both with and without chemical pretreatment. A swirl concentrator was used as a pretreatment device for solids removal. The filters consisted of 3 feet of No. 12 anthracite over 5 feet of No. 1220 sand. Chemical addition in the swirl concentrator was found to achieve higher removal efficiencies than chemical addition directly before the filter. The importance of contact time for chemical conditioning was identified in this study.

A performance curve for high rate filtration is presented in Figure 5-47. This curve was developed using the Cleveland and Rochester pilot study results (90,106). The suspended solids removal efficiencies are for the filtration step only and do not include the solids removal in the pretreatment step. The addition of polymer at dosages of approximately 1-2 mg/1 was shown to enhance removal efficiency by 20 to 30 percent.

5.5.2.5 Screens and Microscreens

Screens remove suspended material from combined sewer overflow by physically straining the solids from the liquid. They range in size from 3 inch clear openings (bar screens) to openings as small as 15 micron (stainless steel woven microscreens). Screens have been divided into four classifications by Metcalf and Eddy (35) 1, bar screens; 2, coarse screens; 3, fine screens; and 4, microscreens.

1. Bar screens are typically installed prior to storage treatment facilities and pump stations to protect downstream equipment.

2. Coarse screens or static screens are usually used as pretreatment devices prior to other treatment units such as dissolved air flotation, or high rate filtration to remove the coarse solids and increase the operational efficiency of the subsequent treatment steps.

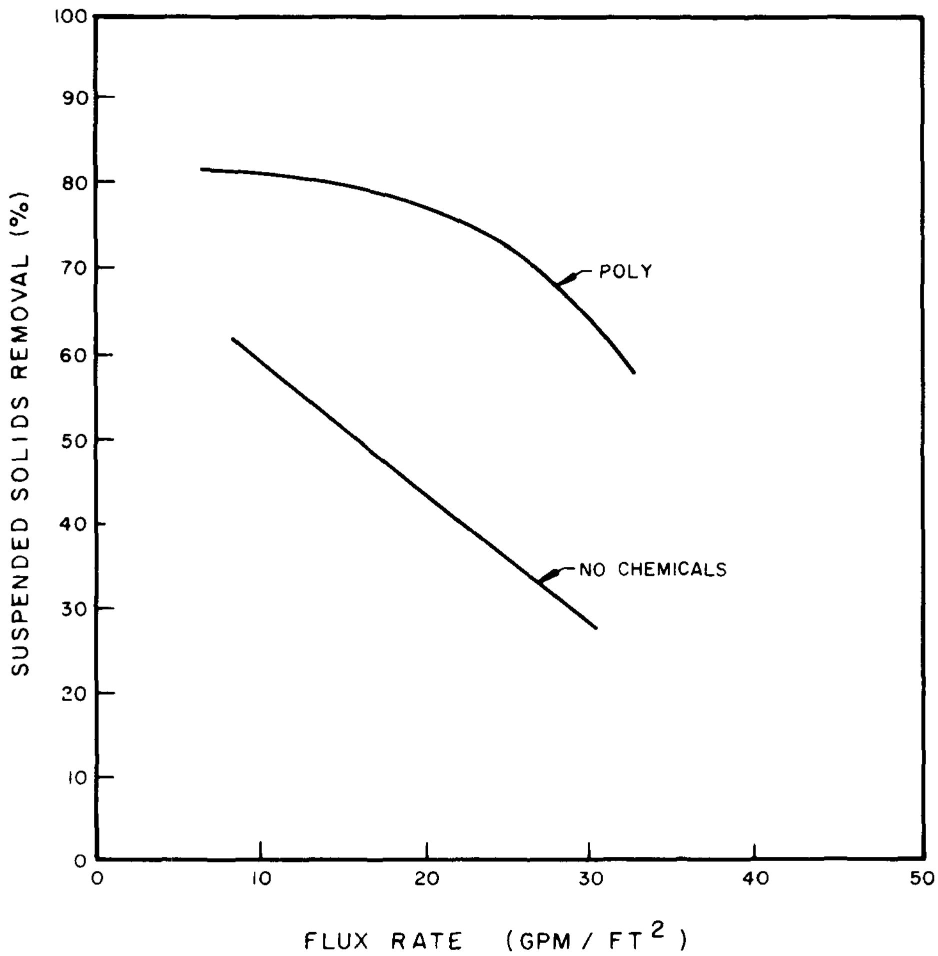


FIGURE 5-47

HIGH RATE FILTRATION PERFORMANCE

3. Fine screens and microscreens are similar in operation but differ in the size of the aperatures. Fine screen devices commonly referred to as drum screens, have a range of aperature sizes from 100 to 600 microns. These screens may rotate on either a vertical, or horizontal axis. Several 105 micron units have been tested.

4. Microscreens, commonly called microstrainers, have aperature sizes ranging from 15 to 65 microns. They provide versatility since they can be designed for specific applications by changing the aperature size of the screen. A typical microstrainer unit consists of a rotating drum fitted with a fine screen operating partly submerged in a tank. The stormwater enters the interior of the drum, through the open end, passes out through the screen and into the outlet chamber. The suspended solids are retained on the screen and develop a mat of screened particles that acts as a strainer, retaining particles even smaller than the screen aperature. As it rotates, the screen and its mat of retained solids passes under a row of backwash jets which wash the solids into a trough for disposal. The backwash water stream is small and is usually sent to a dry weather treatment plant. The backwash water source is usually microstrained effluent. In microstraining applications, the head loss is about 6 inches for a 23 micron screen with a flow rate of 6-8 gpm/ft^2 of gross submerged screen area. Microstraining may be operated at 24 inches head differential and 15-25 gpm/ft^2 hydraulic loading. With polyelectrolyte hydraulic loading rates of 30-40 gpm/ft^2 are reported to be possible with similar effluent quality. Microstrainers should be preceeded by a coarse bar screen and have a bypass arrangement to divert flow in excess of the peak capacity of the treatment equipment.

Several pilot scale and prototype studies (36,76,77,78,82,106,107,108, 109) have been conducted to evaluate the performance of screening devices to remove suspended solids from combined sewer overflows. Detailed description of the studies and results are contained in these reports. In general, these studies demonstrate that the suspended solids removal efficiency of screening devices, especially the fine screens and microscreens, is related to the influent suspended solids concentration and not the hydraulic loading rate. Screens tend to produce a consistent effluent suspended solids concentration independent of the influent solids concentration and hence have higher removal efficiencies at higher influent suspended solids concentrations. Maximum hydraulic loading rates are defined in these studies and vary from a low 16-36 gpm/ft^2 for a 23 micron microscreen (with the higher value possible with polymer addition), to as high as 180 gpm/ft^2 for static screens.

Figure 5-48 shows the performance of various size screens and microscreens as a function of influent suspended solids concentrations. These curves have been developed from performance data from a 23 micron microstrainer tested by Glover (76) on combined sewage in Callowhill, Philadel-

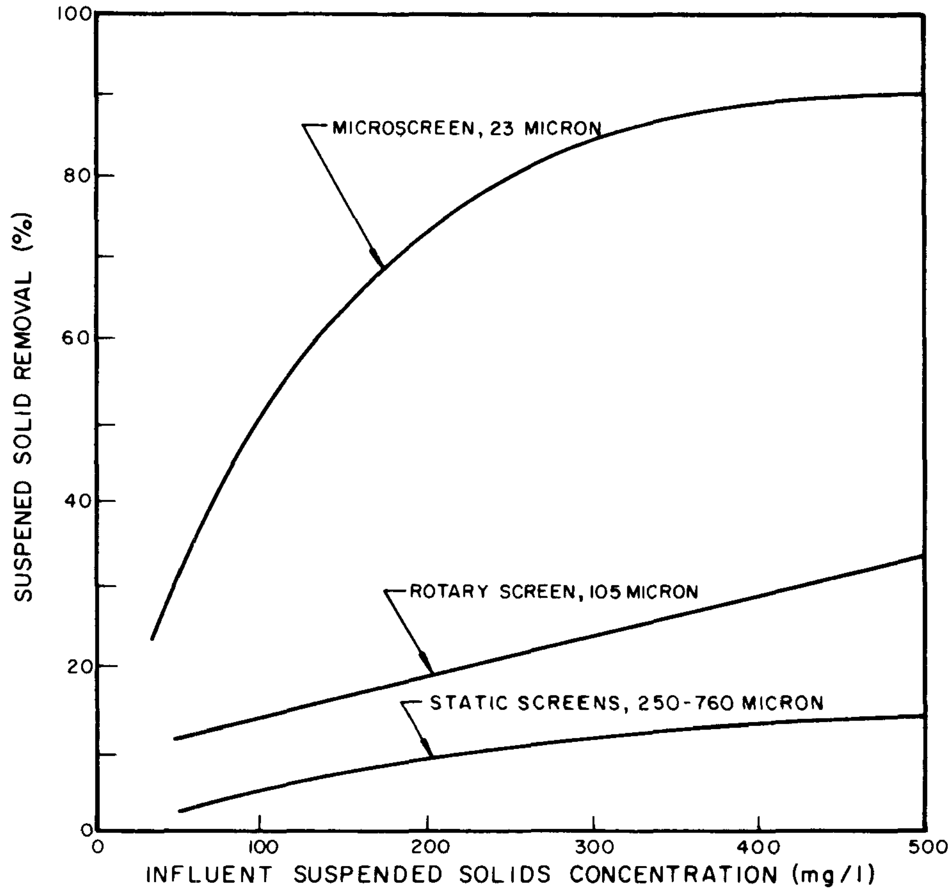


FIGURE 5-48

PERFORMANCE OF SCREENS

phia; 105 micron rotating screens in Belleville, Ontario, (36) Fort Wayne, Indiana (107), and Syracuse, New York (109); and hydrosieves and static screens in Fort Wayne, Indiana (107), and Belleville, Ontario (36) respectively. These curves represents an average performance or removal efficiency at various influent suspended solids concentration. Actual performance data from these pilot and prototype screening studies show a range of ± 15-20 percent at influent suspended solids concentration greater than 200 mg/l and ± 30-40 percent at lower influent suspended solids concentrations.

5.5.2.6 Biological Treatment

Biological treatment systems investigated to treat combined sewer overflows include contact stabilization, high rate trickling filtration, lagoons, and rotating biological contactors. Typical removals, loading rates and detention times are shown for each system in Table 5-14.

Contact stabilization is a modification of the conventional activated sludge process. Since less tank volume is required to yield carbonaceous BOD and suspended solids removals in the range of 75-90 percent, the contact stabilization process is considered more applicable to treat combined sewer overflows than conventional activated sludge (35). In the treatment process, the combined sewer overflow is mixed with return activated sludge for twenty minutes detention at the design flow (35). Removal of BOD is accomplished by adsorption onto the biological sludge. Following the contact period, the flow is settled in a clarifier. The concentrated activated sludge is returned from the bottom of the clarifier to a stabilization basin, where it is aerated for several hours. During this period, the organics from the wastewater are utilized for growth and respiration of the organisms. The overflow from the clarifier may then be chlorinated and discharged. The stabilized sludge is returned to the contact tank to mix with the incoming wastewater, and excess sludge is wasted.

A study at a 20 MGD combined sewer overflow facility in Kenosha, Wisconsin showed overall removal efficiencies of 92% S.S., 83% BOD, 50% Total-N and 50% Total P (35). No correlations are made between performance and parameters such as sludge age, food to microorganism ratio or detention time, because overflows did not last long enough for the plant to stabilize.

High rate trickling filtration is used at a plant in New Providence, New Jersey (98) to treat both domestic sewage and combined runoff. The plant was designed to treat a DWF of 0.6 MGD and a maximum wet weather flow of 6.0 MGD. Both rock and plastic media are used in each of the two filters. Performance of this facility is presented in terms of both hydraulic and organic loading rates and as a percentage of dry weather flow, in Figures 5-49 and 5-50. Under storm conditions, a trickling filter must handle highly varying flows. Applying a varying organic load to a filter does not produce optimum removals. It is generally thought that only sufficient biomass adhers to the supporting medium to oxidize the normal organic load, although, some excess biomass always adheres to the medium and can accept some of the organic load.

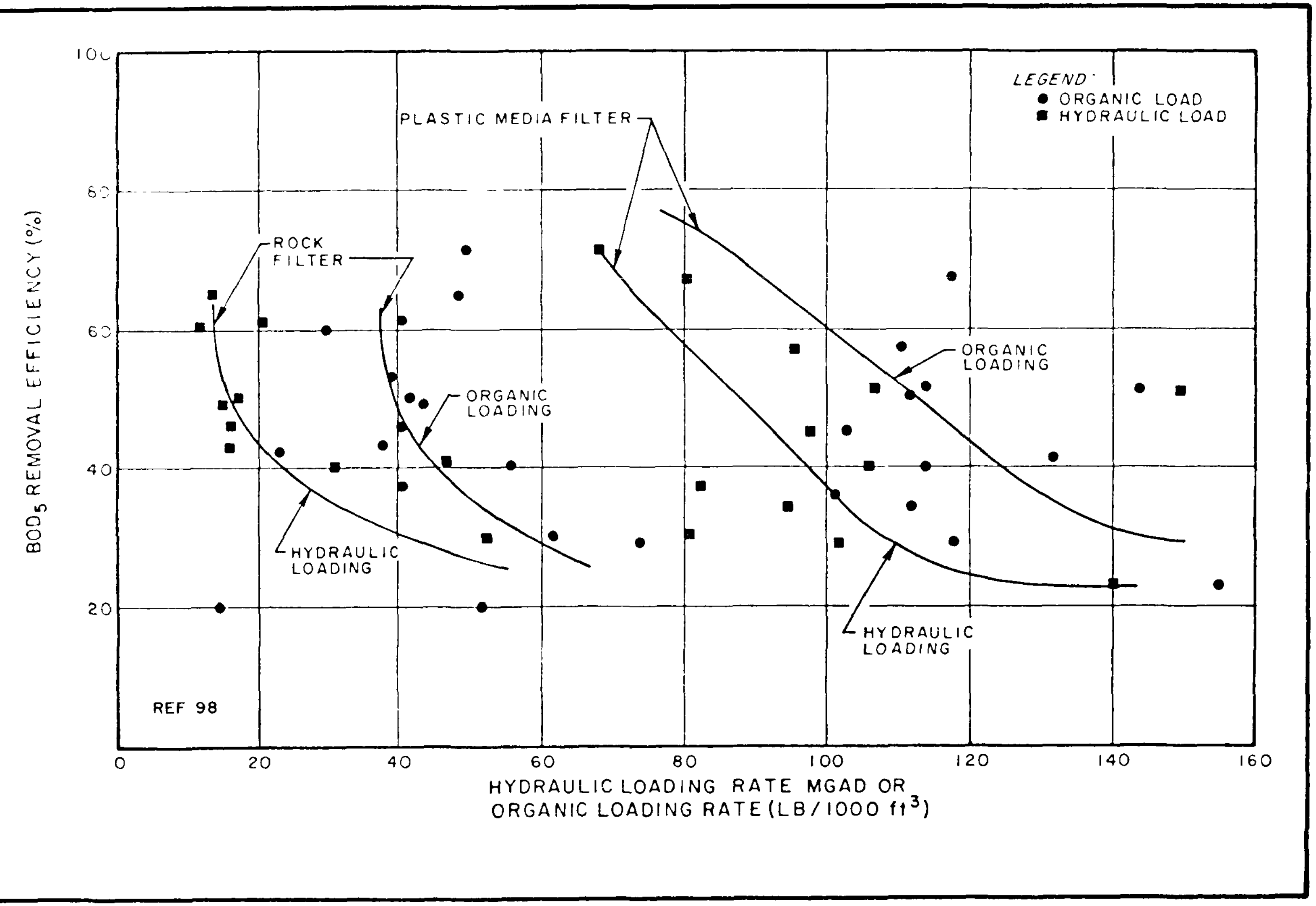


FIGURE 5-49

TRICKLING FILTER PERFORMANCE

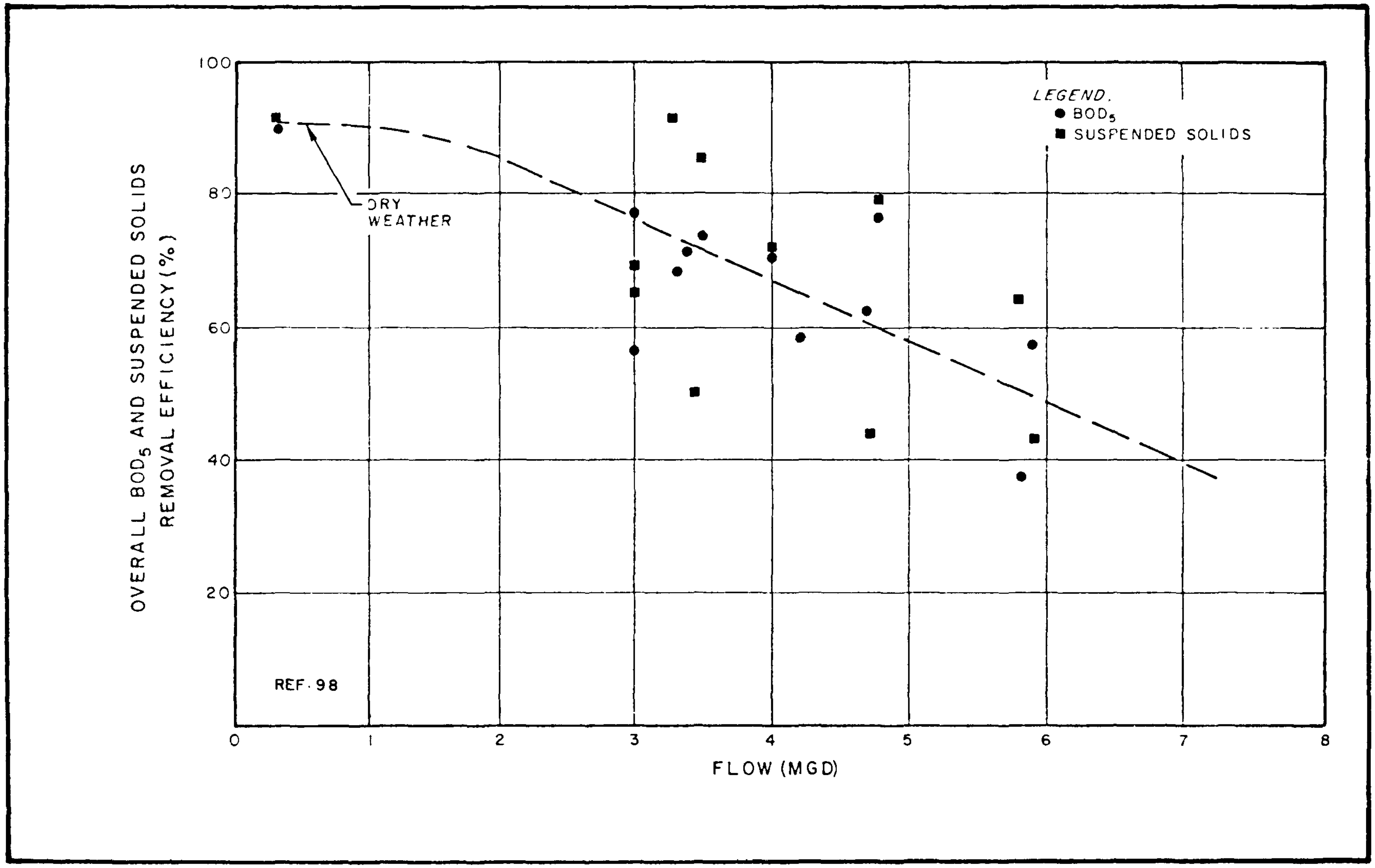


FIGURE 5-50
TRICKLING FILTER PLANT PERFORMANCE

A varying hydraulic load also affects removals. The increased shearing action at high flows causes excess sloughing or washing off of the biomass. To help dampen this effect, filters operating in series under dry-weather conditions can be operated in parallel during storm flows, thereby reducing some of the increased hydraulic load on each filter.

In designing a trickling filter to treat overflows, it must be remembered that dry-weather flow is needed to keep the biomass active between storms. Generally, two or more units should be used to provide high removals by operating in series during dry weather and in parallel during storm events to accommodate the flow variation required.

A unique feature of the New Providence plant is the method of filter operation designed to keep a live biomass available on both filters at all times. To do so, the filters operate in series with the plastic medium filter in a lead position for treating all flows up to 2.8 mgd. At this point, an automatic transfer to parallel operation is accomplished and maintained until flows again drop within the series range. In parallel operation (the normal combined sewer overflow treatment mode), both filters receive equal flow, resulting in a much higher (3 to 1) unit loading on the smaller plastic medium filter.

Lagoons can be used to both store and treat wastewater. Stormwater lagoons can be of several types: oxidation ponds in which oxygen is supplied by natural means from the atmosphere and algae growth, aerated lagoons which use mechanical aeration for mixing and oxygen supply; and facultative lagoons which have an upper aerated zone and a lower sludge decomposition anaerobic zone. A major problem of lagoon treatment is solids and algae carryover into the effluent. Solids removal devices such as microscreens and filtration have been used in some locations to treat lagoon effluents.

Rotating biological discs, a recent development in the biological treatment field, were tested on combined sewer overflows in Milwaukee, Wisconsin, because of their reported ability to handle highly varying flows (99).

The rotating biological contactor consists of a set of rotating discs upon which a biomass is grown. The rotating discs are partially submerged in the wastewater, and baffles are used to prevent shortcircuiting. The waste flow enters the contact tank at one end and is allowed to flow either perpendicular to, or parallel to, one or more units in series for treatment. The removal of organic matter is accomplished by adsorption at the surface of the biological growth covering the rotating discs. Rotational shearing forces cause sloughing of excess biomass. Secondary clarification is normally provided to remove any sloughed biomass from the wastewater. The demonstration pilot plant exhibited an ability to treat storm flows of 14 to 20 times dry weather flow. The performance of the Milwaukee facility for BOD, COD, total nitrogen, and phosphorus is presented in Figure 5-51.

Biological treatment dependent on living organisms, and efficient performance requires that these organisms be sustained by continual application of wastewater. Storm overflows are random in their occurrence, and the

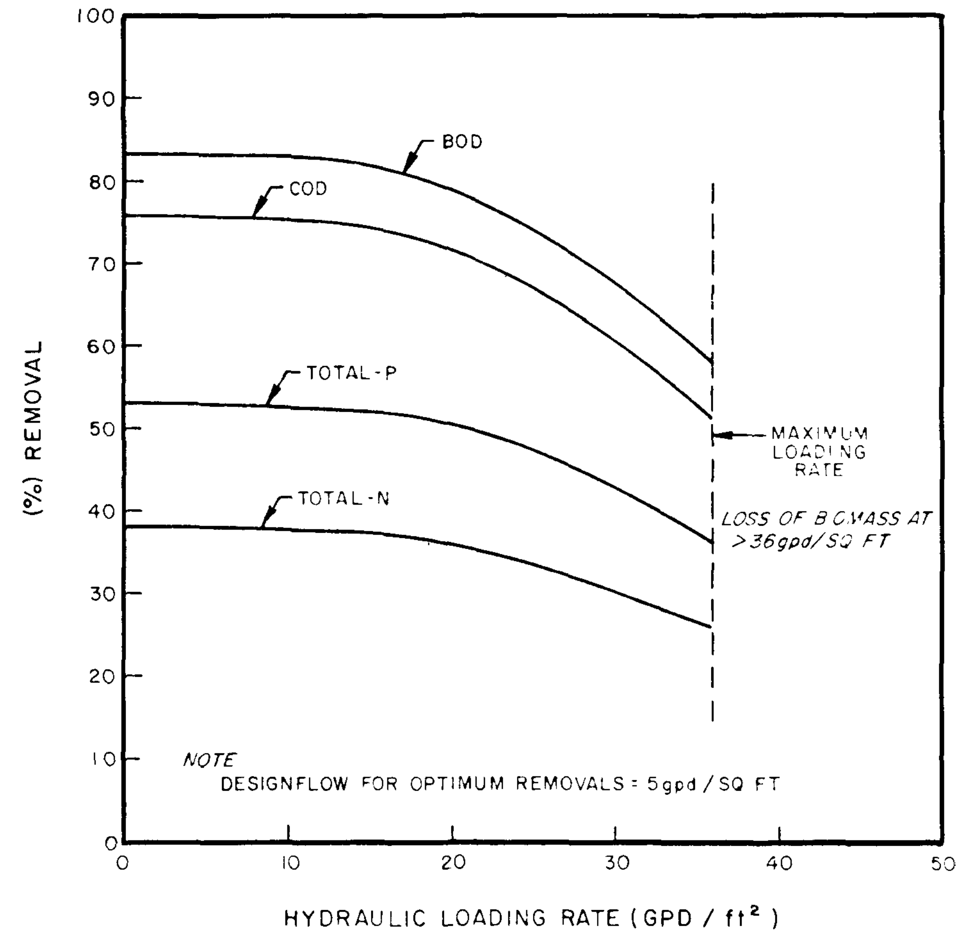


FIGURE 5-51

ROTATING BIOLOGICAL CONTACTORS PERFORMANCE time between overflows can at times be quite long. This situation limits the appropriateness of biological treatment for stormwater runoff to situations where such facilities are operated at the dry weather treatment plant, or sustained by dry weather flow diverted from the principal waste stream. A possible exception to this limitation are lagoons which are less dependent on a constant waste application may go for long periods without discharge.

5.5.2.7 Disinfection

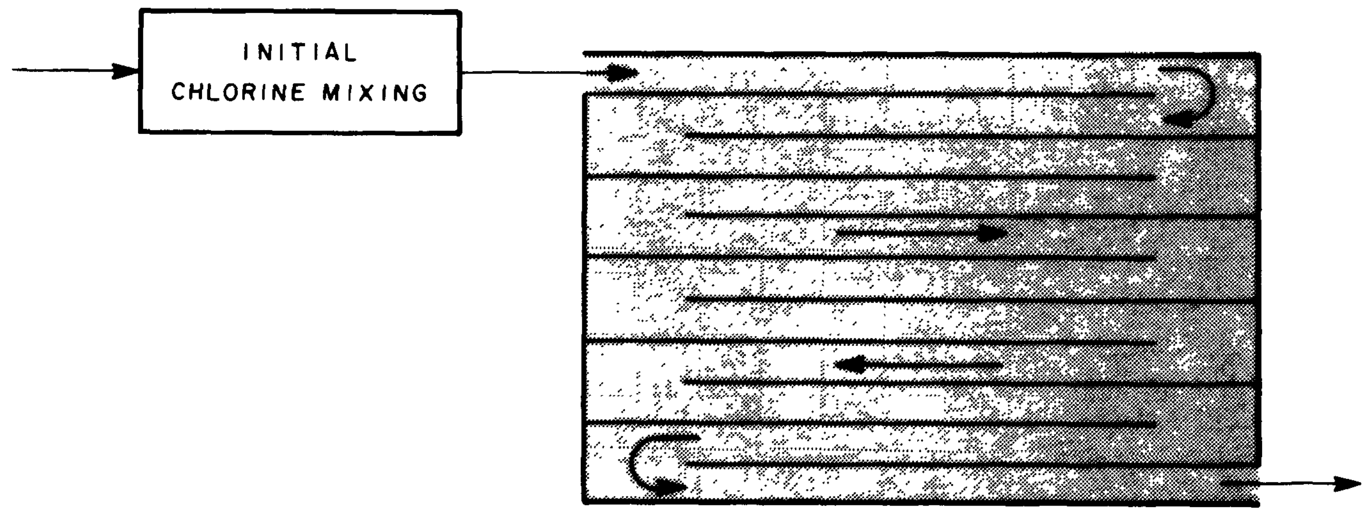
A number of modifications in both the design and analysis of disinfection systems are required when they are applied to the treatment of stormwater runoff or overflows. Nevertheless, disinfection is often considered for wet weather control either by itself or in combination with other treatment devices, particularly when treating combined sewer overflows. Field (110) presents a brief review of recent developments in stormwater disinfection. The methods currently available incorporate either chlorine, sodium and calcium hypochlorite, chlorine dioxide, or ozone. Recent research has emphasized high rate disinfection with the more rapid oxidants (i.e., chlorine dioxide and ozone) two stage disinfection, and methods of obtaining rapid, adequate mixing. The possibility of residual toxicity effects must be considered when large dosages of disinfectant are used for combined sewer overflows.

In this section, the fundamental factors which affect disinfection performance are analyzed to estimate the effect of storm flow variation on treatment efficiency. A generalized approach is developed and may be used to estimate the long term performance of disinfection systems which operate at variable rates, when removals are specified at a fixed flow rate.

The basic mechanisms of disinfection are reviewed by Collins, Selleck, and White (111) where they compare the bacteria removal obtained by an ideal plug flow device (where each element of water has the same contact time, $\bar{t}$) and an ideal completely mixed tank (where contact times are exponentially distributed with mean $\bar{t}$). Each element of water is assumed to behave according to Chicks Law (112), which states that the survival fraction of bacteria organisms equals e^{-kt}, where k is the kill rate and t is the contact time of that element. The overall fraction of bacteria remaining for the plug flow system is thus $e^{-k\bar{t}}$; while for the complete mix device the fraction remaining is $(1 + k\bar{t})^{-1}$. Evaluation of the expected value of these removals over varying influent flows (using the method developed further in this section) confirms the conclusion of Glover and Herbert (76) that wet weather disinfection systems should be designed to approach plug flow rather than complete mix conditions. Glover and Herbert suggest a flow-through device with corrugated baffles to increase the mixing intensity without causing a significant deviation from plug flow conditions. Their design is depicted in Figure 5-52.

In an actual treatment device such as the one depicted in Figure 5-52, the ideal plug flow removals ($e^{-k\bar{t}}$) may not be obtained. There are two primary reasons for the deviation:

i. Chicks Law for the bacteria decay may not be valid;



NOTE:
BASED ON GLOVER AND HERBERT (76)

FIGURE 5-52

TOP VIEW OF DISINFECTION DEVICE DESIGNED TOWARDS PLUG FLOW CONDITIONS

ii. The distribution of contact time will be non-ideal, and have some spread around the mean, $\bar{t}$.

These issues are discussed by Collins, Selleck, and White (111). Their empirical analysis of devices designed towards plug flow indicates that the fraction remaining may be expressed as $(1 + k\bar{t})^{-3}$, where k is an effective kill rate proportional to the concentration of disinfectant, in this case the combined amperometric chlorine residual. This performance may be thought of as being intermediate between the ideal plug flow performance and that obtained in a completely mixed tank.

To determine the performance of the empirical disinfection device treating varying influent flow, the analysis of Collins et al is expanded with a number of simplifying assumptions. It is felt that these assumptions allow for useful solutions which are consistant with the order-of-magnitude accuracy of the overall analysis.

The first of these assumptions concern the chlorine dosage and the effective kill rate.

1. The kill rate, k, is proportion to the chlorine concentration, (Cl), raised to the n^{th} power.

$$k \propto (Cl)^n \tag{5-20}$$

2. The chlorine concentration is equal to the chlorine dosage, W_{Cl}, divided by the flow through the device, q.

$$(Cl) = W_{Cl}/q \tag{5-21}$$

This is a gross simplification of the chemical process of chlorination, ignoring the interaction of free available and combined available chlorine, chlorine demand and chlorine residual, and the temperature and pH. The exponent n is referred to as a coefficient of dilution, with reported values ranging from 0.8 to 1.5 (113). In the current analysis, n is assumed equal to 1.0. If n is greater than 1.0, the kill rate is more sensitive to the concentrations of disinfectant, and subsequently more sensitive to the influent flow rate when the chlorine dosage rate is constant and not metered in proportion to flow. Note that while chlorine is assumed to be the disinfectant, the analysis is also applicable to other disinfectants, as discussed later in the development of an alternative disinfection model. Assumptions 1 and 2 allow a determination of the kill rate given the influent flow and rate of chlorine dosage. If W_{Cl} is constant (i.e., the dosage mechanism is triggered when a storm begins, but dosage is not proportioned to flow) the kill rate will be high during small storms and decrease as the flow increases. If the chlorine dosage is proportioned to the flow, a constant kill rate may be maintained. Note that the improved mixing of the chlorine (resulting in better kills) which may be caused by the higher velocity gradients associated with larger flows is not incorporated. This may make the analysis somewhat conservative, as is discussed further later in this section.

The relationship between the contact time (residence time), the size of the device, and the influent flow is also important.

3. The mean contact time, $\bar{t}$, equals the volume of the disinfection device, V_D, divided by the influent flow.

$$\bar{t} = V_D/q \tag{5-22}$$

This says that if the flow doubles, the runoff will pass through the system twice as fast. This does not allow for the backup of runoff, which should be analyzed separately as a storage device. The disinfection is considered to be completely in line with the varying runoff flow. This assumption is reasonable for devices which must treat the large surges associated with storm runoff.

A storm is assumed to be defined by an average flow. The variation between storms is assumed to be much bigger than the variation within storms. The probability distribution function of average storm flows will determine the long term performance.

4. Runoff flows are assumed to be exponentially distributed, with a mean Q_R. The probability density function of average storm flows is thus:

$$p_q(q) = \frac{1}{Q_R} e^{-q/Q_R} \tag{5-23}$$

This would indicate a coefficient of variation for the flow, ν_q, equals to one. When ν_q is greater than one, or if the within storm variation is large, the disinfection device will perform more poorly than predicted in these analyses.

The final assumption concerns the relationship between the runoff flow and coliform counts.

5. The concentration of bacteria in the runoff (c) is independent of the flow.

If bacteria concentrations are positively correlated with flow (i.e., greater flows have higher bacteria concentrations), the disinfection device will perform more poorly than predicted here. If higher bacteria concentrations are associated with lower flows, performance will be underestimated.

Given the assumptions and the empirical performance equation for the disinfection system designed towards plug flow, the fraction remaining, f(q), of bacteria as a function of flow is determined.

The basic equation for the empirical device is:

$$f(q) = (1 + k\bar{t})^{-3} \tag{5-24}$$

where $k \propto \frac{W_{Cl}}{q}$, and $\bar{t} = \frac{V_C}{q}$

Note that k is constant when the chlorine dose is regulated proportional to the influent flow.

The following terms are now defined:

$(k\bar{t})_R$ = the value of $(k\bar{t})$ at the mean runoff flow, Q_R

f_R = the fraction remaining at the mean runoff flow

$= 1 + (k\bar{t})_R^{-3}$

The expression for f(q) for the flow proportioned dosage (constant k) can be derived as:

$$f(q) = \frac{q^3}{(q+h)^3} \tag{5-25}$$

where $h = Q_R(f_R^{-1/3} - 1)$

The fraction remaining for the case where W_{Cl} is constant is:

$$f(q) = \frac{q^6}{(q^2 + hQ_R)^3} \tag{5-26}$$

Equations (5-25) and (5-26) are plotted in normalized form in Figure 5-53 for the case where $f_R = 10^{-4}$. Note that as expected, the device with the flow regulated chlorine dosage is less sensitive to flow. In both cases, however, the disinfection performance is considerably reduced at high flows (intense storms) where a large fraction of the total bacteria load is discharged.

The long term fraction of the stormwater bacteria load remaining after disinfection is calculated as an expected value:

$$\frac{\bar{M}}{M_R} = \frac{\int_{c=0}^{\infty} \int_{d=0}^{\infty} \int_{q=0}^{\infty} f(q)\ cdq\ p_c(c)\ p_d(d)\ p_q(q)\ dc\ dd\ dq}{\bar{c}DQ_R} \tag{5-27}$$

Assuming q is independent of c and d:

$$\frac{\bar{M}}{M_R} = \frac{1}{Q_R} \int_{q=0}^{\infty} f(q)\ q\ p_q(q)\ dq \tag{5-28}$$

For the flow regulated chlorine dosage:

$$\frac{\bar{M}}{M_R} = \frac{1}{Q_R^2} \int_{q=0}^{\infty} \frac{q^4}{(q+h)^3}\ e^{-q/Q_R}\ dq \tag{5-29}$$

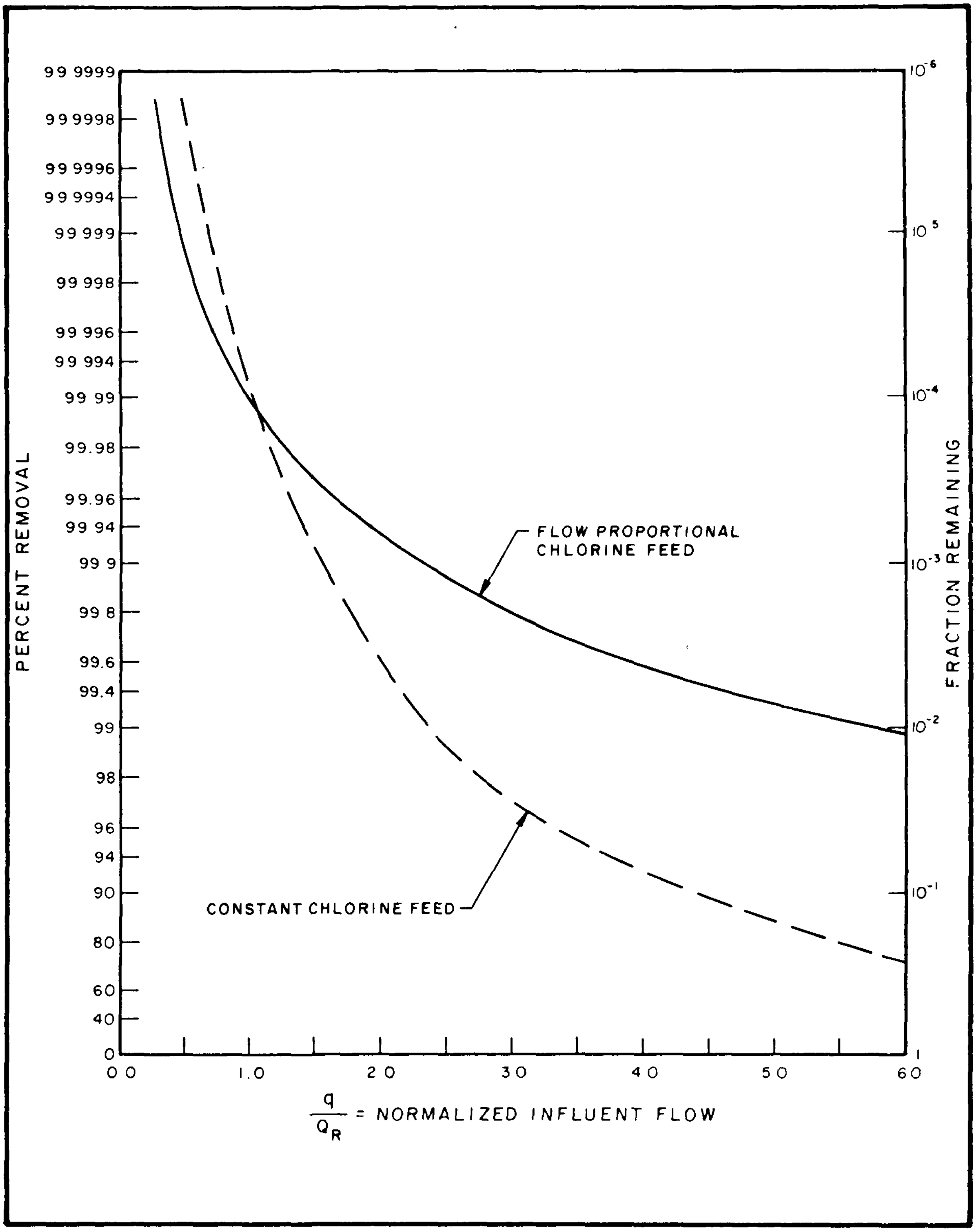


FIGURE 5-53

DISINFECTION RATING CURVE FOR EMPIRICAL DEVICE WITH FOUR ORDER REDUCTION AT MEAN RUNOFF FLOW

For the constant chlorine dosage:

$$\frac{\bar{M}}{M_R} = \frac{1}{Q_R{}^2} \int_{q=0}^{\infty} \frac{q^7}{(q^2+hQ_R)^3} e^{-q/Q_R} dq \qquad (5\text{-}30)$$

Equations (5-29) and (5-30) are evaluated with numerical integration and the results are displayed in Figure 3-24, reproduced as Figure 5-54. As discussed in Section 3.6.1.4.3, Figure 5-54 demonstrates the effect of flow variability on the average performance of disinfection. The long term performance of a system with a proportionally regulated chlorine dosage subjected to varying storm flows is about one order of magnitude poorer than a system receiving a constant flow rate (i.e., the mean runoff flow, Q_R). The long term performance of the system treating varying flows with a constant chlorine dosage is about two to three orders of magnitude poorer than the constant flow device.

Alternative Disinfection Model

In a recent study of high-rate disinfection in Rochester, New York, Geisser and Garver (114) perform multiple regression analysis on disinfection performance data. Their analysis for high-rate chlorine disinfection yields the following equation:

$$\log\left(\frac{N_1}{N_2}\right) = 0.04223\ C_1^{0.66245}\ G^{0.28090}\ T^{0.45611}\ 10^{-0.0043C_2-0.00456C_3} \ldots (5\text{-}31)$$

where $\log\left(\frac{N_1}{N_2}\right)$ = log kill = $\log\left(\frac{\text{influent F. Coli}}{\text{effluent F. Coli}}\right)$

C_1 = concentration of chlorine (mg/l)

G = velocity gradient (minutes^{-1})

T = nominal contact time (minutes)

C_2 = concentration of TKN (mg/l)

C_3 = concentration of BOD_5 (mg/l)

The disinfection performance improves with larger chlorine concentration [C_1 = (Cl)], longer detention time, and higher velocity gradients (improved mixing). Higher concentrations of TKN and BOD_5 reduce disinfection efficiency, reportedly because they react with chlorine, reducing the disinfectant available for bacteria reduction. The study of Geisser and Garver should be referred to for a more detailed discussion of the conditions of the experiment, limitations in the range of the independent variables, and the implications of the results.

To compare Equation (5-31) with the previously developed model, the sensitivity to influent flow of each term in the equation is evaluated.

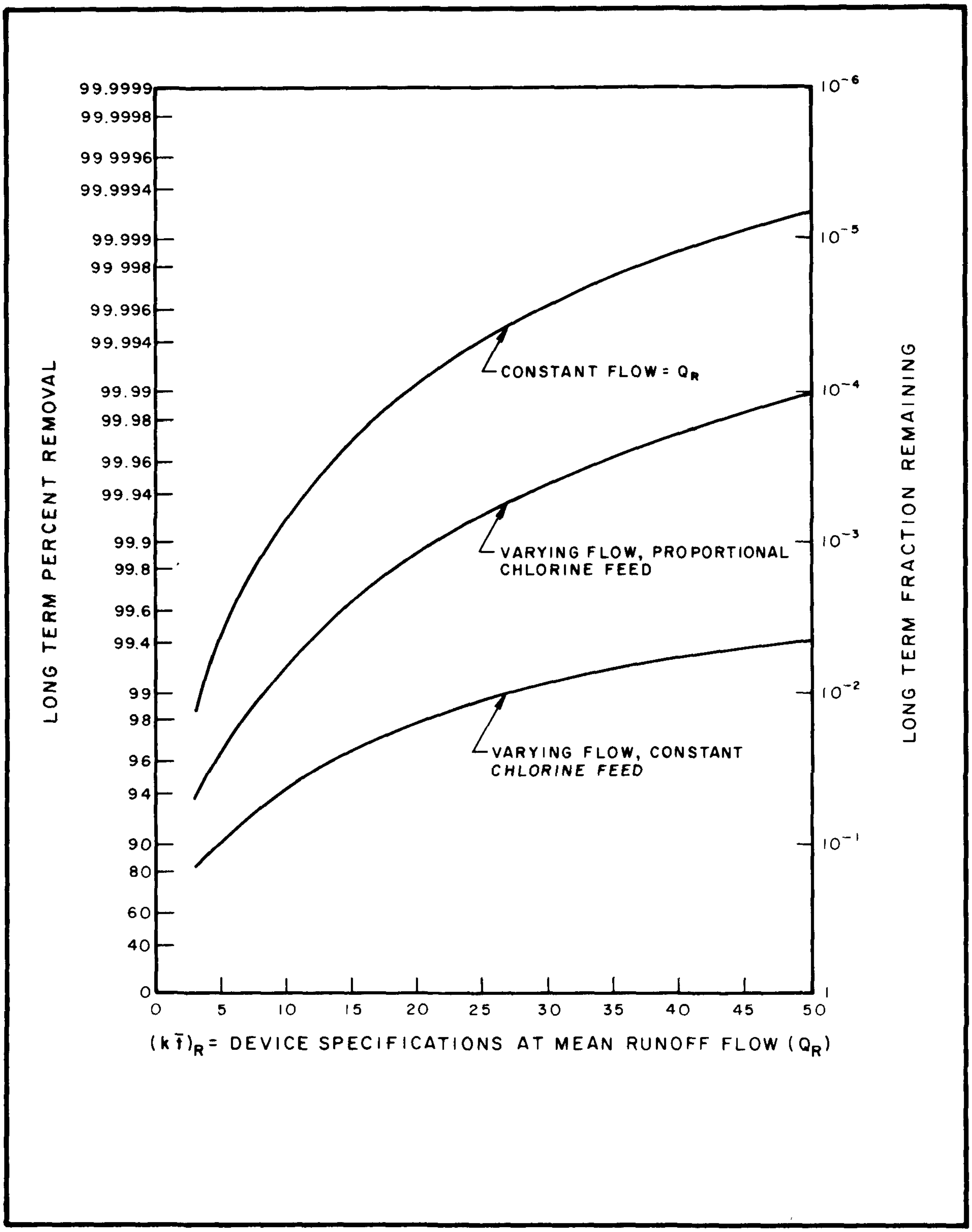


FIGURE 5-54

EFFECT OF STORMFLOW VARIATION
ON PERFORMANCE OF EMPIRICAL DISINFECTION DEVICE

The chlorine concentration is assumed constant when a flow proportional chlorine feed is used, and inversely proportional to the flow when W_{Cl} is constant:

$$C_1 = \text{constant, for proportional chlorine feed} \qquad (5\text{-}32)$$
$$C_1 \alpha \ \frac{1}{q}, \text{ for constant chlorine feed}$$

The contact time is inversely proportional to the flow:

$$T \ \alpha \ \frac{1}{q} \qquad (5\text{-}33)$$

The velocity gradient is proportional to the square root of the velocity, and thus proportional to the square root of the flow:

$$G \ \alpha \ q^{1/2} \qquad (5\text{-}34)$$

The BOD_5 and TKN concentrations are assumed independent of the flow. Combining these assumptions with Equation (5-31) yields the following relationships:

For the device with a flow proportional chlorine feed:

$$\log \left(\frac{N_1}{N_2}\right) \ \alpha \ q^{-0.3} \qquad (5\text{-}35)$$

For the device with a constant chlorine dosage rate:

$$\log \left(\frac{N_1}{N_2}\right) \ \alpha \ q^{-1.0} \qquad (5\text{-}36)$$

This allows a determination of the removal flow relationship given the removal at the mean runoff flow, Q_R. This is demonstrated and compared to the previous model in Figure 5-55, for a device which yields a 4 order bacteria reduction at the mean runoff flow (10^{-4} remaining). The relationships are quite similar and expected values of the long term average performance should likewise be similar (i.e., Figure 5-54 would be similar using the alternative model). This provides further support for the findings of the current study.

Garver and Geisser also evaluated the effectiveness of chlorine dioxide with the following results:

$$\log \left(\frac{N_1}{N_2}\right) = 0.95229 \ C_1^{0.62897} \ G^{0.05022} \ T^{0.07812} \ 10^{0.00314C_2 - 0.00719C_3} \ldots (5\text{-}37)$$

where the terms are the same as in Equation (5-31), except C_1 is the concentration of ClO_2 (mg/l) rather than Cl_2. Note the low level of dependency on contact time, T. The flow relationships are thus:

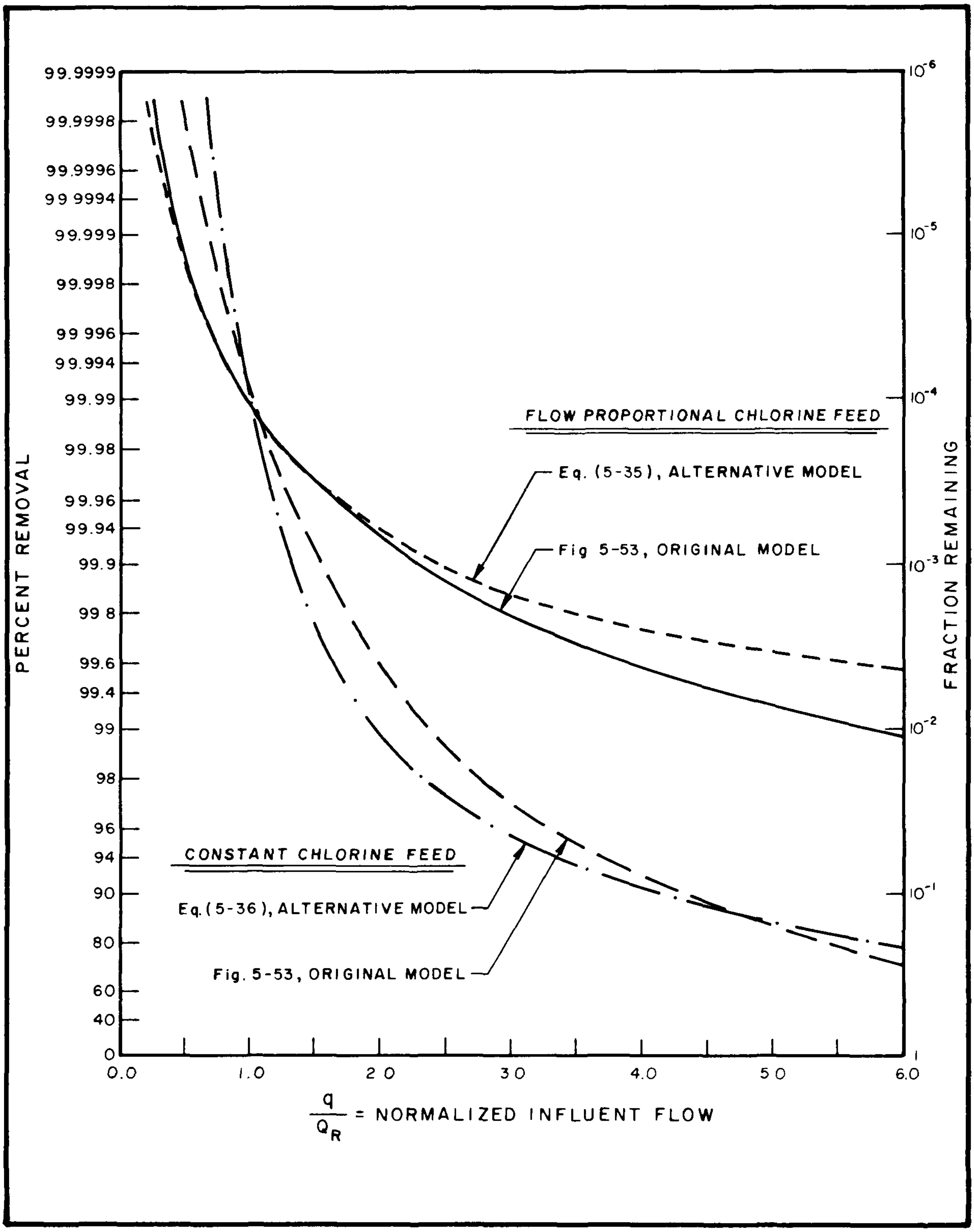


FIGURE 5-55

COMPARISON OF DISINFECTION RATING CURVES
FOR ALTERNATIVE MODELS
(ASSUME FOUR ORDER REDUCTION AT MEAN RUNOFF FLOW)

For a device with a flow proportional chlorine dioxide feed:

$$\log \left(\frac{N_1}{N_2}\right) \alpha \, q^{-0.05} \tag{5-38}$$

For a device with a constant chlorine dioxide dosage rate:

$$\log \left(\frac{N_1}{N_2}\right) \alpha \, q^{-0.7} \tag{5-39}$$

The sensitivity to flow is lower in both cases when ClO_2 is used rather than Cl_2. This would yield flatter performance curves in Figure 5-55 and less subsequent deterioration in the long term average performance due to varying influent flows. This type of conclusion may also apply to ozone due to its rapid oxidation properties.

Garver and Geisser perform a cost/benefit analysis which shows that chlorine is more cost effective than chlorine dioxide for the treatment of Rochester combined sewage. Their analysis assumes a constant, design influent flow rate at the treatment plant. It would be interesting to see if this conclusion is still upheld if an analysis allowing for varying influent flows is performed.

Observed Disinfection Reductions

The primary conclusion of the current disinfection analysis is that long term reductions of bacteria organisms in variable stormwater overflow will probably be about one to two orders of magnitude, rather than the four order reductions usually obtainable under fixed influent conditions. A recent study of a New Orleans wet weather disinfection program tends to support this assertion Pontius, Pavia, and Crowder (115) report on the decrease in coliform bacteria levels in New Orleans stormwater outfall channels after the installation of a sodium hypochlorite (NaOCl) disinfection system. They report that tests of the treated water indicated 99.99% or greater removals of coliform when a 0.5 mg/l chlorine residual (total available) was present. After actual system operation, however, "...long term fecal coliform levels were reduced by one order of magnitude in each outfall canal." (115, Abstract) Total coliform levels were not reduced due to aftergrowth-recovery effects. While aftergrowth and other sources of coliform bacteria (i.e., sediments) may have affected the results, the one order fecal coliform reductions in the outfall canals (as opposed to the four order reductions under test conditions) are consistent with the results presented in this manual.

5.5.2.8 Treatment at Dry Weather Plants

Municipal treatment plants in areas served by a combined sewer system receive a portion of the stormwater runoff. The amount of wet weather flow reaching the plant is dependent on regulator settings in the sewerage system and the influent capacity at the plant. While a treatment plant receiving combined sewage may be modeled by calculating the portion of the runoff captured and conveyed to the plant with DWF, an alternative approach for an initial assessment is to consider the entire system as a flow sensitive treatment device. The overall percent removal decreases with increasing runoff flows due to both the deteriorated performance at the plant and the bypassing of overflows. This section presents considerations for this type of analysis at dry weather treatment plants.

The quantity of wet weather flow which can be handled in a treatment unit operation is dependent on the hydraulic design. Dry weather treatment plants are typically sized for a peak flow rate of about three times the average dry weather flow. Flows in excess of the design result in flooding and overflow of the tanks. For treatment plants in a combined sewer area, hydraulic design flows might be substantially higher (ie. 10-15 times dry weather flow). For treatment plants capable of accepting these excess flows, the performance of the treatment system deteriorates as the flow through the plant increases. There are many types of dry weather plants, including primary or physical treatment plants, secondary in the form of activated sludge and its several modifications, and tertiary treatment plants with biological nutrient removal. Conventional activated sludge treatment is discussed here for simplicity, however, the effect of storm flows on other treatment systems is similar and is governed by the same factors.

Increased storm flows result in an increased loading rate on primary settling tanks, resulting in higher suspended solids concentrations in the primary effluent. This results in increased loadings (BOD and SS) to the secondary or biological system. The increased organic loading on the biological system, in conjunction with the shorter detention time in the aeration tanks, result in a reduction in removals of BOD, If the increased organic loading is sustained for a sufficient period of time (long duration storms), the settling characteristics of the activated sludge may be altered due to floc dispersion. This results in poorer settling of the sludge in the secondary clarifiers. For shorter durations of increased flowrates, the increased loading may be absorbed by the micro-organisms, which have a retention time in the system of one to two days.

Increased organic loadings can also cause decreased dissolved oxygen concentrations in the aeration tanks again resulting in lower BOD removals, deterioration in settling characteristics, and odor. Sufficient oxygen must be supplied to the biological organisms for good treatment performance.

The increased organic loading rates result in more biological solids production and together with the increased primary solids from the storm flow, result in an increase in the amount of solids requiring disposal. Solids disposal facilities must be capable of handling these additional loads.

The major problem associated with excess storm flows is the increased loading rates on the secondary clarifier. The increased overflow into the secondary clarifier results in a carryover of biomass in the effluent, a serious problem since if sufficient biomass is washed out, the treatment effectiveness will deteriorate. Although treatment effectiveness deteriorates during storms, some degree of treatment will be maintained if sufficient secondary clarification capacity is provided for these excess storm flows.

Since there are many parameters that effect the performance of activated sludge and other dry weather treatment plants, no attempt has been made to relate overall performance to flowrate. This relationship would be a specific one for each plant evaluated. Many factors should be considered in evaluating the amount of flow a dry weather plant can accomodate. An evaluation would also suggest possible modifications to the treatment facilities to increase the capacity to handle stormwater flow. Some of the possible modifications are discussed in the remainder of this section.

Flow equalization by storage would dampen the variation in hydraulic and organic loading rates and minimize some of the problems discussed previously. Modification of the activated sludge system to a contact stabilization process is another possibility. A portion of the existing system could be used for a contact tank in which a good portion of the influent BOD is adsorbed onto the biological floc. The sludge would then be settled and sent to a stabilization tank. The overflow from the clarifier could be chlorinated and discharged. Sufficient clarification and recycle capacity must be provided to prevent solids carryover at peak flowrates, and to pass the settled biomass from the clarifier to the sludge stabilization tank. The typical operating parameters for contact stabilization should be checked to determine if additional tank volume, recycle pumping, or additional oxygen capacity are required. Additional units could be provided next to the dry weather plant as done in Kenosha, Wisconsin (35).

Modification to a Step Aeration system is another possibility for increasing the capacity of an existing plant. Piping arrangements can be made to introduce the wastewater to a number of points along the aeration tank to reduce the initial oxygen demand at the head end and make more efficient utilization of the biomass. Equivalent performance could be obtained by treating more flow in the same tank volume.

5.6 Cost Estimates for Treatment Alternatives

This section presents a basis for estimating the capital, operation and maintenance costs of stormwater control systems. Most of the costs presented are based on the work of Culp/Wesner/Culp (CWC) in a report entitled, "Estimating Construction Costs and Operating and Maintenance Requirments for Combined Sewer Overflow, Storage and Treatment Facilities", published by the EPA in May 1976 (116). In the CWC report, average construction, operation and maintenance costs are presented for stormwater treatment facilities ranging from 5 to 200 million gallons per day in capacity, and storage facilities ranging in size from 1 to 240 million gallons. Physcial and chemical treatment costs are presented using June 1975 prices as a basis. The report suggests a careful review of the methodology employed if the data is to be used for specific project planning. The cost data are useful for general planning and evaluation of alternatives, however they do not reduce the need for an understanding of local conditions or a recognition of design requirements for specific applications.

This section presents a simplified presentation of the CWC cost work which can be used to evaluate treatment alternatives for an initial assessment. The CWC report presents the construction cost of individual treatment processes as a function of plant size and flow capacity. To determine the total capital cost, engineering design, site preparation, legal and fiscal adminstration, and loan interest costs during construction are added.

In the CWC report, operation and maintenance requirements are presented in several categories: operation and maintenance labor, power, chemical and miscellaneous supplies, administrative costs, laboratory sampling and yardwork. To obtain the total annual operation and maintenance expenditures presented in this section, these individual cost components were determined and added together for each of the control methods. The operation and maintenance costs consist of a fixed cost and a variable cost, both of which are a function of the plant capacity, the number of storms and the number of hours of operation per year. The fixed cost consists of labor for supplies, routine maintenance and repairs, yardwork, laboratory and administrative costs. The major variable costs are the power, chemical requirements, and labor for plant operation and cleanup.

The labor requirements in the CWC report are based on the number of storm events per year. For this presentation, labor costs are shown as a function of plant capacity and the number of hours of operation per year. Labor costs are estimated as $10 per man-hour and power costs as $0.02 per kilowatt-hour. Chemical costs are obtained from a 1975 EPA Technical Report (117). All the costs presented are based on June 1975 prices. The use of any cost estimating technique requires careful consideration of inflation, which recently has been averaging about nine percent per year. In the construction industry, the most frequently used indices are the Engineering-News Record's (ENR) Construction Cost Index and the Building Cost Index (118). A sewage treatment cost index was developed by the EPA to provide a more specific index (119). Cost data must be adjusted by the EPA Sewage Treatment Cost Index or the ENR Building Cost Index projected to the appropriate construction period.

$$\text{Cost Estimate} = \text{1975 Cost Estimate} \times \frac{\text{Index for construction period}}{\text{June 1975 Index}}$$

June 1975 ENR Construction Index: 1306.7

June 1975 EPA Index: 246.5

5.6.1 Structural Treatment Devices

5.6.1.1 Storage Basins

Capital costs for storage basins are presented in Figure 5-56 as a function of basin volume. Costs for earthen basins, and covered and uncovered concrete basins are presented. Costs for earthen basins include earthwork, liner, paving and fencing. The costs assume on-site embankment soil, no rock excavation or groundwater problem. Provisions for mechanical residue collection are not included in these cost estimates. The capital costs for the concrete basins include the concrete forming, reinforcing steel, and in the case of covered basins, the precast concrete members and roofing material.

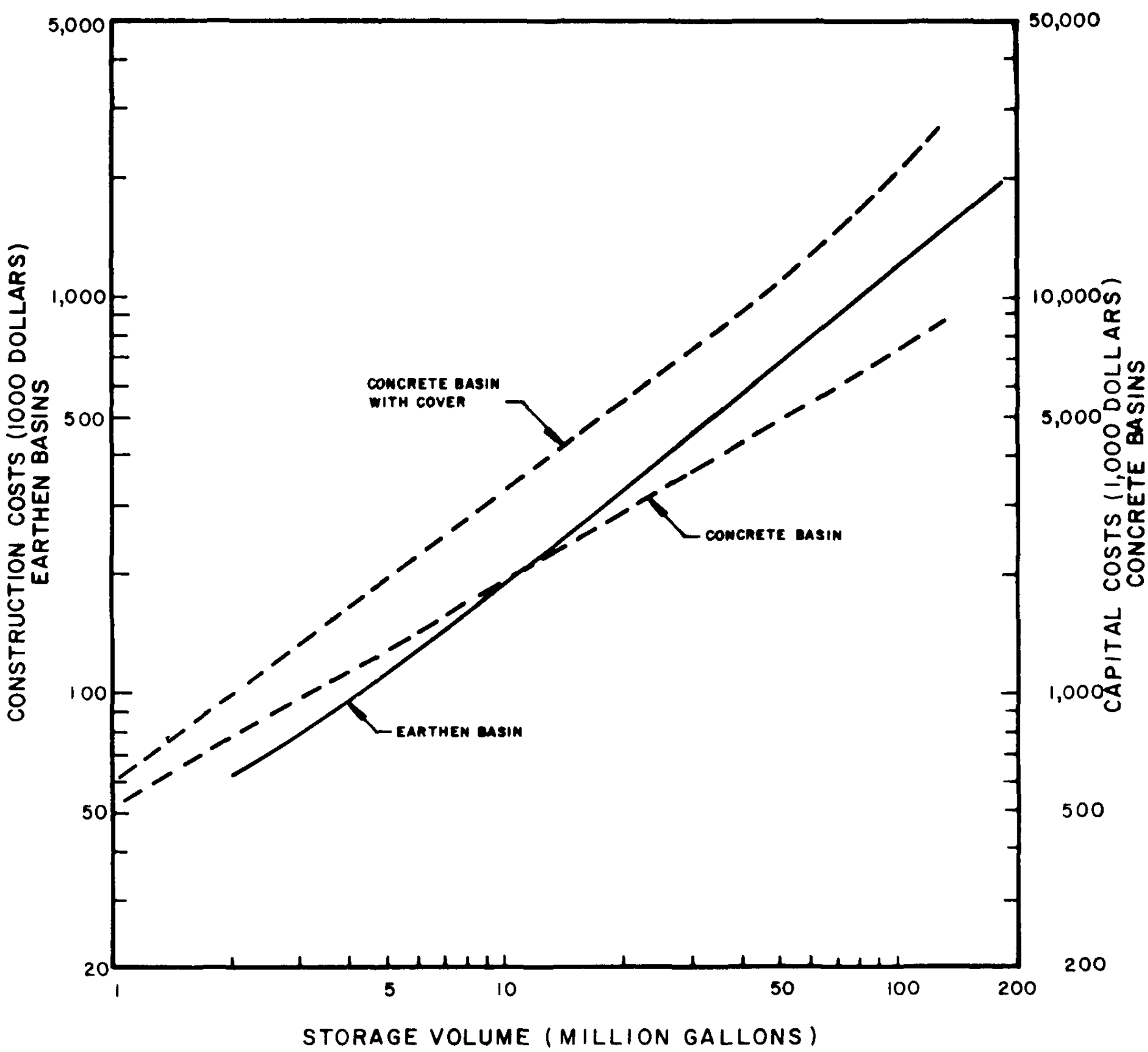


FIGURE 5-56

CAPITAL COST - STORAGE BASINS

(JUNE 1975 COSTS)

The operation and maintenace costs for storage basins are shown in Figure 5-57 for covered concrete basins. These costs include labor at one man-day per storm event, material and supply costs, power for automatic spray cleaning systems, administration, yardwork and laboratory cost.

5.6.1.2 Sedimentation

Capital costs for circular sedimentation basins or settling tanks are presented in Figure 5-58. The costs include manufactured equipment, materials, and sludge collection equipment applicable to circular shaped tanks. The cost should be increased by 15 to 20 percent for straight line sludge collection equipment in rectangular basins. A basin having a 12 foot side water depth and 1.5 foot freeboard were used as a basis for the cost estimates.

The operation and maintenance costs for sedimentation basins are presented in Figure 5-59. The costs include labor requirements for routine visits, plant cleanup and maintenance, materials and supplies, administrative, yardwork and laboratory costs.

5.6.1.3 Air Flotation

Air flotation is a unit process designed primarily on the basis of surface area. Air flotation equipment is furnished in package units which include the tanks and equipment. For field erected systems, the largest practical individual unit is about 20 feet wide and 100 feet long. The capital cost for air flotation units is presented in Figure 5-60 as a function of surface area and design flow capacity. These costs include manufactured equipment, construction materials, piping and equipment gallery, recycle pumping, air retention tanks and flow measurement.

Other unit process typically associated with air flotation for stormwater treatment include chemical feed, rapid mix, flocculation, prescreening, chlorination, and raw wastewater and sludge pumping. Costs for these processes should be added where applicable.

The operation and maintenance costs for air flotation without sludge pumping are presented in Figure 5-61. These costs include labor for routine visits twice per month at two hours each, plant cleanup at 0.004 man-hour per square foot of tank area per storm event, two hours travel time per storm event, an operator at the plant for four hours per event and maintenance at 0.9 man hours per square foot of tank area per storm. Administrative, yardwork, laboratory, power, material, and supply costs are also included.

5.6.1.4 Swirl Concentrator

The capital costs for swirl concentrators ranging from 100 to 2000 square feet of surface area are presented in Figure 5-62. The corresponding design flow for a swirl concentrator is also plotted on the horizontal scale. The costs presented included construction of the basin itself and flow measurement. If influent pumping is required, this cost should be added to the capital cost of Figure 5-62.

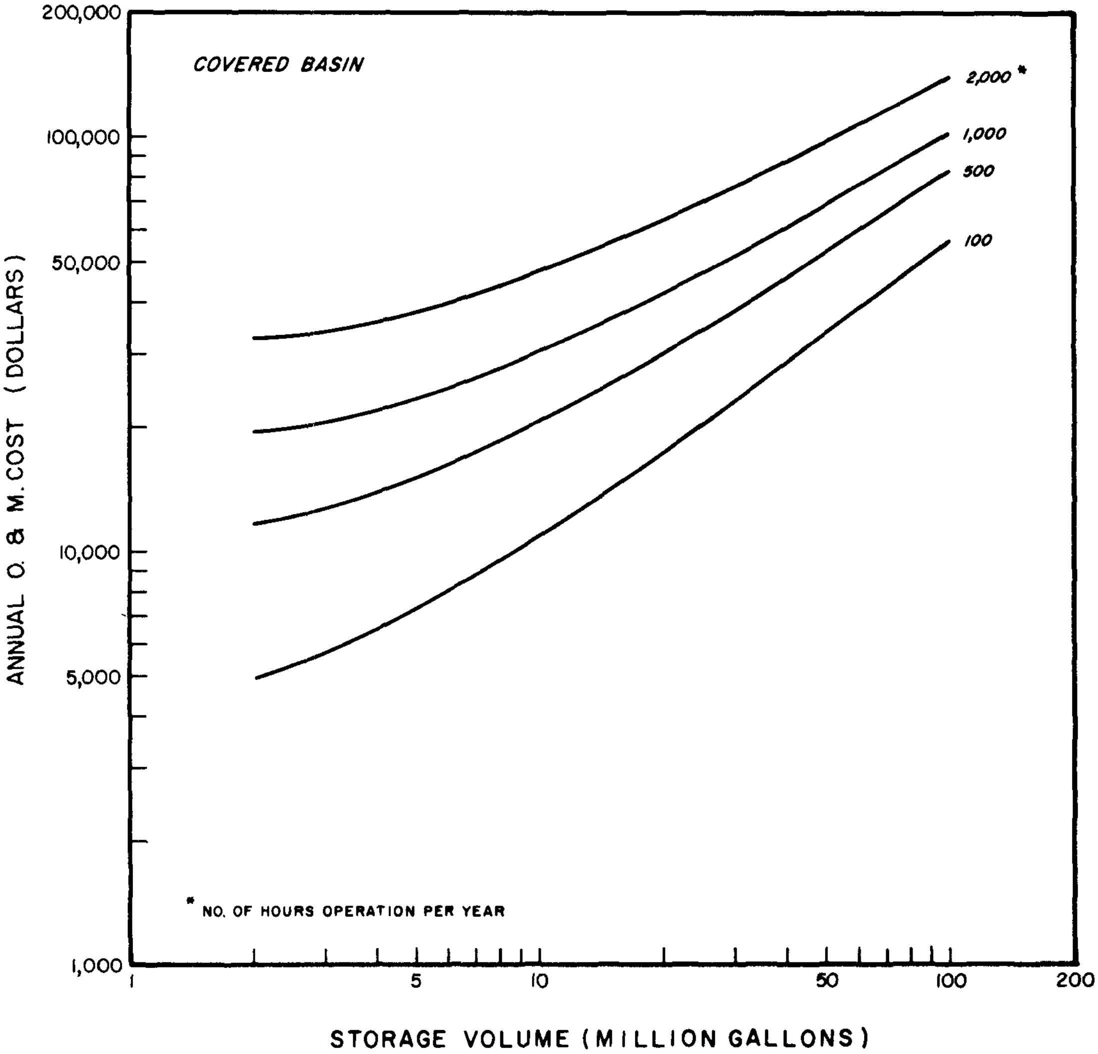


FIGURE 5-57
ANNUAL OPERATION AND MAINTENANCE COST — STORAGE BASINS
(JUNE 1975 COSTS)

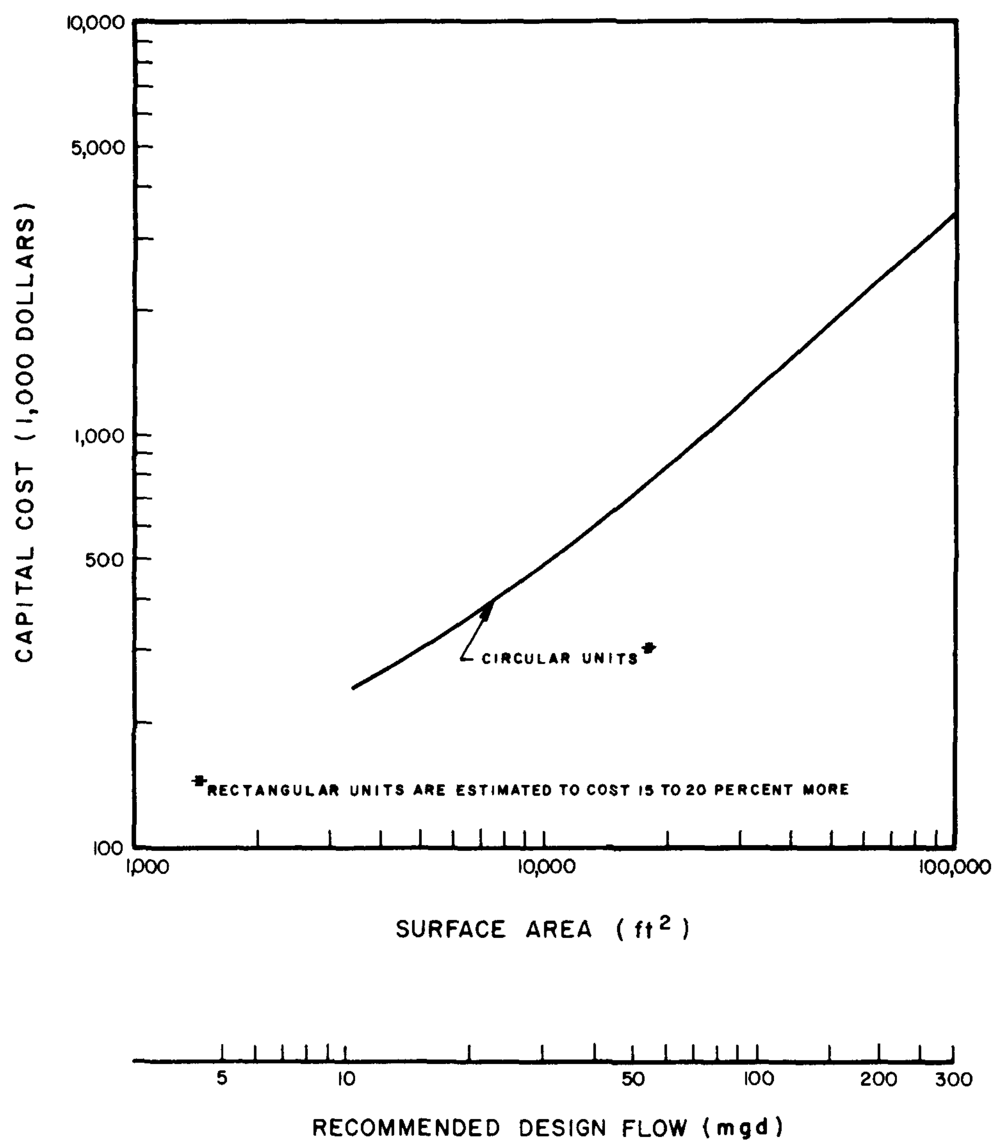


FIGURE 5-58
CAPITAL COST - SEDIMENTATION BASINS
(JUNE 1975 COSTS)

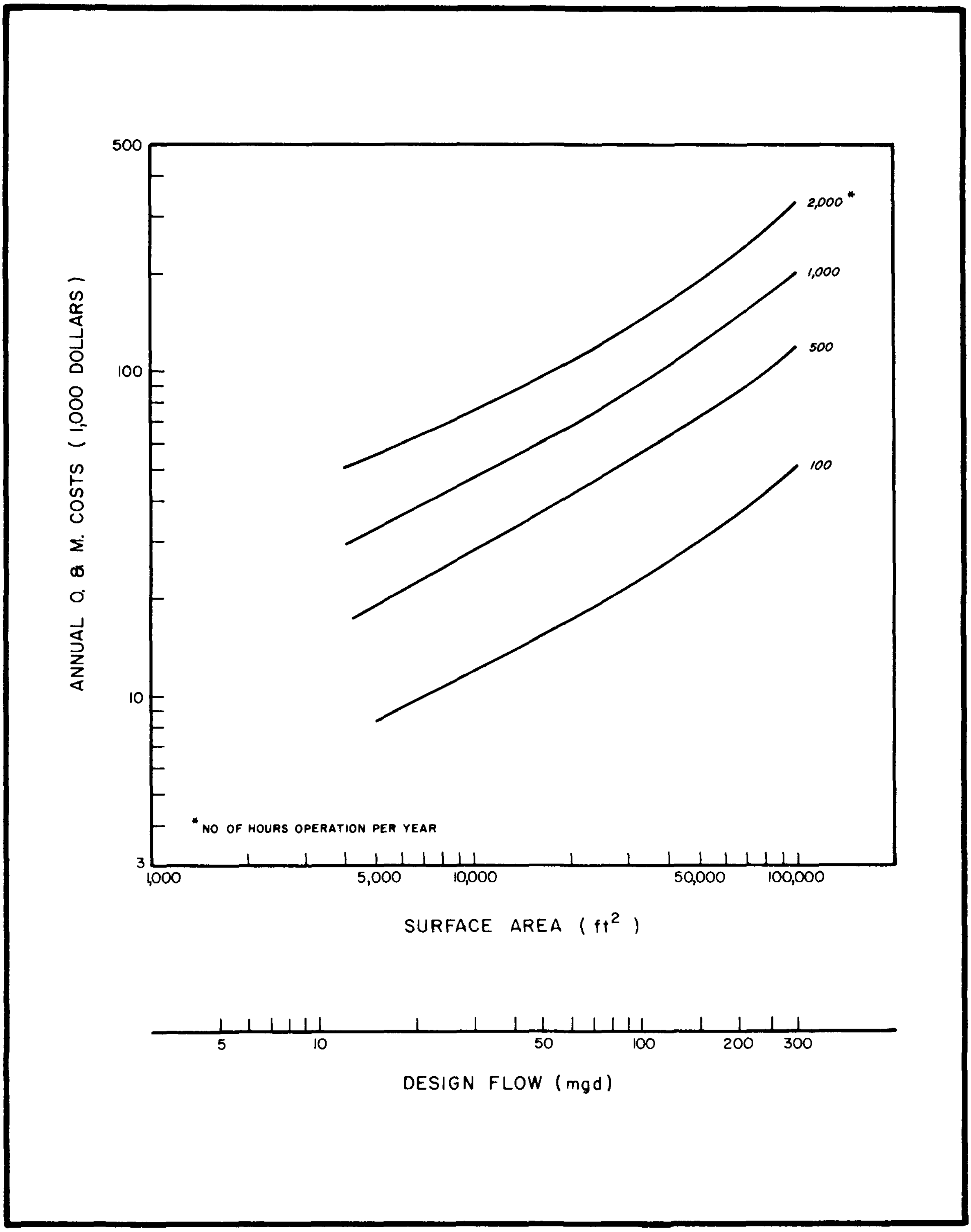


FIGURE 5-59
ANNUAL OPERATION AND MAINTENANCE COST - SEDIMENTATION BASINS
(JUNE 1975 COSTS)

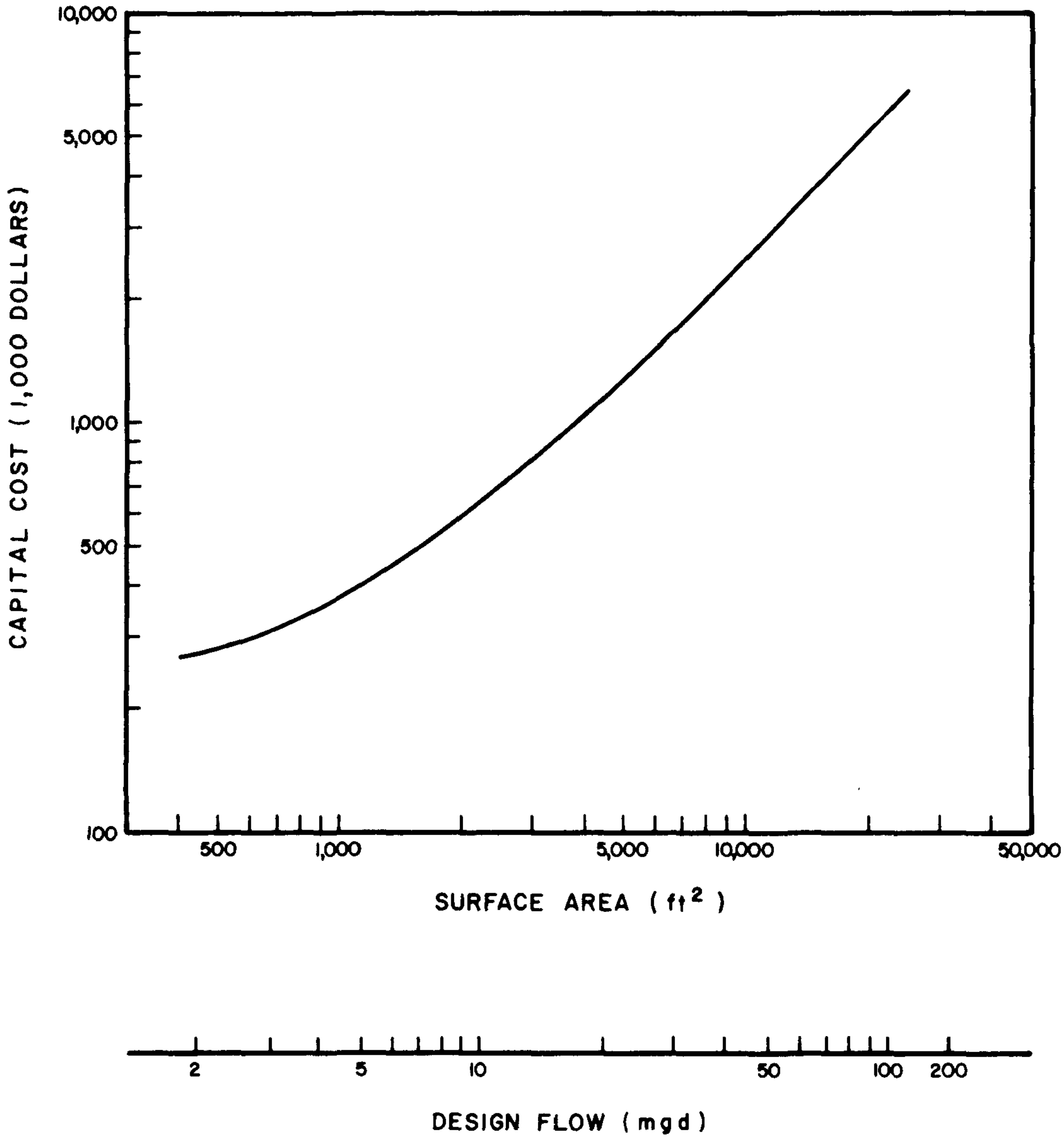


FIGURE 5-60
CAPITAL COST — AIR FLOTATION
(JUNE 1975 COSTS)

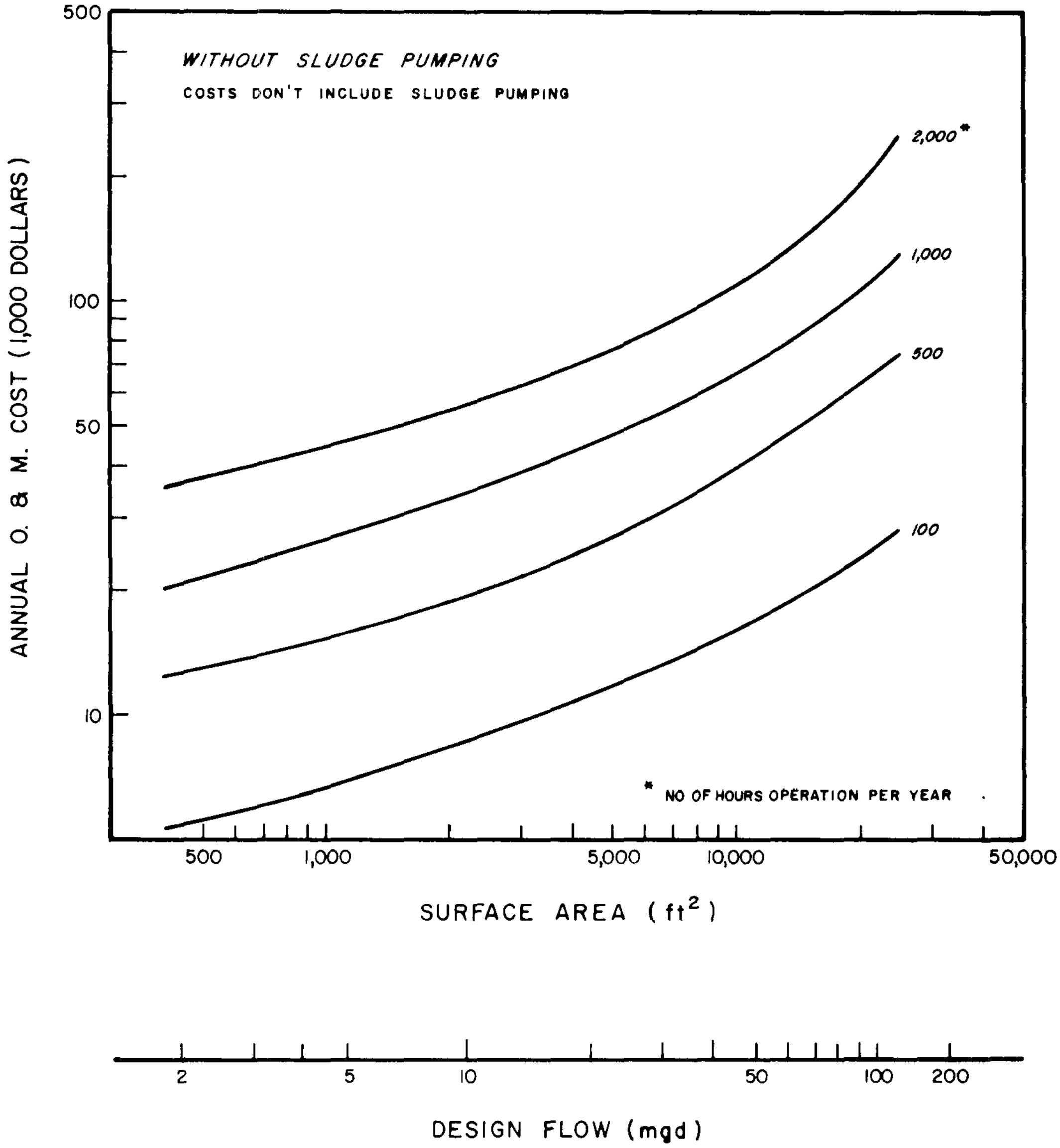


FIGURE 5-61

ANNUAL OPERATION AND MAINTENANCE COST — AIR FLOTATION

(JUNE 1975 COSTS)

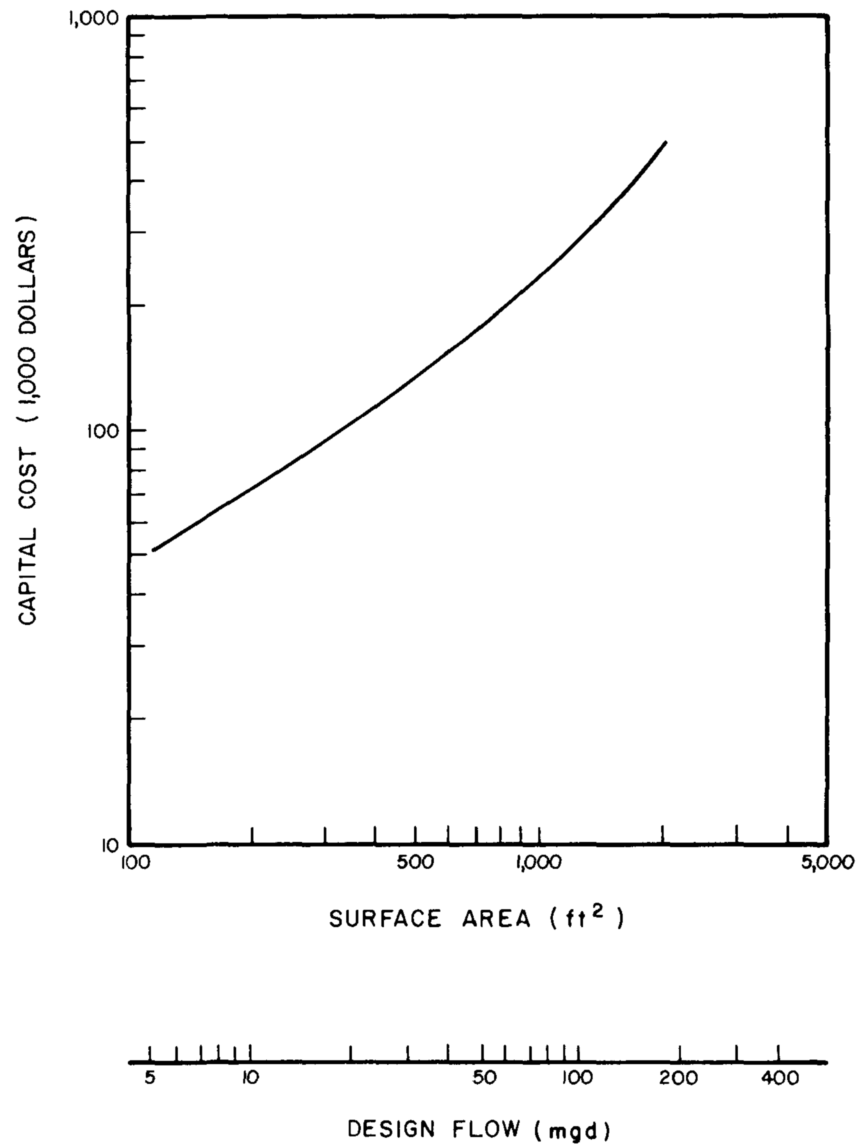


FIGURE 5-62
CAPITAL COST — SWIRL CONCENTRATOR
(JUNE 1975 COSTS)

The annual operation and maintenance costs for a swirl concentrator are shown in Figure 5-63. The costs are presented to include various operational levels and include manpower for routine visits approximately once every other week, cleanup after storms, administrative, laboratory, material and supply costs.

5.6.1.5 High Rate Filtration

The capital costs for high-rate filtration are shown in Figure 5-64 as a function of the surface area. The cost is based on concrete gravity filters and includes dual filter media, construction materials, housing, underdrains, pipes and valves, and instrumentation for automatic filter backwashing. Consideration should be given to pressure filters for operations in excess of 12 to 15 feet of headloss. Other design features should include backwash supply storage and influent flow equalization. This could be incorporated into storage basins preceding the filter.

The operation and maintenance costs for high rate filtration are shown in Figure 5-65. These costs are based on labor costs for routine visits to the facility twice per month for two hours each, plant cleanup after each storm event based on one man-hour per filter, one hour for set-up and shutdown, an operator at the facility for eight man-hours per storm event and plant maintenance of 12 hours per year per filter. Costs for miscellaneous supplies, power, yardwork, laboratory, and admininstration are also included.

5.6.1.6 Stationary Screens

A stationary (static) screen is a wedgewire screen where waste is discharged along a sloping section causing solids to be removed and discharged by gravity, while the screened wastewater flows through the screen to a collector flume below. The capital cost for stationary screens is shown in Figure 5-66 as a function of design flow capacity. A six-foot wide screen having two screen faces is rated at four MGD capacity. The cost includes manufactured equipment, construction materials, collection flumes for sludge and screened effluent, housing, oversized open channels and metal weirs at each screen for flow splitting.

The annual operation and maintenance costs for stationary screens at various operational levels are presented in Figure 5-67. The operation and maintenance costs include: labor for routine visits to the facility of 24 hours per year; labor for two hours of cleanup after each storm event, plus one hour per screen; and costs for materials, supplies, administration, laboratory and yardwork.

5.6.1.7 Horizonal Screens

Horizonal screens, often referred to as microscreens or microstrainers, are installed in chambers designed to permit entry of wastewater to the interior of a drum and discharge of the screened or filtered wastewater from the exterior side of the drum. Construction costs are the same for any of the screen aperature sizes used. The capital cost for horizonal screens is presented in Figure 5-68. The cost includes manufactured equipment,

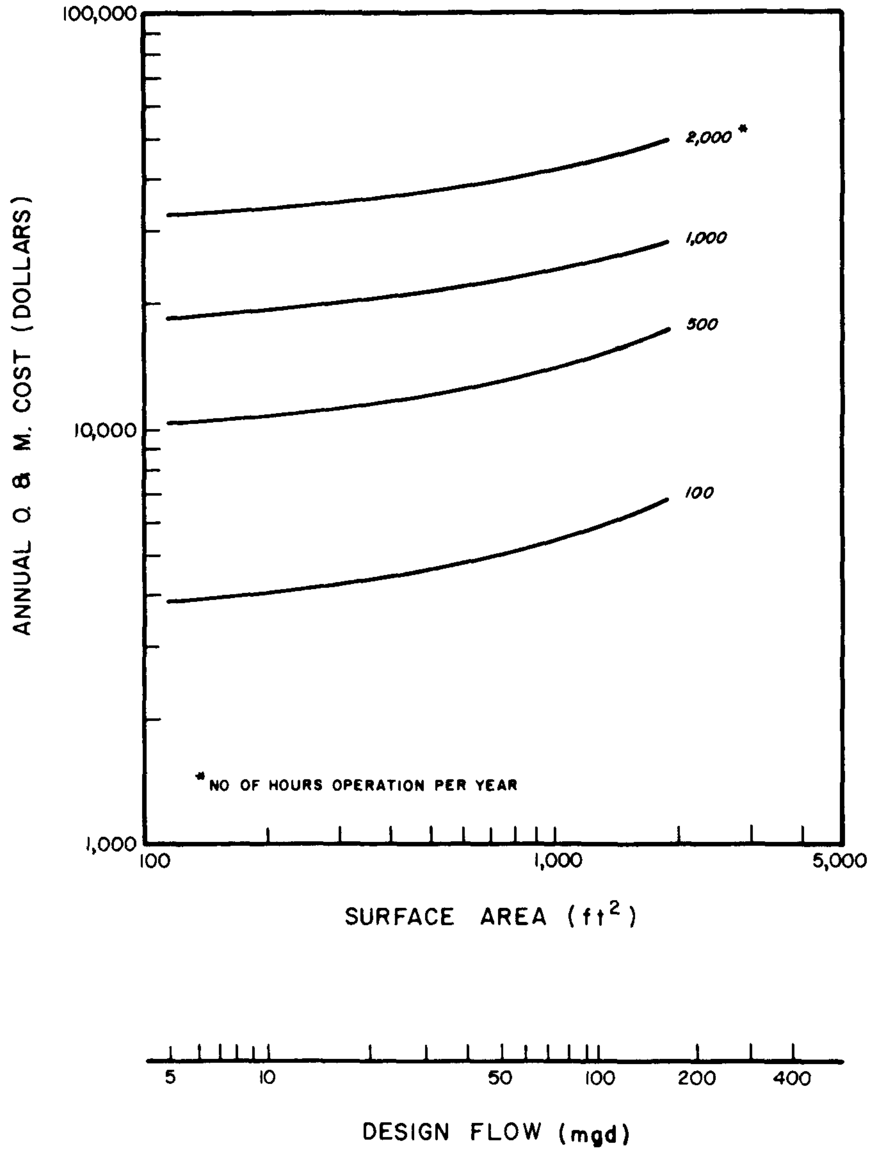


FIGURE 5-63
ANNUAL OPERATION AND MAINTENANCE COST-SWIRL CONCENTRATOR
(JUNE 1975 COSTS)

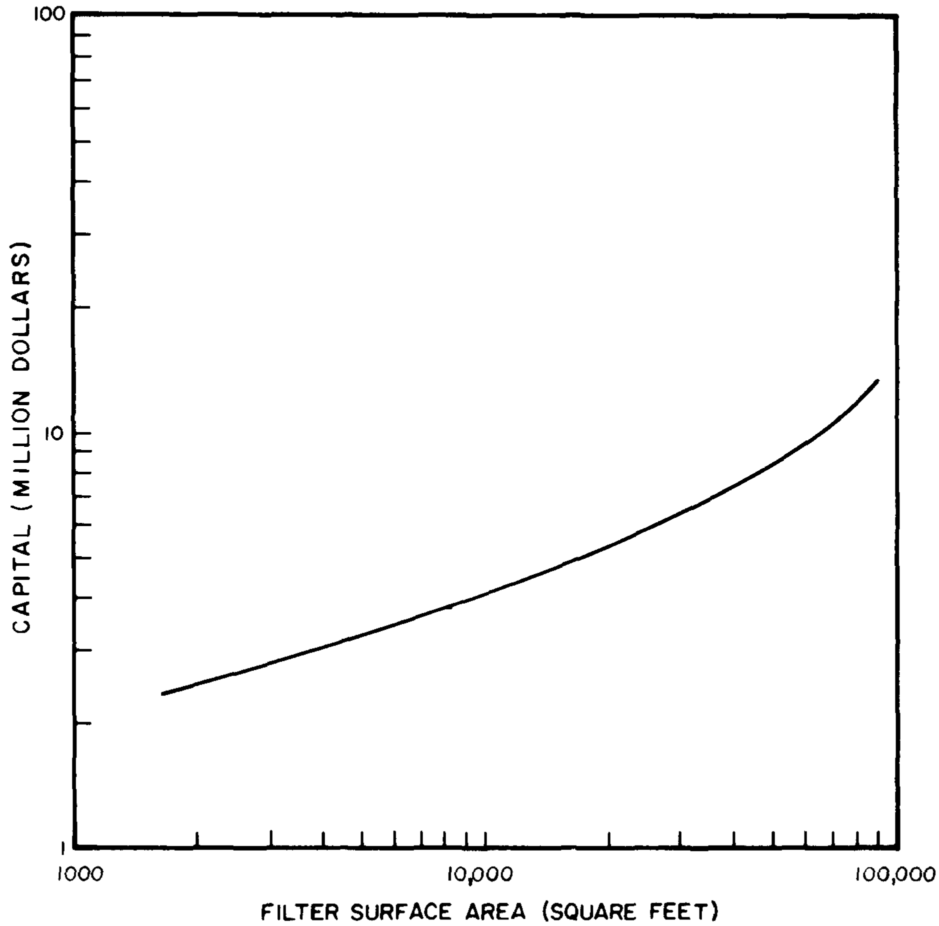


FIGURE 5-64

CAPITAL COST HIGH RATE FILTRATION CONCRETE GRAVITY FILTERS

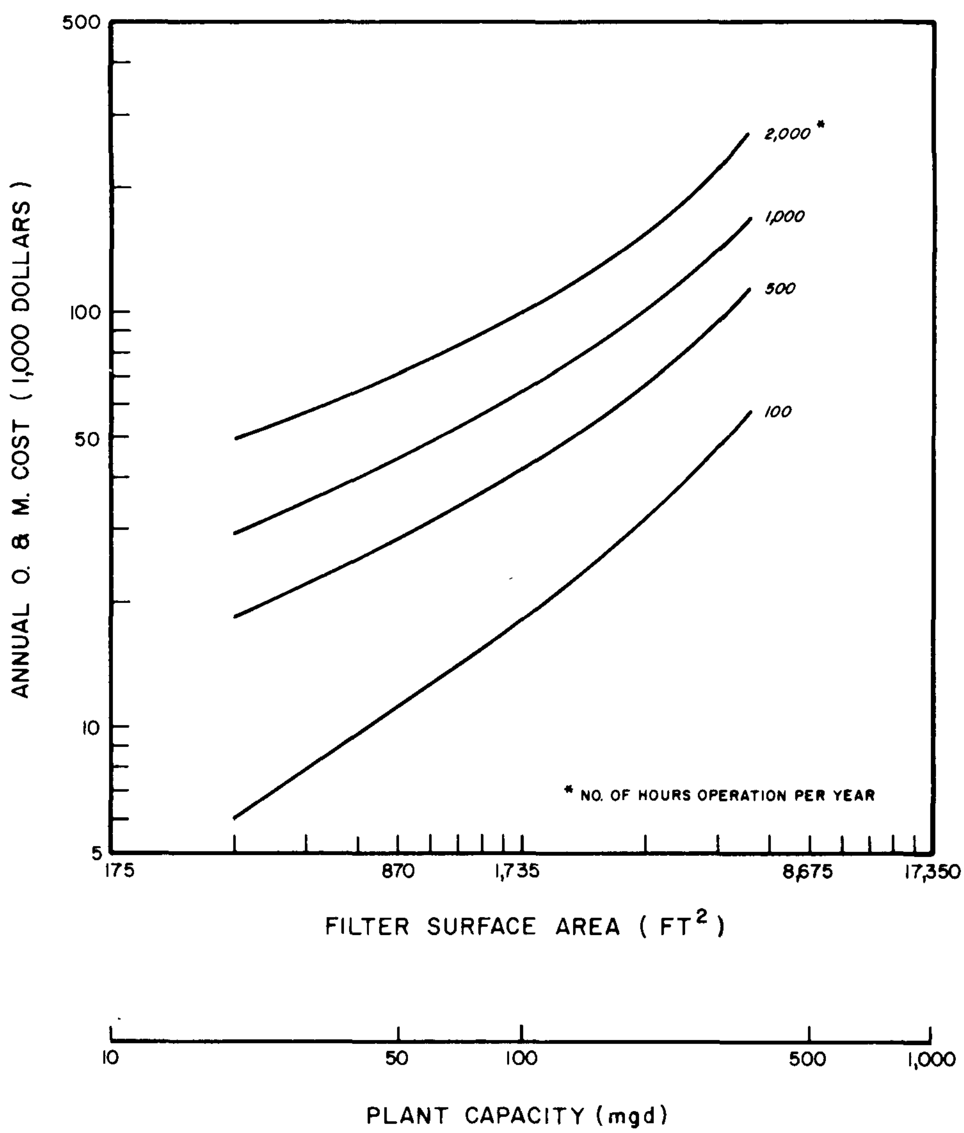


FIGURE 5-65

ANNUAL OPERATION AND MAINTENANCE COST – HIGH RATE FILTRATION

(JUNE 1975 COSTS)

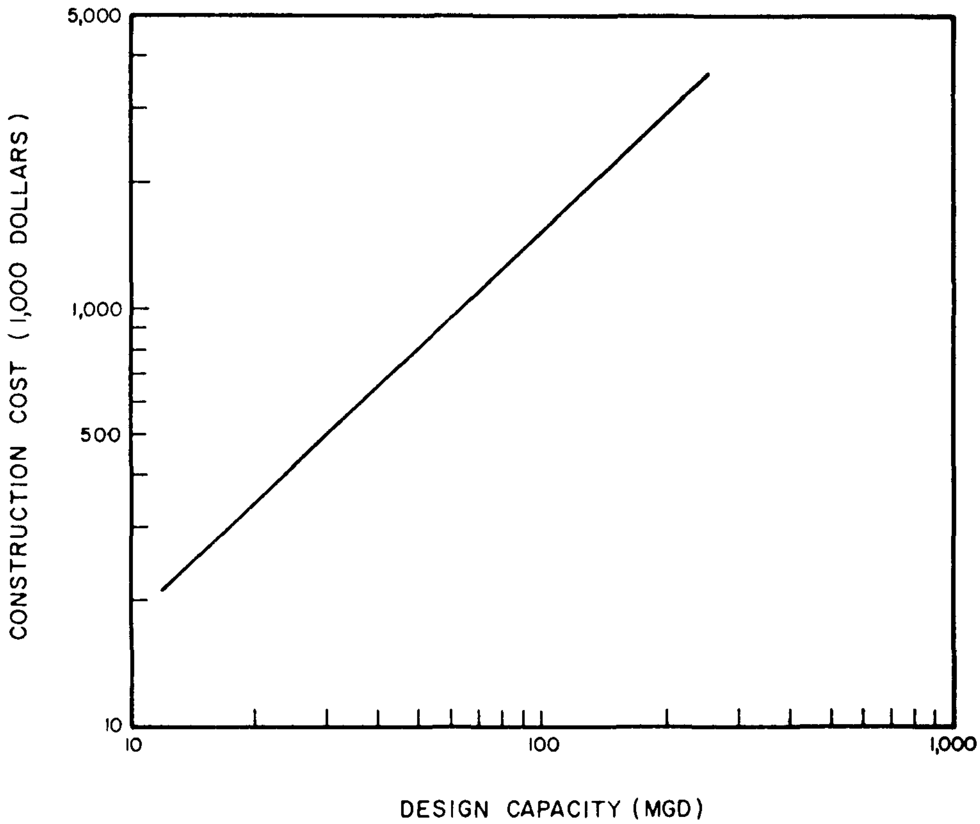


FIGURE 5-66
CAPITAL COST — STATIONARY SCREEN
(JUNE 1975 COSTS)

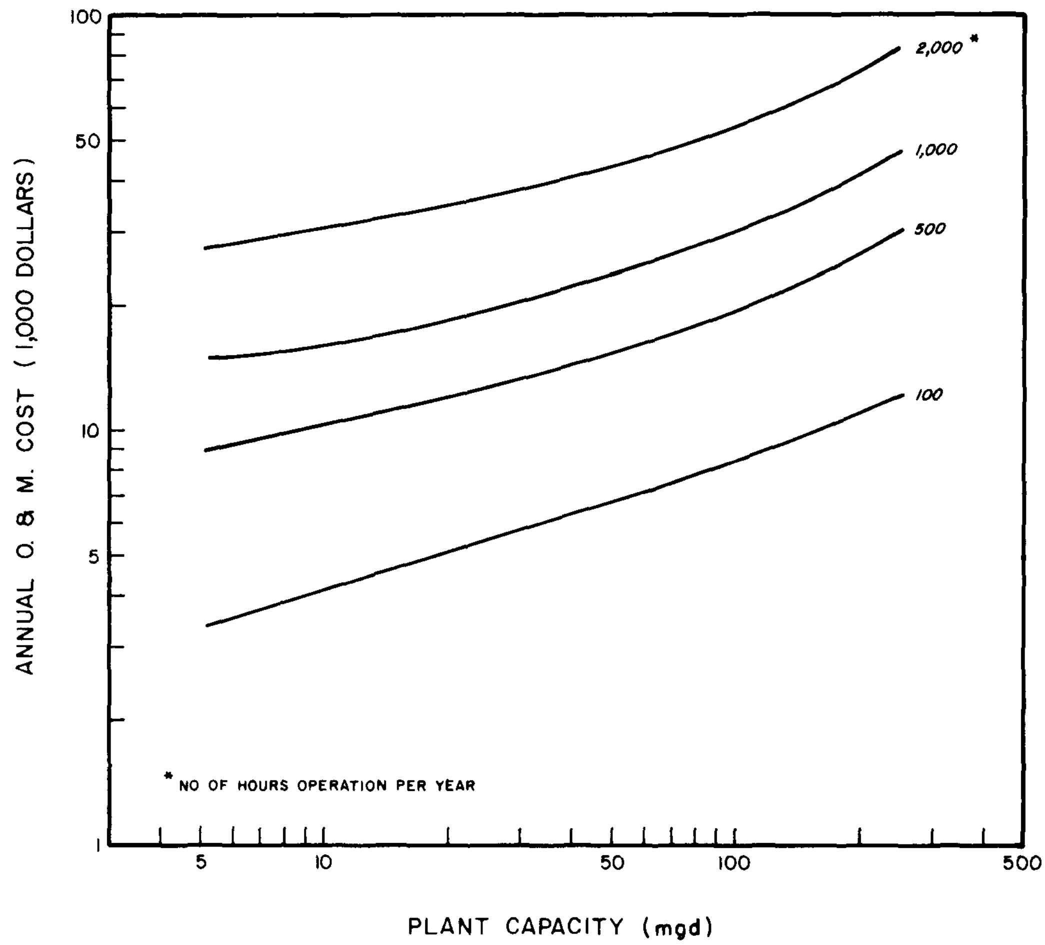


FIGURE 5-67

ANNUAL OPERATION AND MAINTENANCE COST - STATIONARY SCREEN

(JUNE 1975 COST)

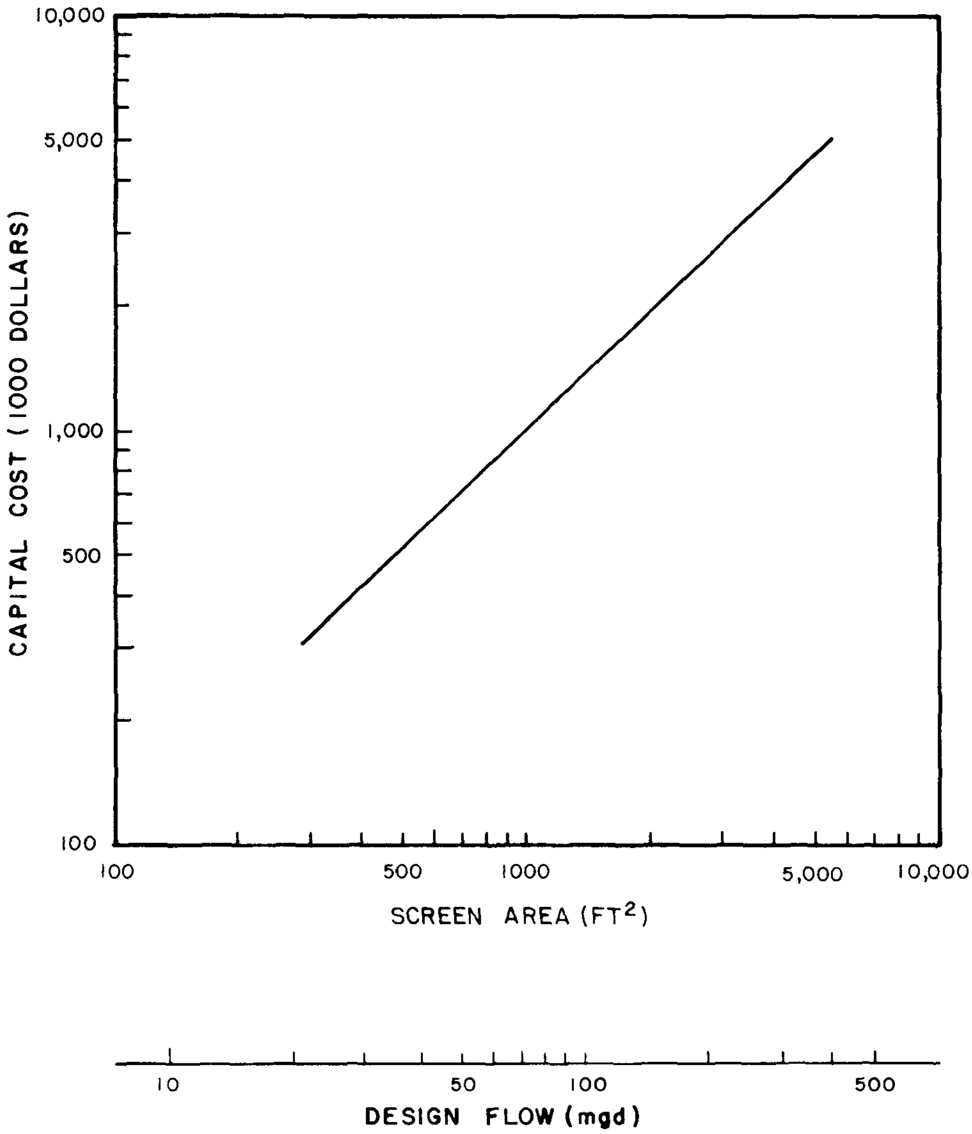


FIGURE 5-68

CAPITAL COST - HORIZONTAL SCREENS

(JUNE 1975 COSTS)

construction material and flow measurement. Concrete construction with a center pipe gallery and housing for the pipe gallery and screen are used to estimate the cost. Ultraviolet light slime growth control, a backwash sprayer, and backwash storage and pumping facilities are also included in the cost estimate.

The operation and maintenance costs for horizonal screens are shown in Figure 5-69. These costs are a function of plant capacity and the number of hours of operation per year. Included are labor for routine visits to the facility twice per month for two hours each, plant cleanup after storms, an operator present at the facility eight hours per day for each event, and maintenance of 24 man-hours per year. Administration, yardwork, laboratory, power, material and supply costs are also included.

5.6.1.8 Chemical Coagulation

The use of chemicals in combined sewer overflow treatment facilities has been employed in conjunction with air flotation, high rate filtration and sedimentation. To estimate the capital cost of chemical treatment, the cost of chemical feed systems should be added to the cost of the unit treatment process which is being utilized. A rapid mix tank should also be included in a chemical coagulation system and the appropriate cost should be added. Capital costs for chemical feed systems are presented in Figure 5-70. These costs include feeder and storage equipment, housing, electrical and instrumentation costs and miscellaneous items. Costs are shown for lime, ferric chloride, alum and polyelectrolyte feed systems.

The operation and maintenance costs for chemical feed systems are shown in Figures 5-71 to 5-74. These costs include chemicals, labor for the unloading of chemicals, operation of chemical feeding equipment, power, materials and supplies. The following chemical costs are used: $25/ton lime, $100/ton alum, $80/ton ferric chloride, and $2/lb polymer.

5.6.1.9 Chlorination Feed Equipment

The capital cost for chlorination feed equipment is presented in Figure 5-75 as a function of the feed capacity. This cost includes manufactured equipment, construction materials, housing, distribution panels, a standby chlorinator, monorail trolley and hoist equipment. These costs are based on the use of gaseous chlorine feed equipment and do not include a mixing or contact basin.

The operation and maintenance costs for chlorination are shown in Figure 5-76 as a function of chlorine usage. These costs include power, chlorine, labor, materials and supplies. Generation and feed costs (per pound of active disinfectant) are approximately:

(a) for chlorine dioxide - 2 times costs for chlorine gas
(b) for hypochlorite - 10 times costs for chlorine gas.

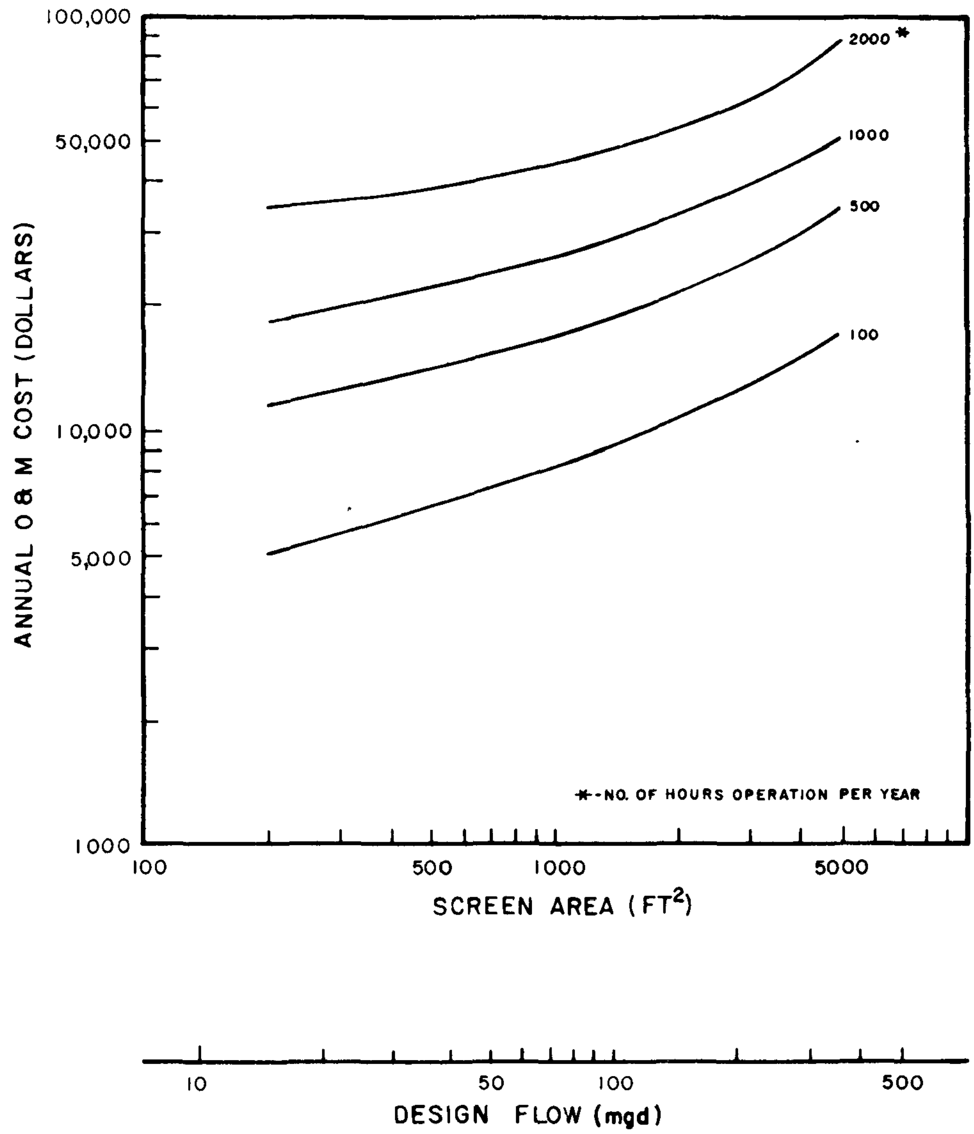


FIGURE 5-69

ANNUAL OPERATION AND MAINTENANCE COST
HORIZONTAL SCREENS
(JUNE 1975 COSTS)

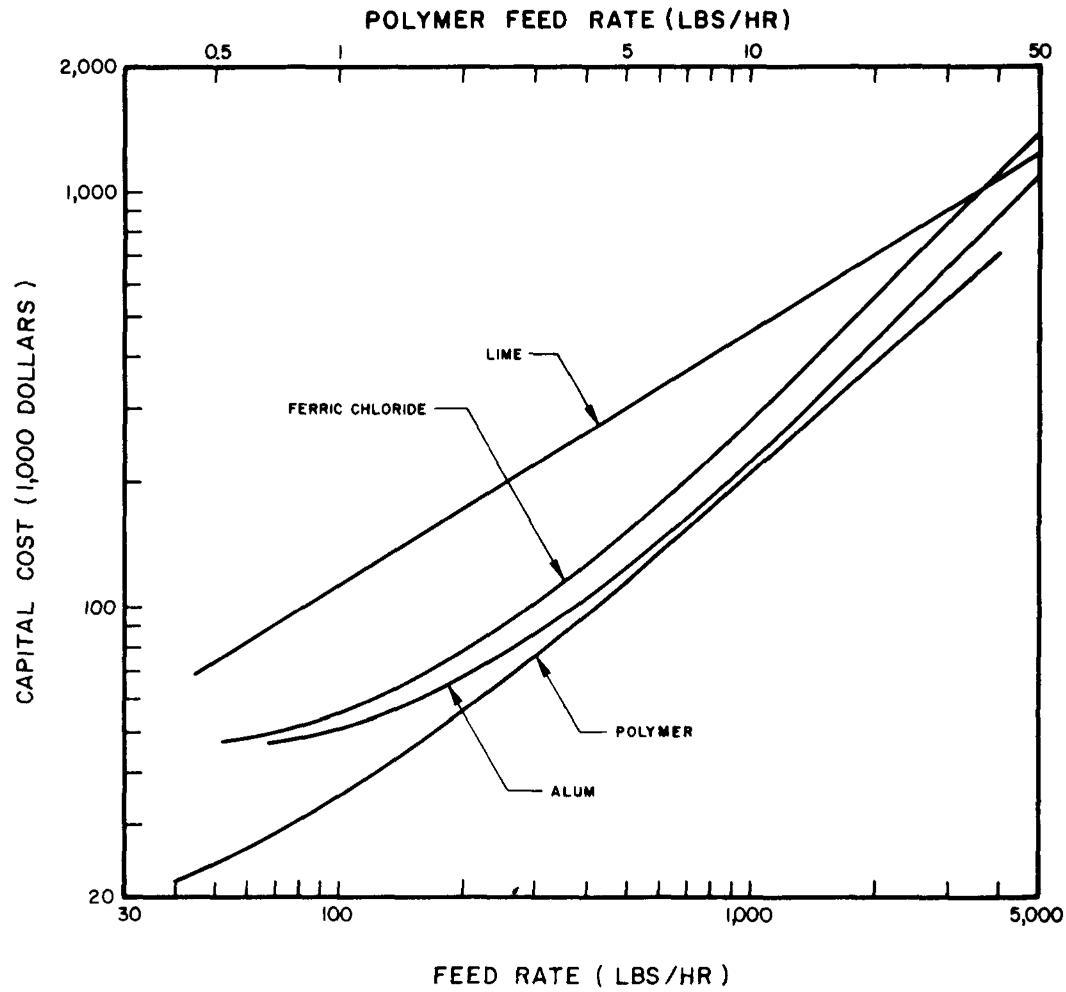


FIGURE 5-70
CAPITAL COST — CHEMICAL FEED SYSTEMS
(JUNE 1975 COSTS)

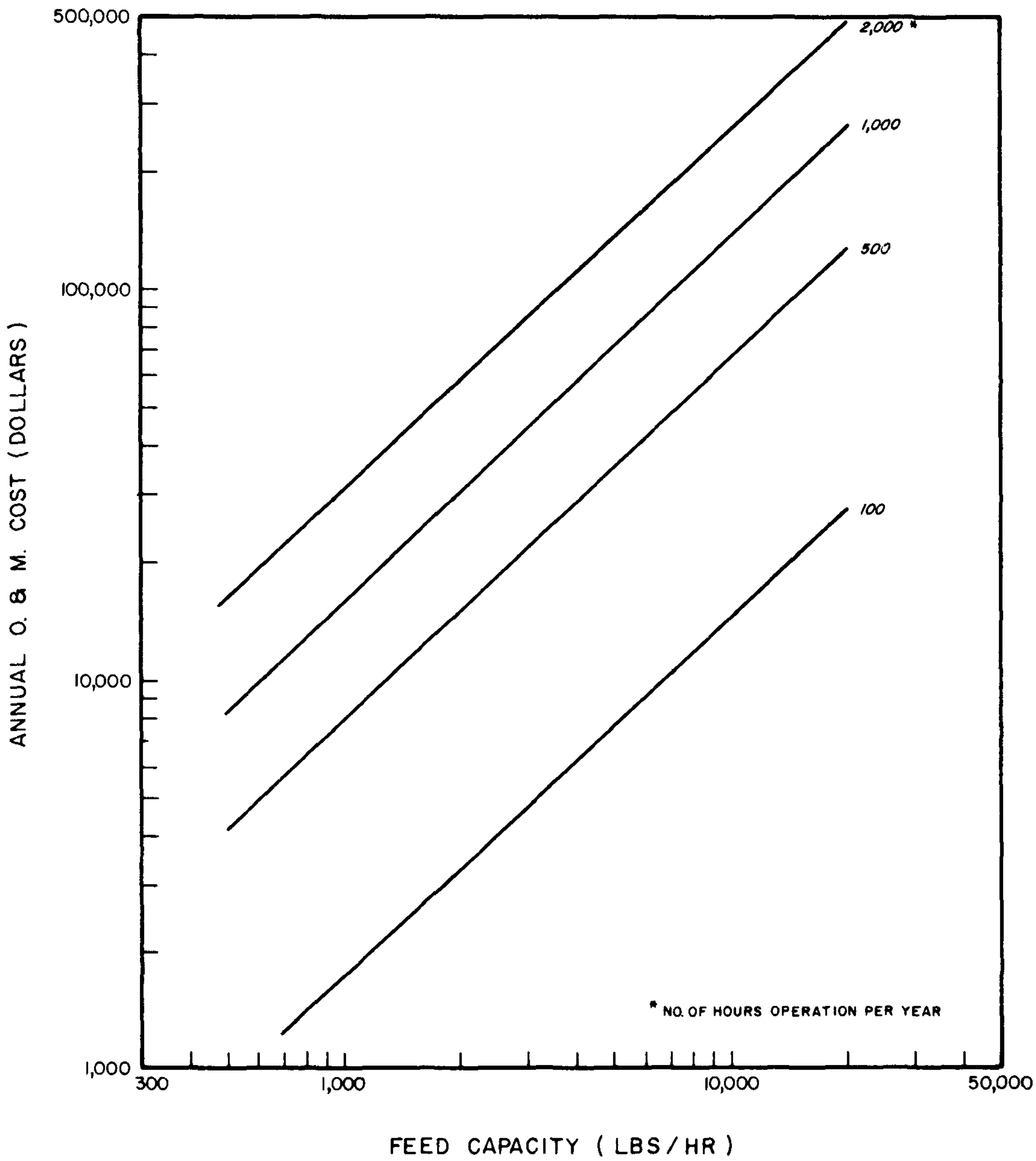


FIGURE 5-71
ANNUAL OPERATION AND MAINTENANCE COST – LIME FEED
(JUNE 1975 COSTS)

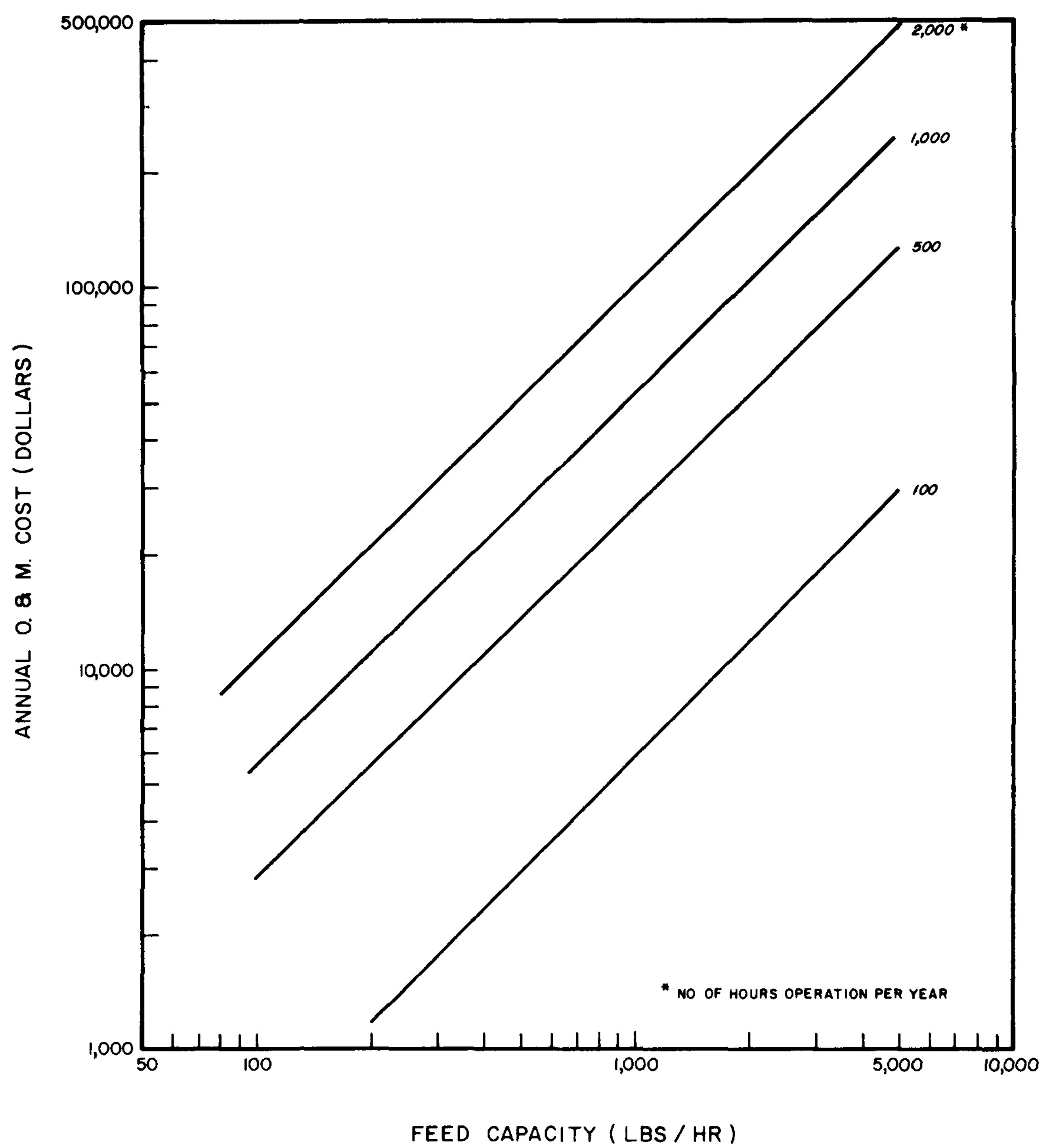


FIGURE 5-72

ANNUAL OPERATION AND MAINTENANCE COST - ALUM FEED

(JUNE 1975 COSTS)

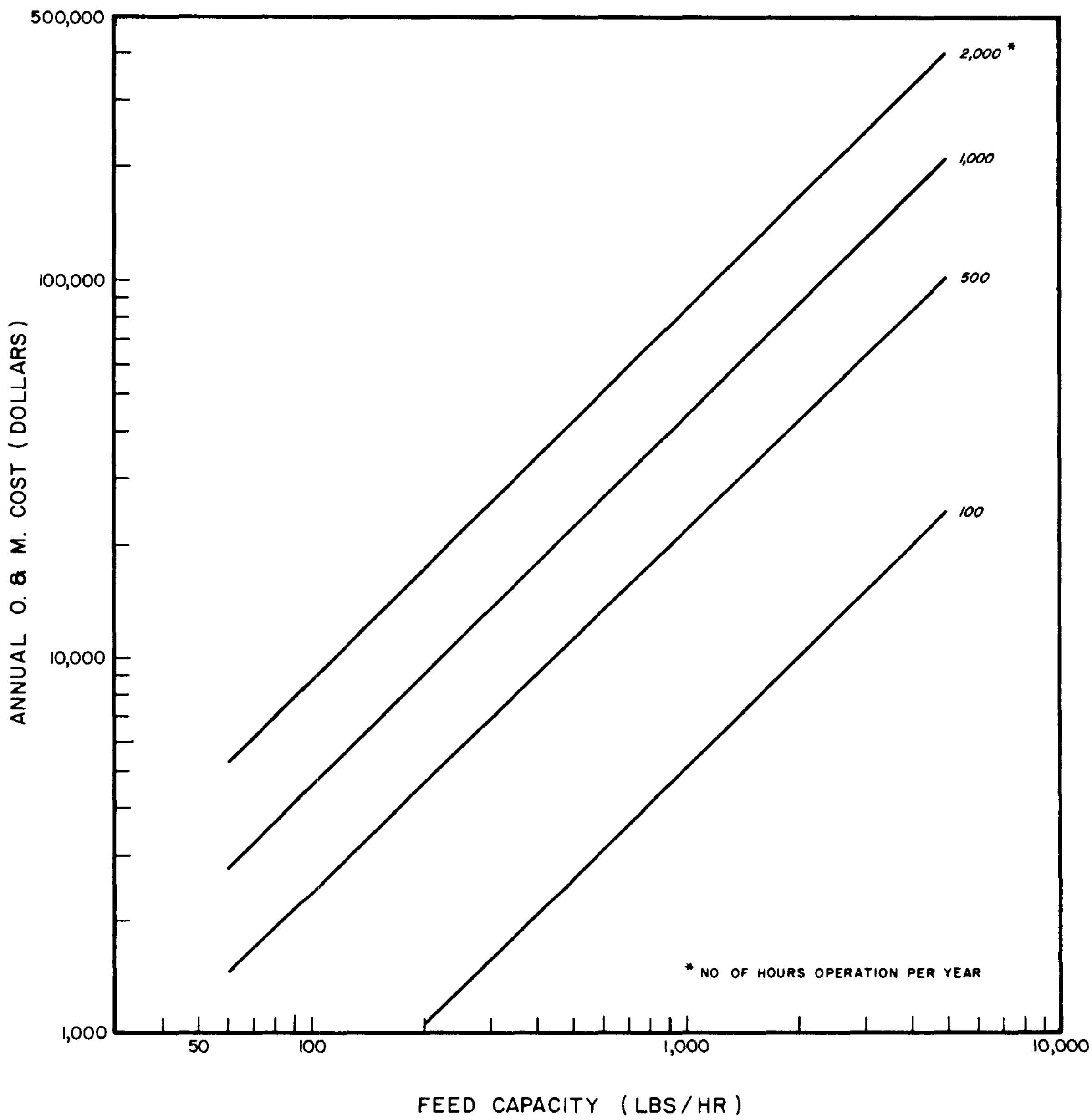


FIGURE 5-73

ANNUAL OPERATION AND MAINTENANCE COST – FERRIC CHLORIDE FEED

(JUNE 1975 COSTS)

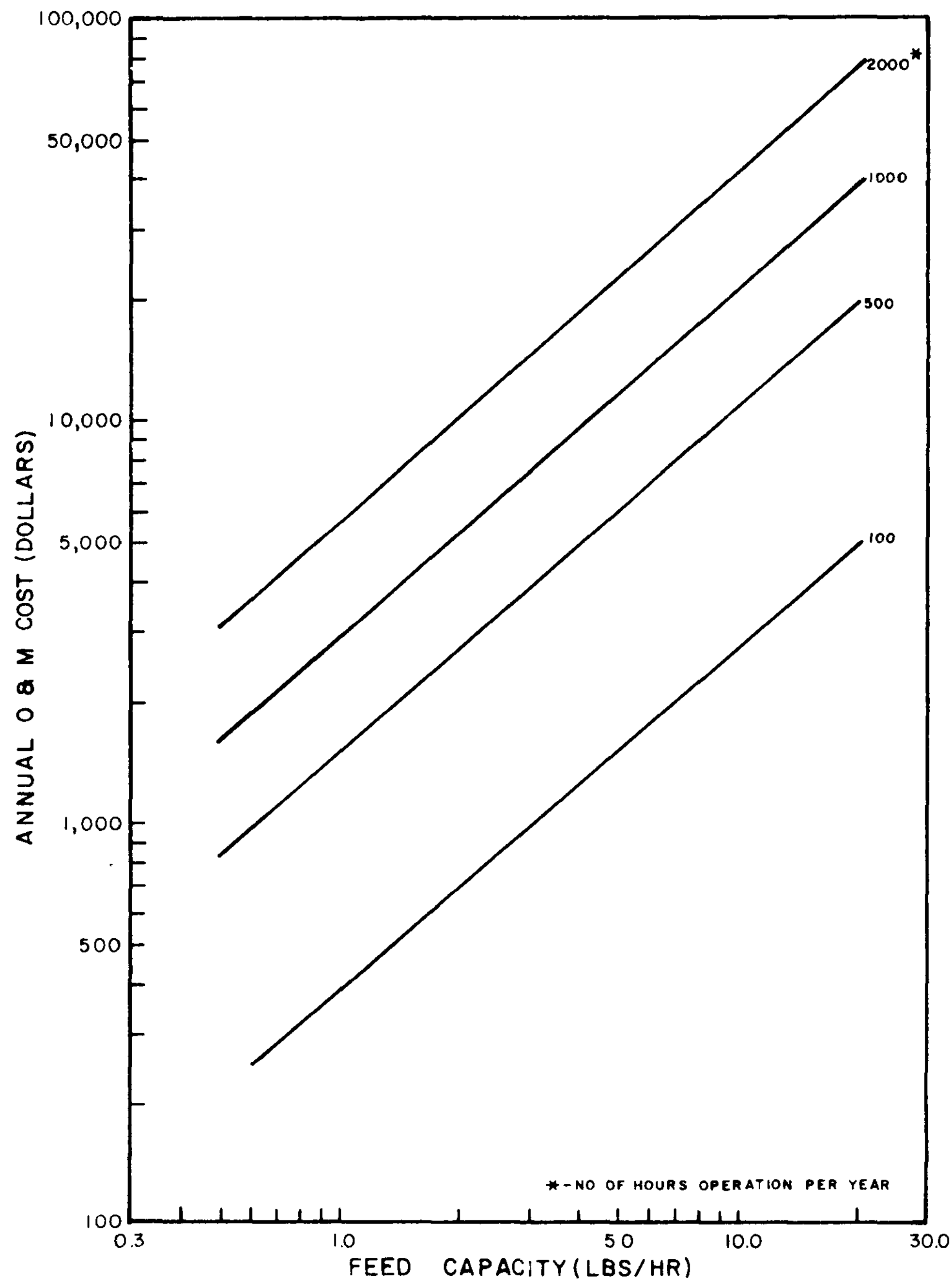


FIGURE 5-74

ANNUAL OPERATION AND MAINTENANCE COST POLYMER FEED

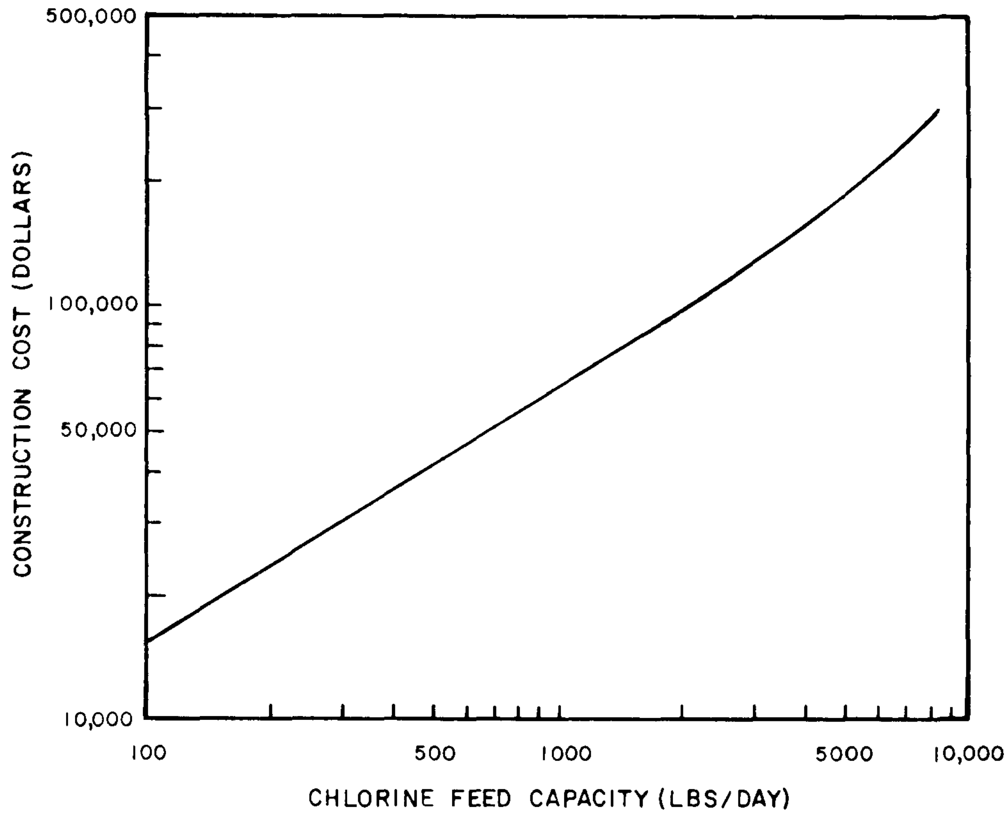


FIGURE 5-75

CAPITAL COST - CHLORINE FEED EQUIPMENT
(JUNE 1975 COSTS)

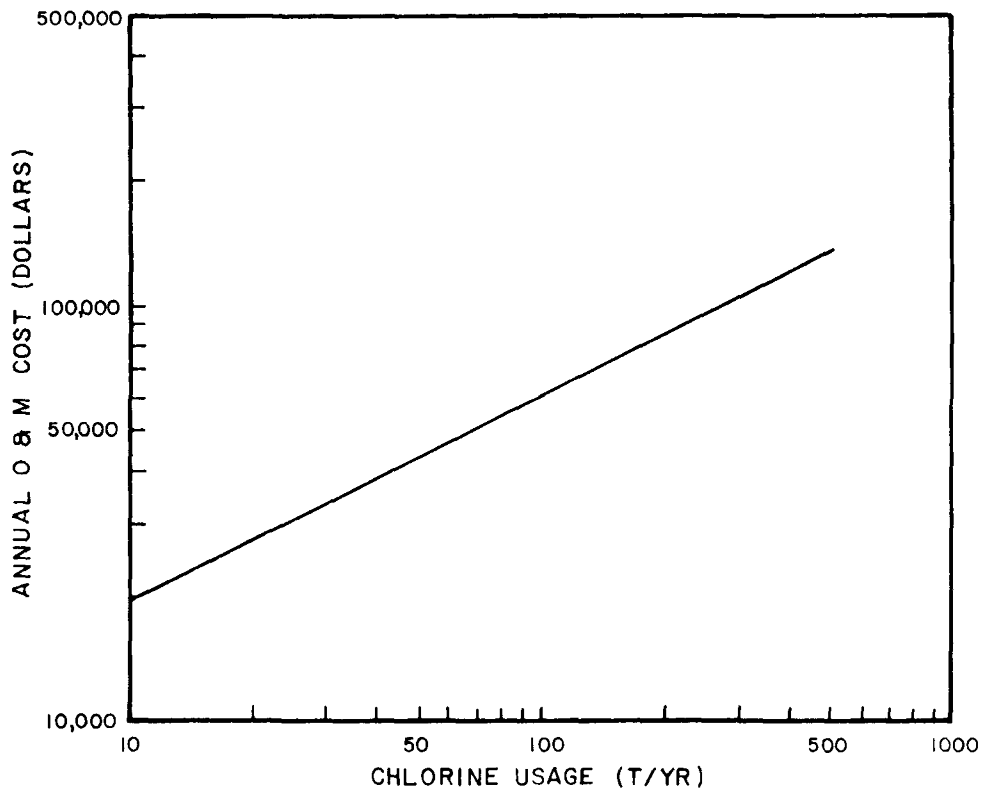


FIGURE 5-76

ANNUAL OPERATION AND MAINTENANCE COST
CHLORINE FEED EQUIPMENT
(JUNE 1975 COST)

5.6.1.10 High Intensity Mixing/Chlorine Contact Basin

For application of chlorination facilities to the treatment of stormwater, consideration should be given to detention times shorter than the conventional 15 minutes used in sewage treatment. Utilizing high mixing intensities will improve chlorine utilization and effectiveness of treatment at lower detention times. For high intensity mixing, a rapid mix basin similar to that used in water treatment for chemical coagulation can be used. The capital cost of a rapid mix basin for chlorination and chemical flash mixing is presented in Figure 5-77 as a function of volume. This cost includes manufactured equipment, a concrete basin and stainless steel mixers.

The operation and maintenance costs for rapid mix basins are presented in Figure 5-78. These costs include labor at one man-day per storm event, material and supply costs, and power for the mixer.

5.6.1.11 Raw Wastewater Pumping

The capital cost of a raw wastewater pumping system is presented in Figure 5-79 and is based on a facility which includes rough screening, dry well arrangement, housing for the pumps, piping, electrical control equipment, auxiliary power and surface access.

The operation and maintenance costs for raw wastewater pumping are presented in Figure 5-80. These costs include labor for clean-up after storms at eight man-hours per storm event, 24 hours per year inspection and equipment checking, materials, supplies and power. Power costs are based on an average flow of 45 percent design capacity, 35 feet of head and 65 percent efficiency.

5.6.1.12 Sludge Pumping

Sludge pumping stations are required in specific stormwater treatment facilities where gravity return to the dry weather sewer is not possible. The capital cost for sludge pumping is presented in Figure 5-81. This cost is based on positive displacement pumps appropriate for high solids concentration and intermittent use. The facility cost is based on underground installation and piping constructed in conjunction with the unit process for solids separation. A superstructure is included to provide access to the station from the ground level and to house electrical control equipment.

The operation and maintenance costs for sludge pumping are presented in Figure 5-82. These costs include labor, materials, supplies, and power costs based on a pumping head of 25 feet.

5.6.1.13 Biological Treatment

It is unlikely that biological treatment of stormwater overflows will be found to be cost effective in many applications. While biological treatment can be applied to the treatment of stormwater, the variability of flow and waste characteristics make this method of treatment expensive and difficult to operate. Also the bacterial population must be sustained

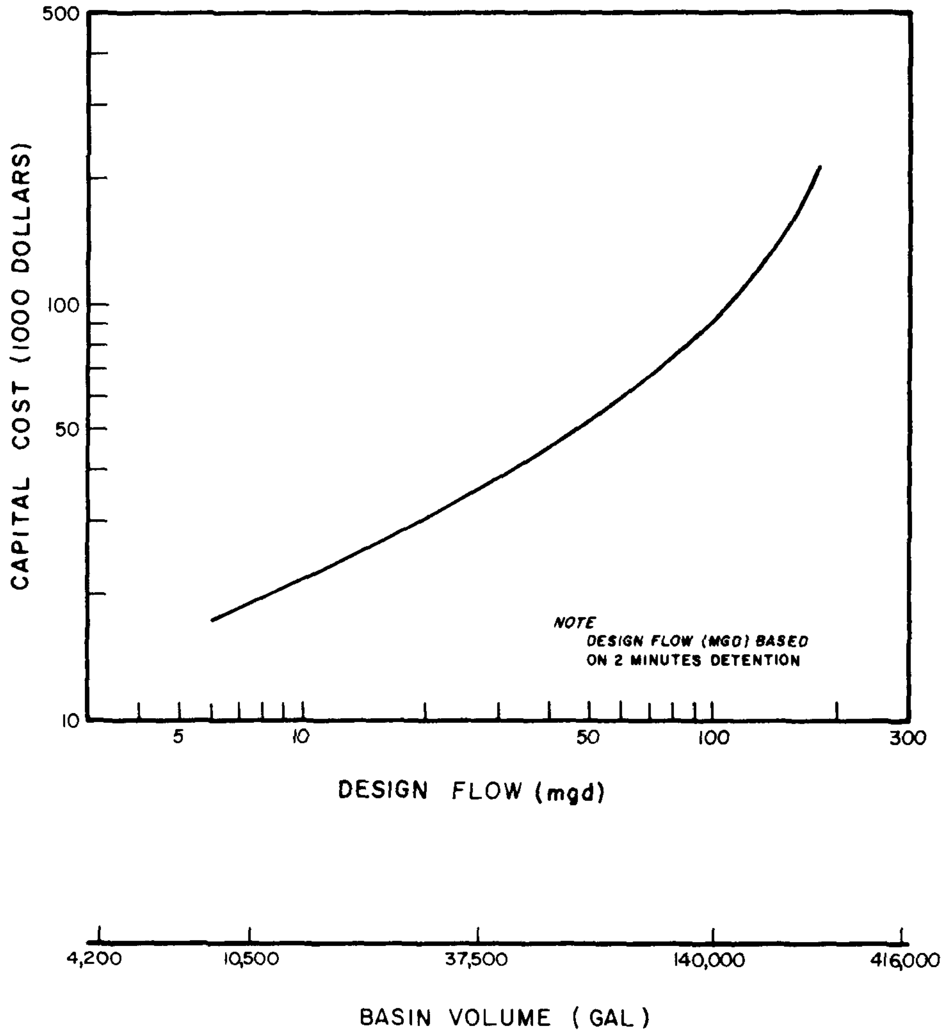


FIGURE 5-77
CAPITAL COST — RAPID MIX BASIN
(JUNE 1975 COSTS)

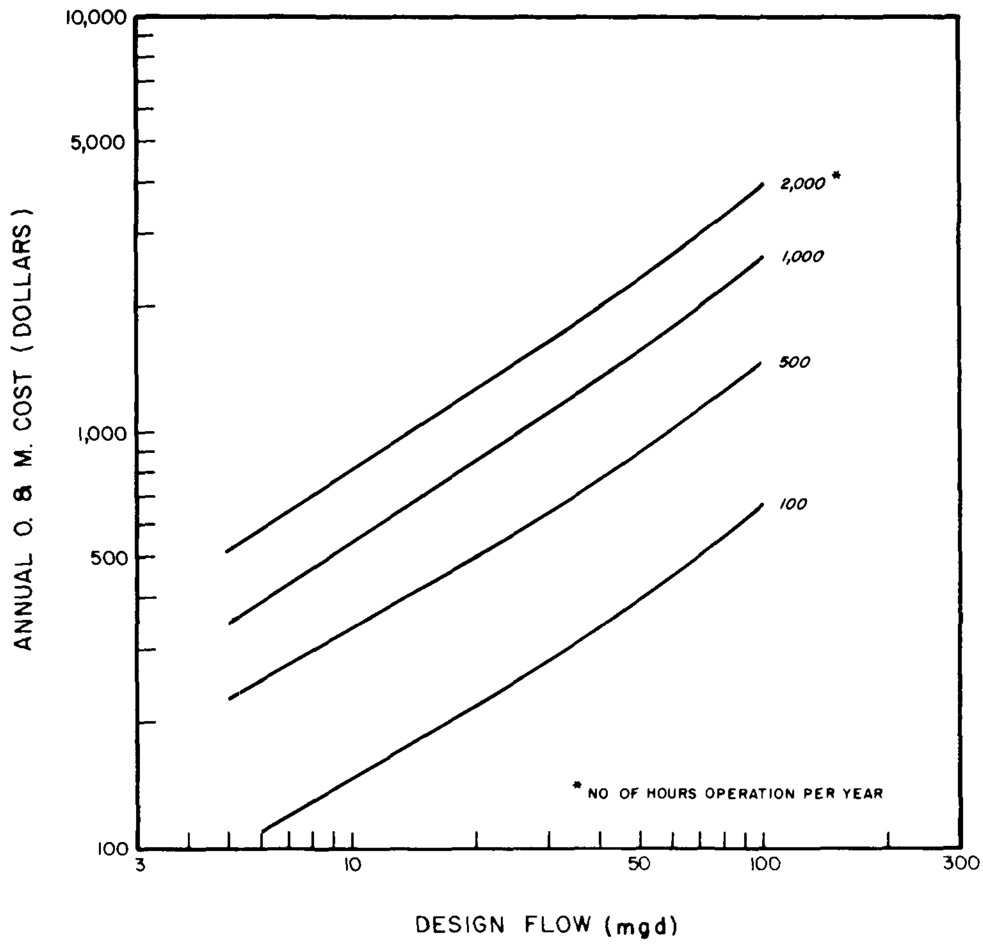


FIGURE 5-78
ANNUAL OPERATION AND MAINTENANCE COST - RAPID MIX BASIN
(JUNE 1975 COSTS)

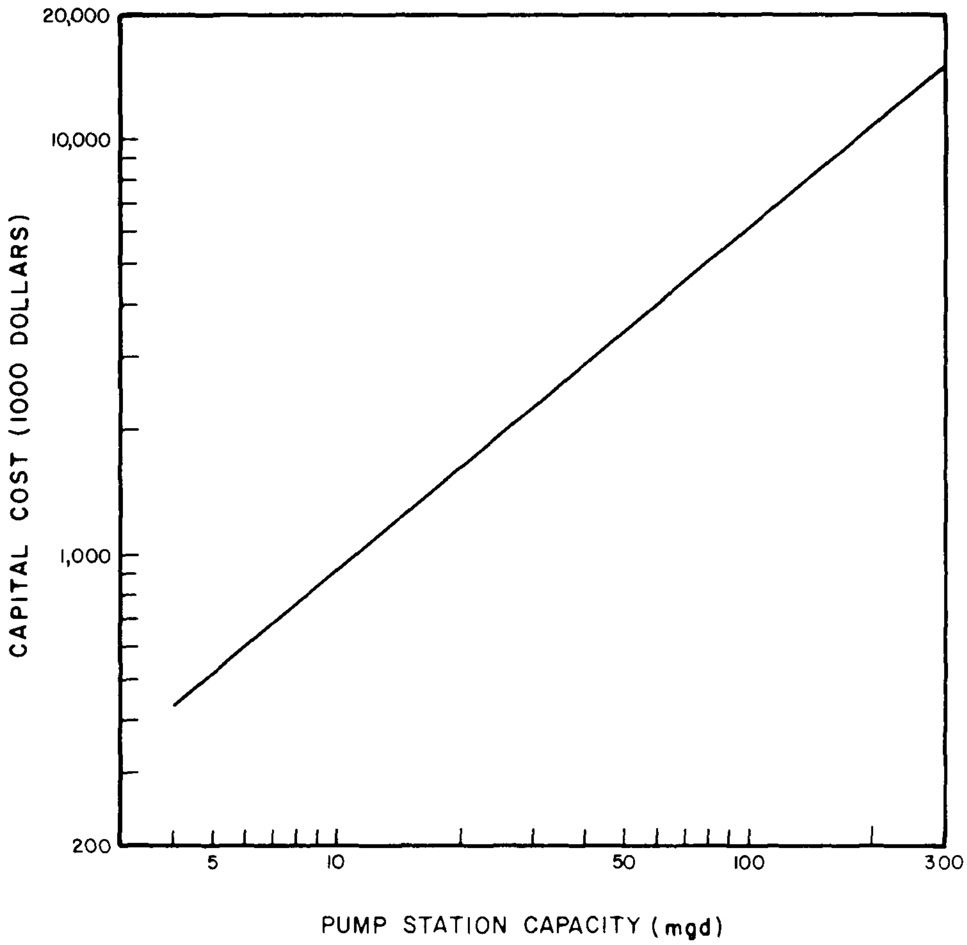


FIGURE 5-79
CAPITAL COST — RAW WASTE WATER PUMPING
(JUNE 1975 COSTS)

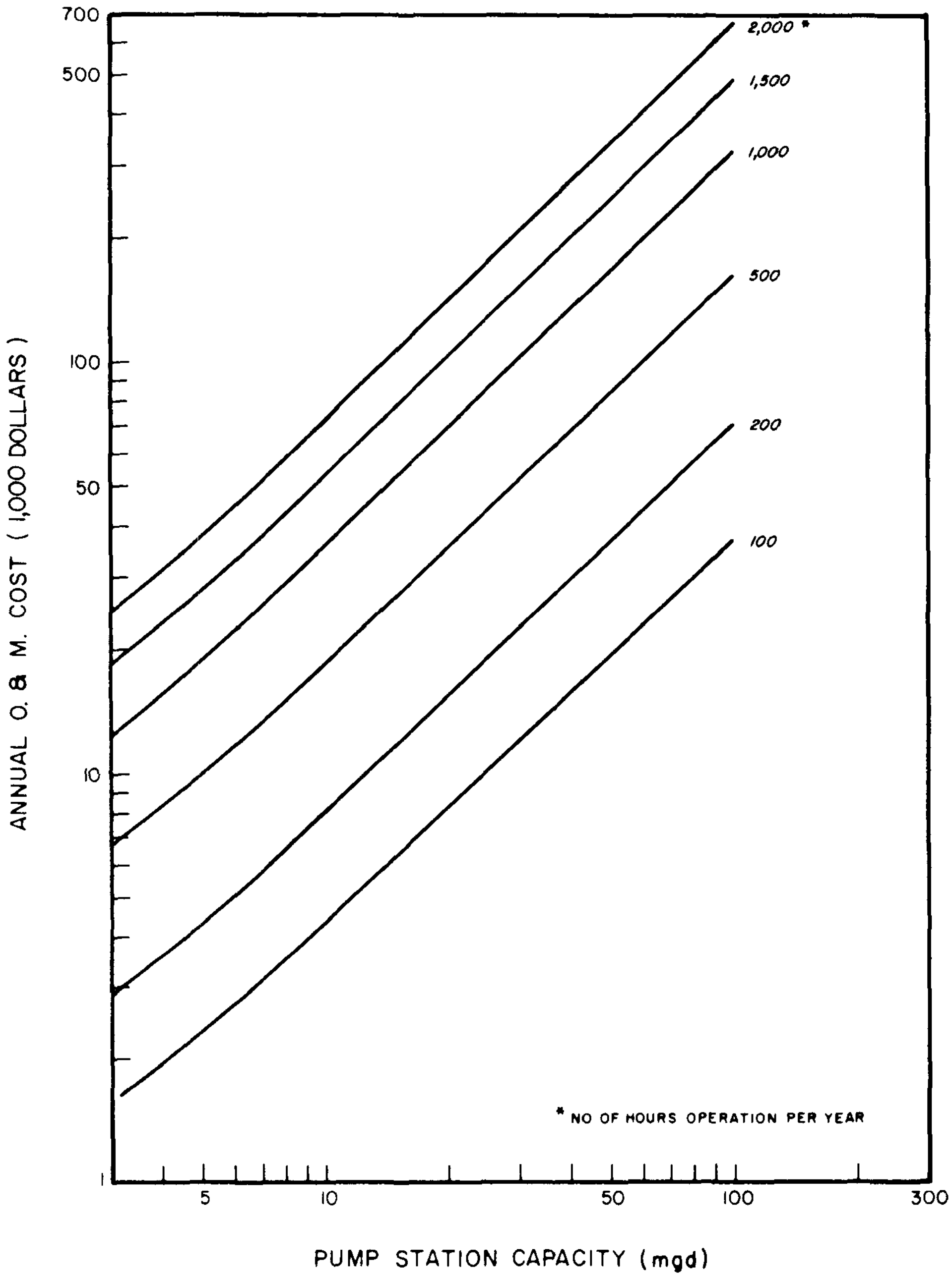


FIGURE 5-80
ANNUAL OPERATION AND MAINTENANCE COST
RAW WASTE WATER PUMPING
(JUNE 1975 COSTS)

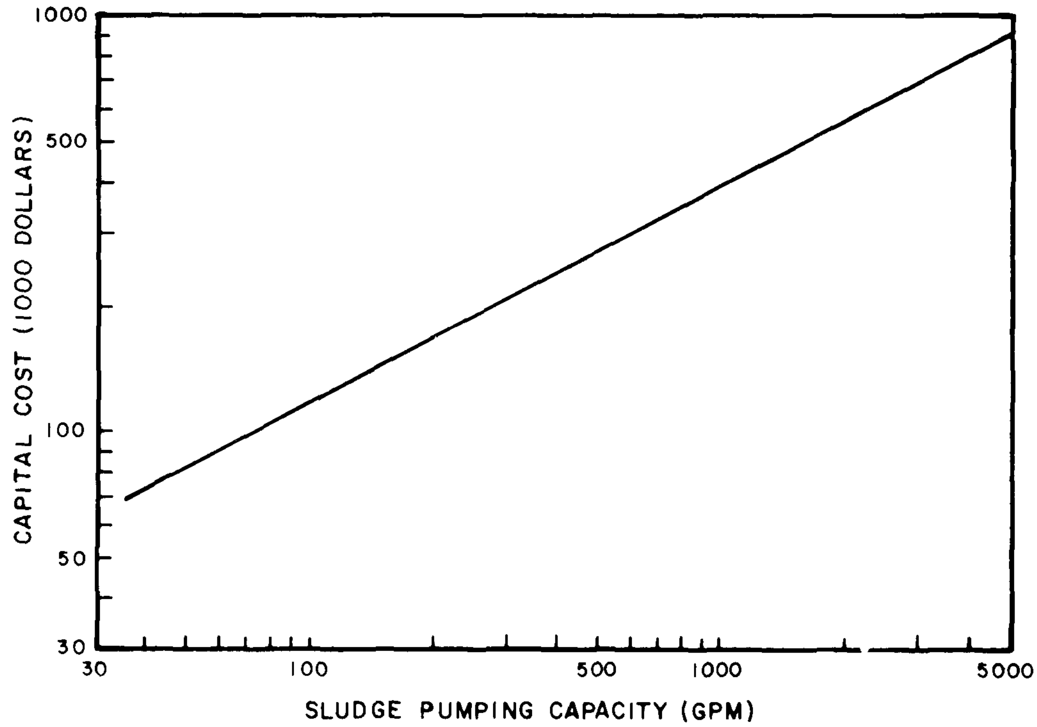


FIGURE 5-81

CAPITAL COST — SLUDGE PUMPING
(JUNE 1975 COSTS)

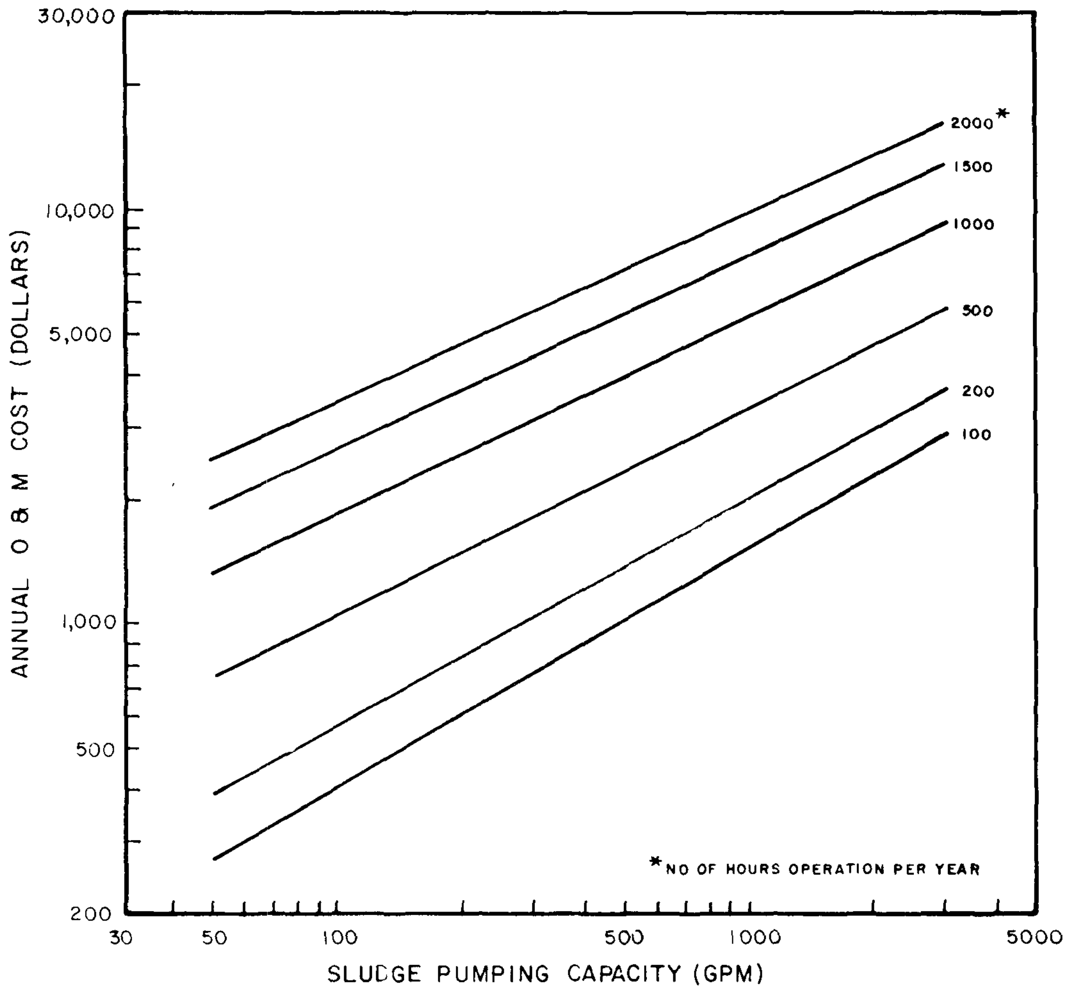


FIGURE 5-82

ANNUAL OPERATION AND MAINTENANCE COST - SLUDGE PUMPING

(JUNE 1975 COSTS)

during dry weather periods so that it will be available for the intermittent stormwater loads.

The CWC report did not provide cost information for biological treatment systems and more general guidelines for estimating these costs will be presented.

The economics of biological methods for the treatment of stormwater should be evaluated on a case by case basis. Figure 5-83 provides a general guide for estimating the cost of biological treatment. Capital costs related to plant capacity are based on the activated sludge process. This curve is based on information taken from Water and Wastewater Engineering (113) and provides a rough estimate for building a new biological treatment plant which includes primary and secondary clarification aeration, chlorination, and sludge disposal. The costs of adding individual unit processes to a dry weather treatment plant would involve a more detailed study on a case by case basis.

A rough estimate of operation and maintenance costs for biological treatment range from five to 12 cents per 1,000 gallon treated for a trickling filter and eight to 21 cents per 1,000 gallons treated by activated sludge. These general costs could be associated with the total volume of stormwater treated annually to roughly estimate the annual operation and maintenance costs.

5.6.1.14 Example Cost Calculation

To illustrate the use of the cost estimating procedure, assume the analysis of treatment device performances has indicated that the following combination of facilities would be beneficial:

1. Wastewater pumping with capacity of 50 MGD (77 cfs),

2. Horizonal screen with a maximum flow accepted of 50 MGD, and

3. Chlorination Facilities: a. Rapid mix tank with two minute detention time and maximum flow accepted of 50 MGD; b. Chlorine feed equipment with a feed rate proportional to flow. Average feed rate during storms of 2,085 lb/day chlorine.

Assume there is an average of 500 hours of rain (and runoff) per year. The capital costs as of June 1975 are shown in Table 5-15. The total capital cost estimate includes land, site work, engineering design, legal, fiscal and administrative costs and interest during construction. The operation and maintenance costs are also estimated in Table 5-15. Results of the sample calculation show that the capital costs for the treatment system are about $4,274,000 and the annual operation and maintenance costs are $128,400. These are for June 1975 prices and should be adjusted for known or predicted inflation. The comparison to other treatment processes may then be based on either the total present worth or the equivalent annual cost. A thorough comparison requires some knowledge of the principles of engineering economics.

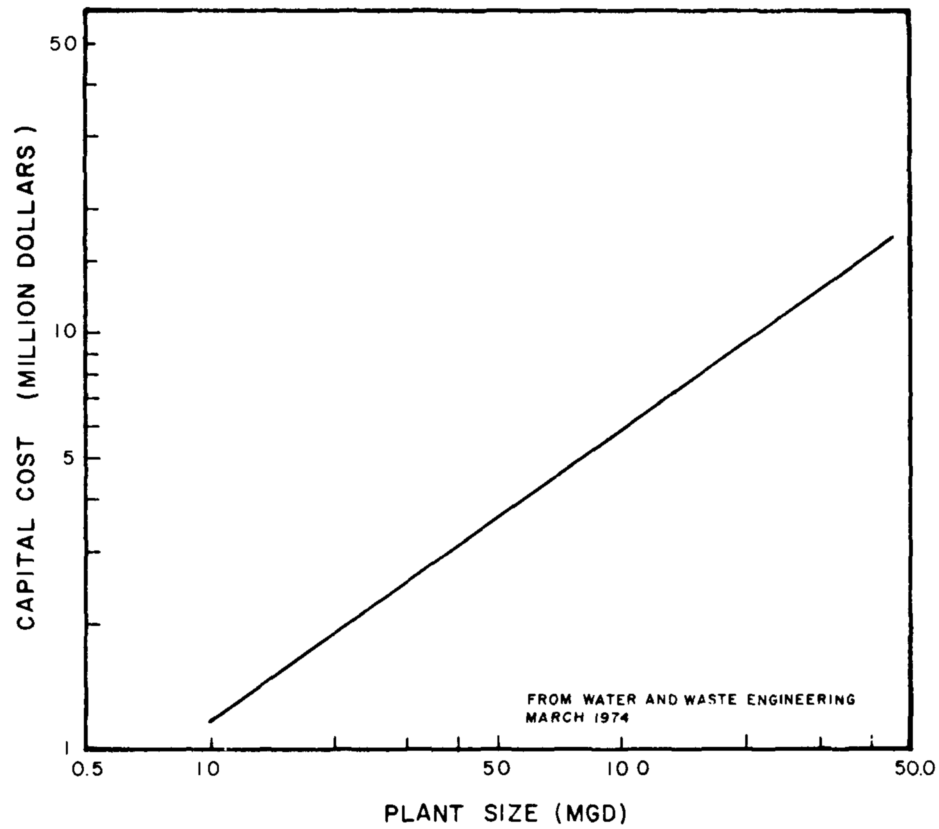


FIGURE 5-83
CAPITAL COST — SECONDARY BIOLOGICAL TREATMENT
(JUNE 1975 COSTS)

TABLE 5-15

EXAMPLE CALCULATION OF STORMWATER TREATMENT COST

(June 1975 Costs)

CAPITAL COST:

Process Unit	Unit Parameter	Reference Figure	Cost Estimate (dollars)
Wastewater Pumping	50 MGD	5-79	3,400,000
Horizonal Screen	50 MGD	5-68	720,000
Rapid Mix Tank	50 MGD	5-77	54,000
Chlorine Feed Equipment	2,085 lb/day	5-75	100,000
		Total	$4,274,000

ANNUAL OPERATION AND MAINTENANCE COSTS:

Process Unit	Unit Parameter	Reference Figure	Cost Estimate (dollars)
Wastewater Pumping	50 MGD	5-80	84,000
Horizonal Screen	50 MGD	5-69	15,500
Rapid Mix Tank	50 MGD (70,000 Gal)	5-78	900
Chlorine Feed Equipment	22 tons/year	5-76	28,000
		Total	$ 128,400

5.6.2 Management Practices

A number of numerical estimates for the cost of implementing stormwater management practices have been developed. Heaney and Nix present a summary of unit costs for street sweeping (120), reproduced as Table 5-16. Note the wide variation in the cost per curb mile from one location to another. Koplan (121) estimates the capital cost required for a street sweeping program to be in the order of $10 to $15/curb mile cleaned per year, for the sweepers alone (ENR 2000). The cost of various methods of catch basin clean ing are estimated on the order of $3 to $4 per catch basin (122).

Costs for combined sewer flushing are reviewed by Heaney and Nix (120) based on a study in the Boston area. The annual cost of flushing per unit cost of sewer line is calculated as $11.78/ft or $62,200/mile.

The cost of other management practices will vary depending upon the particular application, materials used, frequency of maintenance, etc. Wanielista (123) estimates the construction cost of small ponds in Florida at about $1.00/cu. yd. and the cost of swales (drainage channels with vegetative stabilization) at about $1.00 per foot of 4 foot wide swale. Local factors should be considered for particular studies.

5.7 References

1. Heane, James P., et al., *Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume II: Cost Assessment and Impacts*, University of Florida for U.S. Environmental Protection Agency, EPA-600/2-77-064, March 1977.

2. Howard, Charles D.D., *Theory of Storage and Treatment Plant Overflows*, Journal of the Environmental Engineering Division, ASCE Proc. Paper 12310, August 1976.

3. Thom, H.C.S., *A Frequency Distribution for Precipitation*, Abstract, Bulletin American Meteorology Society, Vol. 32, No. 10, December 1951.

4. Thom, H.C.S., *A Note on the Gamma Distribution*, Monthly Weather Review, Vol. 86, No. 4, April 1958.

5. Chow, Ven Te, Ben Chie Yen, *Urban Stormwater Runoff: Determination of Volumes and Flowrates*, University of Illinois for U.S. Environmental Protection Agency, EPA-600/2-76-116, Cincinnati, Ohio, May 1976.

6. Yen, Ben Chie, Discussion of *Theory of Storage and Treatment Plant Overflows*, Journal of the Environmental Engineering Division, ASCE, Vol. 103, No. EE3, June 1977.

7. Eagleson, Peter S., *Dynamic Hydrology*, McGraw-Hill Book Company, New York, 1970.

TABLE 5-16

UNIT COSTS OF STREET SWEEPING

	Street Sweeping Costs			
	\$/Curb-mile (\$/curb-km)	\$/yd^3 (\$/m^3)	\$/ton (\$/metric ton)	\$/capita/year
Mean	86.61 (53.82)	22.14 (28.96)	31.31 (34.51)	1.54
Median	7.00 (4.35)	13.79 (18.04)	14.28 (15.74)	1.23

Source: Unpublished data from American Public Works Association, 1976.

8. Smith, Robert, *Use of the Gamma Distribution Function for Fitting Rainfall Data,* Memorandum to Record, U.S. Environmental Protection Agency, Systems and Economic Analysis Section, Cincinnati, Ohio, March 24, 1977.

9. Rodriguez-Iturbe, Mejia, *On the Transformation of Point Rainfall to Areal Rainfall,* Water Resources Research, Vol. 10, No. 4, August 1974.

10. Linsley, R.K., M.A. Kohler, J.L.H. Paulhus, *Hydrology for Engineers,* McGraw-Hill, New York, 1958.

11. Miller, Clayton R. and Warren Viessman, Jr., *Runoff Volumes from Small Urban Watersheds,* Water Resources Research, Vol. 8, No. 2, April 1972.

12. Lager, John A., Theodor Didriksson, George B. Otte, *Development and Application of a Simplified Stormwater Management Model,* Metcalf and Eddy, Inc., for U.S. Environmental Protection Agency, EPA-600/2-76-218, Cincinnati, Ohio, 1976.

13. *Nonpoint Sources, An Assessment of Pollutant Loadings to Lakes and Rivers in North Central Texas,* Hydroscience, Inc. for North Central Texas Council of Governments, Arlington, Texas, 1977.

14. Feuerstein, Donald L. and William O. Maddaus, *Wastewater Management Program, Jamaica Bay, New York, Volume I, Summary Report,* H. F. Ludwig and Associates and Engineering-Science Inc. for U.S. Environmental Protection Agency, EPA-600/2-76-222a, September 1976.

15. *Final Report - Year 1, Spring Creek Auxiliary Water Pollution Control,* H.F. Ludwig and Associates and Engineering-Science, Inc. for City of New York, May 1970.

16. Betson, Roger, *Urban Hydrology, A Systems Study in Knoxville, Tennessee,* Tennessee Valley Authority Division of Water Management, Knoxville, Tennessee, June 1976.

17. Gameson, A.L.H., R.N. Davidson, J.M. Threlfall, *Storm Flows From Combined Sewerage Systems In Three Areas,* Journal of the Institution of Public Health Engineers, London, July 1965.

18. Kluesener, John W. and G. Fred Lee, *Nutrient Loading From a Separate Storm Sewer in Madison, Wisconsin,* Journal of the Water Pollution Control Federation, Vol. 46, No. 5, May 1974.

19. *Storm Water Pollution from Urban Land Activity,* AVCO Economic Systems Corporation for FWQA, U.S. Department of Interior, Report 11034FKL07/70, July 1970.

20. Colston, Newton V., *Characterization and Treatment of Urban Land Runoff,* North Carolina State University for U.S. Environmental Protection Agency, EPA-670/2-74-046, December 1974.

21. Donigian, A.S. Jr. and Norman H. Crawford, *Modeling Non-Point Pollution from the Land Surface,* U.S. Environmental Protection Agency, 1976.

22. *Engineering Investigation of Sewer Overflow Problems, Roanoke, Virginia,* Hayes, Seay, Mattern and Mattern, U.S. Environmental Protection Agency, Report 11024DMS05/70.

23. Ragan, R.M. and T.J. Jackson, *Use of Satellite Data in Urban Hydrologic Models,* Journal of the Hydraulics Division, ASCE, Vol. 101, No. HY12, December 1975.

24. Stankowski, S.J., *Magnitude and Frequency of Floods In New Jersey with Effects of Urbanization,* Special Report 38, US Geological Survey, Water Resources Division, Trenton, New Jersey, 1974.

25. Reiner, Franzini, *Urbanizations Drainage Consequence,* Journal of the Urban Planning and Development Division, American Society of Civil Engineers, Vol. 1, December 1971.

26. Graham, Costello, Mallon, *Estimation of Imperviousness and Specific Curb Length for Forecasting Stormwater Quality and Quantity,* Journal Water Pollution Control Federation, Vol. 46, No. 4, April 1974.

27. American Public Works Association, *Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume III: Characterization,* US Environmental Protection Agency Publication, EPA-600/2-77-064, March 1977.

28. American Public Works Association and University of Florida, *Evaluation of the Magnitude and Significance of Pollution Loading From Urban Stormwater Runoff, Ontario,* Department of Environment, Toronto, 1976.

29. Heaney, Huber, Nix (University of Florida), *Storm Water Management Model, Level I, Preliminary Screening Procedures,* US Environmental Protection Agency Publication, EPA-600/2-76-75, Cincinnati, Ohio, October 1976.

30. Brater, E.F., J.D. Sherrill, *Rainfall-Runoff Relations on Urban and Rural Areas,* University of Michigan for U.S. Environmental Protection Agency, EPA-670/2-75-046, May 1975.

31. Whitehead, H.C., and J.H. Feth, *Chemical Composition of Rain, Dry Fallout, and Bulk Precipitation at Menlo Park, California, 1957-1959,* Journal of Geophysical Research, Vol. 69, No. 16, August 1964.

32. Pearson, F.J. and D.W. Fisher, *Chemical Composition of Atmospheric Precipitation in the Northeastern United States,* U.S. Geological Survey, WSP 1535-P, 1971.

33. Cogbill, C.V. and G.E. Likens, *Acid Precipitation in the Northeastern United States,* Water Resources Research, V. 10, No. 6, December 1974.

34. Ruppert, H., *Geochemical Investigations on Atmospheric Precipitation in a Medium-Sized City,* Water, Air and Soil Pollution (4) 1975.

35. Lager, J.A. and W. E. Smith, *Urban Stormwater Management and Technology: An Assessment,* Metcalf and Eddy, Inc. for U.S. Environmental Protection Agency, EPA-670/2-74-040, December 1974.

36. Lager, John A., W.G. Smith, W.G. Lynard, R.M. Finn and E.J. Finnemore, *Urban Stormwater Management and Technology: Update and Users Guide,* Metcalf and Eddy, Inc. for U.S. Environmental Protection Agency, EPA-600/8-77-014, September 1977.

37. Huber, Wayne C., James P. Heany, *Urban Rainfall-Runoff-Quality Data Base,* University of Florida for the U.S. Environmental Protection Agency, EPA-600/8-77-009, July 1977.

38. Texas Tech University, Water Resources Center, *Variation of Urban Runoff Quality With Duration and Intensity of Storms,* WRC-73-2, 1973.

39. *Humboldt Avenue Pollution Abatement Demonstration Project,* Consoer, Townsend and Associates and Hydroscience, Inc., for the City of Milwaukee and the U.S. Environmental Protection Agency, September 1974.

40. *Report Upon Overflow Analysis to Passaic Valley Sewerage Commissioners, Passaic River Overflows,* Elson T. Killam Associates, Inc., 1976.

41. Sartor, James D., and Gail B. Boyd, *Water Pollution Aspects of Street Surface Contaminants,* U.S. Environmental Protection Agency, EPA-R2-72-081, November, 1972.

42. URS Research Company, *Water Quality Management Planning for Urban Runoff,* U.S. Environmental Protection Agency, EPA-440/9-75-004, December, 1974.

43. Vilaret, M.R., et al, *Storm and Combined Sewer Pollution Sources and Abatement,* Black, Crow, and Eidsness, Inc., NTIS, PB 201 725, Gainesville, Florida, January 1971.

44. Wells, Dan M. et al, *Variation of Urban Runoff Quality with Duration and Intensity of Storms-Phase III, Volume 4 - Project Summary,* OWRT and The City of Lubbock, Project No. B-177-TEX, December 1975.

45. Haith, Douglas A., *Land Use and Water Quality in New York Rivers*, ASCE, Journal of Environmental Engineering Division, Vol. 102, No. EE1, February 1976.

46. Darby, William P., Francis Clay McMichael, Robert W. Dunlop, *Urban Watershed Management Using Activity Indicators to Predict Water Quality*, Water Resources Research, Vol. 12, No. 2, April 1976.

47. Gerghty, Miller, Van Der Leeden, Troise, *Water Atlas of the United States*, A Water Information Center Publication, 1973.

48. *Simplified Mathematical Modeling of Water Quality*, Hydroscience, Inc., and Mitre Corporation, Environmental Protection Agency Publication, March 1971.

49. Fischer, Hugo B., *Dispersion Predictions in Natural Streams*, Journal of the Sanitary Engineering Division, ASCE, Vol. 94, No. SA5, October 1968.

50. Bansal, Mahendra K., *Dispersion in Natural Streams*, Journal of the Hydraulics Division, ASCE, Vol. 97, No. HY11, November 1971.

51. McQuivey, R.S. and T.N. Keefer, *Simple Method for Predicting Dispersion in Streams*, Journal of the Environmental Engineering Division, ASCE, Vol. 100, No. EE4, August 1974.

52. Lice, Henry, *Predicting Dispersion Coefficient of Streams*, Journal of the Environmental Engineering Division, ASCE, Vol. 103, No. EE1, February 1977.

53. Mancini, John, *Numerical Estimates of Coliform Mortality Rates Under Various Conditions*, Research and Development Report, Hydroscience, Inc., May 1977.

54. *Mathematical Modeling of Natural Systems*, Summer Institute Notes, Environmental Engineering and Science Division, Manhattan College, Bronx, N.Y., May 1976.

55. O'Connor, D.J. and W.E. Dobbins, *Mechanism of Reaeration in Natural Streams*, Transactions, ASCE, Vol. 123, Paper No. 2934, 1958.

56. Churchill, M.A., H.L. Elmore and R.A. Buckingham, *Prediction of Stream Reaeration Rates*, Journal of the Sanitary Engineering Division, ASCE, Vol. 88, No. SA4, July 1962.

57. Isaacs, W.P. and A.F. Gandry Jr., *Atmospheric Oxygenation in a Simulated Stream*, Journal of the Sanitary Engineering Division, ASCE, Vol. 94, No. SA2, April 1968.

58. Steele, Timothy D. and Marshall E. Jennings, *Regional Analysis of Streamflow Chemical Quality in Texas*, Water Resource Research, Vol. 8, No. 2, April 1972.

59. McElroy, Chiu, Nebgen, Aleti, Bennet, *Loading Functions for Assessment of Water Pollution From Non-point Sources*, Midwest Research Institute for U.S. Environmental Protection Agency, EPA-600/2-76-151, May 1976.

60. *Influence of Land Use on Stream Nutrient Levels*, U.S. Environmental Protection Agency, EPA-600/3-76-014.

61. Betson, R.P. and W.M. McMaster, *A First Generation Nonpoint Source Mineral Water-Quality Model*, Journal of the Water Pollution Control Federation, Vol. 47, No. 10, October 1975.

62. Hydroscience, Inc., *208 Areawide Assessment Procedures Manual, Volume I, Chapter 2*, U.S. Environmental Protection Agency, EPA-600/9-76-014, Cincinnati, Ohio, July 1976.

63. *Analysis of Upstream Background Loadings*, Internal Memorandum, Hydroscience, Inc., 1976.

64. Ledbetter, J.E. and E.F. Gloyna, *Productive Techniques for Water Quality Inorganics*, Journal of the Sanitary Engineering Division, ASCE, Vol. 90, No. SA-1, February 1964.

65. Gunnerson, C.G., *Streamflow and Quality in the Columbia River Basin*, Journal of the Sanitary Engineering Division, ASCE, Vol. 93, No. SA6, 1967.

66. Hall, F.R., *Dissolved Solids - Discharge Relationships, 1, Mixing Models*, Water Resources Research, Vol. 6, No. 3, 1970.

67. Hall, F.R., *Dissolved Solids - Discharge Relationships, 2, Applications to Field Data*, Water Resources Research, Vol. 7, No. 3, 1971.

68. Hem, John D., *Study and Interpretation of the Chemical Characteristics of Natural Water*, U.S. Geological Survey Water-Supply Paper 1473, 1959.

69. *Urban Storm Runoff and Combined Sewer Overflow Pollution, Sacramento, California*, Envirogenics Company for U.S. Environmental Protection Agency, Report 11024 FKM 12/71, December 1971.

70. Wang, Wun-Cheng and Ralph L. Evans, *Dynamics of Nutrient Concentrations in the Illinois River*, Journal of the Water Pollution Control Federation, Vol. 42, No. 12, December 1970.

71. *National Assessment of Trends in Water Quality*, Enviro Control, Inc., National Technical Information Service, June 1972.

72. Cahill, T.H., P. Imperato, and Francis H. Verhoff, *Evaluation of Phosphorus Dynamics in a Watershed*, Journal of the Environmental Engineering Division, ASCE, Vol. 100, No. EE2, April 1974.

73. *Assessment of the Effects of Nutrient Loadings on Lake Ontario Using a Mathematical Model of the Phytoplankton*, Hydroscience, Inc. for the International Joint Commission, Great Lakes Water Quality Board, March 1976.

74. Metcalf and Eddy, University of Florida and Water Resources Engineers, *Storm Water Management Model (SWMM) - Volume I Final Report*, Environmental Protection Agency, 11024 DOC 07/71, July 1971.

75. University of Florida, *Urban Stormwater Management Modeling and Decisionmaking*, prepared for National Environmental Research Center, May 1975.

76. Glover, G.E., and G.R. Herbert, *Microstaining and Disinfection of Combined Sewer Overflows - Phase II*, Environmental Protection Agency, EPA-R2-73-124, January 1973.

77. Glover, G.R., *Application of Microstraining to Combined Sewer Overflow*, contained in Combined Sewer Overflow Seminar Papers, Environmental Protection Agency, EPA-670/2-73-077, November 1973.

78. Ecology Division, Rex Chainbelt, Inc., Milwaukee, Wisconsin, *Screening/Flotation of Combined Sewer Overflows*, Environmental Protection Agency, 11020 FDC 01/72, January 1972.

79. Eckenfelder, W.W., and D.J. O'Connor, *Biological Waste Treatment*, Pergamon Press, 1961.

80. American Society of Civil Engineers, *Sewage Treatment Plant Design*, ASCE Manual No. 36, 1959.

81. Envirogenics Company, *Urban Storm Runoff and Combined Sewer Overflow Pollution*, Environmental Protection Agency, 11024 FKM 12/71, December 1971.

82. Gupta, M.K., and R.W. Agnew, *Screening/Dissolved Air Flotation Treatment of Combined Sewer Overflows*, contained in Combined Sewer Overflow Seminar Papers, Environmental Protection Agency EPA-670/2-73-077, November 1973.

83. White, Robert L., and T.G. Cole, *Dissolved Air Flotation for Combined Sewer Overflows*, Public Works, February 1973.

84. Rhodes Technology Corporation, Houston, Texas, *Dissolved-Air Treatment of Combined Sewer Overflows*, Environmental Protection Agency, 11020 FKI, January 1970.

85. American Public Works Association, *The Swirl Concentrator as a Combined Sewer Overflow Regulator Facility*, Environmental Protection Agency EPA-R2-72-008, September 1972.

86. Sullivan, Richard H., et al, *The Swirl Concentrator as a Grit Separator Device*, Environmental Protection Agency, EPA-670/2-74-026, June 1974.

87. Sullivan, Richard H., *The Swirl Concentrator as a Combined Sewer Overflow Regulator*, contained in Combined Sewer Overflow Seminar Papers, EPA-670/2-73-077, November 1973.

88. Dalrymple, R.J., S.L. Hodd, D.C. Morin, *Physical and Settling Characteristics of Particulates in Storm and Sanitary Wastewater*, Beak Consultants Limited for U.S. Environmental Protection Agency, EPA-670/2-75-011, April 1975.

89. Field, R.I., and P.E. Moffa, *Treatability Determinations for a Prototype Swirl Combined Sewer Overflow Regulator/Solids-Separator*, paper for Environmental Protection Agency, 1975.

90. Nebolsine, R., P.J. Harvey and C.Y. Fan, *High-Rate Filtration of Combined Sewer Overflows*, Hydrotechnic Corporation for U.S. Environmental Protection Agency 11023 EYI 04/72, April 1972.

91. Harvey, P.J., *High-Rate Multi-Media Filtration*, contained in Combined Sewer Overflow Seminar Papers, EPA-670/2-73-077, November 1973.

92. Glover, G.E., *High Rate Disinfection of Combined Sewer Overflow*, contained in Combined Sewer Overflow Seminar Papers, EPA-670/2-73-077, November 1973.

93. Shuckrow, A.J., G.W. Dawson and W.F. Bonner, *Physical-Chemical Treatment of Combined and Municipal Sewage*, Environmental Protection Agency, EPA-R2-73-149, February 1973.

94. Black & Veatch, Consulting Engineers, *Process Design Manual for Phosphorus Removal*, Environmental Protection Agency, October 1974.

95. *Process Design Manual for Upgrading Existing Wastewater Treatment Plants*, Environmental Protection Agency, October 1974.

96. Pitt, R.E., and G. Amy, *Toxic Materials Analysis of Street Surface Contaminants*, Environmental Protection Agency, EPA-R2-73-283, August 1973.

97. Argo, D.G., and G.L. Culp, *Heavy Metals Removal in Wastewater Treatment Processes: Part 1 and Heavy Metals Removal in Wastewater Treatment Process: Part II - Pilot Plant Operation*, Water and Sewage Works, August-September 1972.

98. Hommack, P., K.L. Zippler and E.C. Herbert, *Utilization of Trickling Filters for Dual Treatment of Dry and Wet Weather Flows*, Environmental Protection Agency, EPA-670/2/73-071, September 1973.

99. Autotrol Corporation, Milwaukee, Wisconsin, *Combined Sewer Overflow Treatment by The Rotating Biological Contactor Process*, Environmental Protection Agency, EPA-670/2-24-050, June 1974.

100. *Process Design Manual for Suspended Solids Removal*, Environmental Protection Agency, January 1975.

101. Schmid, L.A., and R.R. McKinney, *Phosphate Removal by a Lime-Biological Treatment Scheme*, JWPCF, July 1969.

102. Culp, R.L., and G.L. Culp, *Advanced Wastewater Treatment*, Van Nostrand Reinhold Company, 1971.

103. Eckenfelder, W.W., *Water Quality Engineering for Practicing Engineers*, Barnes & Noble, Inc., 1970.

104. *Advanced Waste Treatment Design for Wantagh, Long Island Groundwater Recharge Study*, Consoer, Townsend & Associates, Chicago, Illinois, June 1973.

105. Beak Consultants Limited, *Physical and Settling Characteristics of Particulates in Storm and Sanitary Wastewater*, Environmental Protection Agency, EPA-670-2-75-011, April 1975.

106. O'Brien and Gere Engineers, *Pilot Plant Studies, Combined Sewer Overflow Abatement Programs*, EPA Grant New York 005141, Draft Report, November 1976.

107. Prah, D.H., P.L. Brunner, *Combined Sewer Stormwater Overflow Treatment By Screening and Terminal Ponding at Fort Wayne, Indiana*, Municipal Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection, Draft Report, Volume I, June 1976.

108. Maher, M.B., *Microstraining and Disinfection of Combined Sewer Overflow - Phase III*, U.S. Environmental Protection Agency, EPA-670/2-74-049, 1974.

109. O'Brien and Gere Engineers, Inc., *Syracuse CSO Demonstration Study*, Draft Report, Sections III-VIII, March 15, 1977.

110. Field, Richard, et al., *Proceedings of Workshop on Microorganisms in Urban Stormwater*, U.S. Environmental Protection Agency, EPA-600/2-76-244, November 1976.

111. Collins, Harvey F., R.E. Selleck, G.C. White, *Problems in Obtaining Adequate Sewage Disinfection*, Journal of the Sanitary Engineering Division, ASCE, SA5, October 1971.

112. Chick, H., *An Investigation of the Laws of Disinfection*, Journal of Hygiene, Vol. 8, Cambridge, England, No. 92, 1908.

113. Fair, Gordon M., J.C. Geyer, D.A. Okun, *Water and Wastewater Engineering, Volume 1 and 2,* John Wiley and Sons, Inc., New York, 1968.

114. Geisser, Donald F., and S.R. Garver, *High-Rate Disinfection: Chlorine Versus Chlorine Dioxide,* Journal of the Environmental Engineering Division, ASCE, Vol. 103, EE6, December 1977.

115. Pontius, Uwe R., E.H. Pavia, D.G. Crowder, *Hypochlorination of Polluted Stormwater Pumpage at New Orleans,* U.S. Environmental Protection Agency, EPA-670/2-73-067, September 1973.

116. Benjes, H.H., *Cost Estimating Manual-Combined Sewer Overflow Storage Treatment,* Culp, Weaner, and Culp, Inc. for U.S. Environmental Protection Agency, EPA-600/2-76-286, 1976.

117. *A Guide to the Selection of Cost-Effective Wastewater Treatment Systems,* U.S. Environmental Protection Agency, EPA-730/9-75-002, July 1975.

118. *Engineering News-Record,* New York, Monthly.

119. *Construction Cost Indexes,* Office of Water Program Operations, U.S. Environmental Protection Agency, Quarterly.

120. Heaney, James P., Stephen J. Nix, *Storm Water Management Model: Level I -Comparative Evaluation of Storage Treatment and Other Management Practices,* U.S. Environmental Protection Agency, EPA-600/2-77-083, April 1977.

121. Koplan, Michael, *Preventive Approaches to Stormwater Management,* Chapter 3, U.S. Environmental Protection Agency, EPA-440/9-77-001, January 1977.

122. American Public Works Association, *Water Pollution Aspects of Urban Runoff,* Federal Water Pollution Control Administration, WP-20-15, 1969.

123. Wanielista, Martin P., *Nonpoint Source Effects,* Florida Technological University, Orlando, Florida, January 1976.

CHAPTER 6

MONITORING FOR STORMWATER ASSESSMENT

A monitoring program is an integral part of any stormwater assessment study. The attributes and characteristics of urban runoff vary significantly from one location to another, and handbook or extrapolated values from sources such as Chapter 5 of this manual, or so called "default values" employed in some mathematical storm runoff models, should be utilized with caution and judgement. Normally, site specific data will be required for an adequate understanding and characterization of local conditions, and to permit published values or ranges to be utilized in a sensible manner.

Chapter 6 presents considerations and guidelines for developing a stormwater monitoring program. Aspects of the program discussed include rainfall monitoring, drainage basin characterization, monitoring of runoff and overflows, and receiving water monitoring. The emphasis is on the identification of the important components of a stormwater monitoring program, and on discussing some relevant aspects, which should aid in structuring an effective overall program. Information important to the actual execution of a monitoring program is available from other sources and is not covered here. A number of such reference sources which describe monitoring, sampling techniques, and sampling equipment in detail, are identified and briefly reviewed at the end of this chapter.

6.1 Purpose of Monitoring Program

The collection of data is not in itself a goal in a stormwater assessment program. Large data bases are of little value unless they aid in the planning and decision making process. The purpose of any monitoring program is to provide the data necessary for application of the methods of analysis which will be used to study the defined set of water quality problems. The data needs will vary depending on the problem, method of analysis, and the degree of confidence desired or achievable. Generally, the more sensitive a parameter is to the solution, the more accurately it should be defined.

The design and implementation of a program which will maximize the effectiveness of an effort with specific time and budgetary constraints is a challenging task. Because of the substantial variability of the data which will be obtained, and the physical problems associated with securing and analyzing the samples which the program will specify, it is most important that the monitoring program be carefully designed. It will be much more effective to the overall program to implement a monitoring effort which is relatively modest in scope, but which provides complete data sets, reliable results, and the type of data which improves estimates of the important parameters in the analytical procedure subsequently used. A program which generates large quantities of data may at the same time do little to contribute to an understanding of the system, or improve on the reliability of, or confidence in, projections to be made.

As a basic principle, the design of a stormwater monitoring program should reflect how the information obtained will be used in the analysis to follow. It should be developed by "working backwards" from a prior identification of the problem to be corrected, and the analytical procedure which will be used to characterize loads and impacts and examine the effectiveness of control measures.

Existing water quality data for the area should be collected and reviewed. Data from dry and wet weather periods should be compared for indications of stormwater effects. A visual inspection of the study area may indicate the existance of some water quality problems which normal data bases will not reflect. Shortcomings in the existing information and data base should be noted and the significant time and space scales of the problems should be determined.

Initial screening using methods outlined in Chapter 3 may be helpful in providing an overall perspective and in defining any parameter inputs that are particularly important to the problem solution. The basic elements in any stormwater analysis include the rainfall characteristics of the area, the amount of rainfall which reaches the receiving stream as surface runoff, the contaminants carried by the runoff together with a measure of the amount associated with runoff events, and the impacts of runoff loads on receiving water quality. Some pertinent scientific and technical criteria are presented for each of these elements in the sections which follow.

6.2 Rainfall Monitoring

Rainfall data are utilized in stormwater analyses in two significantly different ways.

1. To determine runoff characteristics of the study area, rainfall data for specific storm events are used in conjunction with measured runoff flows.

2. To project runoff loads from the study area for evaluation of water quality impacts and the effectiveness of control measures, a characterization of the overall rainfall patterns for the area is required. Long term characteristics and statistical properties of a record representative of the area are important rather than single event data for subcatchments.

6.2.1 Use of Rainfall Data to Define Site Characteristics

An important requirement of stormwater studies is that rainfall and runoff measurements be made by a monitoring program to characterize the runoff ratio for the area, i.e., the proportion of rain which falls on an area which will result in surface runoff. A limited number of site events will be monitored in this manner, even in a relatively comprehensive program; so long term rainfall characteristics will not be defined by such data. Rainfall data used for defining site runoff characteristics must reflect as accurately as possible the amount of rain falling on monitored catchments during specific storm events. This almost invariably requires local gages, or a network of gages in close proximity to the monitored catchment, with relatively small distances between gages.

The ratio of runoff to rainfall volume determined from such monitoring efforts is used in the statistical method or simplified simulation models, such as those discussed in Chapter 4, or as direct input to more sophisticated, event oriented simulations.

Since rainfall depths vary spatially, there is some error in the areal estimate of rainfall volume for individual events, even for small areas and local gages. This error is related to the size of the drainage area, the rainfall pattern, and the density of raingages. One of the practical effects of the error is to cause variability in the measured ratio of runoff to rainfall. This is illustrated schematically in Figure 6-1 for two hypothetical storms. Although the same amount of rainfall occurs over the drainage area and the measured runoff volume is the same, the apparent ratio of runoff to rainfall is 0.8 in the first case, and 0.33 in the second case. A monitoring program consisting of a number of storms (for example, 10 or 20) may find considerable variation in the ratio of runoff to rainfall from storm to storm. While a portion of this variation due, for example, to changing antecedent conditions is real, in the sense that it would be present even if the average areal rainfall depth during a storm was exactly determined, a major portion may also be due to inaccurate areal rainfall estimates. The larger the number of events which are monitored and averaged, the better will be the estimate of the mean ratio. When it is possible to monitor only a limited number of events, the error in the estimate of areal storm depths, and hence the runoff ratio, may be reduced by increasing the raingage density.

6.2.2 Use of Rainfall Data for Projections and Evaluation of Impacts

Different criteria are important when the purpose is to project runoff loads under the entire range of storm events which can occur in the study area. Such determinations require reliable long-term records of precipitation, and a record of adequate duration for definition of the statistical properties of the storm events. U.S. Weather Bureau records normally provide both the desired reliability and period of record, although they are often some distance removed from specific study sites. For any but quite closely spaced gages (or a gage within a relatively small drainage area) such records are normally unsuitable for defining site characteristics due to poor correlation on an individual event basis. On the other hand, when monthly, annual, or particularly, when long period records are compared, quite satisfactory correlations may exist even for gages spaced on the order of 25 to 50 miles apart. Thus, barring the presence of defined local influences such as orographic effects caused by local topography, U.S. Weather Bureau records

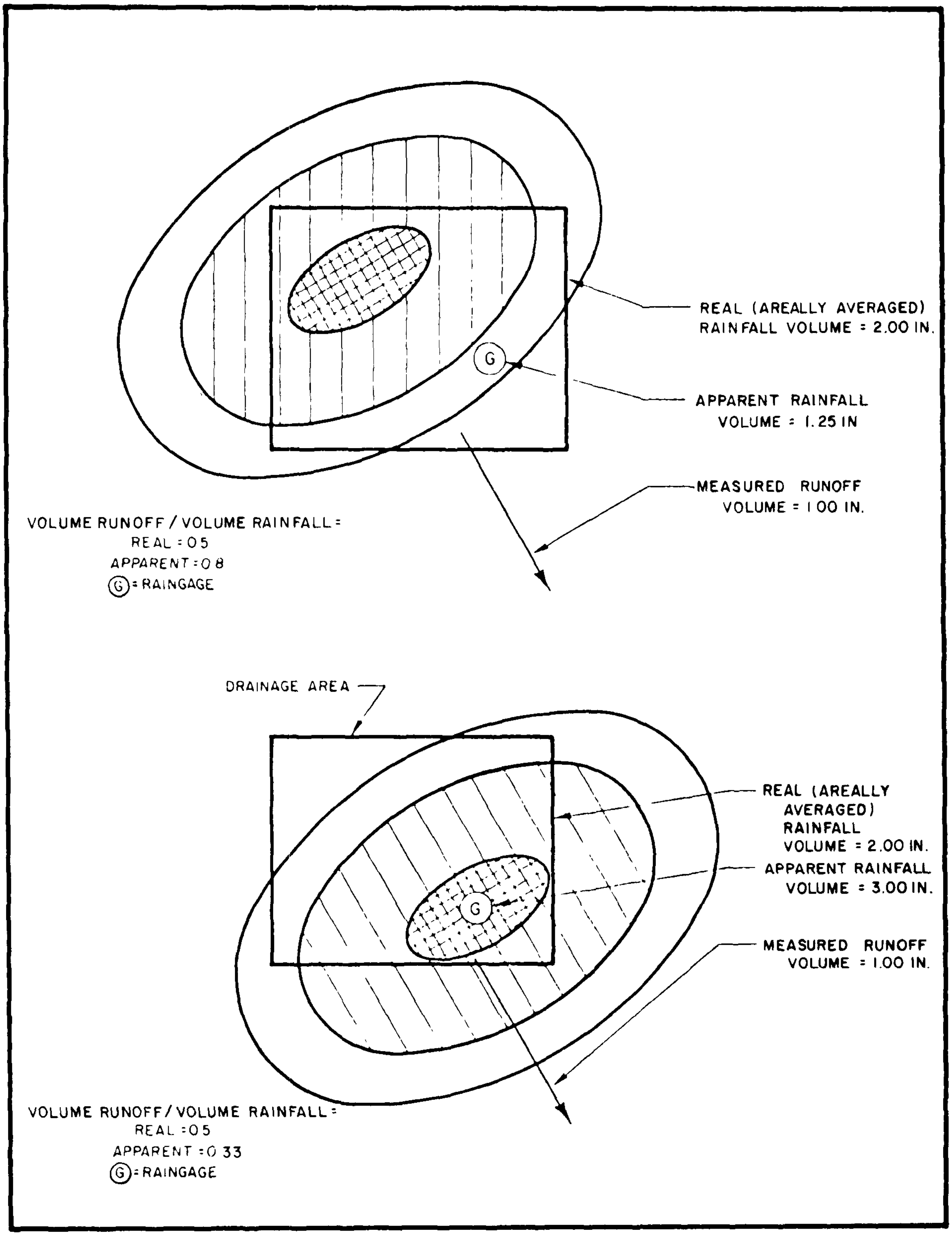


FIGURE 6-1
EFFECT OF AREAL DISTRIBUTION OF RAINFALL
ON REAL VS. APPARENT RATIO OF RUNOFF TO RAINFALL

can be used effectively for projections, even where gage locations are not within the study area.

Guidelines have been given to enhance the accuracy of such determinations by integrating the data from a network of gages, or by adjusting point rainfall records to better reflect areal distributions (see Section 5.1.4). Such refinements increase in importance with the size of the study area and the remoteness of reliable long term gaging stations.

The characterization of the long term event statistical properties is thus usually based upon existing recording raingages which report to the National Weather Service. If none, or too few of these gages are available, their records may be supplemented by placing additional continuously recording raingages in the study area. These data will be of little use in the long term characterization of precipitation until at least one year of records are accumulated. After one year, the data from the new gages may be compared to the existing gages to identify differences within the study area and to study the areal variability of rainfall by aggregating the records, as described in Section 5.1.4.1. Factors determined for converting point rainfall to areal rainfall during the study year may be applied to the longer records of the previously existing raingage(s), until more years of data are collected at the new gage(s). Therefore, there may be considerable value in having additional raingages for the statistical characterization of precipitation.

6.2.3 Raingage Density

A brief discussion of some basic relationships between the density of raingages in the study area and the accuracy of rainfall estimates is provided below. As may be expected, at higher raingage densities, the rainfall on a study area is determined with greater accuracy. However, the intent is not to specify a minimum raingage density, or the design of a network which would result in a given maximum error of estimate. At this stage of the examination of stormwater loads and impacts, it is neither practical nor desirable to commit resources to the design, establishment and operation of elaborate or extensive networks of raingages. By and large, stormwater studies will work with what is available. The value of an understanding of the raingage density relationships discussed here is that they help provide an appreciation of the limitations of the existing rainfall record.

It is always preferable to use data from several raingages to determine the average areal storm depth during a particular storm, rather than data from a single gage. Standardized techniques, such as the averaging method, Theissen method, and isohyetal method (1,2) are available for this type of determination.

A useful method for evaluating the ability of a network of raingages to represent the rainfall over a given area is to compare the areal rainfall depth estimated by a subset of the network to that estimated by the entire network. The rainfall depth estimated by the entire network is assumed to be the true storm depth. Figure 6-2 summarizes the results of such an analysis using data from a dense raingage network in Illinois, as reported by

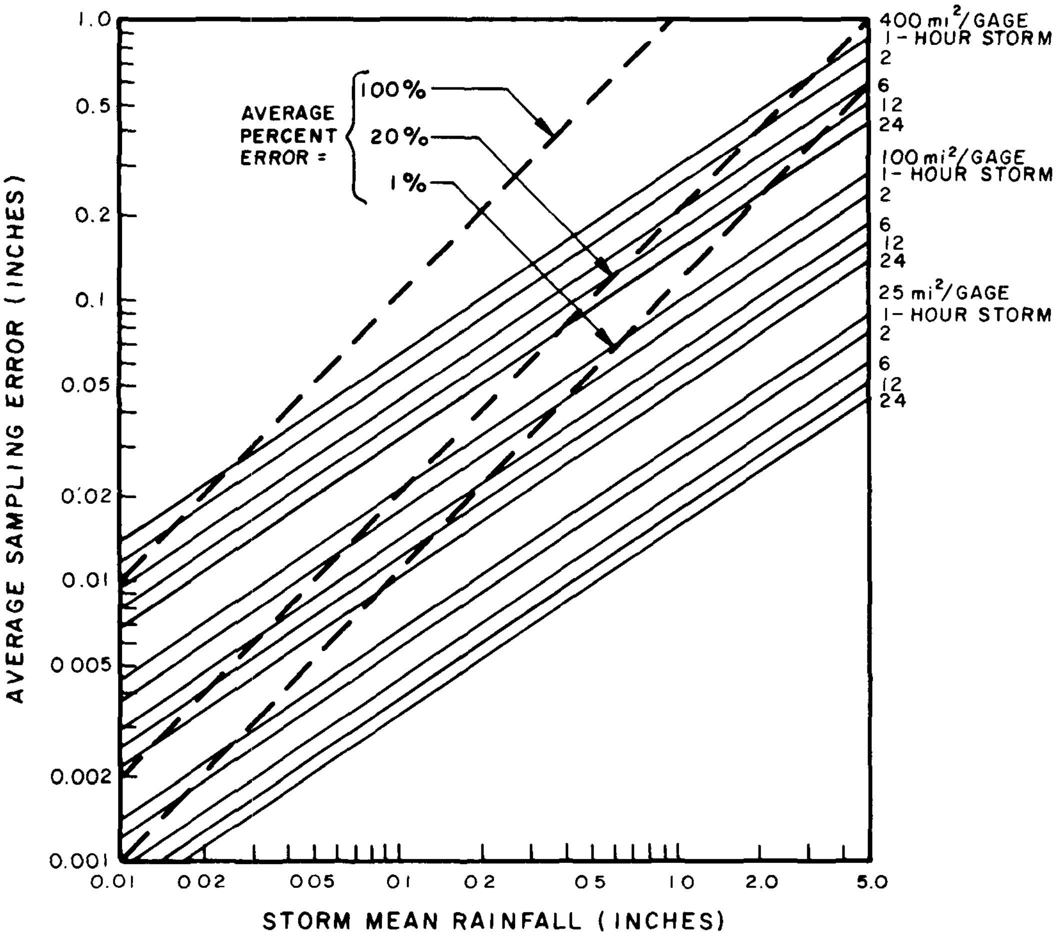


FIGURE 6-2

ESTIMATE OF RAINFALL SAMPLING ERROR IN 400 mi^2 ILLINOIS AREA

Huff (3). The average sampling error of the storm depth is seen to decrease with larger volume, longer duration storms, and with a more dense network (the curves were determined with regression analysis). The calculated average percent error has been superimposed upon Huff's curves. To provide a perspective, we can assume that we are most interested in storm events with mean rainfall volumes between 0.1 and 1.0 inches, because smaller storms contribute less significantly to waste loads and impacts, and larger storms occur relatively infrequently. For this range of storm events, in the Illinois study area, the average percent error is approximately nill for a raingage density (RGD) of 25 square miles per gage, nill to 20 percent for a RGD of 100 square miles per gage, and 10 to 60 percent for an RGD of 400 square miles per gage. Note, however, that this is the average percent error, and a few of the storms may still be estimated with a significantly larger error.

A similar type of analysis was conducted by Hydroscience, Inc. (4) using data collected by the U.S. Geological Survey and the Texas Water Development Board (5,6,7) on seven small watersheds in Texas.

The results of this analysis are summarized by Figure 6-3, indicating the effect of raingage density on the error which can be expected in determining the distribution of rainfall on an area, based on data from a gage or a network of gages. These results of the Texas study are displayed differently than those from Illinois, and use a different basis for defining accuracy. In the Texas area, raingage densities in the order of 10 to 25 square miles per gage produce a standard error of estimate in the order of 10 percent. This is related to, though not the same measure as the average percent error which describes the Illinois results. A standard error of 10 percent signifies that the estimated rainfall for a storm in the study area will be within ± 10 percent of the true rainfall about 2/3 of the time.

A conclusion which appears appropriate based on a review of published literature on the subject (8), and as illustrated by the two examples presented, is the following. The number of raingages in relation to the size of the study area (the raingage density) can have an appreciable effect upon the estimates of rainfall on a study area by using one or a series of raingage records. As indicated by the difference between the Illinois and the Texas results, and further by the differences between individual basins in the Texas study, the geographic location will influence the relationship between the raingage density and the accuracy of estimates which are made. Seasonal effects in an area which, for example, may be prone to summer thunderstorm activity can also be expected to influence this relationship. It is, accordingly, possible to provide only some general guidelines concerning raingage density in tnis discussion.

Raingage densities in the order of 10 to 25 square miles per gage (gages spaced four to five miles apart), would generally appear to provide a very satisfactory degree of accuracy for utilizing long term records, or for estimating the distribution of rainfall over a study area, if one is content to accept the fact that the error might be great on any one specific storm, but that over a period of time results average out such that the

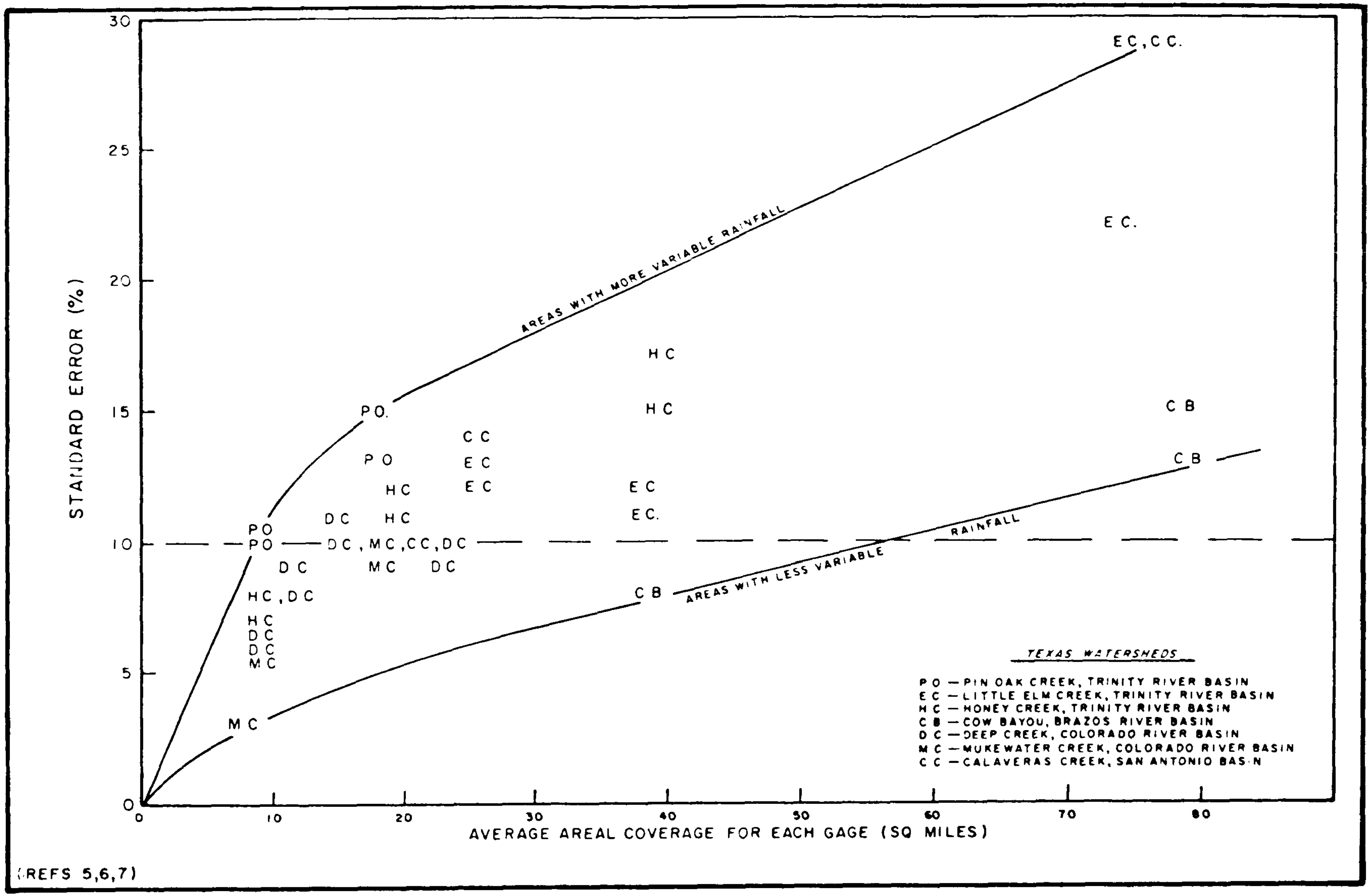


FIGURE 6-3
EFFECT OF RAINGAGE DENSITY ON RAINFALL ESTIMATES

overall analysis has a high level of accuracy. On this basis, errors in the order of 10 percent can be considered quite acceptable.

At a density of about 100 square miles per gage (about 10 miles spacing between gages), accuracy in most cases will probably prove acceptable for the type of analysis being performed, recognizing the accuracy of other elements of the analysis, and particularly given the sensitivity of water quality impacts to uncertainties in the waste loads in the order of 10 or perhaps 20 percent. It would appear, from available information, that when raingage densities reach or exceed on the order of 400 square miles per gage (20 miles spacing between gages), that the accuracy of estimates of the areal distribution of rainfall on a study area become more uncertain and are likely to result in a large error for many of the rainfall events.

The guidelines just presented refer to the use of a long term records, or the application of rainfall records in a manner which takes advantage of the fact that errors on individual storm events will average out in the long term. It is quite a different matter when such an "averaging out of results" cannot be tolerated. This is the case when the purpose of the rainfall measurement is for comparison with measured runoff from a specific catchment to determine the characteristics of that site. Here, the requirement is for the best estimate of how much rainfall occurred on the monitored area during the particular storm event for which runoff flow and quality are measured. Considerations such as the average error or the standard error of the estimate are less meaningful in this case, since even acceptably low average errors provide no confidence in the error of a specific individual event.

For this reason, it is strongly recommended that for the determination of site characteristics, the monitoring program include at least one gage located in the catchment being monitored for quality and flow. For this purpose, relatively simple gages which provide only the total volume during the storm event are appropriate. Since monitoring teams will be active in the area for measuring flows and collecting quality samples, the servicing of such simple gages will not usually add a significant burden to the effort. If and when the stormwater analyses make use of sophisticated, event oriented simulation models which require more temporal detail than the methods presented in this manual, continuous recording gages would be required rather than the simple volumetric gages suggested above.

Marsalek (14) cites Linsley (26) as recommending the use of two or more raingages for even the smallest watershed, with an absolute minimum of one gage located within the watershed. He considers two gages sufficient for areas up to 4 square miles, and three for an area up to 20 square miles.

6.2.4 Other Raingage Considerations

The selection of sites for raingage locations is important, particlarly in highly developed urban areas. Ground level locations are preferable, however, roof tops may be used as long as the gage is not placed near the edge, or near obstacles on the roof (9). It is generally agreed that gages not be placed within a distance of three or four times the height of the nearest obstacle (i.e., building, tree, etc.). However in densely settled urban areas, distances of at least one height may serve as a minimal yet reasonable criterion (9). Although shielding is used at most first order stations, it is generally not required (9).

There are several types of recording raingages available (10). The tipping bucket gage is recommended because of its commercial availability and proven ability. Recordings are made for each 0.01 inch of rain. The tipping bucket raingage may underestimate very high rainfall intensities, however, and is not appropriate for snowfall measurements. As previously discussed, budgetary constraints may limit the use of recording gages in each area, and less expensive manual funnels, which measure only the total storm depth at the end of an event, may be appropriate.

6.3 Site Selection and Drainage Basin Characterization

Ideally the runoff and overflows from the entire study area should be monitored for a large enough number of events to accurately define the properties of stormwater pollution loads. Since this may not be economically feasible, the selection of several representative areas is often necessary. There are a number of criteria for selecting monitoring sites. One approach is to select small drainage areas, each having one predominant characteristic or land use. Data collected from each monitored area are then synthesized and extrapolated to represent similar areas (i.e. land use) in the rest of the community. This approach is directed towards identifying relationships between drainage basin characteristics and runoff properties. Characteristics which should be considered include land use, type of conveyance system (i.e. combined vs. separate), conveyance system characteristics such as sewer slope, and geographic location. Sites are selected to represent the range of conditions found in the planning area.

Another approach to site selection is to monitor larger areas with mixed characteristics. The runoff properties are more representative of the average, typical conditions found throughout the area and the total load from the urban area may be better estimated as a larger portion of the region is monitored.

One promising method for determining the quality of combined sewage in larger, integrated areas is to monitor the influent to combined municipal treatment plants during storms (11). The disadvantage of sampling large, mixed areas is that it is impossible to isolate the effects of pertinent drainage basin characteristics (such as land use) on runoff quantity and quality.

There are advantages and disadvantages to each of the approaches to site selection described above. A comprehensive monitoring program can incorporate both types of sampling (12): a number of small, homogeneous drainage areas can be monitored to attempt to identify and isolate the effects of land use, sewer characteristics, etc.; and a few larger areas should be monitored to better represent the total loadings from the study region.

A number of other factors should be considered when selecting monitoring sites. These include:

1. A complete flow balance must be established in areas where the ratio of runoff to rainfall (R_V) is to be calculated. This limits the possible sites to hydraulically tight systems for which complete flow balances are possible. These tend to be smaller, less complex basins.

2. Sites need to be suitable for the placement of necessary sampling equipment. Prospective sites can be observed during storm periods to check for possible problems.

3. Sites that are readily accessible, particularly during adverse weather conditions are preferred. The availability of electrical power is an important consideration.

4. The potential for vandalism is also a consideration. Positive public awareness of the program should be encouraged.

5. Written legal permission should be obtained for land rights and easements.

A more detailed discussion and methodology for site selection is included in the Guide for Collection, Analysis, and Use of Urban Stormwater Data (10).

Characterizations of basins and catchments within the urban area are necessary for both the site selection and the data analysis procedures. A comprehensive listing of the elements of a basin characterization is presented below. The level of detail with which the various elements are addressed in a study is subject to flexibility, based on the scope and depth of analysis in the overall effort. This is particularly true of the detail with which the collection system is characterized.

The size of each area and the predominant land characteristics are determined. The percent of each land use type and population density are noted, and may be used to estimate the percent impervious area in each catchment. Land use information and a direct estimate of the percent impervious area may be obtained from areal photographs, satelite photographs (Landsat)(13), or local inspection, though local inspection is impractical for large study areas. Some information on local soil types, permeability, and ground slopes should be obtained. If a site specific relationship between the percent impervious area and the ratio of runoff to rainfall (R_V) is being determined to further refine the general range shown in Figure 5-20, a wide range of imperviousness should be obtained throughout the city.

Information on the collection system is also required. Tributary areas are delineated and overflow points located. The capacity of major interceptors is catalogued and an estimate of internal storage is made. The potential for deposition in the system should be estimated using factors such as average sewer slopes and dry weather flow velocities. An estimate of the dry weather flow in combined systems (or in separate storm systems with illegal connections or infiltration) is necessary. Certain situations where the sewer maps do not provide an adequate representation of actual conditions, or where there are faulty or inoperative regulators, illegal connections, or cross connections between systems are best avoided. Local sewer maintenance personnel can provide guidance.

Management practices which are currently employed in the drainage areas are of interest, such as the street sweeping frequency and catch basin cleaning programs. Construction sites, residual disposal sites, highway deicing practices, and special industrial facilities (i.e. storage ponds, transfer sites, etc.) are of particular note. If significant activities occur on a seasonal basis, samples collected during the different seasons can be checked for this influence.

6.4. Monitoring of Runoff and Overflows

Stormwater load determinations are based on estimates of quantity and quality, and both aspects must be monitored in an assessment program. As discussed previously, the monitoring program is tailored to collect data for use in a specific modeling procedure for a specific set of water quality problems, although some consideration can be given to possible future uses of the data. The guidelines presented in the following sections are primarily directed towards programs collecting information for studies using assessment models such as the statistical method (Chapter 3) and broad scale simulators (Chapter 4). The required information is the total storm runoff volume, average flow rate, average concentration, general temporal concentration profiles (average duration and magnitude of the first flush), and some indication of flow-concentration correlations. Requirements for programs collecting data for more sophisticated models, which predict hydrographs and pollutographs at finer time scales, are also briefly discussed. Factors considered include the length of the study period, sampling frequency, and the sampling procedure.

6.4.1. Study Period and Sampling Frequency

Monitoring programs can vary from a few typical events during a few critical months, to many events over one or more years. The length of the study period varies depending upon the purpose of the study, availability of budget and manpower, and the magnitude and complexity of the study area.

It is important to plan a program to obtain needed data within the constraints of the study circumstances and to yield a maximum of useful information for the monies spent. In a statistical evaluation of rain events performed by the National Oceanographic and Atmospheric Administration (NOAA), one of the findings is that there is a 90 percent confidence level in the expectation that 85 percent of the various intensity and duration rain events for a given location will be experienced within a 2.8 year period (14). It would therefore be a rather long term and costly project to study even most of the possible rainfall events. Some decision must be made on how long a study period should last and how many events should be monitored.

For assessment models, the first purpose of the monitoring program is to determine the average ratio of runoff to rainfall (R_V) and average values for pollutant concentrations ($\bar{c}$). A consequence of statistical sampling theory, based on the Central Limit Theorem, is that the sampling distribution of the average tends to become normal as the number of samples increases, no matter what the distribution of the variable being sampled (15). This property may be used to estimate the number of storms necessary for an adequate estimate. The number is dependent upon the variability of the parameter being measured (in this case, R_V and $\bar{c}$), the maximum desired error in the estimate of the average, and the confidence that one has that the sampled average falls within the particular range. Figure 6-4 shows the relationship between the number of site events monitored and the resulting level of accuracy. The curves presented are based on a coefficient of variation of 0.75 for the sampled parameters, which appears to be a reasonably conservative estimate for both R_V and pollutant concentrations based on observed data (16,17,18). In general, 20 to 40 events will provide a very adequate representation. When time and budget constraints restrict the number of storms which can be sampled, programs may be designed with as few as 10 storms, although much below this number may leave little confidence in the estimated averages, as indicated by the curves in Figure 6-4. It is better to accurately define the properties of fewer sites than to collect an insufficient number of storms in many locations (19). Seasonal or temporary conditions (seasonal industrial operations, leaf fall, construction activity, etc.) may require that a larger number of storms be sampled to adequately characterize a given site. At least one dry weather, diurnal sampling of each combined sewer site should be conducted to identify base conditions and its variation over the day, to better evaluate storm runoff quantity and quality during monitored storms.

6.4.1.1. Sampling Interval Within Storms

The variability of pollutant concentrations within storms tends to be about equal in magnitude to the variability between storms. Therefore, Figure 6-4 may also be used to estimate the number of samples needed within a storm to adequately estimate the mean runoff concentration during the event. This type of characterization is appropriate for the assessment procedures used in this manual. Ideally, shorter storms should be sampled more frequently than longer ones, however, there is no way of knowing the duration of an event until after it is over. An intermediate frequency may be chosen based on the accuracy desired, the average duration of runoff events, and the practical limitations of the sampling equipment and collection procedure. If, for example, runoff durations are typically 5 to 10 hours and at least 10 samples per storm are desired, a maximum sampling interval of one half hour to one hour is indicated. A few short storms may be undersampled and a few longer events sampled more than necessary.

Sampling within a storm may also be directed towards an examination of the first flush effect, and more frequent sampling is required in the early portions of the storm. This more frequent, initial sampling may be routinely incorporated in the monitoring program, or only conducted on a limited number of special surveys, depending on the significance of the first flush effect for the planning study. The frequency required during early portions of the storm is the same as is generally prescribed for sampling programs collecting input for more refined analyses which attempt to accurately reproduce the

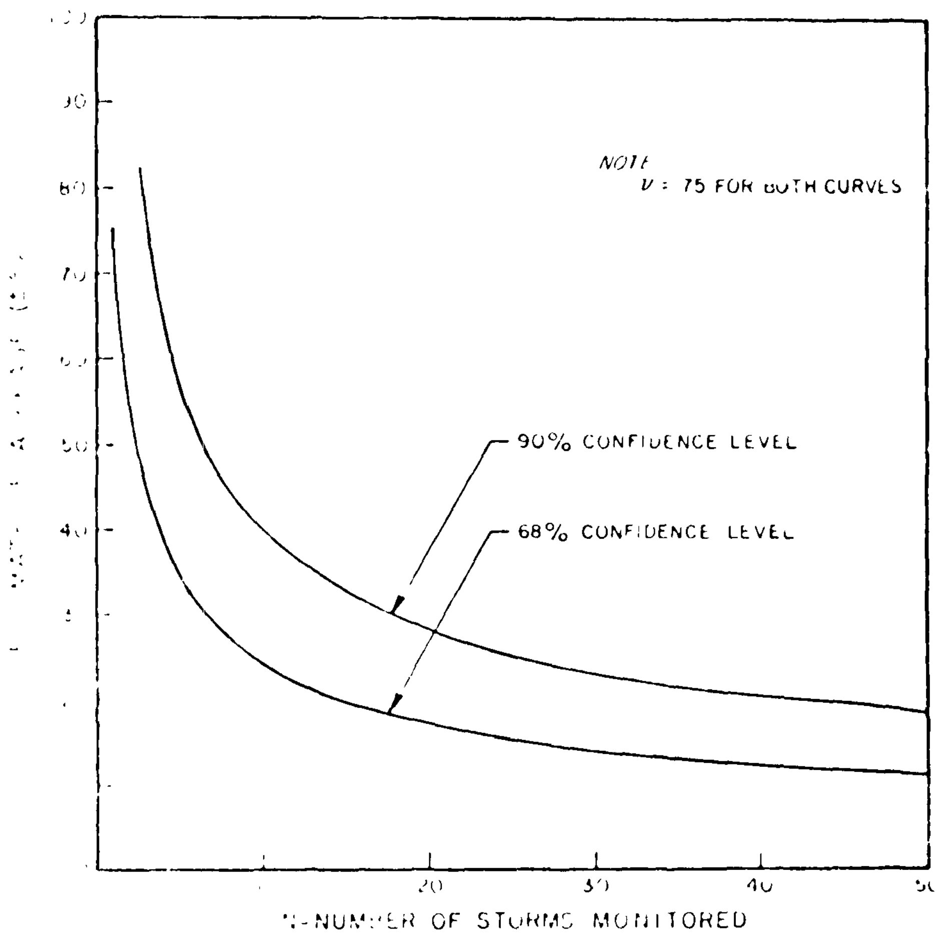


FIGURE 6-4

ERROR IN ESTIMATE OF AVERAGE VERSUS NUMBER OF STORMS MONITORED

storm pollutograph. A review of stormwater runoff and overflow monitoring programs yields the following suggested sampling intervals for this type of program (14):

SAMPLING INTERVALS FOR ACCURATE FIRST FLUSH OR POLLUTOGRAPH DEFINITION

Basin Size (acres)	Sampling Interval (minutes)
10	5
100	5-10
500	10-15
1000	10-15
2000	15
3000	20
5000	25-30

The size of the catchment is important because runoff from smaller areas occurs more rapidly with more strongly defined first flush peaks. However, this short interval sampling is not necessary for all storms when the analysis procedure is based upon the type of assessment methods described in this manual.

6.4.2 Sampling Procedure and Parameters

Sampling procedures can differ, depending on whether the objective is to identify the within-storm variations in concentration, or to determine the total mass of pollutant discharged per storm event.

A sampling program designed to define the mass per storm event will result in a single sample being analyzed for each parameter of interest, representing the particular site and storm event. The pollutant concentration so determined will, when multiplied by the total volume discharged (determined from flow monitoring records) yield the mass discharged. In order that the single sample provide an appropriate representation of the entire storm, a careful compositing procedure must be employed.

Obviously, flow-weighted composites are most desirable. This requires that a series of discrete grab samples collected throughout the duration of the storm event, be combined using a different volume from each discrete sample, the volume of each being determined by the amount of storm flow associated with it. Equal volume samples are taken at regular intervals during the storm event, either manually or with an automatic sampler; monitoring of flow is required. One practical problem with the successful use of this approach, is the delay sometimes encountered in retrieving and analyzing the flow record. To use this approach, flow records must be examined on a sufficiently timely basis to comply with guidelines for acceptable time delays in processing samples for chemical and biological analysis.

In cases where flow records are not taken, where they can not be retrieved rapidly enough, or where they are not available due to malfunction, then simple time compositing of individual samples must be employed. Although less desirable than flow-weighted samples, simple time composited samples can provide useful information on mean concentrations and mean storm loads, provided that:

(a) A sufficiently large number of storm events are analyzed. The variability of within-storm flows and concentrations would tend to average out errors introduced by lack of flow-weighting.

(b) The analyst recognizes that the best estimate provided will be for long-term effects, and that the estimate for an individual storm event may be poor.

The most appropriate approach for securing a flow-weighted storm composite, will involve the use of automatic equipment with sampler operation being controlled by a flow meter. Two approaches can be employed based on the type of automatic equipment selected. The first results in the collection of samples of variable volume, but at regular time intervals. The volume of sample collected is proportioned to the rate of flow at the time it is taken. The other approach employs a flow meter with an integrator (totalizer), so that the sampler is activated on the passage of a specific volume of flow. Thus, uniform sample volumes are collected, but at variable intervals of time.

Programs, or components of programs, designed to secure information which will permit generation of Pollutographs, are simpler to address, since they eliminate the question of compositing. They do however, utilize a much higher level of laboratory analytical resources, because of the large number of samples analyzed for each storm. A series of sequential grab samples are collected, either manually or with automatic samplers, at either constant or variable time intervals related to flow changes. A modification of this approach, which reduces the number of samples to be subjected to laboratory analysis, composites samples for certain periods during the event, so that a series of sequential composite samples is provided for analysis.

A question typically faced in design of a sampling program, is the interval at which discrete samples should be secured. An approach which recommends itself is to adopt an objective of securing some minimum number of samples for each event, which will span the entire event. Samples are taken relatively frequently, such that the desired number will be secured during relatively short storms. During the longer storms, extraneous samples are discarded.

Whether the preferred approach to storm monitoring for a particular project should involve the use of manual or automatic methods of sample collection, is something best decided on an individual basis. Automatic sampling is theoretically superior, however, automatic equipment is subject to malfunction and to physical damage in this application. The financial and manpower resources will be influential in the decision to be made. Either manual or automatic methods can be successfully employed. Both, however, require care and attention in the design and execution of the program.

The water quality constituents that are of interest in stormwater runoff and overflows are dependent upon the particular problems present in the study area. Analytical costs may be high and discretion should be used. Routinely sampled are indicators of oxygen demand, particulate concentration, pathogenic microorganisms, nutrients, and toxic substances. Oxygen demand is measured with the BOD_5 test, however, some long term oxygen demand tests (i.e., 40 days) are run to better estimate the oxidation rate, check for nitrification, and check for possible toxicity effects which may suppress the BOD_5. Long term BOD's may be run with and without agents to suppress nitrification. Measurements may also be made on filtered and unfiltered samples to estimate the fraction of the biochemical oxygen demand in the particulate or dissolved states. Filtered and unfiltered tests are also suggested for other pollutants to aid in the treatability assessment (Section 5.5.1). The particulate concentration is usually determined by the suspended solids test (non-filterable residue), however, selected tests for settleable solids may also be performed to examine deposition and treatability. Total and fecal coliforms are common indicators of pathogenic organisms, and fecal streptococcus may also be measured to examine sanitary quality. Suggested nutrient measurements include nitrate and nitrite, total kjeldahl nitrogen, and total phosphorus. Toxic substances may include heavy metals, pesticides, or herbicides; depending upon local problems and conditions. Chloride measurements are made in coastal areas to check for seawater intrusion due to faulty tide gates. Because the alkalinity of rainwater is very low (near 0 mg/l) and the alkalinity of sewage in a given location is often relatively constant (between 50 and 200 mg/l), alkalinity measurements may serve as a useful tracer for distinguishing between domestic wastewater and stormwater runoff in the sewer system.

6.5. Monitoring of Receiving Water

If the review of existing data and a general knowledge of the problems in an area are insufficient to adequately define the water quality problems and their relevant time and space scales, an initial survey of the receiving water should be conducted. Water quality samples are collected in the receiving water during representative wet and dry weather periods. A water quality survey should be conducted during a dry weather period in the summer months when flows are low and temperatures are high. During these dry weather periods, the small streams and rivers will often approximate steady state conditions and can be sampled spatially at an appropriate time to determine the steady state profiles of various pollutants. Grab samples are collected at a number of locations with more refined spatial sampling near major waste loads where steeper spatial concentration gradients are expected. Composite samples of significant waste loads should also be taken. In estuaries, the water quality varies over the tidal cycle, but may be at steady state intertidally, and samples should be collected at both high and low slack. One or two dry weather surveys should suffice to estimate the magnitude of water quality problems, and the analysis of flow records will normally indicate the likelihood of these problems occurring during other periods of the year.

A wet weather water quality survey should be conducted by collecting water quality samples at various locations and at a number of times during and following a storm event. The sampling interval in the receiving water is determined based on an analysis of predicted responses. In a stream where little dispersion or mixing occurs, storm impacts are pronounced, but are transported rapidly form the point of discharge. In an estuary, the stormwater load is mixed and diluted upon entering the receiving water, but the effects of the storm remain well after the end of the event, and less frequent sampling for a longer period is required.

A preliminary calculation using a receiving water model or equivalent calculation, and utilizing preliminary estimates of pertinent input factors will often prove quite helpful in the design of an effective receiving water monitoring program. Even relatively crude calculated responses for the parameter of interest will aid in establishing an effective time and space framework for the monitoring effort. The samples collected from the dry and wet weather survey should be analyzed for dissolved oxygen and pollutant indicators such as the biochemical oxygen demand, suspended solids, dissolved solids, total and fecal coliforms, phosphorus and nitrogen, heavy metals, and other pollutants which may be important in the study area. Areas with suspected algal problems should be sampled throughout the day for dissolved oxygen, particularly in the early morning and the late afternoon. Chlorophyll 'a' measurements are often made as an indicator of phytoplankton levels. Areas where oxygen depressions are not readily explainable, particularly near combined sewer overflows or major waste discharges, should be checked for sediment oxygen demand. Chloride measurements are taken in estuarine and coastal areas to help identify the transport properties of the water body.

A review of available information on the physical and hydrological characteristics of the receiving water should also be made. If certain information necessary for the modeling effort are lacking, these data should be collected as part of the monitoring program. These include the geometry of the water body (cross-sectional area and depth) in a number of locations, flow and velocity, temperature, and dispersive properties (dye studies) if these are important. For small streams and rivers, the cross-sectional area varies throughout the year depending on the flow, and a number of measurements during different periods are necessary.

After the initial assessment of water quality, further sampling surveys may be appropriate to resolve problems identified by the early efforts, to improve the calibration and verification of receiving water models, and as part of the ongoing monitoring of possible problems. These surveys should be directed towards a better defined set of specific water quality problems.

6.5.1 Continuous Monitors

As methods which predict the long term characteristics of water quality (mean, variability, and frequency distribution) become more advanced, data to verify these predictions become more useful. Continuous monitoring systems for water quality can provide these data. The entire range of water quality is observed, without missing rare or extreme events.

The state-of-the-art of continuous water quality monitors is rapidly changing. Systems range from electrode monitors to automated chemistry units. Although a number of parameters are measurable, currently the most reliable monitors are for temperature, dissolved oxygen, pH, and conductivity. Monitors are still susceptible to malfunction and should be checked and calibrated frequently. Interest has also been recently generated in techniques for estimating water quality from satelite photographs, though this method does not yet appear very promising (20).

6.6 Review of Monitoring Literature

The monitoring section of this manual presents a broad overview of considerations for an effective stormwater monitoring program. A number of reports are available which provide more detail on specific aspects of monitoring. A number of particularly valuable publications are briefly discussed in this section and these or similar reports should be read before initiating a stormwater monitoring program.

A general, comprehensive review of monitoring requirements, methods, and costs is included in Appendix D of the *Areawide Assessment Procedures Manual* (21). This report discusses sources of available data, types of monitoring activities, and the selection of sites, parameters, and frequencies for sampling. Flow measurement and sampling equipment and procedures are analyzed, and cost estimates are provided for various phases of the monitoring program.

The report, *Guide for Collection, Analysis, and Use of Urban Stormwater Data* (10), presents the findings of a unique conference sponsored by the Engineering Foundation, U.S. Geological Survey, and the American Society of Civil Engineers. The final report was rewritten by the approximately 70 attending participant specialists assigned to different work groups. The report discusses stormwater data utilization and modeling analysis, network planning and design, catchment selection, instrumentation, and sampling procedures.

Section VI of *Urban Stormwater Management and Technology - An Assessment* (22) examines various data collection and sampling programs. Guidelines are presented for selecting sampling sites and sampling equipment. Factors such as intake design, collection method, sample transport and storage, and controls and power are discussed. Case histories of large scale sampling systems with specific problems and recommendations are presented. Various types of flow measuring devices are assessed for performance and cost. Research efforts in flow measuring technology are described and suggestions are made for data analysis.

The report, *Methodology for the Study of Urban Storm Generated Pollution and Control* (23), contains recommendations for standard procedures to follow in projects dealing with pollution assessment and abatement of storm generated discharges. The purpose of the project is to develop standard procedures needed to insure that all dischargers and treatment processes are evaluated with the same methods. Issues addressed include methods for sampling and sample preservation, available instrumentation, the choice of water quality parameters, analytical procedures, and methods for evaluating treatment processes.

A study conducted for the Canada Center for Inland Water, *Instrumentation for Field Studies of Urban Runoff* (14), examines monitoring techniques for field studies of urban runoff. Recording precipitation gages, sewer flow measurement instruments and automated wastewater samplers are examined. Comments and discussions are also presented for sampling runoff loads to obtain total storm pollutant loadings and to define the magnitude of the first flush effect. A review of sampling methods is made and comparisons are presented for data during one storm in Washington D.C.. Sampling intervals used in a number of urban runoff studies are summarized.

A section on the "Collection of Field Data for Stormwater Model Calibration" in the *1976 Short Course on Applications of the Stormwater Management Model (SWMM)* (24) reviews the types of field data and the factors involved in wastewater characterization. Flow measurement, sampling, and laboratory analyses are discussed in detail and a review of recent field experience is presented.

The Environmental Protection Agency report, *An Assessment of Automatic Sewer Flow Samplers* (25), presents a general discussion of the purposes and requirements of a sampling program. The favorable characteristics of automatic sampling equipment are set forth and problem areas are outlined. A compendium of over 60 models of commercially available and custom designed automatic samplers is included. A description and characterization of each unit is presented along with an evaluation of its suitability for stormwater applications. A review of field experience with automatic sampling equipment is given and a design guide for the development of an improved automatic sampler for use in storm and combined sewers is presented.

6.7 References

1. Linsley, R.K., Jr., M.A. Kohler, and J.L.H. Paulus, *Hydrology for Engineers*, McGraw-Hill, New York, 1958.

2. Chow, V.T., *Handbook of Applied Hydrology*, McGraw-Hill, New York, 1970.

3. Huff, F.A., *Sampling Errors in Measurement of Mean Precipitation*, Journal of Applied Meteorology, Volume 9, February 1970.

4. *Water Quality Management Planning Methodology for Urban and Industrial Stormwater Needs*, Hydroscience, Inc. for Texas Water Quality Board, Arlington, Texas, 1976.

5. *Hydrologic Studies of Small Watersheds, Pin Oak Creek, Trinity River Basin, Texas, 1956-62*, Texas Water Development Board, Report 54, August 1967.

6. *Hydrologic Studies of Small Watersheds, Cow Bayon, Brazos River Basin, Texas, 1955-64,* Texas Water Development Board, Report 99, October 1969.

7. *Hydrologic Studies of Small Watersheds, Calaveras Creek, San Antonio River Basin, Texas, 1959-69,* Texas Water Development Board, Report 154, August 1972.

8. Rodriguez-Iturbe, I., and J.M. Mijia, *The Design of Rainfall Networks in Time and Space,* Water Resources Research, Volume 10, No. 4, August 1974.

9. Huff, Floyd A., *Sampling, Analysis, and Interpretation of Rainfall for Hydrologic Applications,* Presentation at MSDGC-AMSA Workshop on Water Quality Surveys for 208: Data Acquisition and Interpretation of Non-Point Runoff, Chicago, April 21, 1977.

10. *Guide for Collection, Analysis, and Use of Urban Stormwater Data,* American Society of Civil Engineers, New York, 1977.

11. Mueller, J.A. and A.R. Anderson, *A Mass Balance Method for Estimating Combined Sewer Runoff and Overflow Quality From Sewage Treatment Plant Data,* Environmental Eng. and Science Program, Manhattan College, Bronx, New York, 1977.

12. Grizzard, T.J., J.P. Hartigan, et. al., *Characterizing Runoff Pollution - Land Use Relationships in Northern Virginia's Ocoquan Watershed,* Presented at MSDGC-AMSA Workshop on Water Quality Surveys for 208: Data Acquisition and Interpretation of Non-Point Runoff, Chicago, April 20-22, 1977.

13. Jackson, T.J., R.M. Ragan, W.N. Fitch, *Test of Landsat-Based Urban Hydrologic Modeling,* Journal of the Water Resources Planning and Management Division, ASCE, Volume 103, No. WR1, May 1977.

14. Marsalek, J., *Instrumentation for Field Studies of Urban Runoff,* Environmental Management Service, Canada Center for Inland Waters, Project 73-3-12, Burlington, Ontario.

15. Au, Shane, Hoel, *Fundamentals of System Engineering, Probabilistic Models,* Addison-Wesley, Reading, Massachusetts, 1972.

16. Wells, Dan M., et al., *Variation of Urban Runoff Quality and Quantity with Duration and Intensity of Storms - Phase III, Volume 4 - Project Summary,* Texas Tech University, Water Resource Center, WRC-75-4, Lubbock, Texas, December 1975.

17. *Urban-Runoff Data Base,* University of Florida for U.S. Environmental Protection Agency, Project No. 68-03-0496, In Progress.

18. *Summary of Runoff Data from Durham, N.C., Milwaukee, Wis., Lubbock, Texas, and Washington D.C.,* Internal Memorandum, Hydroscience, Inc., 1976.

19. Shelly, P.E., *Data Collection to Evaluate Management Technique,* Presentation at U.S. Environmental Protection Agency Conference on 208 Planning and Implementation, Reston, Virginia, March 16-17, 1977.

20. Shelly, P.E., *Sediment Measurement in Estuarine and Coastal Areas,* EG&G Washington Analytical Services Center, Inc., for National Aeronautics and Space Administration, NASA CR-2769, December 1976.

21. *208 Areawide Assessment Procedures Manual, Volume II, Appendix D,* EG&G Washington Analytical Services Center for U.S. Environmental Protection Agency, EPA-600/9-76-014, Cincinnati, July 1976.

22. Lager, J.A., W.G. Smith, *Urban Stormwater Management and Technology - An Assessment,* U.S. Environmental Protection Agency, EPA-670/2-74-040, 1974.

23. Wullschleger, R.E., A.E. Zanoni, C.A. Hansen, *Methodology for the Study of Urban Storm Generated Pollution and Control,* U.S. Environmental Protection Agency, EPA-600/2-76-145, August 1976.

24. *Short Course Proceedings Applications of Stormwater Management Models, 1976,* University of Massachusetts for U.S. Environmental Protection Agency, EPA-600/2-77-065, March 1977.

25. Shelley, P.E., and G.A. Kirkpatrick, *An Assessment of Automatic Sewer Flow Samplers,* U.S. Environmental Protection Agency, EPA-R2-73-261, June 1973.

26. Linsley, R.K., *A Manual on Collection of Hydrologic Data for Urban Drainage Design,* Hydrocomp, Inc., Palo Alto, California, 1973.

CHAPTER 7

EXAMPLE STORMWATER ANALYSES

A variety of analytical tools and techniques for evaluating stormwater runoff and overflow pollution problems are developed in this manual. In this section, these tools are applied in two example problems: Salt Lake City, Utah and Kingston, New York. The physical and problem settings are real, however some liberties have been taken to enhance the basic objective of illustrating the practical application of the methodology described in this manual.

The examples selected provide geographic diversity and a range of water quality problems to be addressed. Salt Lake City is located on a river while Kingston is adjacent to an estuary. Salt Lake City is served by separate sewers or natural conveyance while Kingston has a combined sewer system. The principle problem analyzed in the Salt Lake City example is the impact on dissolved oxygen levels necessary for aquatic life, while the problem addressed in Kingston relates to bacteria organisms as they affect bathing beaches. Both examples represent a preliminary assessment. Not all the questions raised are fully answered, and further studies may be appropriate. The examples do demonstrate, however, the utility of the methodologies presented for obtaining a first evaluation of the stormwater problem in urban areas.

7.1 Salt Lake City, Utah

Salt Lake City is located on the Jordan River just upstream of the Great Salt Lake. The principal features of the study area are outlined in Figure 7-1. The Jordan River flows north from its origin at Utah Lake through predominantly agricultural and suburban land until it reaches the Salt Lake City area. A surplus canal in the southern portion of the city was constructed to divert flood flows away from the city, and route them more directly to the Great Salt Lake. It serves to regulate the flow in the lower Jordan River. About 17 miles below the city the Jordan River ends in an undeveloped marsh area near the Great Salt Lake. The principal use of this undeveloped area is as a wildlife and waterfowl habitat. The Jordan River in this reach presently contains a number of rough fish, but there are proposals to stock the river with game fish to provide additional recreational benefits. The analysis presented in this section is directed towards evaluating the effect of urban runoff from Salt Lake City on the dissolved oxygen concentration in the Jordan River, and the possible subsequent limitation of beneficial uses of the waterway. The system is represented in a

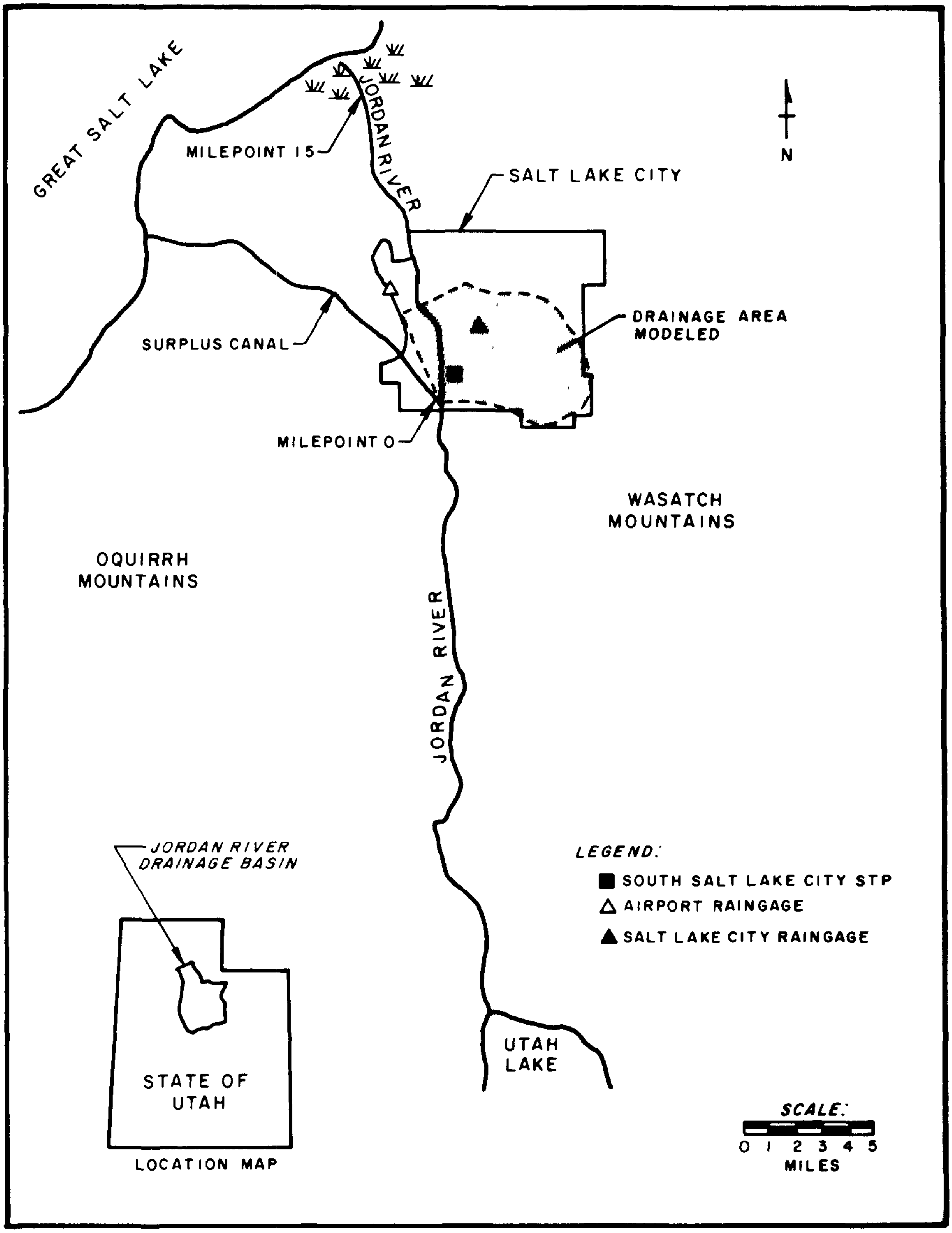


FIGURE 7-1
SALT LAKE CITY STUDY AREA

simplified manner by assuming that all stormwater runoff from Salt Lake City enters the Jordan River just downstream of the surplus canal. This example assessment demonstrates techniques useful for areas with separate or unsewered runoff, dissolved oxygen problems, and river or stream receiving water systems.

7.1.1 Rainfall Analysis

Hourly rainfall records for the Salt Lake City study area are generated by combining the records from the main station in Salt Lake City (Gage 427598) and the Airport (Gage 427603). Equal weights are used to produce a record from 1950 through 1965 which is essentially the average of the two individual gages (See Section 5.1.4.1). This procedure helps account for the areal variation of rainfall over the study area.

A synoptic rainfall analysis (using program SYNOP) is performed on the combined record to calculate the storm statistics. A minimum interevent time of 13 hours is found to yield a coefficient of variation of time between storms (ν_δ) nearly equal to 1.0 throughout most of the year and is thus chosen for event definition. The results are shown in Figure 7-2. Figure 7-2 is used to determine the appropriate rainfall statistics for the preliminary period of interest, July-September. These are shown in Table 7-1.

TABLE 7-1

SUMMER RAINFALL STATISTICS (JULY-SEPTEMBER)
SALT LAKE CITY

	Mean	Coefficient of Variation
Intensity	I = 0.030 in/hr	ν_i = 1.10
Duration	D = 6.5 hr	ν_d = 1.10
Unit Volume	V = 0.16 in	ν_v = 1.60
Time Between Storms	Δ = 175 hr	ν_δ = 1.10

The predominant characteristic of Salt Lake City summer rainfall is its infrequency. The mean time between storms (measured from storm midpoints), Δ = 175 hours (7.3 days) indicates that an average of 12.6 storms occur during the three month summer period.

7.1.2 Drainage Basin Characterization

Previous studies (1,2) have characterized the predominantly urbanized portion of Salt Lake City which drains into the Jordan River near, or downstream of, the surplus canal. The area, outlined in Figure 7-1, is estimated to have a total area of 13,830 acres (21.6 square miles) and an average ratio of runoff to rainfall, R_v = 0.46. The drainage basin is served entirely by separate or unsewered runoff conveyance. The population density in the area is approximately 6,000 persons/square mile.

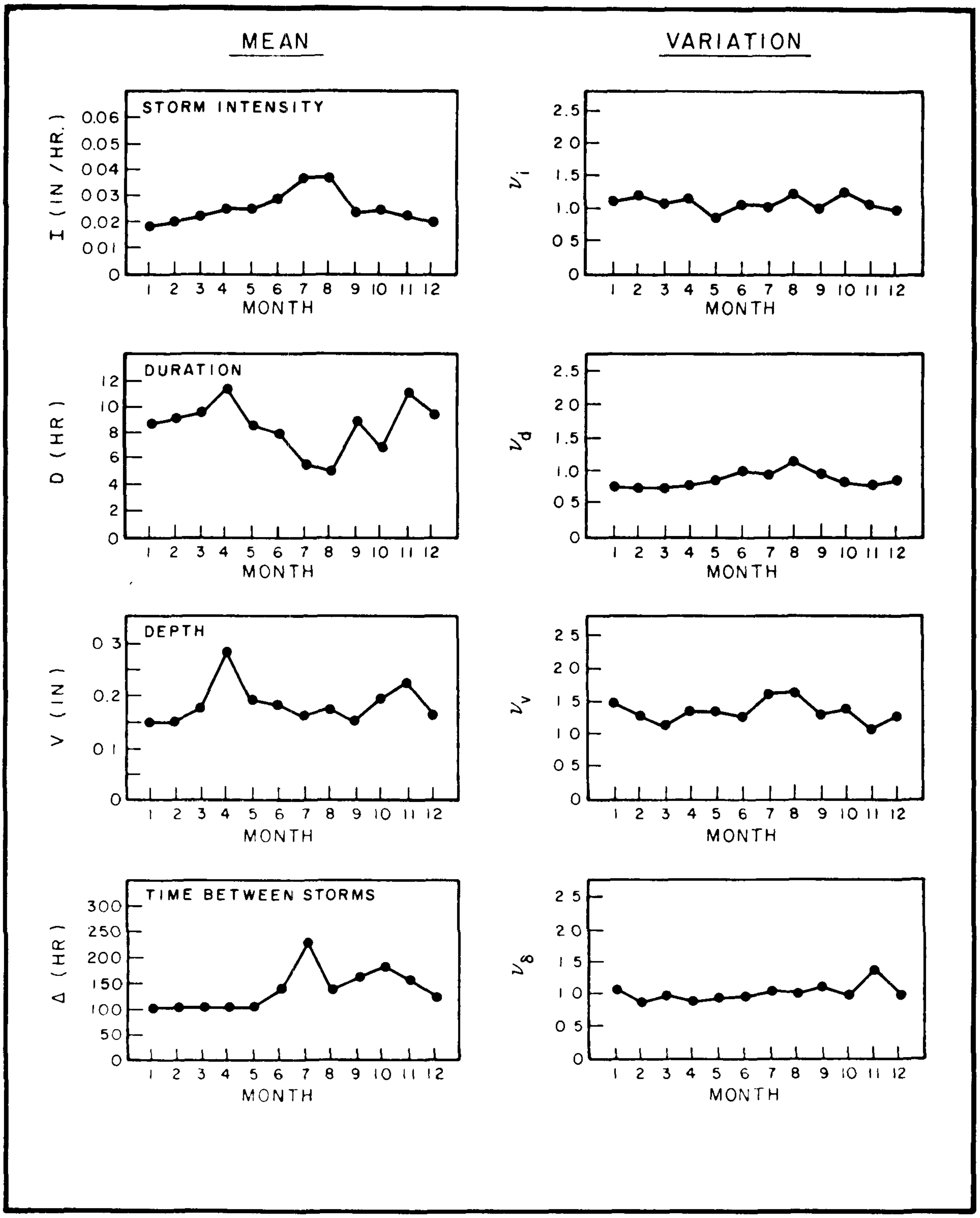


FIGURE 7-2

MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
SALT LAKE CITY
COMBINED RECORD, STATIONS 427598 AND 427603

7.1.3 Runoff Quantity

The mean runoff volume from the Salt Lake City area (V_R) is calculated from Equation 3-12 as follows:

$$V_R = 0.027\ R_V VA$$
$$= (0.027\ \text{MG/acre-in})(0.46)(0.16\ \text{in})\ (13{,}830\ \text{acre})$$
$$= 27.5\ \text{MG}$$

The mean runoff flow (Q_R) is estimated from Equation 3-13:

$$Q_R = R_V IA(D/D_R)$$

All of the variables in this equation are known at this point, except D_R, the mean duration of runoff events. Using the estimating procedure outlined in Section 5.2.2, D_R is determined as follows:

D = mean rainfall event duration = 6.5 hr.

V_d = estimated initial abstract = 0.10 inches

V_d/DI = 0.10 in/(6.5 hr x 0.030 in/hr) = 0.51

D_e/D = 0.45 (from Figure 5-23, reproduced as Figure 7-3)

D_e = 0.45 x 6.5 hr = 2.9 ≈ 3.0 hr.

PD = population density = 6,000 people/mi^2

W_{25} = width of unit hydrograph at 25 percent of peak

= 7 hr (from Figure 5-24, reproduced as Figure 7-4)

D_R = $D_e + W_{25}$ = 3.0 + 7.0 = 10.0 hr

Therefore, Q_R is estimated as:

$$Q_R = (0.46)(0.030\ \text{in/hr})(13830\ \text{acre})(6.5\ \text{hr}/10.0\ \text{hr}) = 124\ \text{cfs}$$

Note that the correction factor for the time of travel flow attenuation (D/D_R) is equal to 0.65. The flow rates estimated to occur from different size storms are tabulated in conjunction with the runoff loading rates in the following section, and are adopted for subsequent analysis because they are consistent with the general level of flow changes observed to occur in the Jordan River. The average summer contribution of runoff flow from the study area (including both rain and non-rain periods) is estimated as:

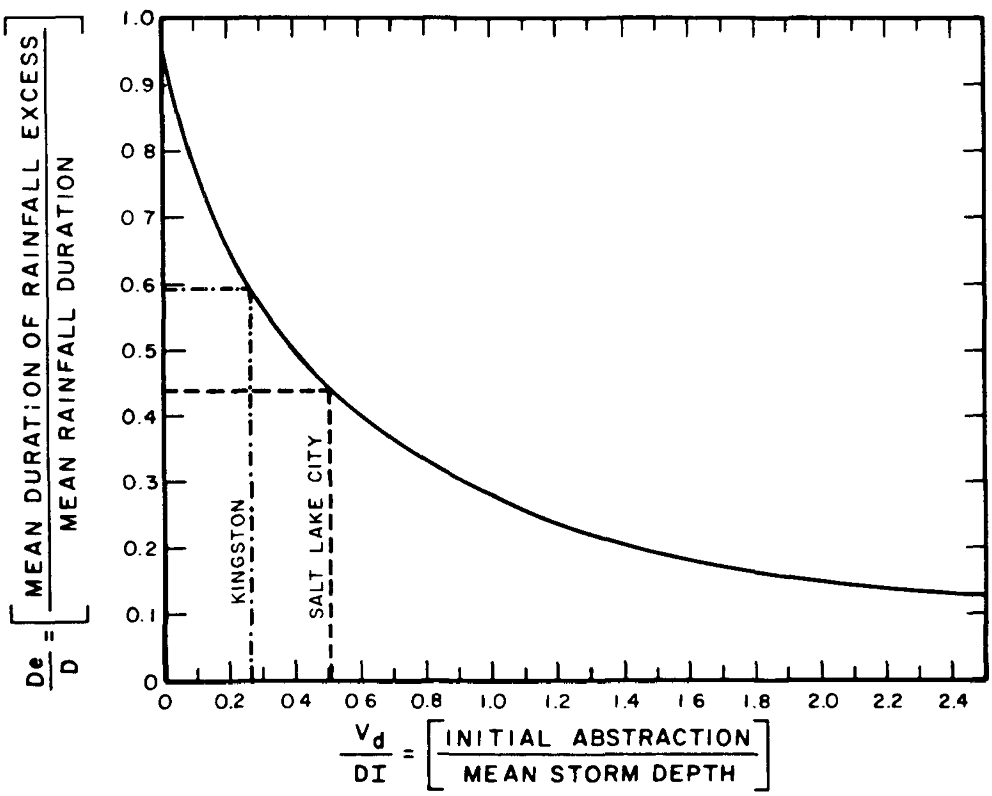
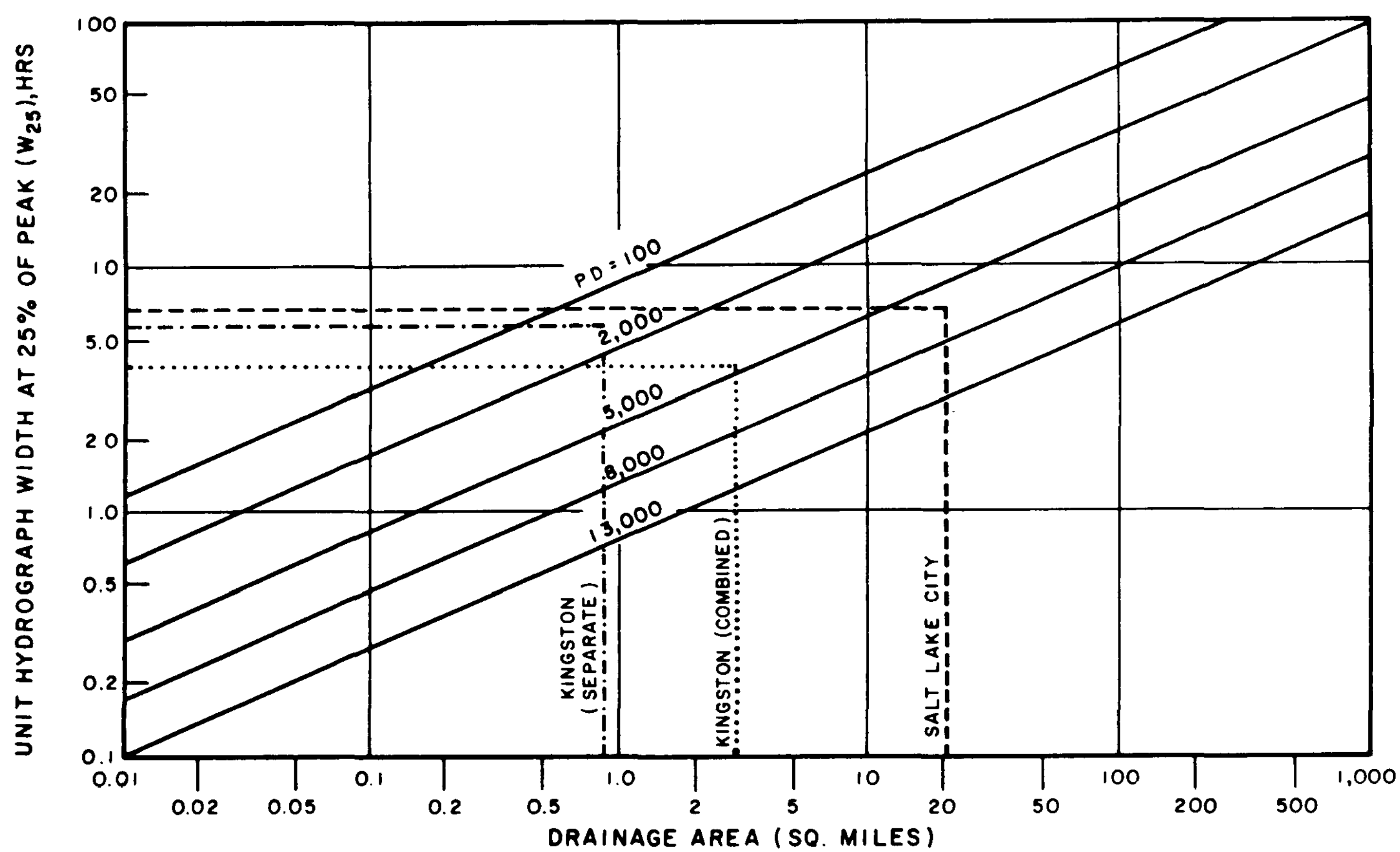


FIGURE 7-3

ESTIMATE OF MEAN DURATION OF RAINFALL EXCESS



FROM BRATER AND SHERRILL (30)

FIGURE 7-4
WIDTH AT 25 PERCENT OF PEAK VERSUS AREA-DESIGN CURVES

$$Q_o = Q_R(D_R/\Delta)$$

$$= 124 \text{ cfs } (10.0 \text{ hr}/175 \text{ hr})$$

$$= 7.1 \text{ cfs}$$

7.1.4 Runoff Pollutant Loads

To determine wet weather organic pollutant loading rates from the Salt Lake City area, the following estimates (1) of average runoff concentrations are used:

The carbonaceous biochemical oxygen demand,

$$CBOD_5 = 40 \text{ mg/l}$$

The total kjedahl nitrogen,

$$TKN = 2.0 \text{ mg/l}$$

The $CBOD_5$ is converted to an ultimate oxygen demand by assuming a factor of 1.5:

$$CBOD_u = 1.5 \ (40 \text{ mg/l})$$

$$= 60 \text{ mg/l}$$

Assuming all of the TKN is oxidizable in the Jordan River, and using the stochiometric factor of 4.57:

$$NBOD_u = 4.57 \ (2.0 \text{ mg/l})$$

$$= 9.1 \text{ mg/l}$$

The mean runoff loading rate (during storms) is calculated as:

$$W_R = 5.4 \ \bar{c} \ Q_R$$

For $CBOD_u$: $W_R = 5.4(\text{lb/day})/(\text{cfs-mg/l}) \ (60 \text{ mg/l})(124 \text{ cfs})$

$$= 40{,}200 \text{ lb/day}$$

For $NBOD_u$: $W_R = 5.4(\text{lb/day})/(\text{cfs-mg/l}) \ (9.1 \text{ mg/l})(124 \text{ cfs})$

$$= 6{,}100 \text{ lb/day}$$

The average summer loading rate (including rain and non-rain periods) is estimated as:

$$W_o = W_R(D_R/\Delta)$$

or $CBOD_u$: W_o = 40,200 lb/day (10.0 hr/175 hr)

= 2,300 lb/day

For $NBOD_u$: W_o = 6,100 lb/day (10.0 hr/175 hr)

= 350 lb/day

These values may be used to estimate the total seasonal load.

The loading rate which occurs during storms of various frequencies is estimated by assuming $\nu_o = \nu_i = 1.10$, and that runoff concentrations are constant. Figure 7-5 is then used to estimate the factor by which the mean flow or loading rate is multiplied to estimate the flow or loading rate of storms with different likelihoods of occurance. The result is tabulated in Table 7-2.

An additional pollutant load considered in this assessment is that generated by the South Salt Lake City municipal treatment plant. The organic load from the plant is estimated as (1):

$CBOD_u$ = 1900 lb/day

$NBOD_u$ = 2000 lb/day

Note that the long term average stormwater loading rate in the summer (W_o) is nearly equal to the treatment plant load in the case of CBOD, and about 6 times smaller than the treatment plant load in the case of NBOD. However, the intermittent nature of the stormwater loads, which occur only about 6 percent of the time (D_R/Δ = 10/175 x 100% = 5.7%), results in the runoff loading rates being much higher than the municipal loading rate during storms. The impact of these loads on the dissolved oxygen concentration of the Jordan River is now evaluated.

7.1.5 Receiving Water Response

The critical step in the evaluation of stormwater problems is the assessment of impacts in the receiving stream. A simplified model of the Jordan River is used to estimate the dissolved oxygen response during wet weather periods. A one segment model with uniform geometry, flow, and reaction kinetics is applied. The municipal treatment plant discharge and all of the stormwater runoff are assumed to enter the river segment at the upstream end. The resulting dissolved oxygen deficit is due to the carbonaceous and nitrogenous biochemical oxygen demand loads in the municipal discharge, the stormwater runoff, and that transported into the river segment from the upstream boundary condition; as well as the remaining portions of the initial dissolved oxygen deficit present in the stormwater and the incoming river flow. As outlined in Section 3.5.2.1, the Streeter-Phelps stream solution is determined and modified for dispersion to estimate the peak dissolved oxygen deficit which passes a location following a storm.

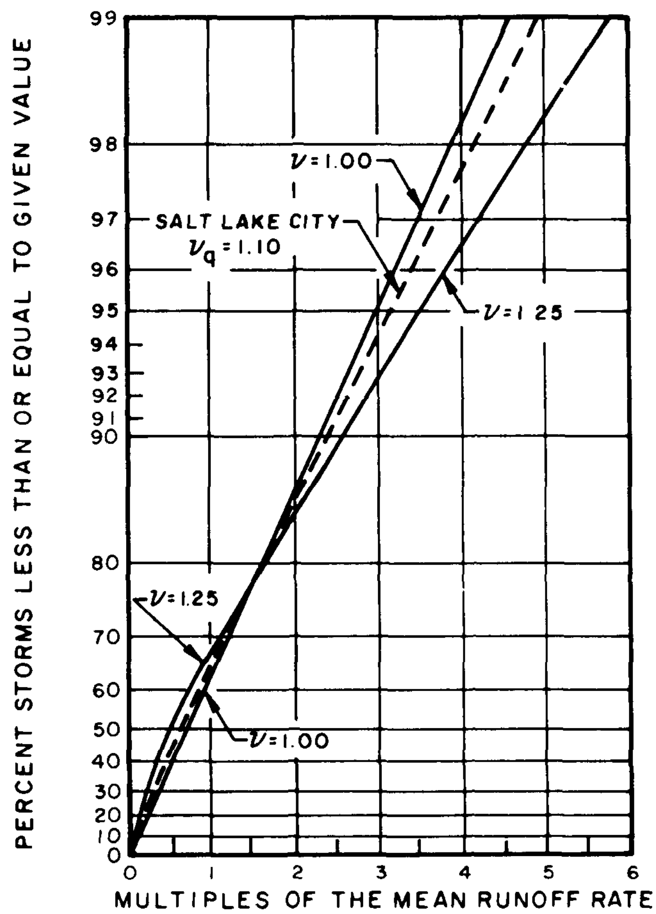


FIGURE 7-5

CUMULATIVE FREQUENCY FUNCTION OF RUNOFF RATES
SALT LAKE CITY SUMMER STORMS

TABLE 7-2

STORMWATER FLOWS AND LOADS GENERATED BY SUMMER STORMS

-SALT LAKE CITY-

Percent of Storms Less Than Given Runoff Rate	Multiples of the Mean	Expected Number Storms Per Summer Greater Than Given Runoff Rate	Runoff Flow During Storm (cfs)	Runoff Loading Rate During Storm $CBOD_u$ (lb/day)	$NBOD_u$ (lb/day)
50	0.60	6.3	74	24,100	3,650
64	1.00	4.5	124	40,200	6,100
80	1.60	2.5	198	64,300	9,750
90	2.40	1.3	298	96,500	14,650

Parameters and coefficients used in the analysis are taken from the recent study of the Jordan River (1). The flow entering the segment of interest in the Jordan River is regulated by the surplus canal to an average value of about 150 cfs during the summer. The total wet weather stream flow (Q_T, cfs) is thus 150 cfs plus the runoff flow. The river depth (H, ft) and velocity (U, ft/sec) are estimated from the flow (Section 5.4.1.1) as follows:

$$H = 0.25\ (Q_T)^{0.5}$$

$$U = 0.20\ (Q_T)^{0.4}$$

The water temperature is 20°C and the oxygen reaeration rate (K_a, per day) is estimated using the O'Connor-Dobbins equation:

$$K_a = 12.96\ U^{1/2}/H^{3/2}$$

The reaction rate for CBOD is $K_r = 0.30$ per day, the CBOD oxidation rate is $K_d = 0.25$ per day (a little smaller than K_r to account for some settling of CBOD without oxidation), and the reaction rate and oxidation rate of NBOD is $K_n = 0.15$ per day.

The longitudinal dispersion coefficient (E, mi^2/day) is derived from Equations 5-14 and 5-15, and a knowledge of the average channel slope, s = 0.0004:

$$E = \frac{0.0605\ Q_T^2}{U^{3/2}\ H^{11/4}}$$

The Jordan River geometry, transport, and reaction parameters are calculated and tabulated for each storm size in Table 7-3.

To select a particular location in the River for analysis, the location of the critical deficit (X_c, miles) is calculated:

$$X_c = \frac{U}{K_r - K_a}\ \ln\left(\frac{K_r}{K_a}\right)$$

The value calculated for X_c ranges from 28 miles for the 50th percentile storm to 48 miles for the 90th percentile storm. Since the length of the River before the confluence with the Great Salt Lake is only 17 miles, it is clear that the dissolved oxygen deficit is continuing to increase as the River flows into the Lake. A location 2 miles from the mouth of the Jordan River (X = 15 miles) is thus chosen for the analysis, to avoid the particularly marshy area near the Lake.

The final factor required for the dissolved oxygen deficit calculations is the reduction in the peak concentration due to dispersion. This is determined from Figure 3-11 (reproduced as Figure 7-6) and the dimensionless factor:

TABLE 7-3

JORDAN RIVER GEOMETRY, TRANSPORT, AND REACTION RATES

DURING SUMMER STORMS

Percent of Storms Less Than Given Runoff Rate	Expected Number Storms Per Summer Greater Than Given Runoff Rate	Q_T (cfs)	H (ft)	U (ft/sec)	U (mi/day)	E (mi^2/day)	K_a (1/day)
50	6.3	224	3.7	1.74	28.5	0.11	2.40
64 (Mean)	4.5	274	4.1	1.89	30.9	0.11	2.15
80	2.5	348	4.7	2.08	34.0	0.11	1.83
90	1.3	448	5.3	2.30	37.6	0.11	1.61

For all conditions, Temperature = 20°C

K_r = 0.30 per day K_d = 0.25 per day

K_n = 0.15 per day

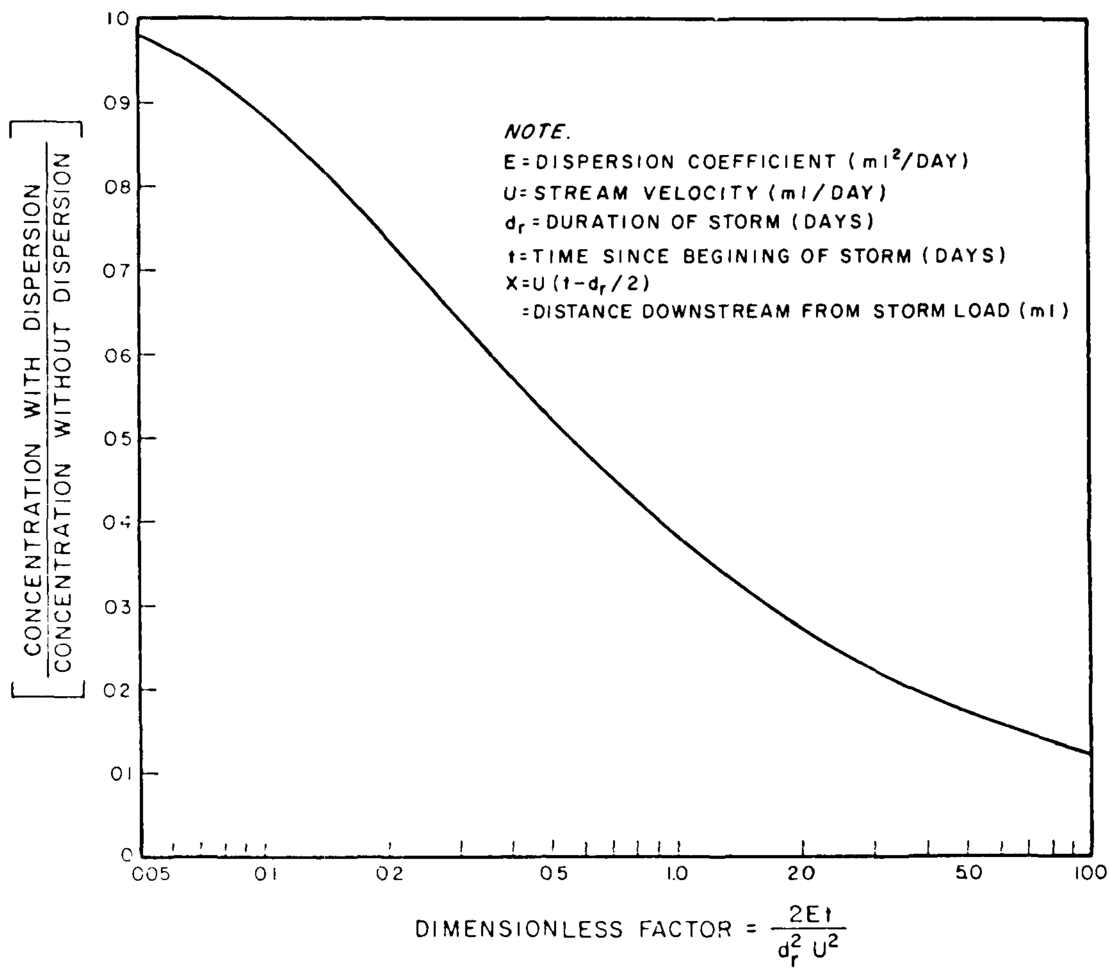


FIGURE 7-6
EFFECT OF DISPERSION ON POLLUTANT
CONCENTRATION AT MIDPOINT OF STORM PULSE

$$\alpha = \frac{2Et}{d_r^2\ U^2}$$

The time of passage of the pulse midpoint to X = 15 miles is (from the beginning of the storm):

$$t = \frac{15}{U} + \frac{d_r}{2}$$

Assuming a 10 hour runoff event ($d_r = 10$), t ranges from 0.61 days (90th percentile storm) to 0.74 days (50th percentile storm). The resulting values of α range from 1×10^{-6} to 2×10^{-6}, all well below the beginning of the abscissa in Figure 7-6.

Even if the value of α were 1000 times greater than calculated, the reduction of the peak deficit due to dispersion would remain negligible. The peak dissolved oxygen deficit calculated from the Streeter-Phelps equation is thus used without modification to estimate the post-storm deficits at X = 15 miles.

The loads generated by stormwater runoff are tabulated in Table 7-2, and the municipal loads are as previously identified. The load transported into the River segment from the upstream boundary is calculated by assigning the boundary concentrations as:

$CBOD_u$ = 13 mg/l

$NBOD_u$ = 9.1 mg/l

Given the upstream flow of 150 cfs, the boundary loads are:

$CBOD_u$, W = 10,500 lb/day

$NBOD_u$, W = 7,400 lb/day

The initial dissolved oxygen deficit (D_o) in both the upstream flow and the stormwater runoff is estimated as 2 mg/l. The saturation value of dissolved oxygen (C_s) is estimated from Figure 5-35 as 9.0 mg/l. (For purposes of example: Jordan River, at over 4000 ft. elevation in fact has a reduced saturation value of approximately 7.5 mg/l D.O. at 20°C.) The portion of the initial deficit remaining at any downstream location is calculated as:

$$D = D_o \exp(-K_a X/U)$$

The deficit caused by each load at any downstream location is:

$$D = \frac{W}{5.4\ Q_T} \quad \frac{K_d}{K_a - K_r} \quad [\exp(-K_r X/U) - \exp(-K_a X/U)]$$

where $K_d = K_r = K_n$ = 0.15 per day for NBOD loads. The total dissolved oxygen deficit (D_T) is the sum of all the component deficits. The resulting dissolved oxygen concentration (DO) is simply:

$DO = C_s - D_T$

The estimated component deficits and resulting dissolved oxygen concentrations following summer storms are calculated at milepoint 15 of the Jordan River and are tabulated in Table 7-4 and shown graphically in Figure 7-7. A recommended (1) dissolved oxygen guideline of 5.5 mg/l for this portion of the River is also indicated. The dissolved oxygen concentration is seen to fall below the guideline following about 40 percent of the storms (5 events per summer). The 90th percentile storm results in about a 1 mg/l violation at milepoint 15.

The components of the dissolved oxygen deficit are also shown in Figure 7-7. The upstream boundary loads and the remains of the initial deficit are considered basically uncontrollable for the current analysis and are grouped together. As indicated in Table 7-4, the decrease in the upstream boundary load impact associated with larger storms (due to the dilution by the larger storm flows) is balanced by the larger remaining portion of the initial deficit during the large storms (due to the shorter time of travel and smaller K_a during the large storms); and the uncontrollable portion of the deficit remains relatively constant. The impact of the South Salt Lake City treatment plant is shown to be quite small, particularly during large storms. The dissolved oxygen deficit resulting from the organic pollutant load in the stormwater runoff is the principal factor, increasing from about one half the total deficit during the 50th percentile storm, to about two thirds the total deficit during the 90th percentile storm. This preliminary assessment of the Jordan River indicates that stormwater loadings may in fact result in small or moderate dissolved oxygen problems during large summer storms.

The resulting dissolved oxygen deficit (at milepoint 15) is shown to increase with larger (less frequent) storm runoff rates. However, caution should be exercised before concluding that the deficit associated with the 90th percentile runoff rate is not exceeded by 90 percent of the summer storms. This is true only if the resulting deficit increases monotonically with increasing runoff rates. If larger runoff rates are calculated to result in less deficit, the direct mapping of probability no longer applies. Situations where this may occur include:

1. Where the decrease in the dissolved oxygen deficit associated with dilution of continuous sources (municipal treatment plant loads, background BOD, etc.) at higher runoff rates is greater than the deficit increase associated with the larger storm runoff load. There are in fact situations where larger storms improve the pollution problem. This usually corresponds to areas where dry weather, continuous problems are more severe than wet weather impacts.

2. Where the time of travel decrease associated with larger storms does not allow the dissolved oxygen sag sufficient time to develop. This is not a problem for first order pollutants (e.g., coliforms) where the maximum concentration occurs at the point of discharge. These problems can be checked by calculating the impact for a wide range of runoff rate frequencies. When the runoff rate-stream

TABLE 7-4

WET WEATHER DISSOLVED OXYGEN RESPONSE

OF JORDAN RIVER TO SUMMER STORMS

(mg/l)

Percent of storms Less Than Given Runoff Rate	Expected Number of Storms Per Summer Greater Than Given Runoff Rate	Dissolved Oxygen Deficit From							Total Deficit D_T	Dissolved Oxygen (DO)
		Municipal Discharge		Stormwater Runoff		Upstream Boundary Load				
		C	N	C	N	C	N	Initial Deficit		
50	6.3	0.11	0.07	1.35	0.13	0.59	0.26	0.57	3.1	5.9
64 (Mean)	4.5	0.09	0.06	1.88	0.18	0.49	0.22	0.70	3.6	5.4
80	2.5	0.07	0.05	2.40	0.23	0.39	0.17	0.89	4.2	4.8
90	1.3	0.05	0.04	2.75	0.26	0.30	0.13	1.05	4.6	4.4

Note: C indicates the deficit caused by the CBOD load

N indicates the deficit caused by the NBOD load

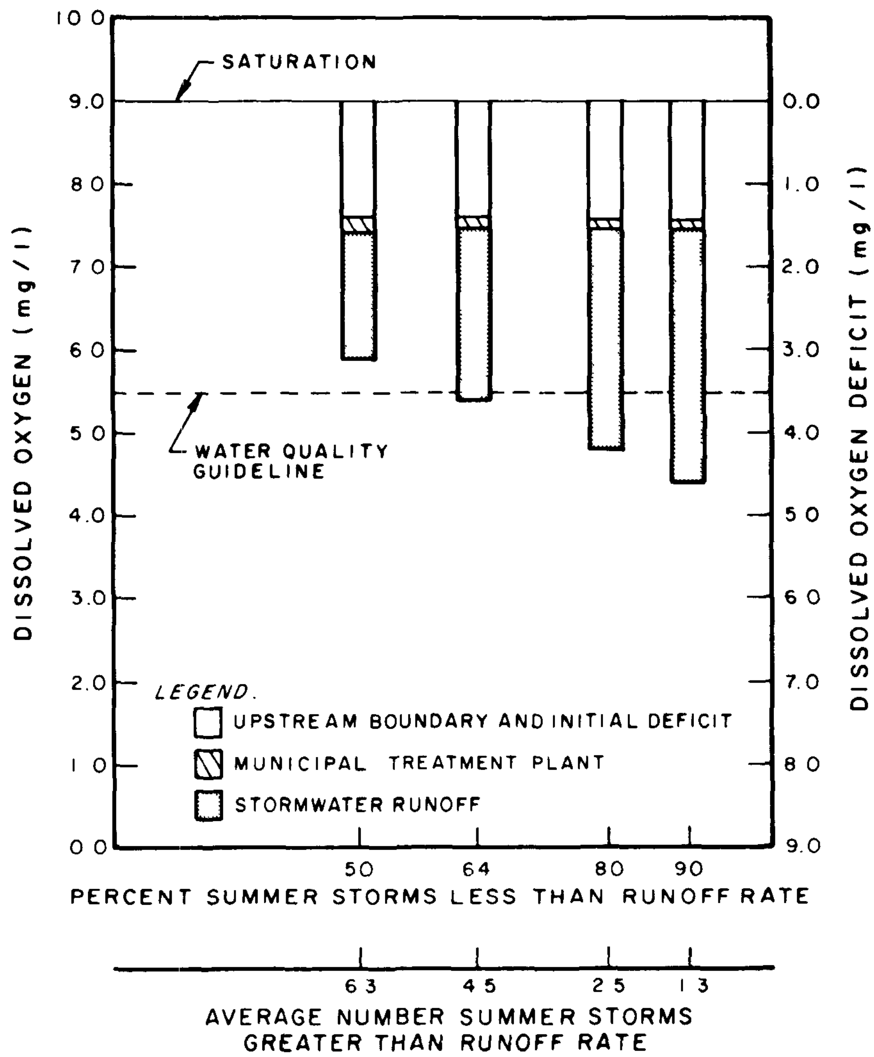


FIGURE 7-7

MINIMUM DISSOLVED OXYGEN AT MILEPOINT 15 OF JORDAN RIVER FOLLOWING SUMMER STORMS concentration relationship deviates significantly from the monotonic assumption, long term simulation is required for a complete probabilistic assessment of wet weather stream impacts.

7.1.6 Stormwater Control Alternatives

A variety of control alternatives are available to reduce the wet weather impacts identified. Two possibilities: storage and management practices, are briefly analyzed in this section.

Storage for stormwater runoff could be provided in the Salt Lake City area with a series of detention ponds, lagoons, or by utilization of existing irrigation canals. The storage performance may be estimated using the procedure outlined in Section 3.6.1.2. Assuming the total capacity of the retention system is V_B = 40 MG and the average release rate from storage is Ω = 25 MGD, the calculations proceed as follows:

$$V_B/V_R = 40 \text{ MG}/27.5 \text{ MG}$$

$$= 1.45$$

$$\Omega\Delta/V_R = (25 \text{ MGD})(7.3 \text{ days})/27.5 \text{ MG}$$

$$= 6.6$$

V_E/V_R = 1.40 (Figure 3-17, reproduced as Figure 7-8)

f_V = 0.40 (From Figure 3-18, reproduced as Figure 7-9. Note that $\nu_{VR} = \nu_V = 1.60$)

Assuming the presence of a small to moderate first flush, the storage performance is improved such that,

f_V = 0.35 (From Figure 3-19, reproduced as Figure 7-10)

The storage system is thus estimated to capture about 65 percent of the stormwater load. Furthermore, given the coefficient of variation of storm volumes ($\nu_{VR} = \nu_V = 1.60$), the fraction of storms totally captured may be estimated. As indicated in Figure 7-11, 79 percent of the summer storms are completely captured. Therefore, only an expected 2.6 storms per summer have any direct overflow. To develop a modified version of Figure 7-7, indicating the expected dissolved oxygen condition in the Jordan River after the implementation of storage treatment, more detailed simulation analyses incorporating storage, overflow, treatment in the basins, release from storage, etc., are appropriate.

The cost of the retention system is estimated by assuming that four 10 MG earthen basins are used to provide the storage capacity. Figure 5-56 (reproduced as Figure 7-12) estimates the cost of each basin at about $200,000 (June, 1975 costs). The annual operation and maintenance cost is estimated to be similar to the covered basin, and Figure 5-57 (reproduced as Figure 7-13) is used. The hours of operation per year are estimated as:

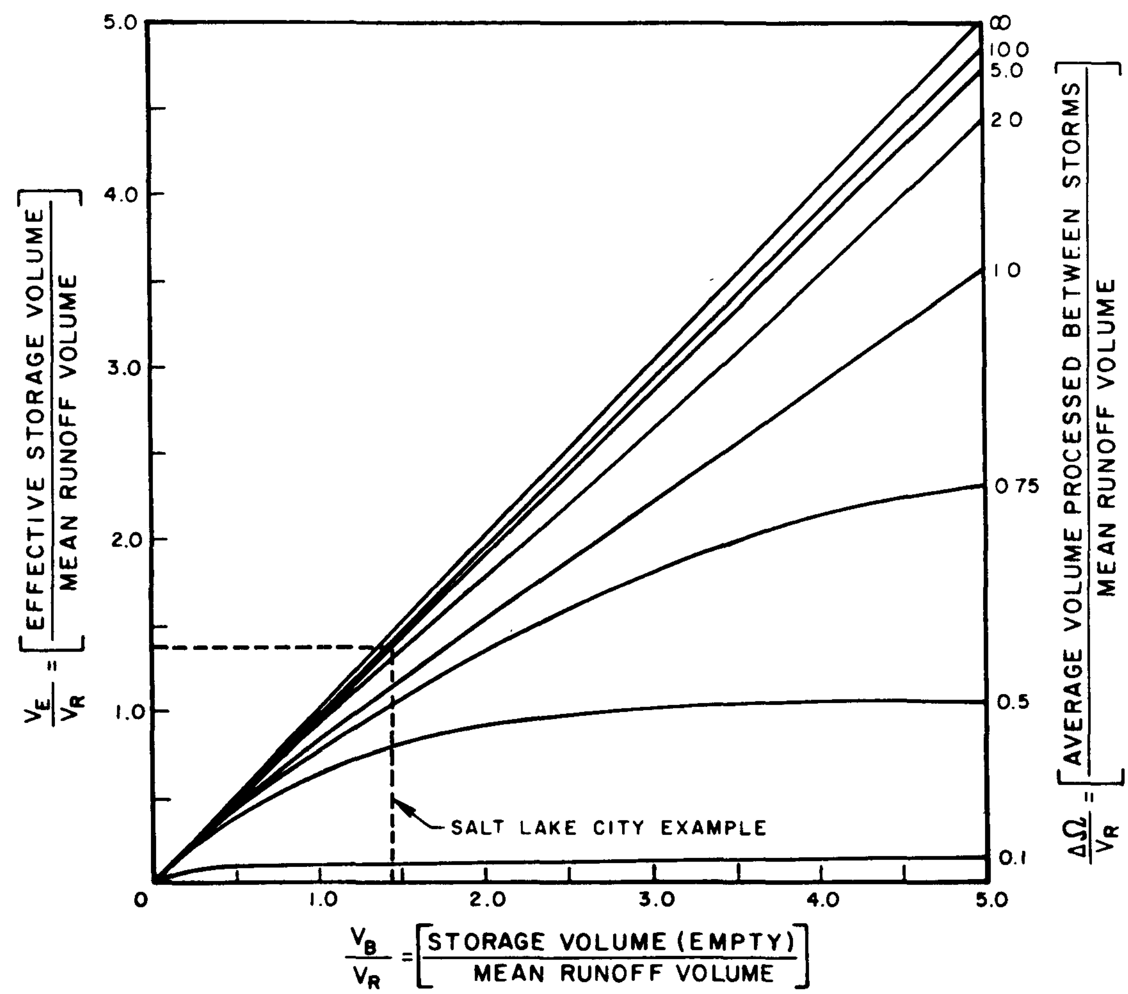


FIGURE 7-8

EFFECT OF PREVIOUS STORMS
ON LONG TERM EFFECTIVE STORAGE CAPACITY

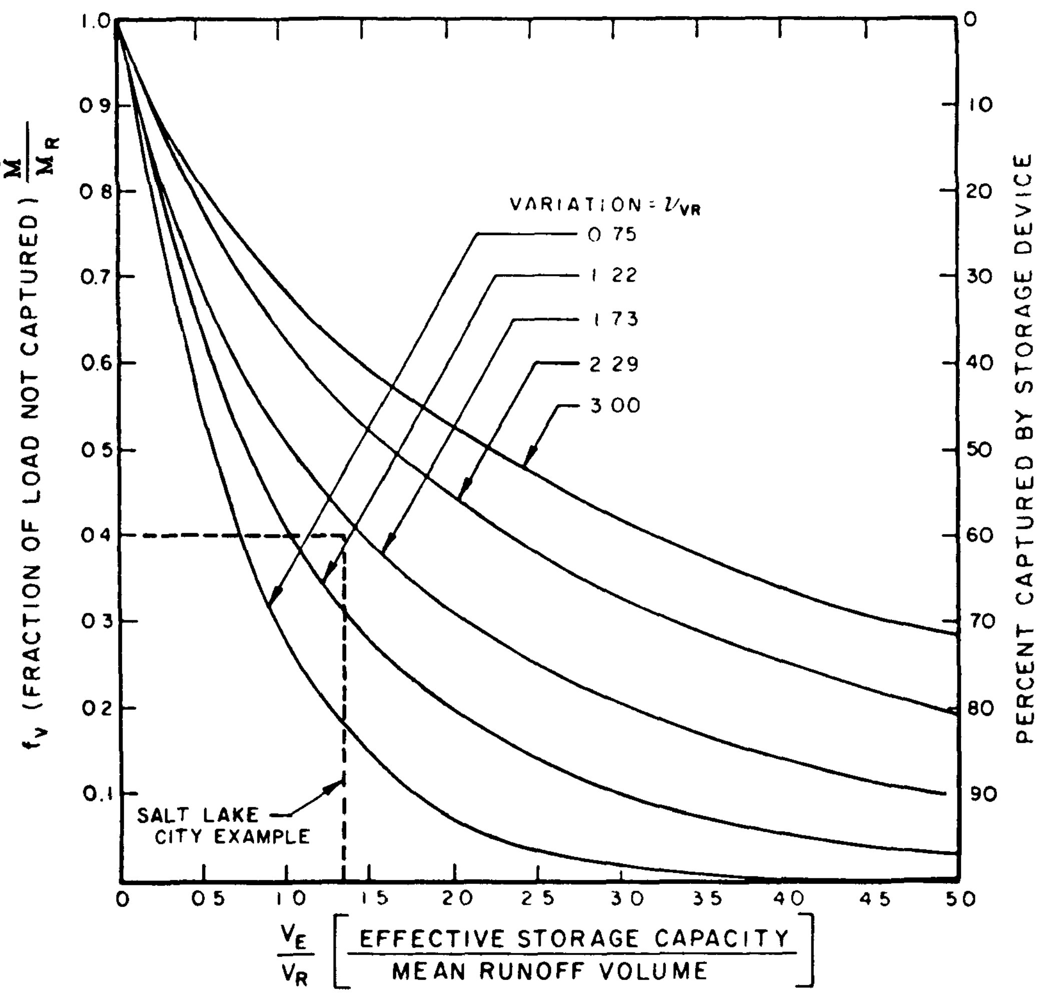


FIGURE 7-9
DETERMINATION OF LONG TERM STORAGE DEVICE PERFORMANCE

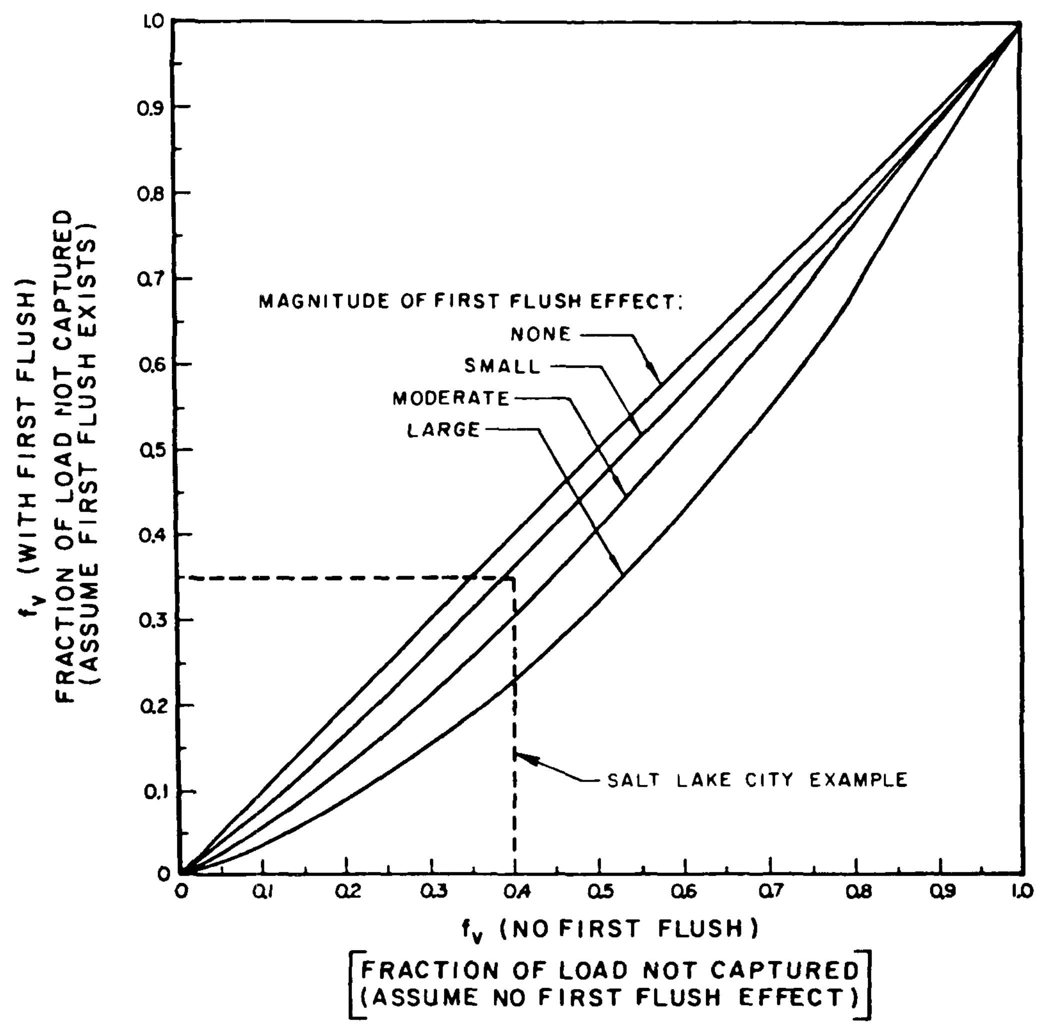


FIGURE 7-10
IMPROVEMENT IN LONG TERM STORAGE DEVICE PERFOMANCE DUE TO FIRST FLUSH EFFECT

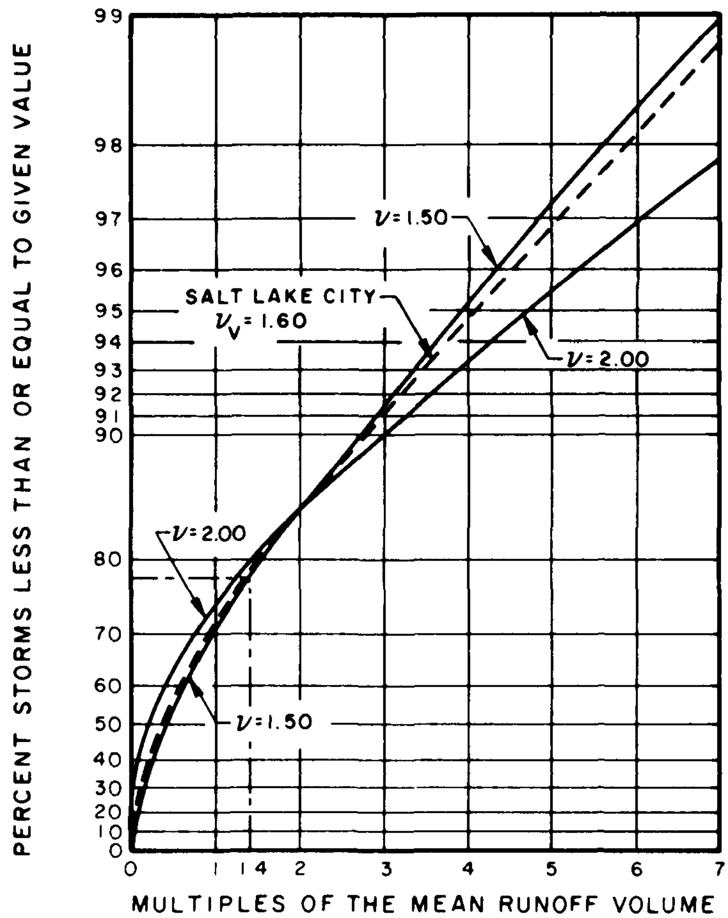


FIGURE 7-11

CUMULATIVE FREQUENCY FUNCTION OF RUNOFF VOLUMES
SALT LAKE CITY SUMMER STORMS

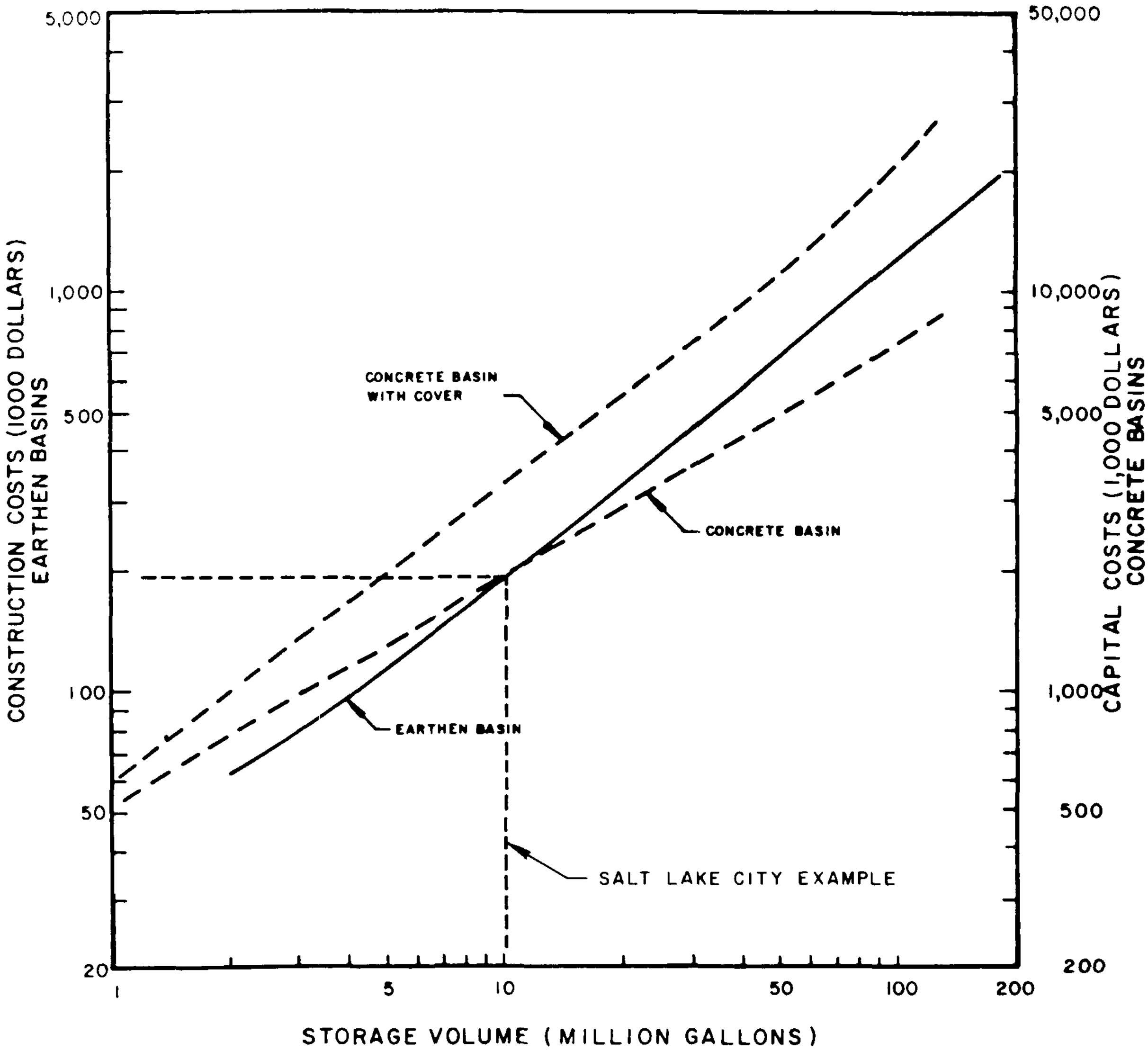


FIGURE 7-12

CAPITAL COST - STORAGE BASINS

(JUNE 1975 COSTS)

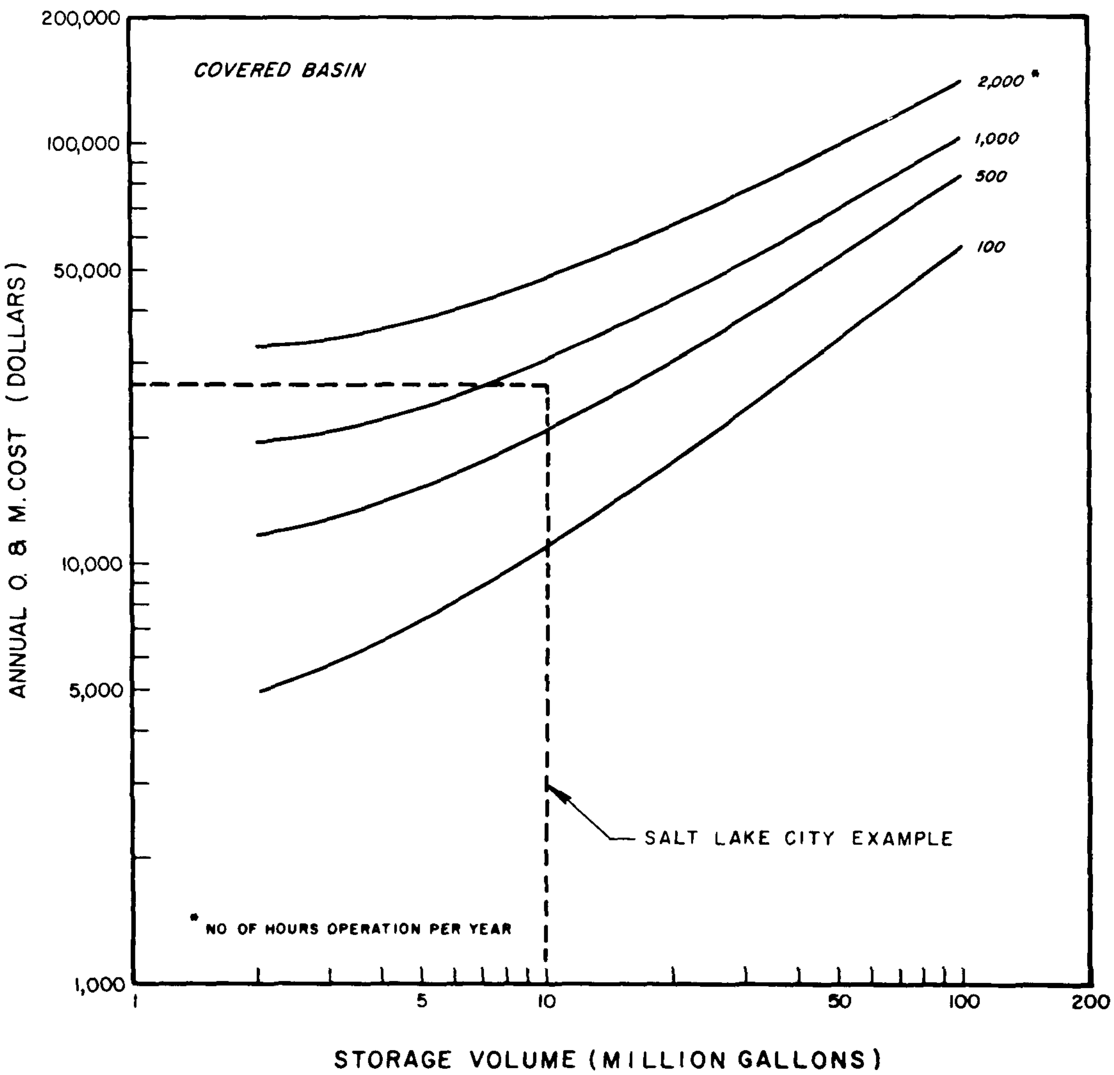


FIGURE 7-13
ANNUAL OPERATION AND MAINTENANCE COST — STORAGE BASINS
(JUNE 1975 COSTS)

$$\text{Hours of operation per year} = \left(\frac{D_R}{\Delta}\right) 8766 \text{ hr/year}$$

where D_R and Δ are chosen to be representative of yearly averages (from Figure 7-2).

$$\text{Hours of operation per year} = \frac{12 \text{ hr}}{130 \text{ hr}} (8766 \text{ hr/year})$$

$$= 810 \text{ hr/year}$$

The annual operation and maintenance cost for each basin is thus estimated as $25,000 (June, 1975 cost). The total cost of the storage system (June, 1975 cost) is therefore:

Capital Cost: $800,000

Operation and Maintenance: $100,000/year

These are rough, order of magnitude estimates, intended only to provide perspective on the stormwater treatment issue.

An alternative to the storage system analyzed is the implementation of management practices to reduce the pollutant concentration of runoff from the Salt Lake City area. These practices could include erosion control, localized detention through landscaping or small ponds, modified street sweeping, etc. If a 35 percent reduction in the CBOD and NBOD concentrations is achieved, the dissolved oxygen concentration following summer storms is improved as indicated in Table 7-5.

TABLE 7-5

WET WEATHER DISSOLVED OXYGEN
AFTER IMPLEMENTATION OF MANAGEMENT PRACTICES

Percent of Storms Less Than Given Runoff Rate	Expected Number of Storms Per Summer Greater Than Given Runoff Rate	DO (mg/l)
50	6.3	6.4
64	4.5	6.1
80	2.5	5.7
90	1.3	5.5

The 90th percentile storm is seen to just meet the water quality guideline. A determination of the feasibility and cost of management practices requires a more detailed evaluation of local conditions and current practices in the Salt Lake City area.

The initial assessment of wet weather impacts in the Jordan River indicates the possible presence of a small but potentially manageable stormwater related dissolved oxygen problem near the River mouth. All available water quality records for the lower Jordan River should be reviewed to determine whether available data is generally consistent with the assessment. The analysis which has been performed will help to focus attention on the most appropriate station to examine. A complete preliminary assessment would perform a similar estimate of receiving water impact for other contaminants which may be significant, and perhaps for other seasons where this may be pertinent.

If the magnitude and frequency at which violations of standards are estimated to occur are judged to be significant and if control is judged to be feasible and justifiable both technically and economically, the preliminary assessment would normally be supplemented by more detailed studies. Such studies can be organized more efficiently based on the perspective gained from the results of the preliminary assessment.

7.2 Example Stormwater Analysis: Kingston, New York

Kingston, New York is a small city of approximately 28,000 people located on the Hudson River Estuary about midway between New York City and Albany. The City and its principal drainage and receiving water systems are shown in Figure 7-14. Runoff from the major portion of the city drains through a combined sewer system into Rondout Creek, a small tributary of the Hudson River. Separate runoff also drains north into the Esopus River or west directly into the Hudson.

The Hudson River and Rondout Creek are tidally influenced with about a three and one half foot range in stage from low to high tide. Bathing beaches are located at Kingston Point and Point Ewen, upstream and downstream respectively of the confluence of the Rondout and the Hudson. The analysis presented in this section is directed towards evaluating the effect of stormwater overflows on the sanitary quality of these beaches. The water quality constituent of interest is fecal coliform bacteria. The system is represented in a simplified manner by assuming that all of the combined stormwater load from the areas tributary to the Rondout enters the Hudson River at the Lighthouse indicated in Figure 7-14. The Hudson River is then modeled as a one-dimensional tidal river or estuary. This may overestimate the impact somewhat by negating reaction of the fecal coliform in the Rondout, however, a reasonable estimate appropriate for an initial problem assessment is expected. This example demonstrates techniques useful for combined sewer areas, coliform bacteria problems, and estuarine receiving water systems.

7.2.1 Rainfall Analysis

Hourly rainfall records for Kingston (U.S. Weather Bureau Gage 304424) from May 1948 through December 1972 were obtained and analyzed using the synoptic rainfall analysis program with a minimum dry interevent time of 4 hours. The resulting monthly rainfall statistics are shown in Figure 7-15. Because the summer months are of particular importance for the bathing beach analysis, the storm statistics for the period of June-September are estimated as follows:

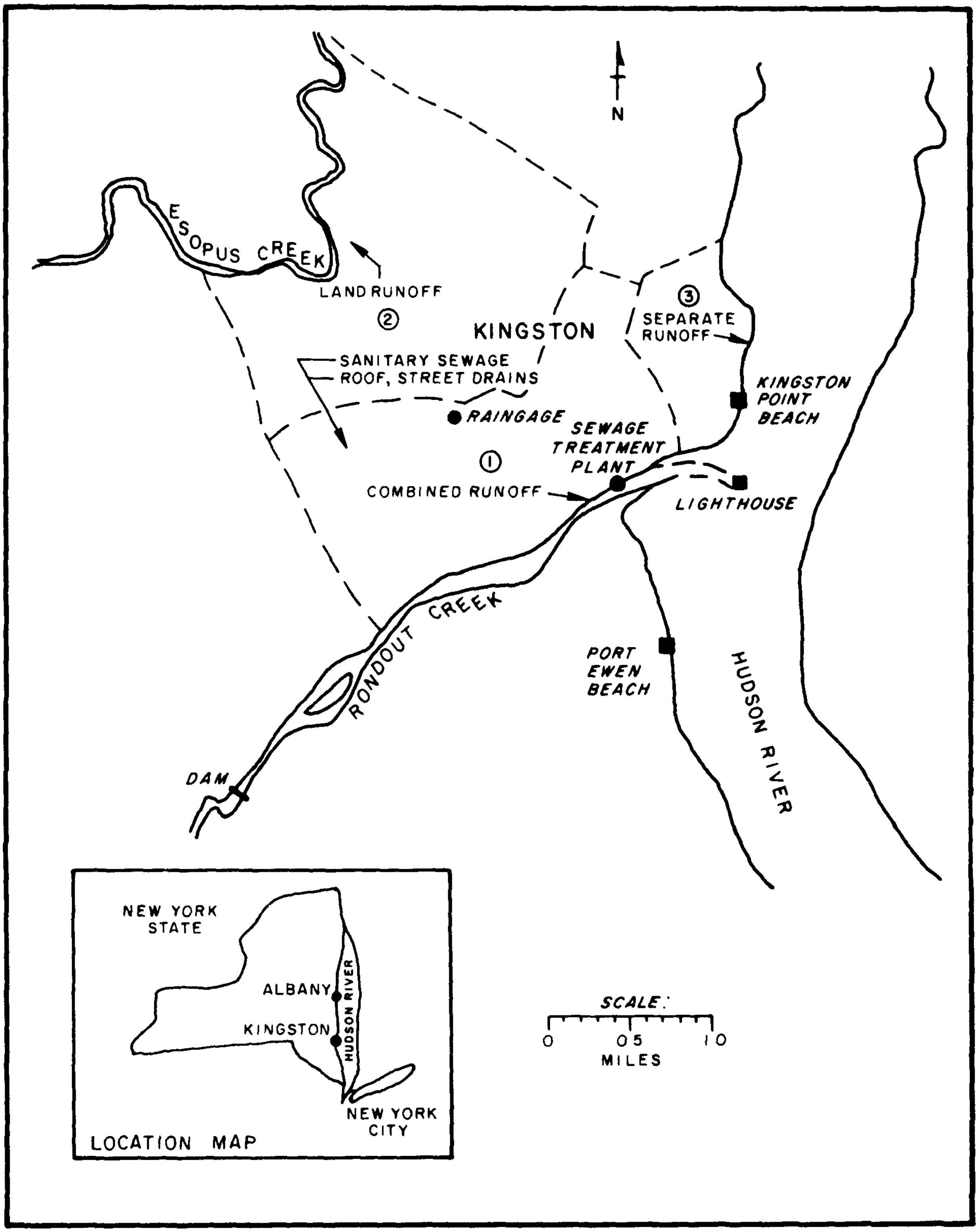


FIGURE 7-14
MAJOR FEATURES OF KINGSTON STUDY AREA

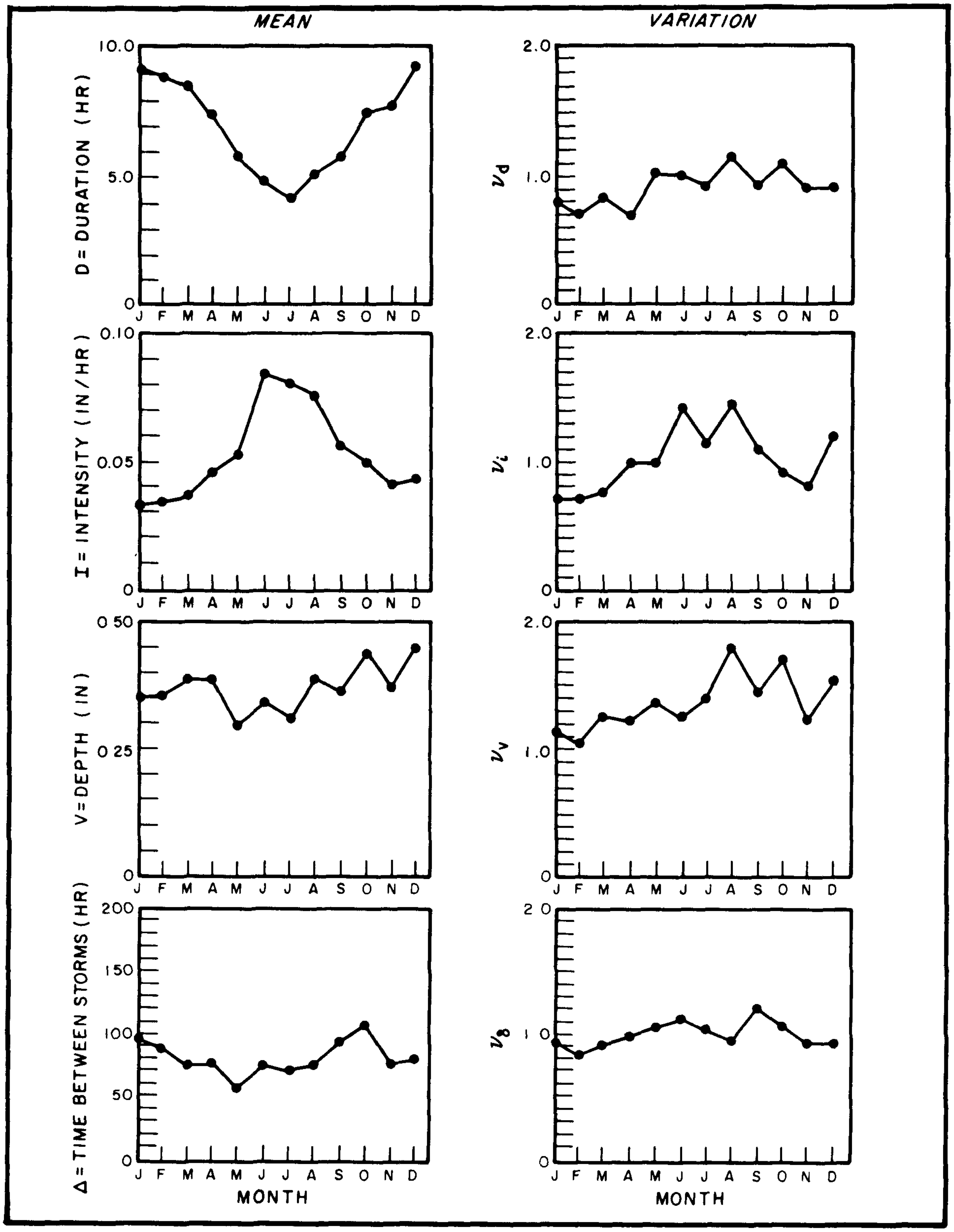


FIGURE 7-15

MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
KINGSTON, NEW YORK-STATION 304424

SUMMER STORM STATISTICS

KINGSTON, NEW YORK

	Mean	Coefficient of Variation
Intensity	$I = 0.075$ in/hr	$\nu_i = 1.3$
Duration	$D = 5.0$ hr.	$\nu_d = 1.1$
Unit Volume	$V = 0.35$ in	$\nu_v = 1.5$
Time Between Storms	$\Delta = 80$ hr.	$\nu_\delta = 1.1$

Because a relatively small drainage area is considered, analysis of a single gage is sufficient.

7.2.2 Drainage Basin Characterization

Three tributary drainage areas are considered for the current analysis. The first is the combined sewer area tributary to the Rondout. The portion of Kingston draining directly into the Rondout is 1950 acres. In addition, because sanitary sewage from the northern Esopus area is pumped to the Kingston treatment plant, and a considerable number of roof and street drains are connected to this system, an additional 5 percent of the northern area (5 percent of 2220 acres = 110 acres) is added to the combined Rondout tributary basin. The land use breakdown and the percent impervious assumed for each land use category are as follows:

KINGSTON COMBINED SEWER AREA LAND USE

Land Use	Percent of Area	Assumed Impervious Percent
Residential	70%	40%
Commercial	10%	80%
Open	20%	0%
		Net = 36%

The area weighted imperviousness is estimated as 36 percent. Figure 5-20 indicates a wide range of possible runoff to rainfall ratios for a 36 percent impervious basin, however, $R_V = 0.30$ is chosen as a typical value.

Because the area tributary to the Rondout is serviced by a combined sewer system and treatment plant, some of the system capacity is available for stormwater capture. The total capacity of the system is 10.2 MGD while the average dry weather flow is 4.5 MGD. This leaves 5.7 MGD ($Q_I = 10.2 - 4.5 = 5.7$ MGD) as the estimated available interceptor capacity.

The 510 acre area directly tributary to the Hudson River has 60 percent light industrial land use and 40 percent undeveloped land. Assuming the light industrial area is 25 percent impervious, the net overall imperviousness is 15 percent with an estimated runoff to rainfall ratio, $R_V = 0.10$. The entire 510 acre area has separate or natural drainage.

7.2.3 Determine Runoff Volumes

For the area triburary to the Rondout, the mean runoff volume (V_R) is calculated from Equation 3-12 as follows:

$$V_R = 0.027\ R_V\ VA$$

$$= (0.027\ \text{MG/acre-in})(0.30)(0.35\ \text{in})\ (2060\ \text{acre})$$

$$= 5.84\ \text{MG}$$

The mean runoff flow (Q_R) is estimated from Equation 3-13:

$$Q_R = R_V IA(D/D_R)$$

All of the variables in this equation are known at this point, except for D_R, the mean duration of runoff events. Using the estimating procedure outlined in Section 5.2.2, D_R is determined as follows:

D = mean rainfall event duration = 5.0 hr.

V_d = estimated initial abstraction = 0.10 inches

V_d/DI = 0.10 in/(5.0 hr x 0.075 in/hr) = 0.27

D_e/D = 0.6 (Figure 5-33, reproduced as Figure 7-3)

D_e = 0.6 x 5.0 hr = 3.0 hr = mean duration of excess rainfall

PD = population density = 27,630 people/(1950 + 2220 acre)

= 6.63 people/acre

= 4,250 people/mi^2

W_{25} = width of unit hydrograph at 25 percent of peak

= 4.0 hr (Figure 5-24, reproduced as Figure 7-4)

D_R = $D_e + W_{25}$ = 3.0 + 4.0 = 7.0 hr

Therefore, Q_R is estimated as:

Q_R = (0.30)(0.075 in/hr)(2060 acre) (5.0 hr/7.0 hr)

= 33 cfs

$= \quad 21$ MGD

Note that the factor for correcting for runoff attenuation ($D/D_R = 5/7$) does not result in a major change in Q_R.

For the area directly tributary to the Hudson the mean runoff volume is:

$$V_R = 0.027\ R_V\ VA$$

$$= (0.027\ \text{MG/acre-in})(0.10)(0.35\ \text{in})(510\ \text{acres})$$

$$= 0.48\ \text{MG}$$

To determine D_R, assume again that $V_d = 0.10$ inches, $D_e = 3.0$ hr, a low PD of 500 people/mi^2, $W_{25} = 6.5$ hr (Figure 7-4) and $D_R = 3.0 + 6.5 = 9.5$ hr. The mean runoff flow is thus:

$$Q_R = R_V IA(D/DR)$$

$$= (0.10)(0.075\ \text{in/hr})(510\ \text{acres})(5.0\ \text{hr}/9.5\ \text{hr})$$

$$= 2.0\ \text{cfs}$$

$$= 1.3\ \text{MGD}$$

7.2.3.1 Capture by Treatment Plant

For the combined sewer area tributary to the Rondout, a portion of the overflow is captured by the treatment plant. To estimate this portion, Figure 3-15 (reproduced as Figure 7-16) is used with:

$$Q_I/Q_R = 5.7\ \text{MGD}/21\ \text{MGD}$$

$$= 0.27$$

and $\nu_q = \nu_i = 1.3$

The estimated percent captured is thus 20 percent ($f_I = 0.80$), and the modified mean runoff volume to the Rondout is therefore:

$$V_R = 0.80\ (5.84\ \text{MG})$$

$$= 4.67\ \text{MG}$$

The runoff directly tributary to the Hudson is unmodified by capture.

7.2.4 Determine Stormwater Loads

To determine wet weather fecal coliform loadings from Kingston the following concentrations are assumed:

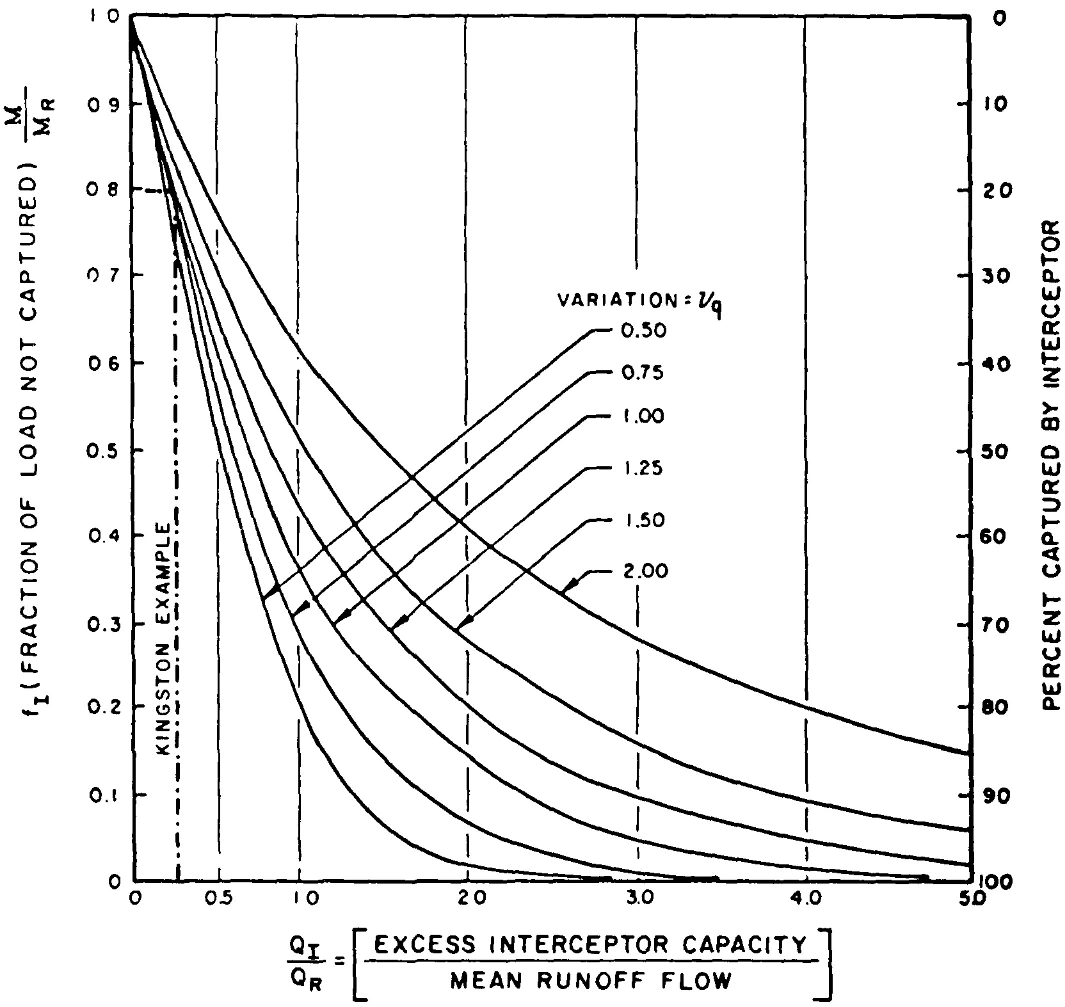


FIGURE 7-16
DETERMINATION OF LONG TERM INTERCEPTOR PERFORMANCE

Combined Overflow, $\bar{c}$ = 800,000 MPN/100 ml

Separate Runoff, $\bar{c}$ = 10,000 MPN/100 ml

Assuming 10^8 MPN/100 ml = 1 mg/1, the concentrations are converted to standard units, 8×10^{-5} and 1×10^{-4} mg/1 respectively. (The actual conversion value used is immaterial relative to the final receiving water concentration calculated, so long as the same conversion back to MPN/100 ml is used.) The mean runoff load from the combined sewer area tributary to the Rondout is calculated using Equation 3-19:

$$
\begin{aligned}
M_R &= 8.34\ \bar{c}\ V_R \\
&= (8.34\ \text{lb/MG - mg/1})(8 \times 10^{-3}\ \text{mg/1})\ (4.67\ \text{MG}) \\
&= 0.31\ \text{lb fecal coliform}
\end{aligned}
$$

For the separate Hudson area:

$$
\begin{aligned}
M_R &= 8.34\ \bar{c}\ V_R \\
&= (8.34\ \text{lb/MG - mg/1})(1 \times 10^{-4}\ \text{mg/1})(0.48\ \text{MG}) \\
&= 0.00040\ \text{lb fecal coliform}
\end{aligned}
$$

The variability of both loads are assumed equal to the variability of storm volumes:

$$\nu_m = \nu_v = 1.5$$

7.2.4.1 Receiving Water Response

The response of the Hudson River to the intermittent fecal coliform loadings from Kingston is modeled using the procedure outlined in Section 3.2.2.2. The characteristics of the Hudson River in the reach adjacent to Kingston are taken from a study of the Hudson (3):

a = cross-sectional area = 115,000 ft^2

H = average depth = 22 ft

E = dispersion coefficient = 1.0 mi^2/day

A typical flow in the Hudson during the summer months is:

Q = 9000 cfs

which indicates an average freshwater velocity:

$$
\begin{aligned}
U &= Q/a = 9000\ \text{cfs}/115{,}000\ \text{ft}^2 \\
&= 0.078\ \text{ft/sec} = 1.28\ \text{mi/day}
\end{aligned}
$$

The reaction rate for fecal coliform is estimated as:

$$k = 1.5/\text{day}$$

The mean summer fecal coliform concentration due to Kingston stormwater loads on the Hudson is calculated using Equation 3-30:

$$\bar{c} = \frac{M_R/}{Uam} \exp\left(\frac{Ux}{2E}(1 \mp m)\right)$$

At the assumed discharge location at the Lighthouse (x = 0), the exponential term equals one and the mean concentration is:

$$\bar{c} = \frac{M_R/\Delta}{Uam}$$

The m term is first calculated:

$$m = \sqrt{1 + 4kE/U^2}$$

$$= \sqrt{1 + 4(1.5/\text{day})(1\ \text{mi}^2/\text{day})/(1.28\ \text{mi/day})^2}$$

$$= 2.16$$

Therefore:

$$\bar{c} = \frac{0.31\ \text{lbs}/3.25\ \text{days}}{(0.078\ \text{ft/sec})(115{,}000\ \text{ft}^2)(2.16)\ 5.39(\text{lb/day})/\text{cfs-mg/l})}$$

$$= 9.13 \times 10^{-7}\ \text{mg/l}$$

To convert back to MPN/100 ml use the conversion outlined in the previous section (10^8 MPN/100 ml = 1 mg/l):

$$\bar{c} = (9.13 \times 10^{-7}) \times 10^8$$

$$= 91\ \text{MPN/100 ml}$$

The mean concentration at the Kingston Point Beach (x = -0.5 mile) and the Port Ewen Beach (x = + 1.0 mile) are then estimated.

At Kingston Point:

$$\bar{c} = \frac{M_R/\Delta}{Uam} \exp\left(\frac{Ux}{2E}(1 \mp m)\right)$$

$$= 91\ \text{MPN/100 ml}\ \{\exp[\frac{(1.28\ \text{mi/day}(-0.5\ \text{mile})}{2\ (1\ \text{mi}^2/\text{day})}(1 + 2.16)]\}$$

$$= 33\ \text{MPN/100 ml}$$

At Port Ewen:

$$\bar{c} = 91\ \text{MPN/100 ml}\ \{\exp[\frac{(1.28\ \text{mi/day})(+1.0\ \text{mile})}{2(1\ \text{mi}^2/\text{day})}(1\quad 2.16)]\}$$

$$= 44\ \text{MPN/100 ml}$$

This represents the portion of the average summer fecal coliform concentration in the Hudson River at the given locations which is due to the combined sewer overflows. To calculate the mean impact of the separate area runoff the same procedure is followed using an M_R of 0.00040 lbs fecal coliform instead of 0.31 lbs. As the mean result is linear with M_R, the concentrations are 0.0040/0.31 = 0.013 (or about one percent) of the $\bar{c}$'s resulting from the combined overflows. The separate runoff is clearly insignificant relative to the combined overflow.

Another source of fecal coliform bacteria in the Hudson River is the relatively continuous discharge from the Kingston municipal sewage treatment plant. Records from the summer of 1976 (4) indicate an average flow rate of about 4.8 MGD and a fecal coliform density of 115 MPN/100 ml. This effective level of disinfection leads to an estimated contribution to the mean fecal coliform concentration of less than 0.1 MPN/100 ml, and combined sewer overflows remain the primary source of fecal coliform from the Kingston area. The final sources of fecal coliform to the Hudson River: loads from other towns and drainage basins, and the portion transported into the area by the river flow itself are considered background, and no quantitative estimates of their magnitude are made.

7.2.4.2 Variability in the Hudson

A method for estimating the intertidal variability of pollutant concentrations due to stormwater loads to a tidal river or estuary is presented in Section 3.5.2.2. The standard deviation of the concentration is determined using Equation 3-31:

$$\nu_c = \frac{\sqrt{\sigma_m^2 + M_R^2}}{a\sqrt{2\pi E\Delta}} \exp\left(\frac{Ux}{2E}\right) K_o^{1/2}\left(\frac{Uxm}{E}\right)$$

The equation is indeterminent at x = 0, but may be solved for x = -0.5 miles and x = + 1.0 miles, respectively. Considering only the combined sewer load, the standard deviation of the load is:

$$\begin{aligned} \nu_m &= \nu_m M_R \\ &= 1.5\ (0.31\ \text{lb}) \\ &= 0.47\ \text{lb fecal coliform} \end{aligned}$$

The solution for σ_c is therefore:

$$\sigma_c = \frac{\sqrt{(0.47)^2 + (0.31)^2}\ \left(3.03\ \frac{\text{mg/l}}{\text{lb/ft}^2\text{-mi}}\right)}{(115{,}000)\ \sqrt{2\pi(1.0)(3.33\ \text{day})}} \exp\left(\frac{(1.28)x}{2\ (1.0)}\right) K_o^{1/2}\left(\frac{(1.28)x(2.16)}{(1.0)}\right)$$

where x = - 0.5 miles for Kingston Point and + 1.0 miles for Port Ewen. Note that 3.03 is a conversion factor to make the units consistent. Figure 3-12 is used to solve for the K_o term, and the results are:

At Kingston Point (x = -0.5 miles),

$$\sigma_c = 1.20 \times 10^{-6} \text{ mg/1}$$

$$= 120 \text{ MPN/100 ml}$$

At Port Ewen (x = 1.0 miles),

$$\sigma_c = 1.34 \times 10^{-6} \text{ mg/1}$$

$$= 134 \text{ MPN/100 ml}$$

The standard deviations determined are about 3 or 4 times greater than the means ($\bar{c}$ = 33 and 44 MPN/100 ml respectively), indicating the high variability which can be expected from intermittent loads of a highly reactive pollutant.

7.2.4.3 Observed Water Quality

Water quality data have been collected at the Kingston Lighthouse, Kingston Point Beach, and the Port Ewen Beach (5). The results of these weekly surveys for fecal coliform during the summer of 1976 are shown in Table 7-6.

The purpose of examining these results is not to accurately calibrate or verify the model. The model used to represent the system is too simple, the data upon which it is based (particularly the estimate of overflow concentrations) are too uncertain, and the receiving water data are too sparse for rigorous verification. Furthermore, the model is based on general summer conditions and not those specific to 1976. Nevertheless, the data are useful for checking whether the model has properly estimated the order of magnitude of the impact. The 1976 data and the model results are compared in Table 7-7.

The predicted coliform concentrations at all three locations are seen to be of the same order as observed data. Furthermore, both actual data and calculations show a similar pattern, whereby highest concentrations occur at the lighthouse, and diminish both upstream and downstream. One may conclude from this that beach coliform concentrations are predominantly influenced by discharges to the Hudson River from Roundout Creek, rather than from sources elsewhere on the Hudson. Combined sewer overflow loads are indicated to be a significant component of the input from Roundout Creek. The analysis further suggests that coliform loads from this source are greater than (perhaps double) the CSO loads utilized in the calculation. If the levels at the beaches were considered significant enough to constitute a problem which warrented further effort, the analyst would be directed to identify the apparently significant source of additional fecal coliform. Both the actual concentration in the CSO's would be checked, as well as discharges from the

TABLE 7-6

HUDSON RIVER FECAL COLIFORM (MPN/100 ml)
SUMMER 1976

Date	Kingston Lighthouse	Kingston Point Beach	Port Ewen Beach
5/20	50	50	50
6/2	380	30	90
6/9	25	500	25
6/16	180	5	45
6/23	170	95	15
7/7	120	65	65
7/14	310	45	45
7/28	40	5	35
8/5	165	15	60
8/11	50	55	130
8/18	400	70	330
8/25	150	45	40
9/1	1000	35	150
	$\bar{c}$ = 234	78	83
	σ_c = 262	129	84

TABLE 7-7

HUDSON RIVER FECAL COLIFORM (MPN/100 ml)

Location	Model Estimate		Summer 1976 Data	
	$\bar{c}$	σ_c	$\bar{c}$	σ_c
Kingston Lighthouse	91	-	234	262
Kingston Point Beach	33	120	78	129
Port Ewen Beach	44	134	83	84

treatment plant, background loads in the Roundout, or the possibility of other presently unidentified sources.

7.2.5 Simulation of Fecal Coliform Response

To demonstrate an alternative method for estimating the Hudson River response to fecal coliform loadings from Kingston stormwater, the broadscale simulator described in Chapter 4 is used. Hourly loads are generated using a simple landside simulator with a drainage area of 2060 acres, a runoff to rainfall ratio (R_V) of 0.30, and an overflow fecal coliform concentration of 800,000 MPN/100 ml. The treatment plant capture is simulated by a continuous excess interceptor capacity (Q_I) of 5.7 MGD. The Kingston raingage (Gage 304424) is used for the summer of 1965 (July 3, 1965 - August 29, 1965). This 57 day period falls within the maximum allowable for hourly simulation using the broadscale receiving water simulator.

The receiving water response is simulated using the estuary option with a freshwater flow, Q = 9000 cfs; cross-sectional area a = 115,000 ft^2; dispersion coefficient, E = 1 mi^2/day; and fecal coliform death rate, k = 1.5/day. Again note that though the loading and calculations are hourly, intratidal transport is not modeled. The input deck for the receiving water simulation is shown in Table 7-8 (see the users manual in Appendix A for a description of the input cards).

The printed output from the computer simulation is summarized in Figures 7-17 (a-e). Figure 7-17(a) indicates the input parameters used in the run. Figure 7-17(b) shows a plot of the first 45 hours of the simulation. The fecal coliform load (in pounds) is calculated by assuming 10^8 MPN/100 ml = 1 mg/l. The response concentration is shown as MPN/100 ml with a base 10 log scale (the maximum point shown as 3 thus represents 10^3 = 1000 MPN/100 ml). Symbol 1 represents the Kingston Point Beach (x = -0.5 miles) while Symbol 2 represents the Port Ewen Beach (x = 1.0 mile). Note that a day with rainfall is chosen to begin the simulation. This is to avoid too long a period of zero concentrations at the beginning of the simulation, which may not be correct if previous days had rainfall. The output shown in Figure 7-17(b) continues for the full 1367 hours simulated. Figure 7-17(c) shows the next portion of the output which is a listing of the response concentrations. This listing also continues for the full period simulated.

The final and most useful output from the simulator are the location summaries for the receiving water points of interest. These are shown in Figures 7-17(d) and 7-17(e) for Kingston Point and Port Ewen respectively. The summary includes a tabulation of the cummulative density function, the number of times that a selected standard (in this case 200 MPN/100 ml) is violated, and the average, standard deviation and maximum of the observed concentrations. Note that the means calculated at Kingston Point ($\bar{c}$ = 39 MPN/100 ml) and Port Ewen ($\bar{c}$ = 51 MPN/100 ml) are about 17 percent higher than those calculated for the typical summer using Equation 3-30, as shown in Table 7-7. This is consistent with the fact that the total rainfall during the 57 day period in 1965 (7.15 inches) is 19 percent higher than would have fallen during the "typical" summer. (Typical total summer rainfall = 57 days x V/Δ = 57 (0.35/3.33) = 5.99 inches).

```
KINGSTON FECAL COLIFORM SIMULATION -HUDSON R - GAGE 304424 - SUMMER 1955

RECEIVING WATER TYPE = ESTUARY        WATER QUALITY CONSTITUENT = COLIFORM

E =   1.00 SQ.MI/DAY      A = 115000.0 SQ.FT.          U =  0.07 FPS =   1.2 MPD

        COLIFORM DEATH RATE = 1.50 /DAY
```

FIGURE 7-17 (a)

SIMULATOR OUTPUT
SPECIFICATION OF INPUT PARAMETERS

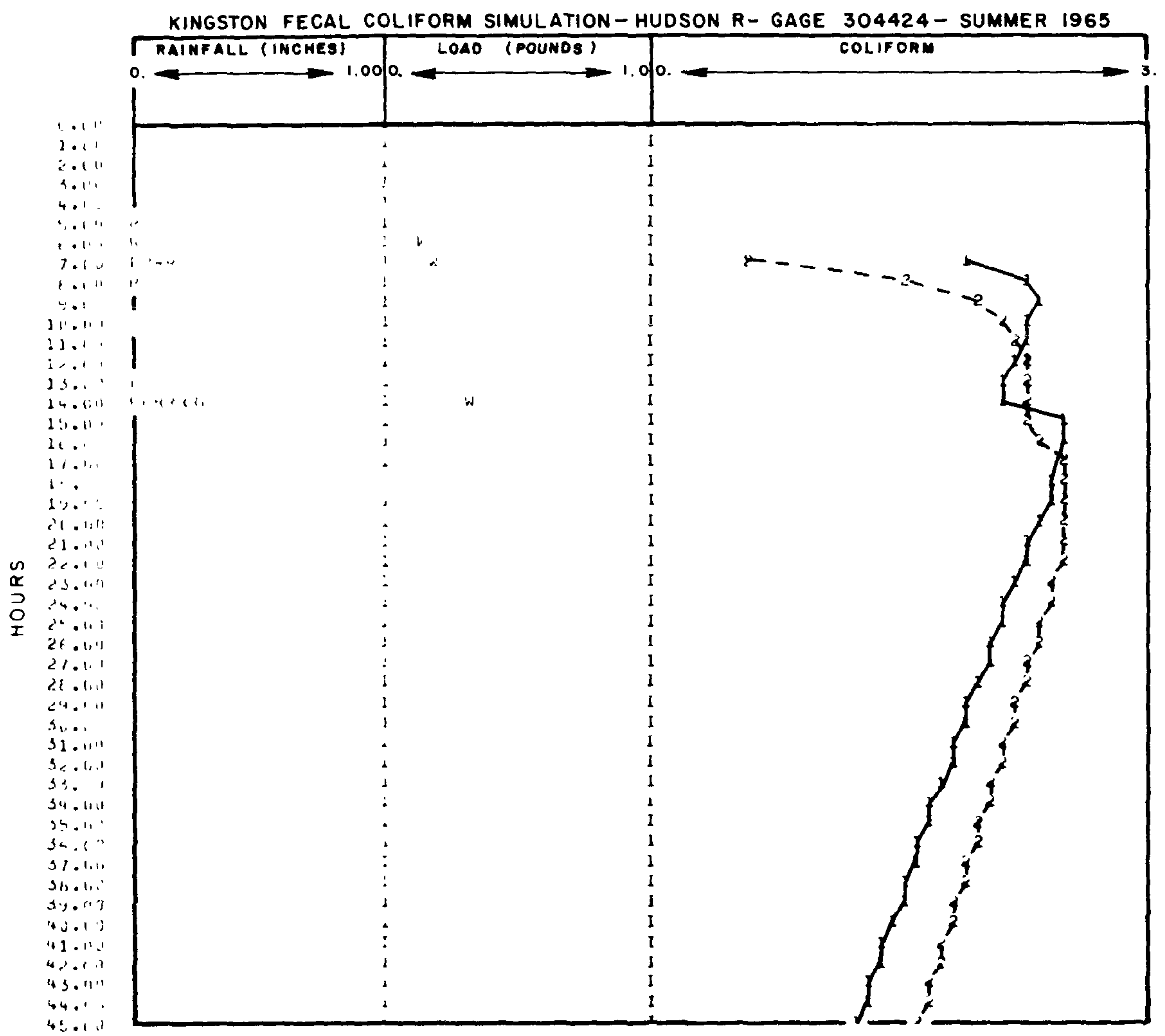


FIGURE 7-17 (b)
SIMULATOR OUTPUT
TIME HISTORY PLOTS

TIME	-0.50 = X	1.00 = X
0.00	0.0000E 00	0.0000E 00
1.00	0.0000E 00	0.0000E 00
2.00	0.0000E 00	0.0000E 00
3.00	0.0000E 00	0.0000E 00
4.00	0.0000E 00	0.0000E 00
5.00	0.0000E 00	0.0000E 00
6.00	0.3572E-03	0.2677E-22
7.00	0.8315E 02	0.3641E 01
8.00	0.1961E 03	0.3661E 02
9.00	0.2136E 03	0.9770E 02
10.00	0.2018E 03	0.1477E 03
11.00	0.1811E 03	0.1754E 03
12.00	0.1609E 03	0.1864E 03
13.00	0.1425E 03	0.1872E 03
14.00	0.1265E 03	0.1819E 03
15.00	0.3077E 03	0.1819E 03
16.00	0.3499E 03	0.2390E 03
17.00	0.3234E 03	0.2989E 03
18.00	0.2958E 03	0.3300E 03
19.00	0.2636E 03	0.3387E 03
20.00	0.2343E 03	0.3333E 03
21.00	0.2033E 03	0.3200E 03
22.00	0.1835E 03	0.3024E 03
23.00	0.1655E 03	0.2829E 03
24.00	0.1430E 03	0.2628E 03
25.00	0.1325E 03	0.2430E 03
26.00	0.1108E 03	0.2239E 03
27.00	0.1037E 03	0.2059E 03
28.00	0.9605E 02	0.1689E 03
29.00	0.8650E 02	0.1732E 03
30.00	0.7800E 02	0.1586E 03
31.00	0.7040E 02	0.1451E 03
32.00	0.6361E 02	0.1327E 03
33.00	0.5752E 02	0.1214E 03
34.00	0.5206E 02	0.1110E 03
35.00	0.4716E 02	0.1014E 03
36.00	0.4274E 02	0.9276E 02
37.00	0.3877E 02	0.8479E 02
38.00	0.3519E 02	0.7749E 02
39.00	0.3195E 02	0.7083E 02
40.00	0.2903E 02	0.6474E 02
41.00	0.2639E 02	0.5919E 02
42.00	0.2400E 02	0.5411E 02
43.00	0.2184E 02	0.4948E 02
44.00	0.1935E 02	0.4525E 02
45.00	0.1810E 02	0.4138E 02
46.00	0.1649E 02	0.3785E 02
47.00	0.1505E 02	0.3462E 02
48.00	0.1370E 02	0.3168E 02
49.00	0.1249E 02	0.2899E 02
50.00	0.1140E 02	0.2653E 02
51.00	.1040E 02	0.2423E 02
52.00	0.9495E 01	0.2222E 02
53.00	0.8658E 01	0.2034E 02
54.00	0.7915E 01	0.1863E 02
55.00	0.7231E 01	0.1706E 02
56.00	0.6603E 01	0.1562E 02

FIGURE 7-17 (c)

SIMULATOR OUTPUT
RESPONSE CONCENTRATIONS

KINGSTON FECAL COLIFORM SIMULATION -HUDSON R - GAGE 304424 - SUMMER 1965
DENSITY FUNCTION FOR LOCATION, X = -0.50
VARIABLE = COLIFORM
CUMLATIVE DENSITY FUNCTION (PCT .LE. CONC)

	PERCENT	CONC
1	0.50	0.00000E 00
2	1.00	0.00000E 00
3	2.00	0.00000E 00
4	3.00	0.00000E 00
5	5.00	0.00000E 00
6	7.00	0.00000E 00
7	10.00	0.00000E 00
8	15.00	0.85516E-04
9	20.00	0.52368E-03
10	30.00	0.12428E-01
11	40.00	0.11131E 00
12	50.00	0.52179E 00
13	60.00	0.15031E 01
14	70.00	0.59255E 01
15	80.00	0.22418E 02
16	85.00	0.56125E 02
17	90.00	0.10697E 03
18	93.00	0.16268E 03
19	95.00	0.21564E 03
20	97.00	0.33363E 03
21	98.00	0.41732E 03
22	99.00	0.59640E 03
23	99.50	0.84464E 03

NO.OF OCCURANCES VS HOURS OF DURATION
THAT COLIFORM VIOLATED STANDARDS
STANDARD = 200.0

NO.HOURS	NO.OCCURANCES	NO.HOURS	NO.OCCURANCES
2	1	7	2
10	1	8	1
14	1	32	1

CRITERIA IS VIOLATED 5.847 PERCENT OF TIME

LOCATION SUMMARY
AVERAGE CONC = 0.388992E 02 MPN
STANDARD DEVIATION = 0.121469E 03 MPN
MAXIMUM CONC = 0.129996E 04MPN

FIGURE 7-17 (d)

SIMULATOR OUTPUT
LOCATION SUMMARY-KINGSTON POINT

KINGSTON FECAL COLIFORM SIMULATION -HUDSON R - GAGE 364424 - SUMMER 1965
DENSITY FUNCTION FOR LOCATION, X = 1.00
VARIABLE = COLIFORM
CUMLATIVE DENSITY FUNCTION (PCT .LE. CONC)

	PERCENT	CONC
1	0.50	0.00000E 00
2	1.00	0.00000E 00
3	2.00	0.00000E 00
4	3.00	0.00000E 00
5	5.00	0.00000E 00
6	7.00	0.00000E 00
7	10.00	0.00000E 00
8	15.00	0.19977E-03
9	20.00	0.15213E-02
10	30.00	0.29796E-01
11	40.00	0.26678E 00
12	50.00	0.11967E 01
13	60.00	0.39454E 01
14	70.00	0.11460E 02
15	80.00	0.40742E 02
16	85.00	0.85370E 02
17	90.00	0.15705E 03
18	93.00	0.22254E 03
19	95.00	0.26379E 03
20	97.00	0.40645E 03
21	98.00	0.53149E 03
22	99.00	0.74784E 03
23	99.50	0.99535E 03

NO.OF OCCURANCES VS HOURS OF DURATION
THAT COLIFORM VIOLATED STANDARDS
STANDARD = 200.0

NO.HOURS	NO.OCCURANCES	NO.HOURS	NO.OCCURANCES
12	1	11	1
16	1	3	1
10	1	19	1
39	1		

CRITERIA IS VIOLATED 8.040 PERCENT OF TIME

LOCATION SUMMARY
AVERAGE CONC = 0.510496E 02 MPN
STANDARD DEVIATION = 0.142394E 03 MPN
MAXIMUM CONC = 0.127646E 04MPN

FIGURE 7-17(e)

SIMULATOR OUTPUT
LOCATION SUMMARY-PORT EWEN

TABLE 7-8

INPUT DECK
RECEIVING WATER SIMULATION

```
Card
 No.

 1   KINGSTON FECAL COLIFORM SIMULATION - HUDSON R. - GAGE 304424 - SUMMER 1965

 2   ESTUARY   COLIFORM

 3       2

 4        -0.50       1.0

 5          1. 115000.      1.5          9000.

 6       1

 7       200.

 8          1.     200.      57.

 9          1.       1.       3.

10   DISK
```

The tabulated cumulative density functions are normalized and plotted for each location in Figure 7-18. It is of significant interest that the calculated distributions seem to be very well represented by a gamma distribution with the appropriate coefficient of variation. Although no theory has as yet been developed which makes this prediction, the possibility appears promising. This would aid greatly in the formulation of predictions relative to the frequency distribution of water quality constituents and problems in tidal rivers and estuaries.

7.2.6 Control Alternatives

The basis for determining the need for stormwater controls is the assessment of the contribution of stormwater loads to the violation of water quality standards or guidelines. While no formal standards are available for the Hudson River beaches, a guideline for fecal coliforms of 200 MPN/100 ml has been suggested (3). The previous analyses indicate that while this guideline is met most of the time, there are some periods when it is not. (If the fecal coliform guideline of 200 MPN/100 ml is indicated as a median, i.e. the 50th percentile concentration, the guideline is not violated.) This section examines the stormwater control that might be considered if the problem was of sufficient magnitude of warrant remedial action.

High rate chlorination units may be considered for overflow locations in the Kingston combined sewerage system. Assuming for the sake of analysis that four units are located to treat the entire overflow, each with a chlorine feed rate (assumed to be 10 mg/l) sufficient to yield a disinfectant concentration which produces a 99.9 percent kill at a contact time associated with the mean runoff flow, Figure 3-24 indicates that an average long term removal of about 93 percent may be expected (without flow proportioning of the chlorine dosage). This should reduce mean summer stormwater induced fecal coliform levels a corresponding amount. As the standard deviation of the fecal coliform loads is reduced much less than the reduction in the mean, one may still expect occasional violations of the 200 MPN/100 ml criteria, though these will occur much less frequently than the currently predicted frequency of about 5 to 10 percent.

To estimate the order of cost for the four units, the following calculations are made:

$$\text{Mean runoff flow at each unit} = Q_R/4 = 21 \text{ MGD}/4 = 5.25 \text{ MGD}$$

$$\text{Chlorine feeding rate at each unit} = (10 \text{ mg/l})(5.25 \text{ MGD})(8.34 \frac{\text{lb/day}}{\text{MGD mg/l}}) = 438 \text{ lb/day}$$

Assume from this that each device has a chlorine feed capacity of 500 lbs/day, Figure 7-19 indicates the capital cost in June 1975 dollars is about

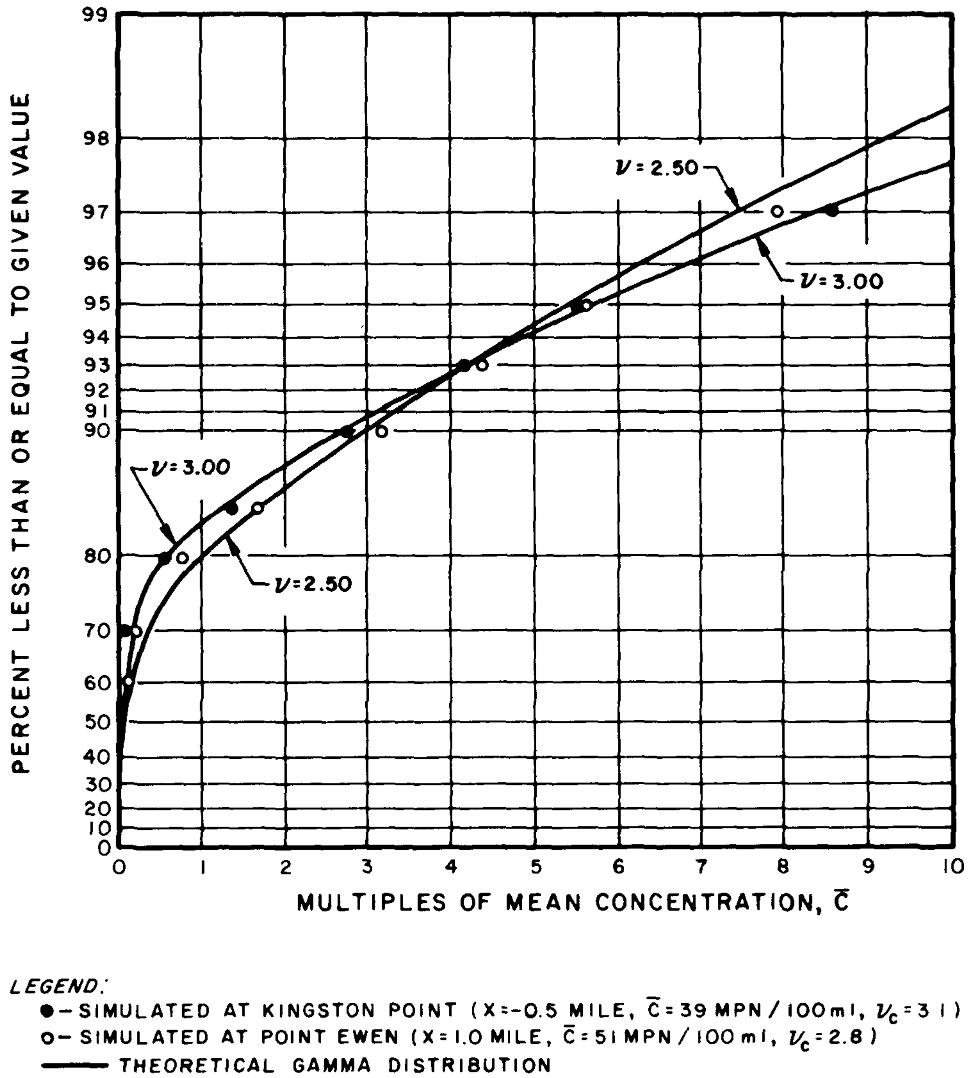


FIGURE 7-18

FREQUENCY DISTRIBUTION OF SIMULATED HUDSON RIVER FECAL COLIFORM RESPONSE (SUMMER, 1965)

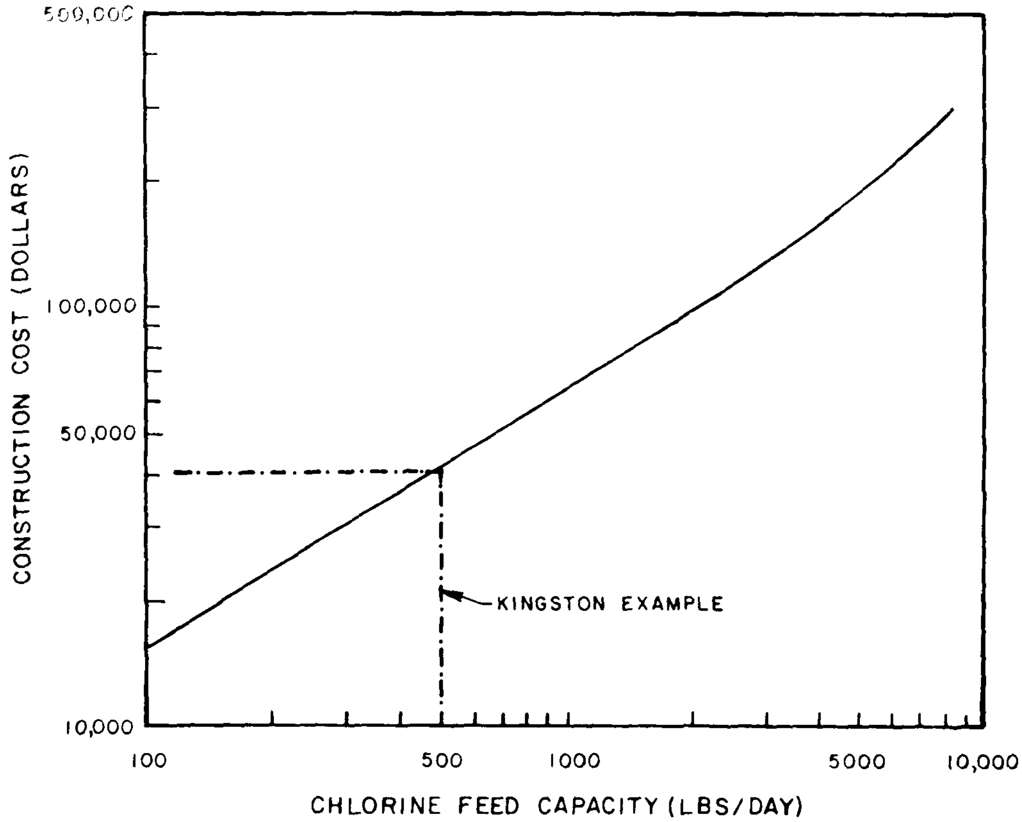


FIGURE 7-19

CAPITAL COST - CHLORINE FEED EQUIPMENT

(JUNE 1975 COSTS)

$40,000 for each device. The operation and maintenance cost for each unit is estimated as follows:

Hours of operation = (No. hours/year)(Fraction of time runoff occurs)

= 8766 (D_R/Δ)

= 8766 (7 hrs/80 hrs)

= 767 hrs/year

Chlorine usage = 767 hrs/year x 438 lb/day x 1 day/24 hrs

= 14,000 lbs/year

= 7 tons/year

Figure 7-20 indicates an annual operation and maintenance cost of $18,000/year for each device. A rapid mixing basin is also required for each chlorination facility. Assuming each has a design flow capacity of four times its mean flow rate (4 x 5.25 = 21 MGD) Figure 7-21 indicates a capital cost of $30,000 for each basin. Finally, Figure 7-22 estimates the operation and maintenance cost for each basin at $600/year. The total capital cost for the chlorination system (chlorine feed and rapid mixing basins) is therefore:

($40,000 + $30,000) x 4 = $280,000

The annual operation and maintenance cost is estimated as:

($18,000 + $600) x 4 = $74,400/year

These are in June 1975 dollars and should be adjusted for other times according to the appropriate inflation-cost index. These values should be viewed only as order of magnitude estimates, without any consideration of particular local conditions.

This concludes the preliminary assessment of stormwater related fecal coliform problems at beaches in the Kingston area. The next step is to decide whether the level of the problem, the projected improvement and associated cost warrant more detailed study of the overflows, their impacts, and possible treatment.

7.3 References

1. *Jordan River Water Quality Projections for Salt Lake County 208*, Hydroscience, Inc., for Salt Lake County Council of Governments, Walnut Creek, California, February 1977.

2. Nielson, Maxwell, Wansgard, Inc., *Memorandum to Hydroscience, Inc. Regarding Computation of Stormwater Runoff Flow for Salt Lake County*, 1976.

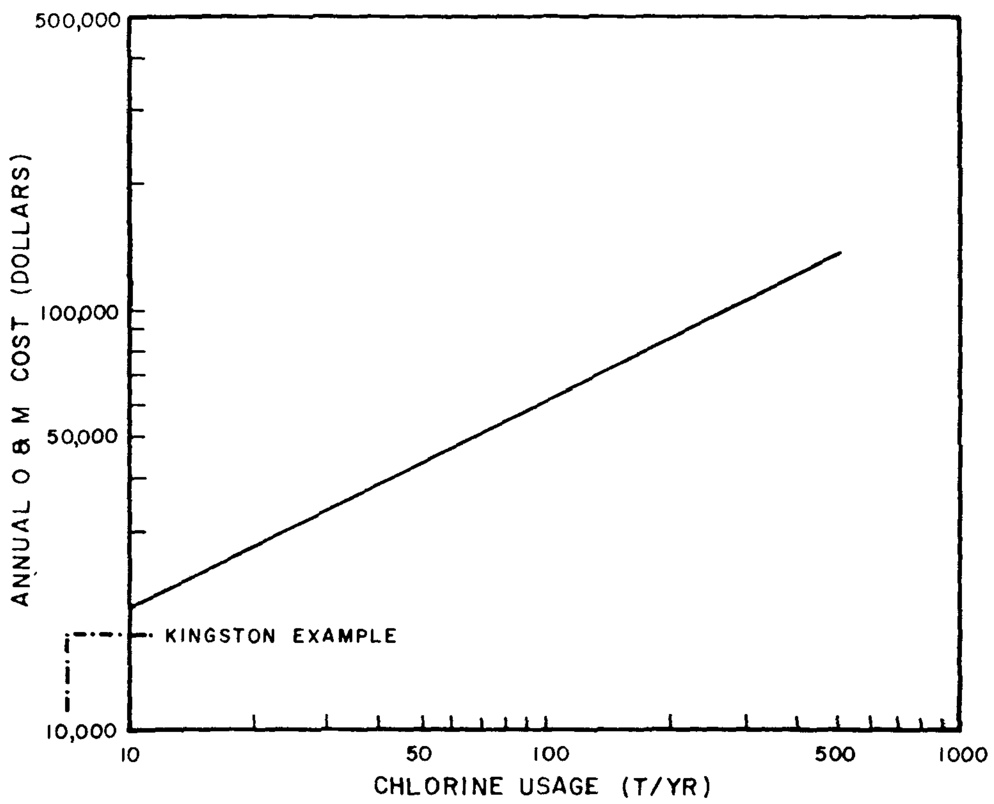


FIGURE 7-20
ANNUAL OPERATION AND MAINTENANCE COST
CHLORINE FEED EQUIPMENT
(JUNE 1975 COST)

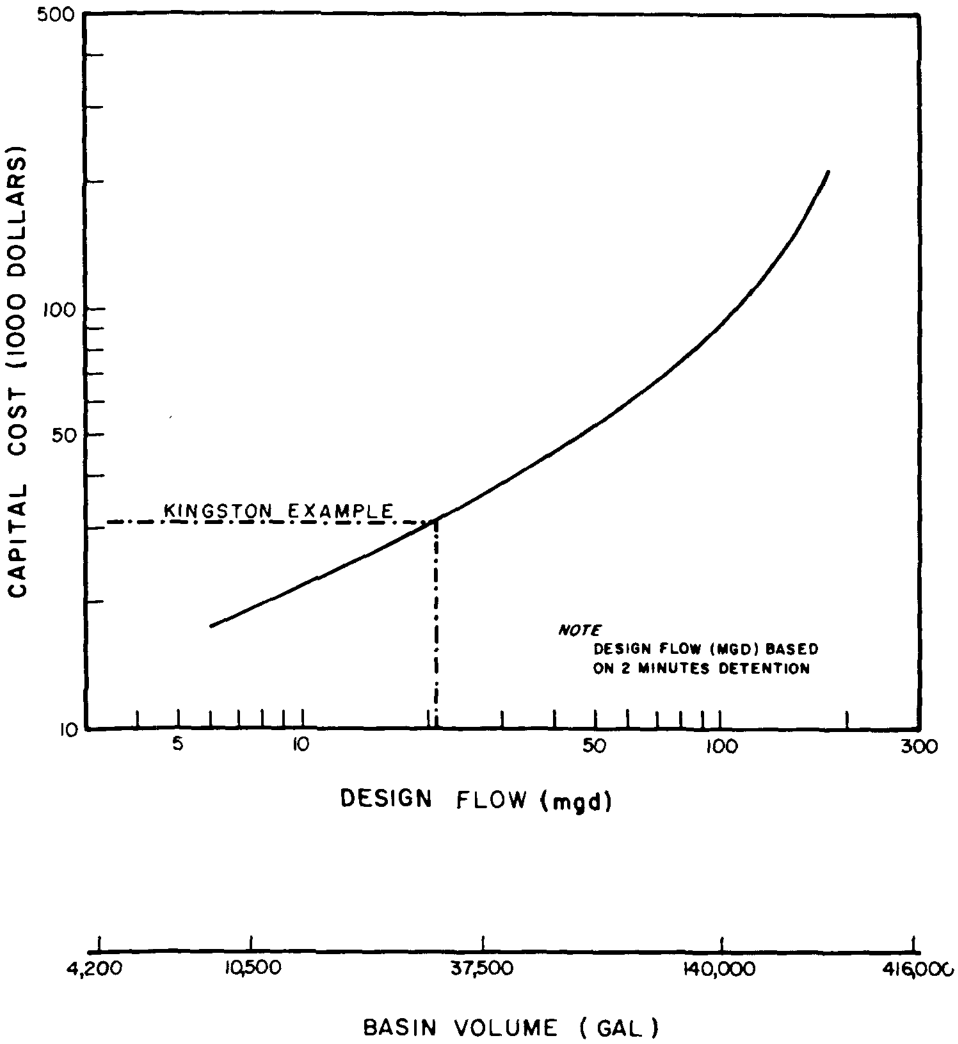


FIGURE 7-21
CAPITAL COST – RAPID MIX BASIN
(JUNE 1975 COSTS)

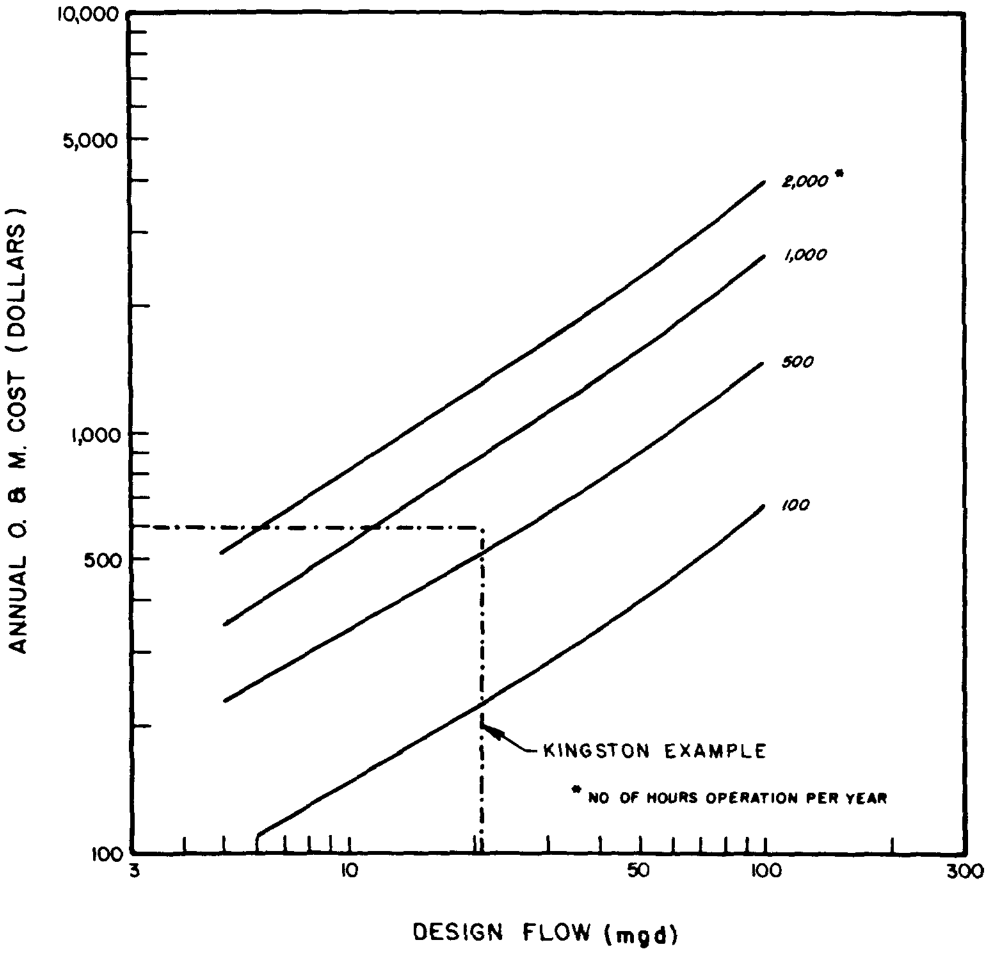


FIGURE 7-22
ANNUAL OPERATION AND MAINTENANCE COST - RAPID MIX BASIN
(JUNE 1975 COSTS)

3. *Hudson River Water Quality and Waste Assimilative Capacity Study*, Quirk, Lawler, and Matusky Engineers for New York State Department of Environmental Conservation, Albany, New York, December 1970.

4. *Monthly Performance Report*, Kingston Wastewater Treatment Facility, Kingston, New York.

5. Water Quality Survey, Ulster County Health Department, Ulster County Health Department, Ulster, New York, 1976.

APPENDIX A

BROADSCALE RECEIVING WATER SIMULATOR

INPUT STRUCTURE

AND

PROGRAM LISTING

APPENDIX A

BROADSCALE RECEIVING WATER SIMULATOR (BRWS) INPUT STRUCTURE

Card 1 - Title Card

1 80
Jackson River June-August 1972
FORMAT (20A4)

TITLE - any alphanumeric text may be used to describe the simulation to be run

Card 2 - Water Body and Water Quality Constituent Types

1 8 11 18
WBTYP WQCON
FORMAT (A8,2X,A8)

WBTYP - RIVER, for river (or streams)
- ESTUARY, for estuaries

WQCON - COLIFORM, for coliform bacteria
- BOD, for BOD
- DO, for dissolved oxygen

Card 3 - Number of Locations to be Monitored

3 5
NLOC
FORMAT (I5)

NLOC - number of locations on the river or in the estuary where concentration profiles are to be computed: minimum of one (1), maximum of six (6)

Card 4 - Milepoints, Relative to Discharge, of Locations to be Monitored

8	16		48
LOCS(1)	LOCS(2)	. . .	LOCS(NLOC)

FORMAT (6F8.0)

LOCS(I) - milepoint relative to discharge location (miles). Convention: (-) for upstream location, (+) for downstream location

Card 5 - Geophysical Parameters

8	16	24	32	40	48	56	64	72
E	A	KR	U	Q	KD	KA	BC	CS

FORMAT (9F8.0)

E - dispersion coefficient (square miles/day). Needed for estuaries only (for rivers = 0)

A - cross sectional area (square feet)

KR - temperature corrected coliform death rate (WQCON = coliform) or BOD removal rate (WQCON = BOD, DO) (day^{-1})

U - freshwater velocity (fps): user may have U calculated by program by inputting Q (cfs) program uses U = Q/A

Q - freshwater flow (cfs): user may have Q calculated by program by inputting U (fps), program uses Q = U*A

KD - temperature corrected deoxygenation rate (day^{-1}), needed only if WQCON = DO

KA - temperature corrected reaeration rate (day^{-1}) needed only if WQCON = DO. NOTE: KR cannot equal KA

BC - upstream background conditions (mg/l needed only if WBTYP = RIVER and WQCON = BOD or DO (for DO reach as dissolved oxygen, not deficit)

CS - dissolved oxygen saturation value (mg/l) needed only if WQCON = DO

Card 6 - Number of Water Quality Standards to be Analyzed

5
NOSTD
FORMAT (I5)

NOSTD - number of water quality standards or criteria to be analyzed: minimum of one (1), maximum of six (6)

Card 7 - Standards

8	16	24
TMPSTD(1)	TMPSTD(2)	TMPSTD(NOSTD)
FORMAT (6F8.0)		

TMPSTD(I) - water quality standard or criteria for coliform units are MPN/100 ml; for BOD or DO units are mg/l

Card 8 - Time Parameters

8	16	24
DT	TTOT	NDAY
FORMAT (3F8.0)		

DT - rainfall observation interval or sampling interval (hours)

TTOT - number of hours to be used for response evaluation (hours): for streams use maximum distance from discharge location divided by freshwater velocity, i.e., time-of-travel to maximum location of interest: for estuaries, use similar approach but remember dispersion coefficient E (suggest two hundred (200) hours)

NDAY - length of rainfall record (days) NOTE: maximum length must meet following constraint:

$(NDAY*24 + TTOT)/DT \leq 1,500$

Card 9 - Plotting Scales

8	16	24
PSCAL(1)	PSCAL(2)	PSCAL(3)
FORMAT (3F8.0)		

PSCAL(1) - plot maximum for rainfall record (inches)

PSCAL(2) - plot maximum for waste discharge record (pounds)

PSCAL(3) - plot maximum for water quality constituent response.

The following scales are recommended:

WQCON	PSCAL
COLIFORM	8 (log base 10) MPN/100 ml
BOD	24 mg/l
DO	12 mg/l

Card 10 - Rainfall and Discharge Record Input Device

```
1   4
DVICE
FORMAT (A4)
```

DVICE - DISK, rainfall record and its associated discharge loads and flows will be read from disk file (generated by a drainage basin simulator). If the user selects disk input, the file should be written as an unformatted file, 8 bytes per record.

DVICE - CARD, rainfall record and its associated discharge will be read from punched cards.

if DVICE = CARD

Card(s) 11 and as necessary - Rainfall and Discharge Record

```
   8  16  24  32
RAIN M  QW  MD
FORMAT (4F8.0)
```

RAIN - rainfall occurring over the interval between this record and the next (inches)

M - pounds of coliform bacteria (WQCON = COLIFORM) or pounds of BOD (WQCON = BOD, DO) occurring over interval (pounds)

QW - stormwater overflow (cfs); needed only if significant when compared to freshwater flow; used to obtain proper mass balance, not used to correct time-of-travel

MD - pounds of DO deficit discharged in runoff or overflow; important if DO in discharge is below saturation

Mass should be calculated as follows:

Coliform - $M = (QC\Delta t) \times 10^{-8} \times 5.39 \times 1/24$

BOD, DO - $M = (QC\Delta t) \times 5.39 \times 1/24$

Q = discharge flow (cfs)

C = concentration of coliform (MPN/100 ml), or
= concentration of BOD or DO (mg/1)

t = time interval between rainfall records (hours)

```
      INTEGER         HLOOP,DLOOP,SLOT
     .                ,OUT,FREQ3(50),FRQ3V(50),THRS
     .          ,ASTER,BLANK,THRSV
     .          ,LINEP(63),ICHAR(9),MAXJP(9),SHIFT(9)
      INTEGER    JKSORT(2),IDSORT(2)
      REAL    TITLE(20),LOCS(6),C(1550,6),M,MD,KR,KD,KA
     .        ,WBTYP(2),WQCON(2),TMPSTD(6),PSCAL(3),NDAY
     .          ,SOBS(1550),TOTAL(6),AVER(6),SD(6),VMIN(6),VMAX(6)
     .          ,PCTILE(23),SCALE(3),P(9)
      EQUIVALENCE   (WBTYP(1),WBT) , (WQCON(1),WQC)
      DEFINE FILE 10(1500,8,U,IREC)
      DATA    COLIF/'COLI'/ , BOD/'BOD '/ , DO/'DO  '/ , RIVER/'RIVE'/
     .        , ESTUR/'ESTU'/
      DATA    DISK/'DISK'/ , CARD/'CARD'/
      DATA    PCTILE/0.5,1.,2.,3.,5.,7.,10.,15.,20.,30.,40.,50.,60.,70.,
     .   80.,85.,90.,93.,95.,97.,98.,99.,99.5/
      DATA       JKSORT/1,0/ , IDSORT/1,0/
      DATA    NPCTL/23/
      DATA    IREC/1/
      DATA    IWQ/0/,IWB/0/
      DATA    LNCNT/0/,MXCNT/0/
      DATA    PI/3.14159/
      DATA    IN/8/,OUT/5/,THRS/0/
      DATA   STDRD/2300./,ASTER/'**'/,BLANK/'  '/
      DATA     NHRSV/0/,THRSV/0/
      DATA    ICHAR/'R','W','D','1','2','3','4','5','6'/
      DATA    SHIFT/0,21,21,42,42,42,42,42,42/
      DATA    MAXJP/20,20,20,40,40,40,40,40,40/
C
      EXPOF(X,T, RATE) = EXP(-((X-U*T)**2./(4.*E*T)+ RATE*T))
C
      CALL SETIA(FREQ3,50,1,0)
      CALL SETIA(FRQ3V,50,1,0)
      CALL SETRA(SOBS,1550,1,0.0)
      READ(IN,1000)   TITLE
 1000 FORMAT(20A4)
      READ(IN,1050)   WBTYP,WQCON
 1050 FORMAT(2A4,2X,2A4)
      IF(WBT.EQ.RIVER)   IWB = 1
      IF(WBT.EQ.ESTUR)   IWB = 2
      IF(WQC.EQ.COLIF)  IWQ = 1
      IF(WQC.EQ.BOD)   IWQ = 2
      IF(WQC.EQ.DO)   IWQ = 3
      IF(IWQ.EQ.0 .OR. IWB.EQ.0)   GO TO 975
      READ(IN,1100)   NLOC
 1100 FORMAT( I5)
C        READ LOCATIONS OF INTEREST(MILEPOINTS ABOVE OR BELOW DISCHARGE)
      READ(IN,1200)   (LOCS(I),I=1,NLOC)
 1200 FORMAT(10F8.0)
      READ(IN,1200)   E,A,KR,U,Q,KD,KA,BC,CS
C        E = SQ.MI/DAY
C        A = SQ.FT
C        KR = 1/DAY
C        U = FPS
C        IF U = 0 CALCULATE U FROM U = Q/A
C        Q = CFS
C        KD = 1/DAY
C        KA = 1/DAY
C        BC = MG/L
C        CS = MG/L
C        READ WATER QUALITY STANDARDS TO BE MET
```

```
      READ(IN,1100)     NOSTD
      READ(IN,1200)     (TMPSTD(IST),IST=1,NOSTD)
C        READ RAINFALL INTERVAL(HRS),NO. OF HOURS TO BE USED FOR
C        RESPONSE EVALUATION, AND LENGTH OF RAINFALL RECORD(DAYS)
      READ(IN,1200)     DT,TTOT,NDAY
C        READ PLOTTING SCALES
      READ(IN,1200)     PSCAL
C        READ INPUT DEVICE
      READ(IN,1000)     DVICE
      IF(U.EQ.0.)     U = Q/A
      IF(Q.EQ.0.)     Q = U*A
      AMOD = A/5280./5280.
      UMOD = U*16.4
      DODBC = CS-BC
      WRITE(OUT,1500)     TITLE,WBTYP,WQCON
 1500 FORMAT('1'///20X,20A4//20X,'RECIEVING WATER TYPE = ',2A4,5X,'WATER
     . QUALITY CONSTITUENT = ',2A4)
      WRITE(OUT,1510)     E,A,U,UMOD
 1510 FORMAT(//16X,'E = ',F6.2,' SQ.MI/DAY      A = ',F8.1,' SQ.FT.
     .     U = ',F5.2,' FPS = ',F5.1,' MPD'//)
      IF(IWQ.EQ.1)     WRITE(OUT,1521)     KR
      IF(IWQ.EQ.2)     WRITE(OUT,1522)     KR,BC
      IF(IWQ.EQ.3)     WRITE(OUT,1523)     KR,KD,KA,BC,CS
 1521 FORMAT(25X,'COLIFORM DEATH RATE = ',F4.2,' /DAY')
 1522 FORMAT(25X,'BOD REMOVAL RATE = ',F5.3,' /DAY      BACKGROUND BOD =
     .',F6.2,' MG/L')
 1523 FORMAT(20X,'KR = ',F5.3,' /DAY      KD = ',F5.3,' /DAY      KA = ',
     .      F5.3,' /DAY'  /  20X,'BACKGROUND DO = ',F5.1,' MG/L      CS =
     .',F5.1,' MG/L')
C
C        INITIALIZATION
C
      E = E/24.
      KR = KR/24.
      KD = KD/24.
      KA = KA/24.
      U = UMOD/24.
      A = AMOD
      Q = U*A
      HLOOP = TTOT/DT+1.5
      DLOOP = FLOAT(IFIX(24./DT+0.5))*NDAY
      IF(DLOOP.GT.1440)     GO TO 900
      SCALE(1) = PSCAL(1)/20.
      SCALE(2) = PSCAL(2)/20.
      SCALE(3) = PSCAL(3)/40.
      IF(IWQ.EQ.1 .OR. IWQ.EQ.2)     CALL SETRA(C,1550,6,0.0)
      IF(IWQ.EQ.3)     CALL SETRA(C,1550,6,CS)
      ISTRT = 0
C        TAKE CARE OF BACKGROUND CONCENTRATION(IF ANY)
      IF(IWB.EQ.2)     GO TO 2
      IF(BC.EQ.0.)     GO TO 2
      DO 1 K=1,NLOC
      X = LOCS(K)
      IF(IWQ.LE.2)      CONC = BC*EXP(-KR*X/U)
      IF(IWQ.EQ .3)     CONC = -DODBC*EXP(-KA*X/U)
      C(1,K) = C(1,K)+CONC
      C1K = C(1,K)
      DO 1  I=2,DLOOP
    1 C(I,K) = C1K
C
    2 DO 100 I=1,DLOOP
```

```
      ISTRT = ISTRT+1
C        'BLANK' PRINT LINE
      CALL SETIA(LINEP,83,1,BLANK)
C        READ RAINFALL RECORD
      IF( DVICE.EQ.CARD)    READ(IN,1200)     RAIN,M,QW,MD
      IF(DVICE.EQ.DISK)   READ(10'IREC)    RAIN,M,QW,MD
C        NOTE...
C        QW = CFS  IS THE STORMWATER RUNOFF     NEEDED ONLY IF IT IS
C                     COMPARABLE TO THE FRESHWATER FLOW
C                     USED ONLY FOR CONC CALULATION, NOT FOR VELOCITY MODIF
      QW = QW*2.44569E-08
      QBAL = Q/(Q+QW)
C        SAVE MASS FOR PLOT
      PM = M
      IF(IWQ.EQ.2)      MD = 0.
      PMD = MD
      IF(M.LE.0. .AND. MD.LE.0.)    GO TO 35
C        MAKE CONVERSION FROM MASS UNITS TO CONCENTRATION UNITS
      M = M*1.0887E-07
      MD = MD*1.0887E-07
C        FOR COLIFORM CONVERT MG/L UNITS TO MPN/100 ML UNITS
      IF(IWQ.EQ.1)    M=M*1.0E+08
C
C        MASS BALENCE AT TIME = 0.
C        LOCATE X = 0.
      DO 3 K=1,NLOC
      IF(LOCS(K).NE.0.)    GO TO 3
C     IF(IWQ.LE.2)    C(ISTRT,K) = QBAL*C(ISTRT,K)+M/(Q+QW)
C     IF(IWQ.EQ.3)    C(ISTRT,K) = CS-(QBAL*(CS-C(ISTRT,K))+MD/(Q+QW))
    3 CONTINUE
C        IF ESTUARY, ADJUST MASS FOR FRESHWATER DILUTION
      IF(IWB.EQ.1)     GO TO 4
      IF(QW.NE.0)     M = M*QW/(Q+QW)
      IF(QW.NE.0)     MD = MD*QW/(Q+QW)
      TERM = M/A/SQRT(4.*PI*E)
      TERMD = MD/A/SQRT(4.*PI*E)
      CALL TSTRT
    4 DO 30  J=1,NLOOP
      IF(J.GT.10)    CALL TSTOP
      T = (FLOAT(J-1)+.10)*DT
      IF(SLOT.GT.DLOOP)    GO TO 100
      SLOT = ISTRT+J-1
      DO 25 K=1,NLOC
      X = LOCS(K)
      GO TO (5,15),IWB
C        RIVER
C        CHECK TO SEE IF MASS SLUG ARRIVES
C        AT -X- DURING THIS TIME INTERVAL
    5 IF(ABS(T-X/U).GE.DT)    GO TO 25
      IF( (T-X/U).GT.0. )    GO TO 25
      IF(IWQ.EQ.3)    GO TO 10
      C(SLOT,K) = QBAL*C(SLOT,K) + M/(Q+QW)*EXP(-KR*X/U)
      GO TO 25
   10 ETERM = EXP(-KA*X/U)
      CONC = QBAL*(CS-C(SLOT,K)) + M/(Q+QW)*KD/(KA-KR)*(EXP(-KR*X/U)-
     . ETERM) +MD/(Q+QW) *ETERM
      C(SLOT,K) = CS-CONC
      GO TO 25
C        ESTUARY
   15 IF(IWQ.EQ.3)    GO TO 20
      C(SLOT,K) = C(SLOT,K)+TERM/SQRT(T)*EXPDF(X,T,KR)
```

```
      GO TO 25
   20 CONC  =  TERM/SQRT(T)*KD/(KA-KR)*(EXPOF(X,T,KR)-EXPOF(X,T,KA))
      IF(MD.NE.0.)    CONC = CONC + TERMD/SQRT(T) *EXPOF(X,T,KA)
      C(SLOT,K) = C(SLOT,K)-CONC
   25 CONTINUE
   30 CONTINUE
C         CHECK FOR TOP OF PAGE
   35 IF(LNCNT.LE.0)    WRITE(OUT,1700)    TITLE,WQCON,ISCAL
 1700 FORMAT('1'  // 14X,20A4 / 14X ,'RAINFALL(INCHES)',10X,
     . 'LOAD(POUNDS)',22X,2A4  /12X,'0.',15X,F6.2,' 0.',11X,F8.1,' 0.' ,
     .      35X,F4.0/12X,'I',9('.'),'I',9('.'),'I I',9('.'),'I',9('.'),
     .          'I I',4('.........I')  )
      P(1) = RAIN/SCALE(1)
      P(2) = PM/SCALE(2)
      P(3) = -1.
      IF(PMD.NE.0.)    P(3) = PMD/SCALE(2)
      DO 40 K=1,NLOC
      IF(IWQ.EQ.1)    P(K+3) = 0.43478*ALOG(C(ISTRT,K))/SCALE(3)
      IF(IWQ.NE.1)    P(K+3) = C(ISTRT,K)/SCALE(3)
   40 CONTINUE
      NP = NLOC+3
      DO 50 J=1,NP
      IF(P(J).LE.0.)    GO TO 50
      IF(P(J).GT.MAXJP(J))    GO TO 50
      JP = P(J)+1.005+SHIFT(J)
      LINEP(JP) = ICHAR(J)
      IF(J.GT.1)    GO TO 50
      DO 45 K=1,JP
   45 LINEP(K) = ICHAR(J)
   50 CONTINUE
      HR = (ISTRT-1)*DT
      WRITE(OUT,1750)    HR,LINEP
 1750 FORMAT(2X,F8.2,1X,'I',21A1,'I',21A1,'I',41A1,'I')
      LNCNT = LNCNT+1
      IF(LNCNT.NE.48)    GO TO 100
      WRITE(OUT,1800)
      LNCNT = 0
 1800 FORMAT(     12X,'I',9('.'),'I',9('.'),'I I',9('.'),'I',9('.'),
     .          'I I',4('.........I')  )
  100 CONTINUE
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
C         END OF RAIN SEQUENCE......PERFORM STATISTICS
      IF(LNCNT.NE.0)    WRITE(OUT,1800)
      WRITE(OUT,1801)    ,
 1801 FORMAT('1')
      WRITE(OUT,1850)    (LOCS(I),I=1,NLOC)
 1850 FORMAT(//5X,'TIME ',6(4X,F7.2,' = X',1X))
      DO 250 I=1,DLOOP
      T = FLOAT(I-1)*DT
  250 WRITE(OUT,1900)    T,(C(I,J),J=1,NLOC)
 1900 FORMAT(1X,F9.2,6E16.4)
C         PERFORM SIMPLE STATISTICAL ANALYSIS ON THE CONC VECTOR
      CALL SETRA(SOBS,DLOOP,1,1.0)
      CALL TALLY(C,SOBS,TOTAL,AVER,SD,VMIN,VMAX,1550,6)
      DO 850 NLOOP=1,NLOC
      DO 825 IST=1,NOSTD
      STDRD = TMPSTD(IST)
C         NHRSV = COUNTER FOR NO. OF CONSECUTIVE HOURS THAT WATER QUALITY
C                 CRITERIA IS VIOLATED
```

```
C        THRSV = TOTAL NO. OF HOURS THAT WATER QUALITY CRITERIA EXCEEDED
C                STANDARDS
C        MHRSV = MAX. NO. OF CONSECUTIVE HRS. THAT WATER QUALITY
C                CRITERIA WAS EXCEEDED
C        MNCUR = MAX. NO. OF OCCURENCES FOR ANY CONSECUTIVE GROUPING
      IF(IST.GT.1)   GO TO 350
C
C        TABULATE DENSITY FUNCTION AND STANDARDS VIOLATIONS
      WRITE(OUT,2500)          TITLE,LOCS(NLOOP),WQCON
 2500 FORMAT('1'/////5X,20A4/5X,'DENSITY FUNCTION  FOR LOCATION, X =',
     .   F7.2/10X,'VARIABLE =',1X,2A4,    /
     .   10X,'CUMLATIVE DENSITY FUNCTION    (PCT .LE. CONC) '/
     .     20X,'PERCENT   CONC'/)
      DO 260 I=1,DLOOP
  260 SOBS(I) = C(I,NLOOP)
      CALL SHELR(SOBS,DLOOP,1550,1,JKSORT,IOSORT)
      DO 300 I=1,NPCTL
      IPOS = FLOAT(DLOOP)*PCTILE(I)/100.+0.05
      CONC = SOBS(IPOS)
  300 WRITE(OUT,3200)     I,PCTILE(I),CONC
 3200 FORMAT(10X,I5,5X,F5.2,E15.5)
  350 DO 500 I=1,DLOOP
      CONC = C(I,NLOOP)
      IF(IWQ.NE.3 .AND. CONC.GE.STDRD)    GO TO 450
      IF(IWQ.EQ.3 .AND. CONC.LE.STDRD)    GO TO 450
  400 IF(NHRSV.EQ.0) GO TO 500
      MXCNT = 0
      ICNT = 1
  410 IF( FRQ3V(ICNT).EQ.NHRSV )    FREQ3(ICNT) = FREQ3(ICNT) + 1
      IF( FRQ3V(ICNT).EQ.NHRSV )    GO TO 430
      IF( FRQ3V(ICNT).NE.0 )    GO TO 420
      FRQ3V(ICNT) = NHRSV
      FREQ3(ICNT) = 1
      GO TO 430
  420 ICNT = ICNT + 1
      IF(ICNT.GT.50)    GO TO 950
      IF( ICNT.GT.50 )    PAUSE  5050
      GO TO 410
  430 CONTINUE
      IF( ICNT.GT.MXCNT )    MXCNT = ICNT
      NHRSV = 0
      GO TO 500
  450 NHRSV = NHRSV + 1
      THRSV = THRSV+1
  500 CONTINUE
      IF(NHRSV.EQ.0) GO TO 510
      ICNT = 1
  501 IF( FRQ3V(ICNT).EQ.NHRSV )    FREQ3(ICNT) = FREQ3(ICNT) + 1
      IF( FRQ3V(ICNT).NE.0 )    GO TO 502
      FRQ3V(ICNT) = NHRSV
      FREQ3(ICNT) = 1
      GO TO 503
  502 ICNT = ICNT + 1
      IF(ICNT.GT.50)    GO TO 950
      GO TO 501
  503 CONTINUE
      IF( ICNT.GT.MXCNT )    MXCNT = ICNT
      NHRSV = 0
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
  510 WRITE(OUT,4550)       WQCON,STDRD
```

```
4550 FORMAT(  ///   18X,'NO.OF OCCURANCES VS HOURS OF DURATION'/20X,
    2  'THAT ',2A4, ' VIOLATED STANDARDS'/29X,'STANDARD = ',F7.1/)
     WRITE(OUT,4600)
4600 FORMAT(10X,2('NO.HOURS    NO.OCCURANCES',4X))
     WRITE(OUT,4700)     (FRQ3V(I),FREQ3(I),I=1,MXCNT)
4700 FORMAT(10X,2(I6,10X,I3,9X)/(10X,I6,10X,I3,9X,I6,10X,I3))
     PCT = FLOAT(THRSV)/FLOAT(DLOOP)*100.
     WRITE(OUT,5000)     PCT
5000 FORMAT(  / 14X,'CRITERIA IS VIOLATED',F8.3,' PERCENT OF TIME'//)
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
C          RESET FREQ. COUNTERS
     CALL SETIA(FREQ3,50,1,0)
     CALL SETIA(FRQ3V,50,1,0)
     THRSV = 0
     MXCNT = 0
 825 CONTINUE
     IF(IWQ.EQ.1)    WRITE(OUT,5500)    AVER(NLOOP),SD(NLOOP),VMAX(NLOOP)
5500 FORMAT(//29X,'LOCATION SUMMARY'/21X,'AVERAGE CONC =',E14.6,' MPN '
    .    /18X,'STANDARD DEVIATION =',E14.6,' MPN '/21X,'MAXIMUM CONC =',
    .    E14.6,'MPN '//)
     IF(IWQ.GT.1)    WRITE(OUT,5520)    AVER(NLOOP),SD(NLOOP),VMAX(NLOOP)
5520 FORMAT(//29X,'LOCATION SUMMARY'/25X,'AVERAGE CONC =', F6.2,' MG/L'
    .    /22X,'STANDARD DEVIATION =', F6.2,' MG/L'/25X,'MAXIMUM CONC =',
    .     F6.2,'MG/L'//)
 850 CONTINUE
C
     CALL EXIT
 900 WRITE(OUT,9000)     NDAY,DT,DLOOP
9000 FORMAT(///20X,'THE STORMWATER DISCHARGE SIMULATOR IS NOT CAPABLE O
    .F PROCESSING A TIME RECORD GREATER THAN 1440 ELEMENTS'/20X,' AND T
    .HE REQUESTED RECORD OF ',F7.2,' DAYS AT ',F6.2,' HRS IS ',I5,' ELE
    .MENTS'/)
     CALL EXIT
9500 FORMAT(///20X,'FREQ3 CORE SIZE EXCEEDED'/)
 950 WRITE(OUT,9500)
     CALL EXIT
 975 WRITE(OUT,9750)
9750 FORMAT(///10X,'IMPROPER RECIEVING WATER BODY SELECTED(NOT EQUAL TO
    . RIVER OR ESTUARY)'//43X,'-OR-'//5X,'IMPROPER WATER QUALITY CONSTI
    .TUENT SELECTED(NOT EQUAL TO COLIFORM, BOD, OR DO)'//)
     CALL EXIT
     END
```

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

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15. SUPPLEMENTARY NOTES

Project Officer: DENNIS N. ATHAYDE

16. ABSTRACT

This manual describes a simplified methodology which can be used to assess the impact of urban stormloads on the quality of receiving waters, and to evaluate the cost and effectiveness of control measures for reducing these pollutant loads. The methodology is particularly appropriate for use at the planning level where preliminary assessments are made to define problems, establish the relative significance of contributing sources, assess feasibility of control, and determine the need for and focus of additional evaluations. It can also be used effectively in conjunction with detailed studies, by providing a cost-effective screening of an array of alternatives, so that the more detailed and sophisticated techniques can examine only the more attractive alternatives.

The methodology is based on the determination of certain statistical properties of the rainfall history of an area. From these statistics, the desired information on loads, performance of controls, and receiving water impacts is generated directly. Procedures are quite simple to apply, using charts and graphs which facilitate screening alternate types or levels of control, testing sensitivity to assumptions concerning drainage area characteristics, stormwater contaminant levels and similar variable factors.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS		b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Runoff	Methodology	Urban Runoff	
Rainfall	Urban Land Use	Urban Hydrology	13 B
Cost-Effectiveness		Receiving waters	
Water Quality		Urban Stormloads	
Water Pollution		Pollutant Loads	
Statistical Analysis		Control Measures	
Computerized Simulation			

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Errata Sheets For

A Statistical Method For The Assessment Of

Urban Stormwater

(EPA 440/3-79-023)

1. Page 1-2. Change the notation for the bibliography to "An extensive bibliography is provided at the end of chapters 2,3,4,5,6, and 7 which can direct the reader..."

2. Page 3-2, equation 3-1. Place a comma before "n = 0,1,..."

3. Page 3-3, equation 3-2. Change to "... = 1 - P_r ($\delta > T$)."

   equation 3-3. Change the last term to "...= $e^{-T/\Delta}$"

   equation 3-4, change to "... = 1 - $e^{-T/\Delta}$"

   equation 3-5. Change to "... = $\frac{1}{\Delta} e^{-\delta/\Delta}$"

   Change the first sentence after equation 3-6 to "The parameter of the Poisson density function, Δ, is thus seen to be the average time between storms, $\bar{\delta}$.

4. Page 3-5, equation 3-7. Change to the following.

$$P_q'(q) = \left(\frac{\kappa}{Q_R}\right)^{\kappa} \frac{q^{\kappa-1}}{\Gamma(\kappa)} \exp(-\kappa q/Q_R)$$

   in which $\kappa = 1/\nu_q^2$ and $\Gamma(\kappa)$ is the gamma function with argument κ

5. Page 3-13, table 3-2. For the precipitation record of:

   5/7/74 -- 7 p.m.; change 0 to 1.

   8 p.m.; change 1 to 0.

   5/16/74 --4 a.m.; change 0 to 2.

6. Page 3-14, table 3-2. Change storm number 2584 to 2694.

7. Page 3-18, equation 3-13. Change the definition of "Q_R" to:

   "Q_R = Mean runoff flow rate (cfs)."

8. Page 3-20. Change the symbol at the top of the page from "ν_{cq}" to "ρ_{cq}".

9. Page 3-21, equation 3-21. Change the term "$(c_o c_p)$" to $(c_p - C_o)$."

Immediately following equation 3-21, add the following.

"The rate of subsidence of the first-flush peak is defined by the rate coefficient β, with units of hours".

Equation 3-23. Change to the following.

"$M_R = E[c(t)qd] = \ldots$"

Equation 3-24. Change the denominator of the second term in the brackets to:

"$(D_R/\beta) + 1$"

10. Page 3-25, equation 3-26. Change to:

"$W_o = M_R/\Delta$"

11. Page 3-25. Following the 10th line from the bottom of the page, add the following. "Length of period of interest (July-September) = 31 + 31 + 30 = 92 days. The assumptions were also made that $D_R/\beta = 1.0$ and $c_p/c_o = 3$."

12. Page 3-26. Change the second line to "... a moderate first flush, with assumed characteristics as identified earlier in this example. Figure 3-6 indicates..."

13. Page 3-27, table 3-3. Add an asterisk to the last two headings of the table. Also add the following footnote.

"* 'Greater than' indicates the percent and expected number of storms which will have flows (Q) and loads (W) greater than the corresponding values in the table."

14. Page 3-33, equation 3-28. Change the units for the symbol "L" to "$(gm/m^2/yr)$."

15. Page 3-40. After the line "U = 10 miles/day", change the following two lines to:

"$\alpha = \dfrac{2Et}{d_r^2 U^2}$

$= \dfrac{2(0.3)(0.42)}{(0.125)^2(10)^2} = 0.16$"

16. Page 3-42. Change the second line from the bottom of the page to "... = (freshwater flow)/A"

17. Page 3-43. Change "K_o = modified Bessel function..." to "$K_o(b)$ = modified Bessel function..." and add "$b = \dfrac{Uxm}{E}$, for this example."

18. Page 3-59, equation 3-35. Add brackets around the center part of the equation as follows.

"$\ldots(q - Q_I)\left[\dfrac{d-V_E}{(q-Q_I)}\right]p_d\ldots$"

19. Page 3-60, equation 3-39. Add brackets around the term following the last integration symbol as follows.

"...[1 - r(q)]..."

20. Page 3-63. Change the third line of the second paragraph to "5-44 to 5-54 in section 5.5.2 of ..."

21. Page 3-86. Add the following footnote.

"The text refers to typical pick up efficiencies of about 50 percent for brush-type and about 90 percent for vacuum type units. Experience from more recent studies suggests that while vacuum devices have higher efficiency with all other things being equal, as a practical matter, other conditions which influence sweeper efficiency are often much more significant than sweeper type. In practical applications, efficiencies in the range of 25 to 50 percent can be expected."

22. Page 3-87, Figure 3-33. Change the note on the figure to
    "STREET SWEEPING SIMULATION, MINNEAPOLIS, MINN. (REF.46)..."
23. Page 3-91. Change the third line from the bottom of the page from
    "...Hendy and Nix..." to "...Heaney and Nix..."
24. Page 5-124. Change the fourth line from the top of the page to:
    "$(k\bar{t})_R$ = the value of $(k\bar{t})$ at the mean runoff flow, Q_R"
25. Page 5-168, reference 1. Change the author's name to "Heaney, James P."
26. Page 6-14, figure 6-4. The description of the ordinate should be
    "ERROR IN ESTIMATE OF AVERAGE (±%)."
    The description of the abscissa should be
    "N = NUMBER OF STORMS MONITORED"
    The scales for the ordinate and abscissa should begin at
    "0" and increase in increments of 10.