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July 1976

AREAWIDE ASSESSMENT
PROCEDURES MANUAL

VOLUME I

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

MANUAL DISTRIBUTION RECORD

The Areawide Assessment Procedures Manual is prepared as one of a number of information documents developed to support the Agency's 208 areawide waste treatment management and planning effort. The complete Manual is presented in a three volume format to facilitate its use. Because the Manual is being prepared and distributed in separate mailings, and because it is anticipated that some chapters or appendices will be revised in the future, it is necessary that the recipient of this portion of the Manual enter a "Register of Manual Users."

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UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

TO: User of the Areawide Assessment Procedures Manual

With the completion of this mailing of the Areawide Assessment Procedures Manual, it is appropriate to review the contents of the three volumes which comprise a complete Manual. By so doing, we can reconcile what was originally proposed in Chapter 1 as the tentative contents of the Manual with the chapters and appendixes which you actually received.

The discussion of the Manual's content in Chapter 1 was prepared at that point in time when details for the preparation of following sections of the Manual had not been finalized. Subsequently, as the later chapters and appendixes were being drafted, modifications to the proposed contents were deemed expedient and the final product is a slightly different but uncompromised version of the Manual that was originally and tentatively outlined. The changes, in fact, have facilitated the continuity of content, data presentation, and format.

The contents of the Areawide Assessment Procedures Manual, specified as to placement in volume I, II, or III, and the chapter or appendix title are as follows:

Volume	Title
I	(Preface material)
I	Chapter 1 - Introduction
I	Chapter 2 - Preliminary Problem Assessment
I	Chapter 3 - Procedures for Assessment of Urban Pollutant Sources and Loadings
I	Chapter 4 - Assessment of Nonurban, Non-point Pollutant Sources and Loadings
I	Chapter 5 - Analysis of Stream Impacts for Urban and Nonurban Sources
I	Chapter 6 - Evaluation and Selection of Control Alternatives

Volume	Title
II	Appendix A - Model Applicability Summary
II	Appendix C - Land Use Data Collection and Analysis
II	Appendix D - Monitoring Requirements, Methods, and Cost
	Appendix D, Part II - Parameter Handbook
II	Appendix E - Documentation for Synoptic Rainfall Data Analysis Program - SYNOP
III	Appendix G, Part I - Urban Stormwater Management Techniques: Performance and Cost
	Appendix G, Part II - "Storm Water Management Model" Report No. EPA-600/2-77-083
III	Appendix H - Point Source Control Alternatives: Performance and Cost
III	Appendix I - Bibliography
III	Appendix J - Glossary

Chapter 7, Examples of Assessment Methodology for Urban and Non-Urban Areas; Appendix B, Water Quality Data Bases; and Appendix F, Water Quality Standards, which were first described in Chapter 1 have been deleted from the contents of the Manual. Due to the increased pressure of the time frame required by the planning cycle, and the reevaluation of priorities which was made as work on other parts of the Manual progressed, the decision was made to concentrate on the most critical remaining sections. As a result, the three sections specified above were not prepared, nor will they be in the immediate future. Therefore, the Areawide Assessment Procedures Manual, as described above by volume and text title, is complete.

DISCLAIMER

This manual has been reviewed by the Office of Research and Development (Municipal Environmental Research Laboratory - Cincinnati, and Environmental Research Laboratory - Athens) and by the Office of Water and Hazardous Materials (Water Planning Division) and is approved for publication.

In approving the first edition of this manual both the Office of Water and Hazardous Materials and the Office of Research and Development emphasize that the information contained herein represents a summarization of selected state-of-the-art assessment procedures and impact analysis techniques that are considered useful and supportive of the objectives of the areawide wastewater planning and management programs.

The contents of the manual are intended to be informative rather than prescriptive in nature and in no way should be considered mandatory. Approval does not signify that the contents necessarily reflect the views of the Environmental Protection Agency nor does mention of trade names or propriety approaches constitute endorsement or recommendation for use.

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

TO: Users of the Areawide Assessment Procedures Manual

The implementation of State and areawide planning under Section 208 of P.L. 92-500 has created a demand for sound technical analyses within a relatively short period of time. As new information becomes available from research efforts, it is important that it be applied where possible to our environmental management efforts. This manual was produced as a joint effort between EPA's Office of Research and Development and the Office of Water Planning and Standards. It provides a statement of procedures available for water quality management, with particular emphasis on urban stormwater. This publication contains the first sections of a manual that will be mailed in three parts. This first mailing includes a description of some of the basic procedures which could be utilized during the early stages of a study to determine whether more complex analyses are warranted.

As point sources are abated, an increased concern has developed on the need for controlling nonpoint sources of pollution. For effective water quality management, it is often necessary to analyze the relative contribution of different pollution sources so that coordinated structural and non-structural control programs can be proposed. This manual suggests procedures which should lead to practical decisions, based on the assumption that the simplest techniques can often produce the necessary information that is to be used. Thus the manual describes several techniques which are representative of different levels of sophistication which may be required for both problem assessment and the evaluation of alternatives. The implementation of environmental programs requires both a sound technical justification as well as local political support; therefore, the desirable plan may not necessarily represent the optimal solution in the strictest sense, but rather a pragmatic solution which will be implemented and result in improved or preserved water quality.

Eckardt C. Beck	Thomas Murphy
Deputy Assistant Administrator for Water Planning & Standards	Deputy Assistant Administrator for Air, Land, and Water Use

ABSTRACT

This manual summarizes and presents in condensed form a range of available procedures and methodologies that are available for identifying and estimating pollutant load generation and transport from major sources within water quality management planning areas. Although an annotated chapter is provided for the assessment of non-urban pollutant loads, the major emphasis of the manual is directed toward the assessment of problems and selection of alternatives in urban areas, with particular concern for stormwater related problems. Also included in the manual are methodologies for assessing the present and future water quality impacts from major sources as well as summaries of available information and techniques for analysis and selection of structural and non-structural control alternatives.

This manual is structured to present problem assessment and impact analysis approaches for several levels of planning sophistication. Simple procedures are recommended for initial analysis to develop the insight and problem understanding to guide the application of more complex techniques where required.

FOREWORD

The enactment of Public Law 92-500 marked a new era of environmental awareness in the United States. A vital part of this legislation is the provision for areawide waste treatment management planning under Section 208. The Congressional intent of Section 208 was to establish a planning framework necessary to meet the 1983 National Water Quality Goals in highly urbanized areas or non-urban environments where complex water quality problems exist.

In establishing an overall wastewater management plan, state and areawide planning agencies must examine the wide variety of pollutant sources and corresponding receiving water impacts in the planning area in light of the necessity and feasibility of their control. The most successful approach will likely be one that integrates ongoing and projected point source pollution control measures with cost-effective combinations of management and structural alternatives for nonpoint source pollution control.

This Areawide Assessment Procedures Manual, produced jointly by EPA's Water Planning Division and Office of Research and Development, is one of a number of guidance and information documents developed to support the Agency's 208 areawide waste treatment management and planning effort. The manual summarizes selected state-of-the-art problem assessment methodologies and approaches that are useful in achieving the goals and objectives of state and areawide water quality management and planning. Future editions of this manual are planned as new areawide assessment procedures and methodologies are further developed and verified.

Francis T. Mayo
Director
Municipal Environmental Research
Laboratory

ACKNOWLEDGEMENTS

This manual has been prepared by the Environmental Protection Agency, Office of Research and Development, Municipal Environmental Research Laboratory, Cincinnati, in collaboration with the Office of Water and Hazardous Materials, Water Planning Division. Significant technical contribution and direction has been provided by the following EPA programs:

- Environmental Research Laboratory, Office of Air, Land and Water Use, Athens, Georgia.
- Environmental Monitoring and Support Laboratory, Office of Monitoring and Technical Support, Las Vegas, Nevada.

In addition to EPA staff contributions, portions of this manual have also been prepared in whole or in part by the organizations listed below:

- Hydroscience, Inc., Westwood, New Jersey.
- Battelle Columbus Laboratories, Columbus, Ohio.
- Roy F. Weston, Inc., West Chester, Pennsylvania.
- EG&G Washington Analytical Services Center, Inc., Rockville, Maryland.
- Metcalf & Eddy, Inc., Palo Alto, California.

CHAPTER 1

INTRODUCTION

1.1 Purpose of the Manual

When Congress passed the Federal Water Pollution Control Act Amendments of 1972 (the Act), it was recognized that a number of water pollution control problems in the United States are so complex that they cannot be solved by traditional engineering evaluation and technology application alone. In most cases, these problems involve urban areas where population and industry are concentrated and where inter-relationships exist between receiving water quality, point source discharges, intermittent point loads from combined sewer overflows, and urban stormwater runoff. The situation may be further complicated and sometimes dominated by nonpoint source contaminant contributions from rural areas outside of the urban fringe, or by the receiving water impacts of construction activities, mining, or residuals management practices.

Section 208 of the Act provides a mechanism for the planning and management necessary to achieve the 1983 goals in these complex regional situations. The purpose of Section 208 is to facilitate the development and implementation of areawide waste management plans at the local level in designated areas and by the state outside such areas. As of September 1976, Federal assistance funds have been provided at 75 to 100 percent of eligible project costs to 176 designated planning agencies for the preparation of initial areawide plans addressing the complex issues.

Early in the water quality management planning program, which began with the award of the first grants in the Spring of 1974, a strong need developed at the designated agency level for technical assistance in the assessment of pollutant loads, receiving water impacts and control alternatives, particularly with regard to nonpoint and intermittent point source loads in the urban environment. As more planning agencies have entered the 208 program, this need has increased even more, in spite of the experience gained by early groups.

To a large degree, the need results from the hydrologic and pollutant generation complexity of urban areas. Each situation is unique, requiring a range of analytic approaches which cannot necessarily be transported from one study area to another. But the major problem has been the fact that there is no established technical framework for analysis of complicated urban wastewater problems. A wide range of methodologies from many sources exists, but many of these are inappropriately applied in planning programs at the sacrifice of time, expense, manpower and plan accuracy.

The objective of this Areawide Assessment Procedures Manual is to provide a unified technical framework for the analysis of complex areawide wastewater problems. A range of useful assessment approaches is evaluated and arrayed within this framework. To the greatest extent possible, an attempt has been made to consolidate selected state-of-the-art information into a single guide. The document stresses approaches to urban problems, but also discusses assessment methodologies for non-urban, nonpoint source pollution problems to the extent needed to put these problems into relative perspective in the overall areawide waste treatment plan. Methods for evaluating the receiving water impact and economic feasibility of alternative pollution control strategies, including non-structural management practices for urban stormwater control, are also provided.

It is especially important to understand that the Areawide Assessment Procedures Manual does not present a required methodology or suggest administrative planning procedures which must be followed. It is a technical assistance and reference document intended for use by designated local planning agencies, state planning bodies and planning and engineering firms involved in areawide waste treatment management planning. It will also be useful to municipal agencies concerned with stormwater management.

The discussion and evaluation of selected present day pollution assessment methods, models, etc., here does not mean that other techniques are not equally useful. The manual will be regularly updated as new techniques are developed and as the results of practical application become available. It is also important to recognize that the document does not specifically concern itself with the institutional, political or legal aspects of areawide waste management planning. These are the subject of other assistance documents soon to be published by the EPA Water Planning Division, Office of Water and Hazardous Materials.

1.2 Relationship of the Manual to Agency Policy, Legal and Regulatory Requirements and Guidance Documents

In assuming its responsibilities for the implementation of the Act, the Environmental Protection Agency has published regulations, policies and guidance documents on water pollution abatement for use by individuals in the private sector and by responsible public officials. While the Areawide Assessment Procedures Manual is not an administrative policy or regulations document, it is important for those who will use it in areawide water quality management planning to understand its relationship to the requirements of such documents. This is particularly true in regard to the question of urban stormwater management policy, which may be less well understood by state and local planners than the more familiar requirements for point source control, effluent limitations, or funding requirements for state and local 208 grants. Several recent developments regarding this policy are discussed briefly below, with special emphasis upon those policies affecting the potential need for application of the assessment procedures of this manual.

1.2.2 Stormwater Management Policy

As the Nation's point source pollution control program nears the end of its first stage, it is becoming more apparent that trade-offs must be made between more advanced treatment of continuous point sources and control of nonpoint source pollution. Among diffuse sources of pollution, stormwater runoff has been identified as one of the major contributors to water pollution in urban areas. Although treatment technology is available for managing the stormwater problem, estimated National costs for implementation of such treatment are prohibitive, ranging from $153 billion to $600 billion. There is little doubt that there may be substantial inaccuracy in these figures. This results from the variability of costing approaches and treatment efficiency assumptions used, and the fact that, in spite of a variety of past assessment studies, the total relationship between urban runoff and receiving water quality is not clearly understood. Nonetheless, it is clear that alternatives other than traditional forms of treatment should be considered.

A more effective approach for stormwater pollution abatement in urban areas is the implementation of nonstructural stormwater management practices in coordination with only the most cost-effective structural control options. This approach may offer pollution abatement potential similar to the various treatment alternatives available, but at a significantly reduced cost.

Stormwater management practices, often termed Best Management Practices (BMPs) fall into two groups: those most useful for existing or developed areas and those more applicable to new or developing areas. In the former instance, BMP embodies "reduction" techniques such as street sweeping, improved waste collection and improved sewer maintenance practices and sewer system management practices to reduce or attenuate stormwater flows to receiving waters. A preventative concept best applies to developing urban areas where the objective should be to manage new development in order to contain or attenuate runoff flows and limit the potential for unnatural pollutant contribution to receiving waters. Techniques in these areas include improved construction site management; provision for groundwater recharge; construction of detention basins; and playground, parking lot, or rooftop storage of stormwater.

1.2.3 Funding and Legal Requirements

The need for the thorough development of comprehensive urban/areawide wastewater management plans has been amplified by a recent court action and by the Agency's further interpretation of stormwater regulations.

As a result of a June 10, 1975, decision of the Federal District Court for the District of Columbia (NRDC versus Train) the Agency is required to apply the National Pollutant Discharge Elimination System (NPDES) permit program to separate storm sewers. Regulations for implementing this decision were finalized in December 1975 (40 FR 56932). In accordance with these regulations certain storm sewers that were once exempt due to nonpoint source status are now considered point sources, for which general permits will be issued. In addition, respective permitting authorities may on a case-by-case basis require the owner-operator of any separate storm sewer to obtain a conventional NPDES permit.

Under these new regulations it has become increasingly important that urban runoff from separate storm sewers be adequately included as a part of the areawide planning process. It is also important for planners to realize that the regulations have expanded the definition of a "separate storm sewer" to mean "a conveyance or system of conveyances (including but not limited to pipes, conduits, ditches and channels) located in an urbanized area and primarily operated for the purpose of collecting and conveying stormwater runoff." (Title 40, Chapter 1, Part 124, Part 125, Federal Register, Vol. 41, No. 54, May 18, 1976).

The Agency has also provided policy direction regarding the use of construction grants for providing treatment and control of combined sewer overflows and stormwater discharges during wet-weather conditions. This policy has significant implications upon the degree of stormwater analysis conducted in 208 planning and upon the nature of stormwater control alternatives proposed in the final water quality plan.

Construction grant funds may be approved for the control of pollution from combined sewer overflows, but only after thorough, detailed wastewater treatment planning for the 20-year planning interval has been completed and has adequately considered the following:

(1) the effectiveness of alternative control techniques or management practices,

(2) the costs of achieving various levels of pollution control with each feasible technique,

(3) the benefits to the receiving waters of a range of levels of pollution control during wet-weather conditions, and

(4) the costs and benefits resulting from the addition of advanced waste treatment processes to dry-weather flows in that area.

Where detailed areawide planning has been completed, treatment or control of wet-weather overflows and by-passes may be given priority for construction grant funds only after provision has been made for secondary treatment of dry-weather flows in the area. For control or treatment of separate discharges of stormwater, however, the Agency's current policy is that construction grants shall not be available except under unusual conditions where a project situation has clearly been shown to meet the detailed planning and evaluation criteria for combined sewer overflow grants.

Projects with multiple purposes in addition to pollution control, such as flood control and recreation, may be eligible for grant amounts not to exceed the cost of the most cost-effective single pollution abatement system.

1.2.4 Relationship to Other Technical Guidance Documents for Areawide Planning

This Areawide Assessment Procedures Manual is one of several publications issued or in preparation to assist the Water Planning Division of the EPA Office of Water and Hazardous Materials to provide technological guidance and information to those involved in areawide water quality management planning.

A distinguishing feature of this manual is that it is more comprehensive in scope than many of the previously issued assistance documents. It represents the first of a series of information documents to be issued for use in technical 208 planning efforts which will also include management practice guidelines and pollution assessment methods for mining activities, non-irrigated agriculture, silviculture, hydrographic modifications, construction activities, and residuals management practices. These documents are currently in preparation.

1.3 Guide for Use of the Manual

1.3.1 Intended Users

The Areawide Assessment Procedures Manual is intended to be used as a comprehensive technical reference and planning assistance document for use by a wide range of individuals involved with various stages of the 208 planning process. The potential user community includes: administrators, planning directors and technical planners at designated 208 agencies; state environmental quality officials and individuals responsible for statewide 208 plans; private firms providing consulting services to 208 planning efforts; Federal and state officials responsible for the review and evaluation of areawide plans; and other public institutions responsible for the management of urban drainage systems.

This is not to imply that the entire manual will be equally useful to all people. Various portions will be useful to each of these groups at various stages of the planning cycle. The manual does not have to be read from cover to cover to be of use in the planning process.

For those designated local agencies in the formative stages of their planning program, the preliminary assessment concepts, technical reference material and cost information throughout the document will be very helpful. Planning directors or administrators will probably find the preliminary problem assessment sections of greatest use, especially in the sense that problem identification in the early project stages will help clarify staffing needs, suggest more efficient allocation of limited resources among highest priority problems, and assist in the generation of an effective work approach. Technical planners, engineers and consultants on the other hand will have a greater interest in the assessment methods, evaluation sections, and the Appendix information.

It is fully recognized that the utility of the manual will also vary from one planning organization to another depending upon its stage in the planning process and its regional environmental planning approach. Overall, the manual will have the greatest impact upon those groups who are still developing their approach to assessing urban/areawide water quality problems. Organizations nearing completion of their 208 plans, or those with well established comprehensive regional assessment approaches will make greater use of the sections dealing with advanced assessment methods, the evaluation of alternative control strategies, and management practices for urban stormwater control. For those designated agencies already contractually committed to a particular planning direction the information throughout the manual will enable more effective technical communication with and direction of their consulting organizations.

1.4 Organization of the Manual

1.4.1 Rationale of Approach

An acknowledged fact of areawide planning is that the problems and needs of no two planning areas are exactly the same. Similarly, problems within a planning area have varying degrees of priority and complexity. Consequently, there is no universally applicable assessment tool for the analysis and solution of these problems. Rather, there are a variety of useful methods of varying cost, accuracy and sophistication which planners must apply in a successful assessment program.

This manual recognizes this fact by establishing a sequential assessment approach beginning with a gross, first-cut analysis designed to determine the relative magnitudes, and spatial and temporal distribution of major pollution problems in an area. These analyses are intended to rely only upon existing or readily available data bases and are designed to help the planner avoid overly sophisticated, expensive and often unnecessary efforts in areas where certain problems are not critical. Once the planner has identified the critical parameters and problem areas for his region, he may then refer to the appropriate higher order analyses which are presented in subsequent sections of the manual.

1.4.2 Chapter Content

The content of each of the respective chapters of this manual is outlined below:

Chapter 1 - "Introduction"

Chapter 2 - "Data Base Inventory and Preliminary Problem Identification": indicates basic data requirements and procedures necessary for preliminary problem identification and assessment in order to define those additional technical steps necessary for development of an effective areawide plan. Techniques are presented to assess the magnitude and impact of various classes of waste sources, urban and non-urban, on water quality under various seasonal and hydrologic conditions.

Chapter 3 - "Procedures for Urban Assessment of Pollutant Sources and Loadings": provides illustrative alternative procedures which are available to assess, in additional detail, the magnitude of urban wastewater loads. Procedures are discussed for definition of continuous municipal and industrial point source loads, intermittent rainfall related combined sewer overflows and stormwater discharges and generalized nonpoint source urban runoff. Two alternative technical approaches are discussed for estimation of stormwater related waste flow and quality.

Chapter 4 - "Procedures for Non-Urban Assessment of Pollutant Sources and Loadings": describes more detailed procedures to determine the quantity and quality of non-urban nonpoint sources. Methods of estimating seasonal variations in the flow and quality of runoff originating from agricultural and forrested areas are discussed. In addition, nonpoint source loadings from other diverse activities including construction, residuals disposal, mining and irrigated return flows are described.

Chapter 5 - "Analysis of Stream Impacts for Urban and Non-Urban Sources": summarizes methods of analysis which are available to determine the impact of urban and non-urban wastewater sources on the quality of receiving waters. Different types of receiving water bodies are described along with time and space scales of water quality problems. Steady state and time varying water quality modeling approaches are discussed as well as single and multi-dimensional analytical networks. Data requirements, parameter evaluation, calibration and verification techniques are described. The use of the various techniques is related to the urban and non-urban loading assessment methodology discussed in Chapters 3 and 4.

Chapter 6 - "Evaluation and Selection of Control Alternatives": a matrix of controllable and uncontrollable sources for critical pollutants from urban and non-urban areas is presented along with methods of ranking structural and nonstructural alternatives for load reduction. Procedures to develop a least cost mix of structural and nonstructural solutions to meet desired water quality goals are identified.

Chapter 7 - "Examples of Assessment Methodology for Urban and Non-urban Areas": a data base inventory and problem identification for urban and non-urban assessments with comparative evaluations of mass pollutant loadings and stream impacts are provided. Additionally, a summary presentation and selection of control alternatives is given.

1.4.3 Use and Content of Appendices

The ten appendices are intended to be supportive of the information presented in the Chapters. However, each appendix is written for separate identifiable subjects and may also be used independent of the text of the manual.

The following briefly describes the content of each appendix.

Appendix A - "Model Applicability Summary": this appendix contains a summary of computer-based mathematical tools available to areawide water quality planners. In addition, an explanation and data input needed for each model is presented in order to help in the evaluation and selection of the most effective model.

Appendix B - "Water Quality Data Bases": the availability and value of various water quality data bases are discussed in terms of their use in the areawide water quality planning process.

Appendix C - "Land Use Data Bases and Methods": a range of specific techniques and qualitative methods for land use data collection, management and analysis in areawide water quality management planning is described. A descriptive survey of alternative approaches to the land use element of the 208 planning process, from relatively simple techniques to complex ones, is provided.

Appendix D - "Monitoring Requirements, Methods and Costs": the best available technical information for the design, management, and execution of water pollution control monitoring of interest to 208 planning agencies is organized and packaged in readily useable form. Descriptions of the manpower, equipment, and technical methodology requirements and associated costs are also presented.

Appendix E - "Statistical Analysis Procedures and Methods": compatible methods for the statistical analysis of climatic data, stream flow, pollutant accumulation and rainfall events are described.

Appendix F - "Water Quality Standards": a summary of current water quality standards is presented along with information and problems for interpretation and incorporation of these standards into receiving water quality analysis.

Appendix G - "Best Management Practices": currently available performance and cost data for urban stormwater management practices is summarized and evaluated in light of its applicability in the development of an areawide stormwater abatement program.

Appendix H - "Structural Cost Analysis Models and Procedures": a concise and definitive summary of the capital and O&M costs of all available structural solutions to waste management problems. This appendix also presents a step-by-step methodology for identification, evaluation, and selection of the most cost-effective combination of structural and non-structural control alternatives in urban areas. This Appendix appears as Volume III of this manual.

Appendix I - "Bibliography": an annotated bibliography of publications frequently useful in areawide water quality management planning is presented.

Appendix J - "Glossary of Terms": the glossary is intended to provide a definition of technical terms used throughout the Areawide Assessment Procedures Manual which might not be readily known by the wide user community anticipated.

1.5 Schedule of Mailings

The need for the Areawide Assessment Procedures Manual is an immediate one. Because of the magnitude of the effort, it would be untimely to withhold distribution of critical portions of the manual until the entire document is completed. Consequently, those portions of the manual which the Agency believes are most essential to the majority of designated agencies at this point in the planning cycle (September 1976) will be distributed first. Table 1-1 describes the anticipated contents and timing of subsequent distributions aimed at completing the manual by January 1977.

TABLE 1-1

SCHEDULE OF MAILINGS FOR THE AREAWIDE ASSESSMENT PROCEDURES MANUAL

Mailing Number 1 - September 1976

Volume I: Chapter 1 - Introduction
Chapter 2 - Preliminary Problem Assessment
Chapter 3 - Procedures for the Assessment of Urban Pollutant Sources and Loads

Volume II: Appendix A - Model Applicability Summary
Appendix C - Land Use Data Collection and Analysis

Mailing Number 2 - December 1976

Volume I: Chapter 4 - Procedures for the Assessment of Non-Urban Pollutant Sources and Loadings
Chapter 5 - Analysis of Receiving Water Impacts of Urban and Non-Urban Sources

Volume II: Appendix B - Water Quality Data Bases
Appendix D - Monitoring Requirements, Methods, and Costs
Appendix E - Statistical Analysis Procedures and Methods
Appendix F - Water Quality Standards
Appendix G - Best Management Practices
Appendix I - Bibliography
Appendix J - Glossary of Terms

Mailing Number 3 - January 1977

Volume I: Chapter 6 - Evaluation and Selection of Control Alternatives
Chapter 7 - Examples of Assessment Methodology for Urban and Non-Urban Areas

Volume II: Appendix H - Structural Cost Analysis Models and Procedures

CHAPTER 2

PRELIMINARY PROBLEM ASSESSMENT

2.1 Introduction

In the past, many wastewater and water quality management studies have been concerned with readily identifiable and controllable point sources of wastewater discharge, principally of municipal and industrial origin. The underlying assumption in many of these studies was that the most severe water quality problems, particularly dissolved oxygen effects, were likely to occur during relatively dry, low flow periods where municipal and industrial treatment plant discharges would have a predominant influence on receiving water quality. In most circumstances, water quality effects which could not be directly related to the point sources under study were considered as natural or background effects to be considered as baseline water quality conditions. Wastewater management activities were then concentrated on readily identifiable and controllable point sources to improve water quality to the extent practical. As a result, many of these studies resulted in the development of waste load allocations for municipal and industrial point sources for water quality control during periods of specified low river flow and background water quality.

In certain cases, the results of this type of analysis are satisfactory for effective water quality management. It is recognized, however, that in many areas, overall long term water quality improvement requires consideration of other factors in addition to municipal and industrial point sources control. Even under low flow conditions, the background water quality used as a baseline in the development of point source waste load allocations may be subject to improvement if the sources which control background water quality can be identified and controlled in a cost-effective manner. Further, during periods of the year other than low stream flow, water quality may be impaired by wastewater contributions from a variety of sources in addition to municipal and industrial effects. Discharges from intermittent point sources such as combined sewer overflows, stormwater drainage, urban runoff, and numerous non-urban non-point sources may all contribute to the degradation of a number of water quality variables in the receiving water in differing proportions at various times due to the yearly hydrological cycle. The analysis and planning problem is compounded by future development activities which are likely to occur which may reduce the effect of some water quality influences and/or intensify and redistribute others.

Under Section 208 of Public Law 92-500 support is provided for the engineering and planning evaluation of such problems in complex urban and industrial settings. The principal purpose of 208 studies is the local development of cost-effective areawide wastewater management plans for initial and longer term water quality control in a framework suitable for modification in a continuing planning process. A broad scale of water quality control options are to be considered with specific evaluation of the effects of municipal and industrial waste, combined sewer overflow and stormwater runoff, and non-point sources from various land usage categories. The wastewater management plan is to be cost-effective and practical; it must focus on principal problems first, provide a procedure to resolve remaining problems in time; and it should include non-structural controls where possible.

In order to produce a quality plan in a complex 208 area, the array of tasks facing the planner can be formidable. A variety of water quality variables and goals may have to be considered throughout a yearly cycle. Numerous point source wastewater discharges, intermittent sources, and non-point sources may have to be assessed, in many cases with little or no direct data. A multiplicity of techniques are available by which to estimate intermittent and non-point source pollutional discharges, each with specific advantages and disadvantages. Receiving waters can differ markedly in complexity from relatively simple streams and rivers to more complex estuaries, embayments, lakes, and coastal zones, each requiring specific methodologies for analysis. The water quality problems may have differing time and space scales from localized short term bacterial problems, to large spatial scale, seasonal time scales characteristic of eutrophication of receiving waters. A wide variety of engineering and management control options may be available for consideration, each with unique levels of effectiveness, cost, and operational reliability. Therefore, the immediate problem facing the 208 planner is to define the appropriate technical steps and procedures which are to be considered in order to evaluate a multiplicity of problems within the time and resources available and yet which will provide adequate technical information for wastewater management planning.

The initial step in the development of a 208 water quality management plan is associated with preliminary problem identification and assessment. The purpose is to provide the 208 planner in the initial phase of the project as broad a view as possible of the water quality problems, the relative magnitude of various waste sources, and the probable impact of the various sources on water quality. With this perspective, appropriate technical procedures can then be selected to focus on the most important waste sources and water quality problems, and the specific additional data needs can be defined. The more detailed technical procedures which are appropriate for further analysis of those waste load sources which are important in a planning area, are described in Chapter 3 for urban areas, and in Chapter 4 for non-urban areas.

Chapter 2 illustrates the basic data and procedures necessary to perform a preliminary problem assessment. The chapter will provide:

1. An identification and description of the basic data and data sources necessary for the preliminary assessment.

2. Methods to identify and estimate the magnitude of urban and non-urban waste sources on an annual average basis, and during selected critical periods.

3. A method of ranking the importance of the various wastewater sources during different points in the hydrologic cycle by an

analysis of the impact on receiving water quality for a number of key variables.

Chapter 2 is structured into several technical subsections as follows:

Water Quality Problems and Standards - The definition of various water quality problems which may be encountered in local areas and common characteristics of water quality standards which will be relevant.

Characteristics of the Planning Area - A description of the fundamental information necessary for preliminary problem assessment including quantification of point source waste discharges, land use categorization, hydrology and receiving water characteristics.

Water Quality Data Base - A description of the types and sources of water quality data which may be useful to identify water quality problems and determine waste load impacts.

Waste Source Identification and Evaluation - A discussion of the procedures to define or estimate the magnitude of various waste sources including continuous municipal and industrial point sources, intermittent urban loadings from combined sewer overflows and storm drainage, and non-urban non-point source for a variety of land uses.

Receiving Water Analysis - The categories and a discussion of various types of receiving waters including streams and rivers, estuaries, coastal embayments, and lakes and the general characteristics of mathematical water quality modeling techniques useful for determination of water quality impacts.

Illustration and Interpretation of Impact Analysis - A presentation of a simplified technique, by use of an example, to illustrate how existing or potential water quality problems can be identified in streams or rivers and how various waste sources can be ranked in order of importance for specific periods in the hydrologic cycle by determination of water quality impact. The guidance is presented for the proper interpretation of the results of preliminary problem assessments so that appropriate methodologies and procedures can be selected for inclusion in the detailed work plan.

Much of the data and procedures discussed in Chapter 2 are of a general character and will be applicable to many local problem settings. However, in order to further explain the utility of this information, an example problem is included in several of the subsections to illustrate the practical application for a particular problem setting. The importance of this sample problem should not be minimized. It is designed both to instruct and to illustrate the preliminary assessment methodology in sufficient detail to enable the reader to perform the analysis for his specific problem area.

2.2 Water Quality Problems and Standards

The water quality of natural water bodies which receive waste loads from point and non-point sources can be degraded resulting in the impairment of beneficial uses. Point and non-point pollution can interfere with man's uses of the water body for recreation and for water supply. In addition, pollution can upset the natural biology of the system. In general, problems encountered in natural water bodies can be classified into the following major catagories: dissolved oxygen depletion; public health risks, eutrophication, and a general category which combines other water use interferences, including siltation and aesthetic considerations.

2.2.1 Water Quality Problems

2.2.1.1 Dissolved Oxygen Depletion

The quantity of dissolved oxygen present provides an overall measure of the general well-being of a receiving water. Non-polluted streams are characterized by dissolved oxygen levels near the atmospheric saturation concentration and exhibit healthy and diverse biological communities. In areas of a water body adjacent to wastewater outfalls of point and non-point waste discharges, less desirable scavanging organisms increase in number while dissolved oxygen is reduced below saturation or may even be completely exhausted. Sedimentation of the suspended portion of the organic matter results in bottom deposits which continue to degrade and thereby remove dissolved oxygen from the overlying waters. Excessive nutrient enrichment of the waters may result in the development of substantial algal or macrophyte biomass with their associated large diurnal fluctuations in the dissolved oxygen.

These effects of point and non-point sources combine to affect the dissolved oxygen content of the receiving water. Thus, the dissolved oxygen concentration of a water body is a valuable indicator of the state of the receiving water.

2.2.1.2 Public Health

There are many aspects to the relationship between waste loads, receiving water quality and public health considerations. For the purposes of this assessment manual, a limited set of potential contributors to public health problems have been considered.

The presence of infectious organisms and toxic substances creates potential health hazards which can severely limit the intended uses of the receiving water. Potable water supply, recreational uses and shellfish beds are among the beneficial uses which may be affected.

Coliform bacteria are generally used as indicator organisms for the possible presence of pathogens. Total and fecal coliform bacteria are the most commonly used indicators. Of the two indicators, fecal coliforms are the more reliable, since they originate in warm-blooded mammals. While less common in data bases than total coliform counts, fecal coliform data should be used whenever possible.

Although the presence of coliforms in receiving waters is the dominant public health concern in the preliminary analysis, it should also be recognized that heavy metals, pesticides and refractory materials may enter the receiving water from municipal and industrial pollution sources or from urban runoff. Pesticides may also originate from application to crops in agricultural areas. These materials may accumulate to harmful levels in the food chain, and should be considered in the context of areawide water quality analyses.

2.2.1.3 Eutrophication

The combination of excessive nutrients, suitable water temperature and adequate sunlight may cause excessive production of algae and higher plant life in natural waters. Problems associated with excessive algal growth may include objectionable taste and odors in water supplies and interference with water treatment operations. In addition, excessive growth of water weeds may reduce the hydraulic capacity of natural conduits, cause flooding of lowlands, and generally obstruct desired uses. They may interfere with recreation by creating conditions which interfere with the attractiveness or usefulness of a water body. As pointed out previously, large diurnal fluctuations in dissolved oxygen may occur as the photosynthetic or respiratory activity dominates.

2.2.1.4 Other Water Use Interferences

The more important contaminants included in this category are suspended, floatable, and dissolved solids and solid particulates whose presence may be harmful in themselves or serve as a transport mechanism for sorbed pollutants. Suspended solids borne by the water may settle out in impoundments and reduce storage capacity. Excessive levels may cause destruction of fish life or benthic organisms. Stream flow used for irrigation may build up high concentrations of total dissolved solids. This results in an economic burden on downstream agricultural areas where crop yields may be severely reduced. Finally, floatable solids are undesirable in any natural watercourse.

In addition to the above, in some situations other parameters such as temperature, pH, oil and grease, and specific ions in excessive concentration sodium, chloride, etc., may also have to be considered.

For this preliminary analysis, the suspended solids are chosen since waste source and receiving water data are generally available and the analysis framework is simple. Total dissolved solids analyses are more complex and are treated in a subsequent chapter.

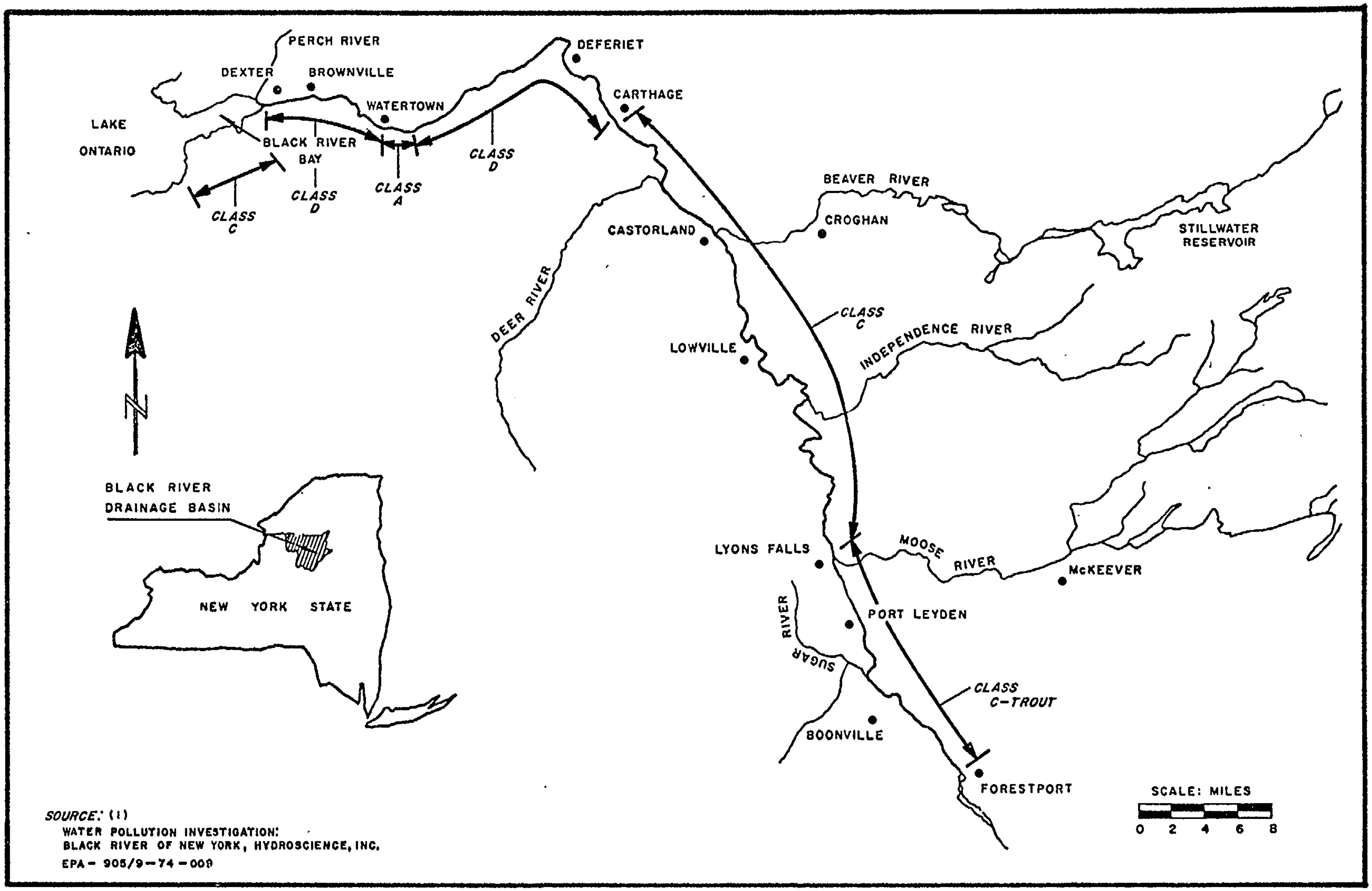
2.2.2 Water Quality Standards

Both the water quality constituents regulated by the standards and the allowable concentration of those constituents are not the same from state to state. In addition, some local agencies may impose water quality regulations which are more stringent than those imposed by the State governments. However, the physical properties and chemical parameters regulated by water quality standards are established for their relationship to the well-being of the water body and the beneficial uses which can be derived from its use.

The water quality standards are obtained at the start of the planning study directly from the state agencies. Usually, the state agency publishes a document which defines the water quality criteria and classifies each surface water body.

After the water quality classification for each water body in the 208 study area is obtained from the State, the classifications for the particular water bodies can then be superimposed on a study area map. For example, the 97 miles of the Black River in New York State are shown in Figure 2-1 along with the water quality classifications. Figure 2-1 shows that the Black River and Black River Bay are divided into 6 water quality segments. The water quality classifications that apply to these segments are A, C, C-trout, and D. Water usage in these stream reaches is intended for water supply (Class A), fishing (Class C), fishing-trout (Class C-trout), and secondary contact recreation (Class D).

Quantitative standards for these classifications are given for pH, dissolved oxygen, dissolved solids and coliform bacteria. The standards are listed in Table 2-1 for Class A, B, C and D waters. In addition to the above water quality standards, New York State policy for the Lake Ontario basin requires wastewater dischargers of 1.0 MGD or larger to reduce effluent phosphorus to 1.0 mg/l or less.



SOURCE: (1)
WATER POLLUTION INVESTIGATION:
BLACK RIVER OF NEW YORK, HYDROSCIENCE, INC.
EPA- 905/9-74-009

FIGURE 2-1
WATER QUALITY CLASSIFICATIONS OF THE BLACK RIVER

TABLE 2-1

NEW YORK STATE WATER QUALITY STANDARDS

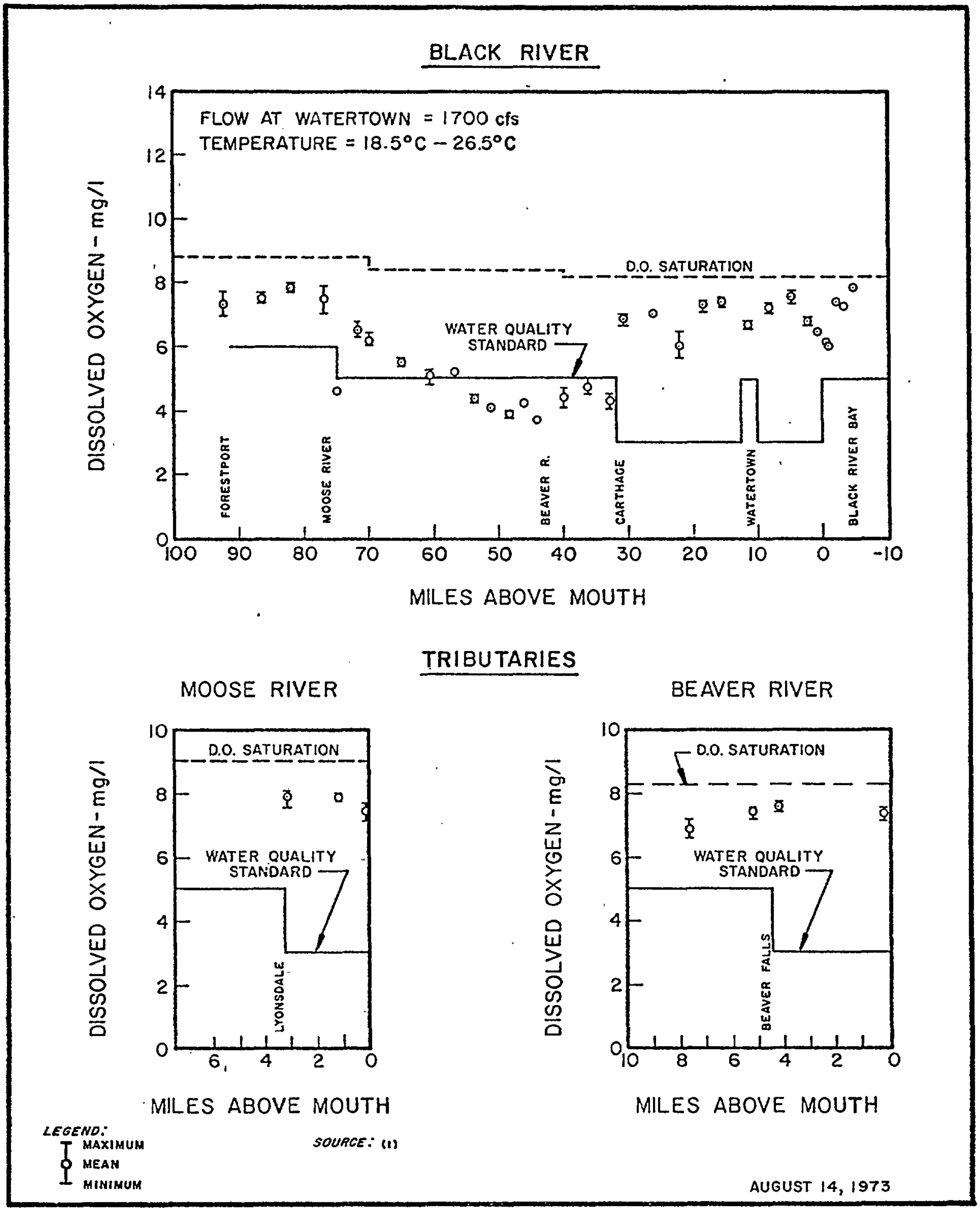
Constituent	Class A	Class B	Class C	Class D
pH	6.5 - 8.5	6.5 - 8.5	6.5 - 8.5	6.0 - 9.5
Dissolved Oxygen - mg/1 (Minimum Daily Avg)	5.0	5.0	5.0 6.0 (Trout)	Never less than 3.0
Dissolved Solids - mg/1 (Maximum)	500	500	500	-
Total Coliform - No./100 ml	Monthly Median 5,000	Monthly Median 2,400	Monthly Geom. Mean - 10,000	-
Fecal Coliform - No./100 ml	Monthly Geom. Mean - 200	Monthly Geom. Mean - 200	Monthly Geom. Mean - 2,000	-
Phenolic Compounds - mg/1	0.005	-	-	-

Source (1)

The standards for dissolved oxygen and coliform bacteria presented in Table 2-1 are for average conditions. For dissolved oxygen, the minimum daily average is reported. For the A, B and C classifications the specifications further stipulate that at no time shall the dissolved oxygen concentration be less than 4.0 mg/l for non-trout waters and no less than 5.0 mg/l for trout waters. In addition to specifying average conditions, the total and fecal coliform standards also include the minimum number of analyses required plus the maximum total coliform counts permitted in 20 percent of the samples.

The water quality classifications and standards can also be shown on the spatial water quality plots to assist in data interpretation. Figure 2-2 shows the spatial dissolved oxygen data observed on August 14, 1973 in the Black River and two of its tributaries. The dissolved oxygen standards are shown on each of the water quality plots. The extent of the six water quality segments and the dissolved oxygen standard for each segment can be readily compared to the observed data in this figure. It is evident from Figure 2-2, that during August of 1973, the dissolved oxygen concentrations were depressed below the standards for 25 miles of the Black River. In the two tributaries, the observed dissolved oxygen levels were well above the dissolved oxygen standards. Similar plots for the other water quality substances showed that acceptable levels of the constituents were present in August.

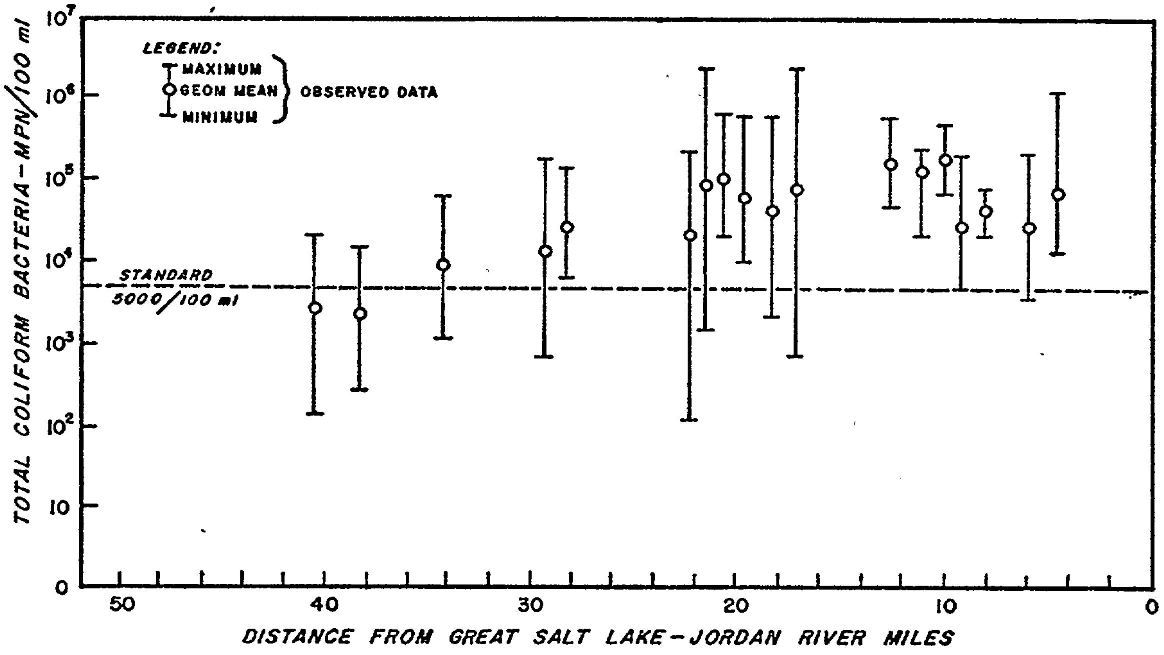
Figure 2-3, is an example for the Jordan River in Utah, which indicates a violation of the total coliform water quality standard. These data are average values of all the STORET summer total coliform data. The total coliform standard is 5,000/100 ml for the entire 52 miles of the river. The comparison of the observed data to the total coliform standard shows that the coliform standard is violated on an average basis for more than 30 miles of the Jordan River.



SOURCE: (1)

AUGUST 14, 1973

FIGURE 2-2
DISSOLVED OXYGEN DATA AND STANDARDS
(BLACK RIVER, N.Y.)



SOURCE: (2)

FIGURE 2-3
INSTREAM TOTAL COLIFORM DATA
(JORDAN RIVER, UTAH)

2.3 Characteristics of the Planning Area

2.3.1 General Description

The 208 planning area study limits are defined at the start of the project and generally conform to political boundaries. However, in the evaluation of all factors that affect water quality within the designated 208 planning area, it may be necessary to look beyond the political boundaries. For example, water quality in rivers is directly related to land use in the entire river basin which might not be entirely incorporated within the designated 208 planning area. In estuaries, water quality at a particular location is also affected by conditions a considerable distance downstream because of tidally induced mixing. Therefore, wastewater loads downstream of a 208 study area might significantly affect upstream water quality.

In the discussions which follow, Figure 2-4 presents a map of the hypothetical 208 planning area for the illustrative problem. The 208 study area boundary is indicated by the dashed line and the drainage basin boundary is defined by the solid line. As shown, the section of the South River within the 208 study area receives runoff from land outside the 208 boundaries. At the upstream end of the study area, near Route 80, the South River water quality is related to point and non-point source loads that enter the river between the headwaters and Route 80. From this example, it can be seen that the general description of the study area should be expanded beyond the limits defined by the designated 208 planning area. It should be emphasized that the only purpose for expanding the extent of the original study area is to better understand those factors that influence water quality within the original designated 208 area.

2.3.2 Land Use

The type and quantity of non-point source loads depend on land use. Land use can be divided into urban and non-urban areas. Generally, the urban land uses are subdivided into residential, industrial, commercial and open areas. The non-urban land uses are divided into areas of

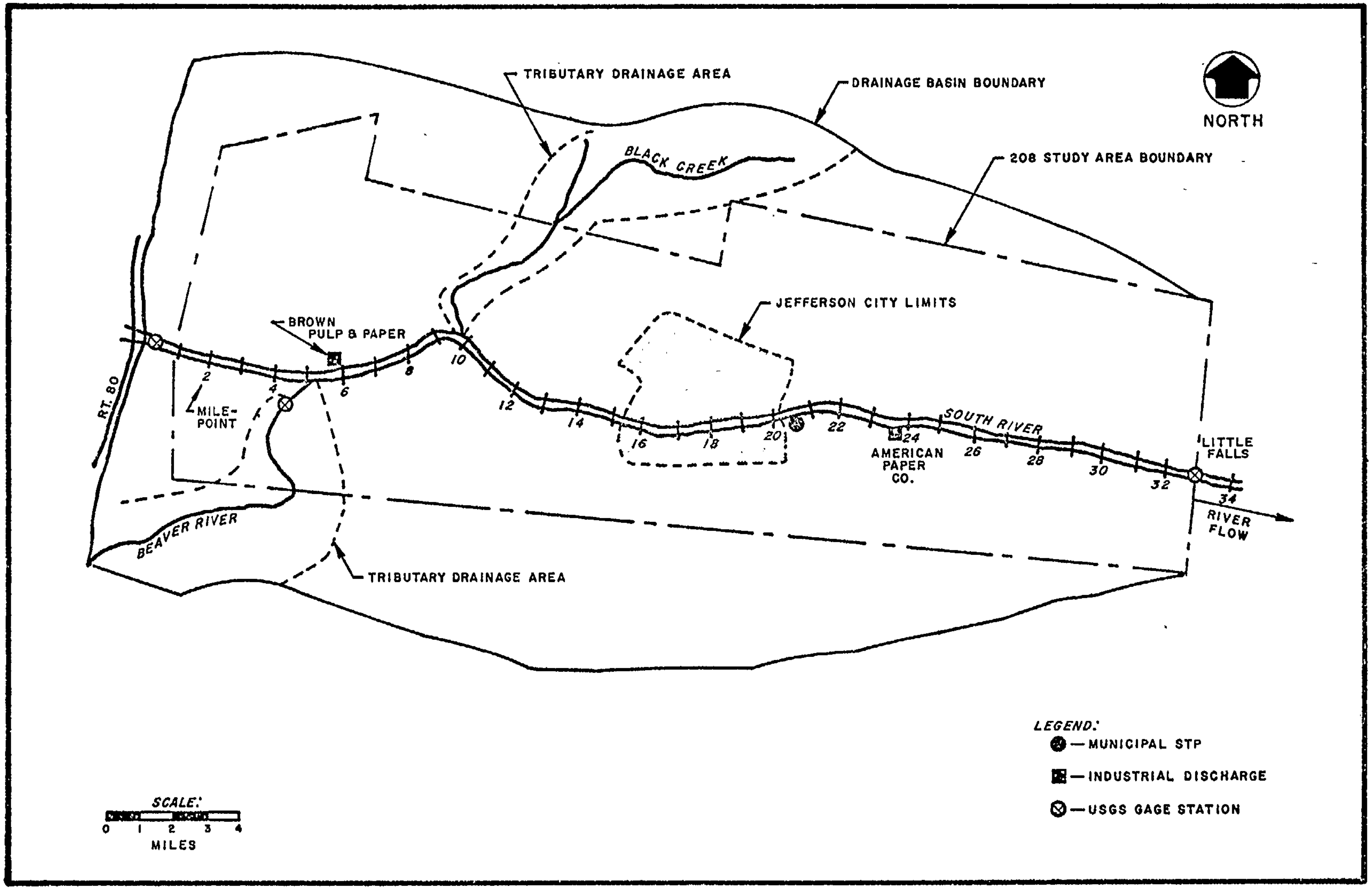


FIGURE 2-4
MAP OF HYPOTHETICAL PLANNING AREA
SOUTH RIVER, USA

agriculture, forest, silviculture, mining, and feedlots. In addition, construction and highway activities in both the urban and non-urban areas generate non-point source (NPS) loads. Residual management practices, such as waste disposal in sanitary landfills, may also be substantial sources of non-point source pollution. In addition to classifying a general land use pattern within the study area, further details concerning land use may be required for the estimation of non-point source loads. For example, soil type and other factors affecting erosion are used in some empirical formulas to compute non-point source loads from open areas. In urban areas, population density can be used in some estimating techniques for calculating storm runoff. In addition, maps detailing pervious and impervious areas, together with handbook relationships can also be useful.

If land use management controls are not practiced, each land use activity has the potential of adding to the degradation of the local and distant waterways. In order to estimate the NPS loads for each major constituent, it is first necessary to determine the distribution of the land use activities in the study area and the areal extent of each land use activity.

Land use data in any 208 study area is available from many sources. These are described in Appendix C of this manual, Land Use Data Collection and Analysis. In general, however, the list would include:

a. Local planning agencies
b. State planning agencies
c. Standard Metropolitan Statistical Area (SMSA) Data
d. H.U.D.
e. Previous basin plans or facilities plans
f. Soil Conservation Service

The calculation of NPS loadings from land use activity and population density will be discussed in more detail in Chapters 3 and 4. As a first step in preliminary problem identification, the definition of general land use patterns is sufficient.

Figure 2-5 presents the general land use pattern for the hypothetical South River 208 study area. As indicated, most of the land bordering the river is agricultural. Forested lands are located at the northern boundary of the drainage area and at the southeast and southwest corners. The urban areas, Jefferson City, is located in the middle of the study area.

The drainage area distribution by land use at five points along the South River is summarized in Table 2-2 and graphically presented in Figure 2-6. At Milepoint 0, the Route 80 Bridge, the drainage area is 500 square miles. For the 208 study, the land use of the upstream point and non-point source loads is defined by water quality measurements at Milepoint 0. At Milepoint 5, the runoff from the Beaver River drainage basin (250 square miles) enters the South River. As with the upstream drainage area, the impact of loads within the Beaver River basin is defined from water quality measurements at the mouth of Beaver River. The increase in drainage area between Milepoints 5 and 33 is agriculture plus forest land with 20 square miles of urban drainage area between Milepoints 15 and 20. Since the flow in Block Creek is minor, the drainage area of the creek is included in the analysis as part of the NPS forest area.

The graphical presentation of the drainage area distribution clearly demonstrates that areas outside of the 208 study area boundaries may significantly influence water quality within the 208 study area. Most of the drainage basin of the Beaver River and all of the upstream drainage area are outside of the 208 study area yet they compose approximately 60% of the total drainage area at Milepoint 33. Figure 206 also graphically shows that the urban area is a relatively small component of the total drainage area.

An analysis of the land use similar to the above should be performed by the 208 planning agency.

2.3.3 Hydrological Data Bases

The local hydrological cycle affects water quality in the drainage basin. The runoff, which depends on the geophysical characteristics of

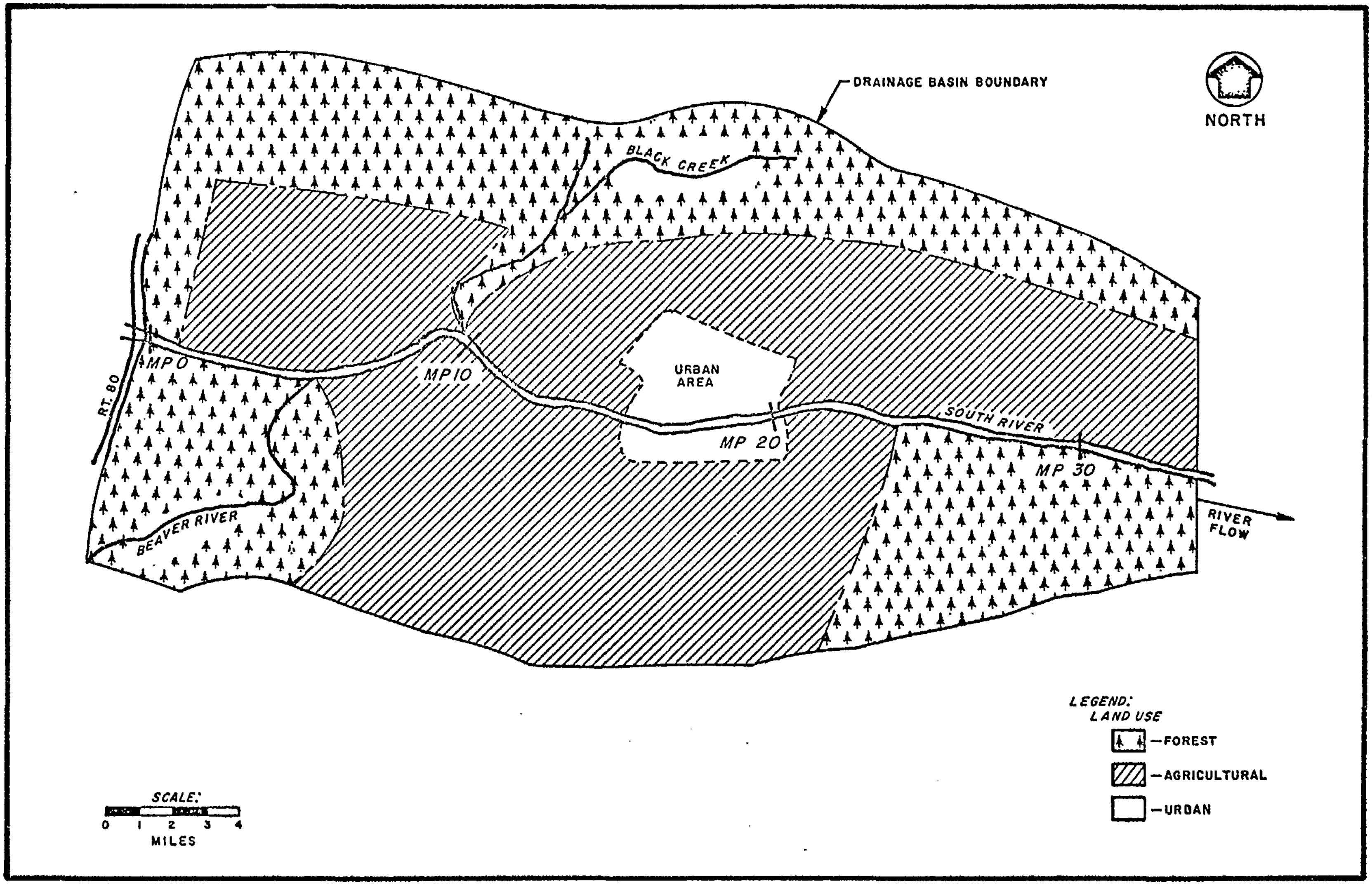


FIGURE 2-5
SIMPLIFIED LAND USE PATTERNS
(SOUTH RIVER, U.S.A.)
HYPOTHETICAL EXAMPLE

TABLE 2-2

DRAINAGE AREA DISTRIBUTION BY LAND USE
(SQUARE MILES)
FOR SOUTH RIVER HYPOTHETICAL EXAMPLE

Milepoint	Upstream	Agriculture	Forest	Tributary	Urban
0	500	0	0	0	0
5	500	30	40	250	0
15	500	160	65	250	0
20	500	210	75	250	20
33	500	280	150	250	20

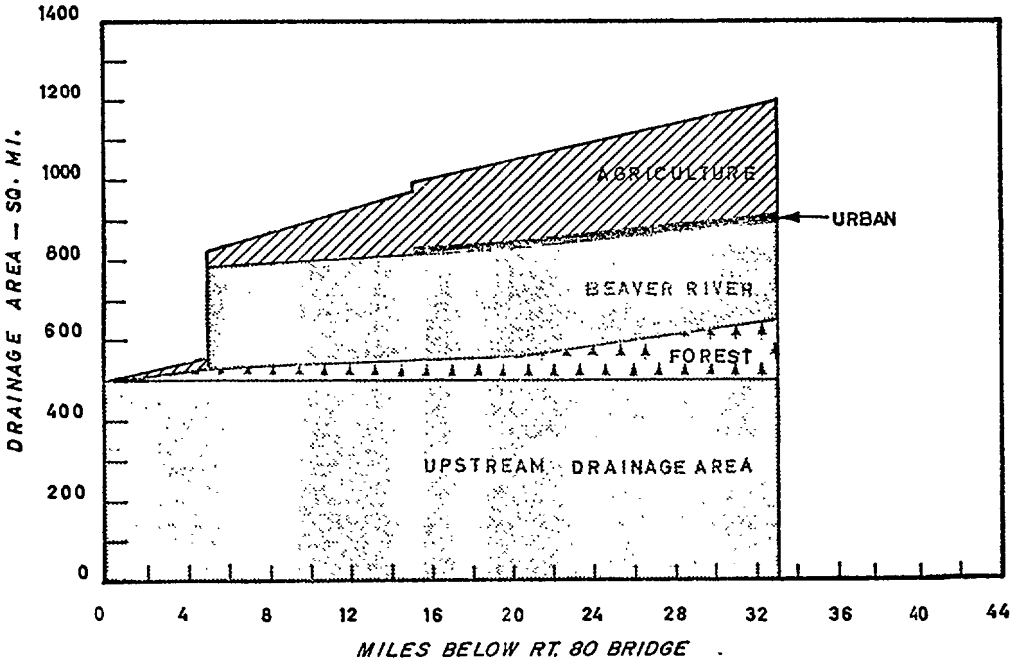


FIGURE 2-6
DRAINAGE AREA DISTRIBUTION BY LAND USE
(SOUTH RIVER, U.S.A.)
HYPOTHETICAL EXAMPLE

the drainage basin, the land uses and control practices, and the amount, frequency, and intensity of rainfall, may improve or degrade water quality.

In order to assess the impact of point and NPS loads on a water body, it is important to know the stream flow and the stream flow patterns. In any one drainage basin, stream flow will vary over the year depending on the rainfall and/or snowmelt. Figure 2-7 shows average monthly distribution of runoff as percent of total runoff over the year for 16 river basins in the country. 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 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 late summer or early fall.

Because of the variation in runoff across the country, it is necessary to collect the 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 2-3(a) is a sample data sheet from the U.S.G.S. surface water records. The data sheet summarizes drainage area, daily flows for the year, monthly average flows, and monthly maximum and minimum flows at a gaging station.

For the preliminary analysis, stream flow data is reduced to monthly average flows for the number of years of record available. The annual average stream flow is calculated in addition to the minimum average 7-consecutive-day, one-in-ten-year low flow. All tributaries of significant size are located with respect to the main channel. If flow information is not available for the tributaries, then estimates of the annual average flow and the seasonal patterns are made by assigning the same runoff yield (cfs/sq. mile) as the major drainage basin as discussed subsequently. After the stream flow data in the main stem and the

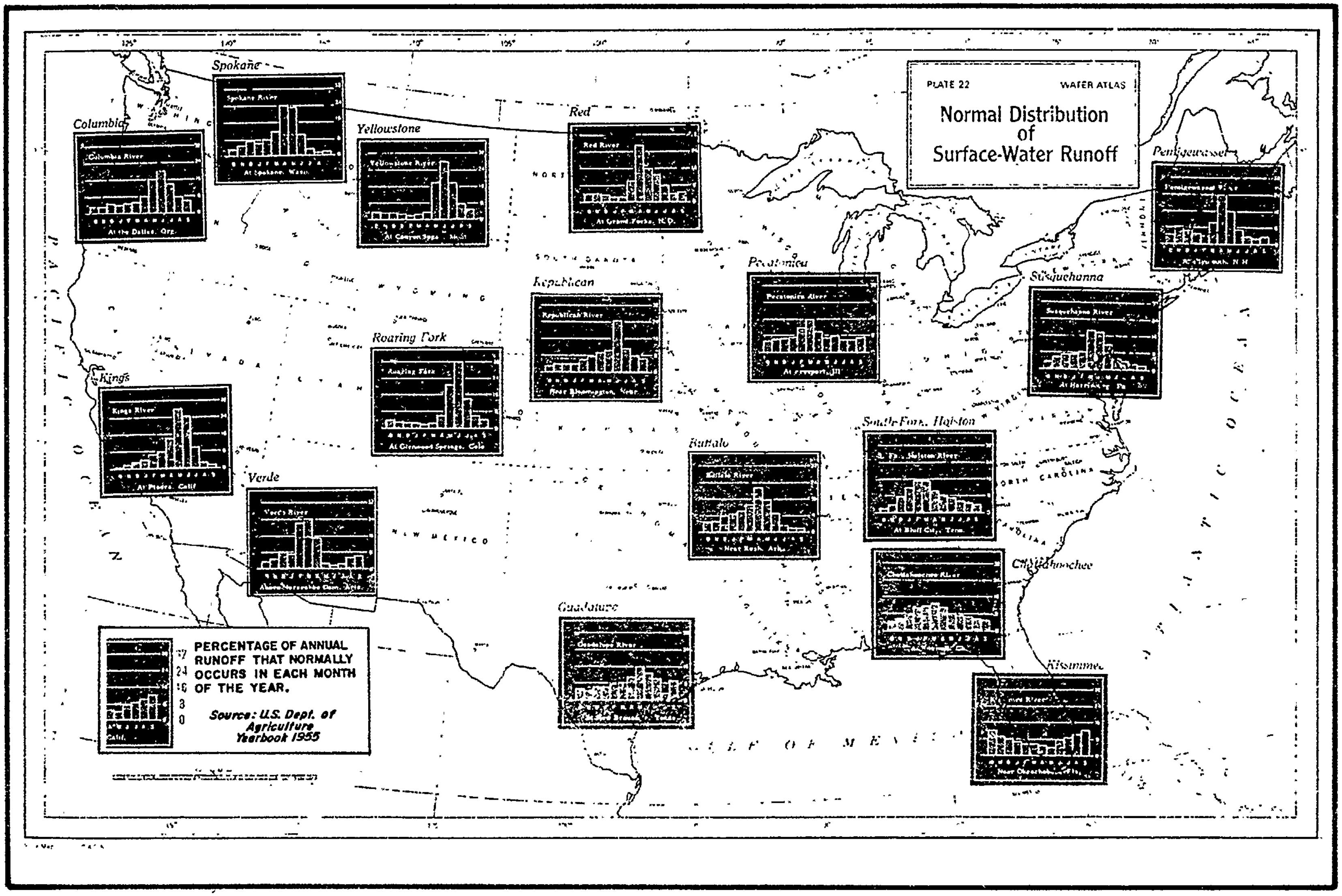
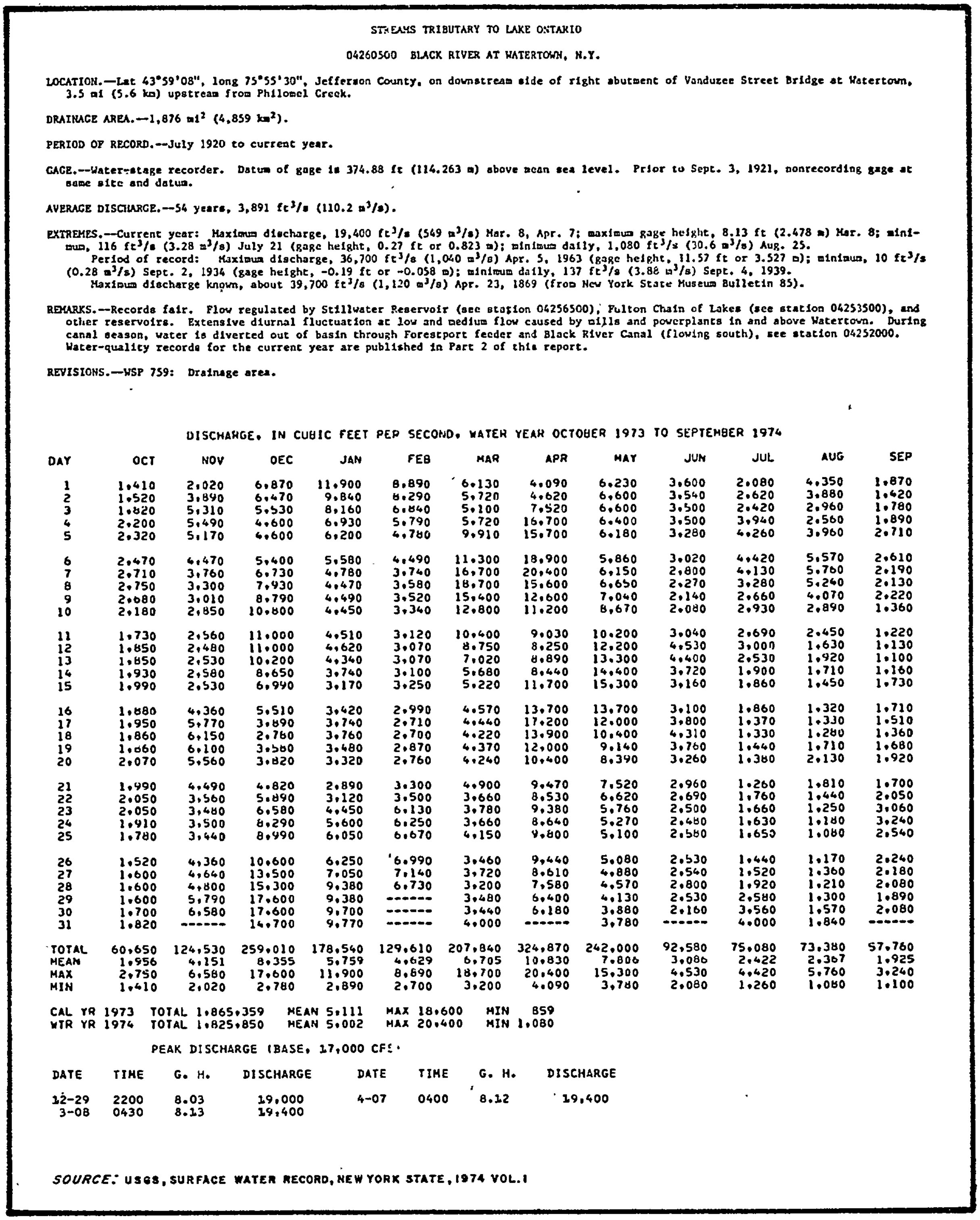


FIGURE 2-7

NORMAL DISTRIBUTION OF SURFACE-WATER RUNOFF



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 0.823 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	1,820	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	2,760	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 2-3(a)

SAMPLE U.S.G.S. SURFACE WATER RECORD DATA SHEET tributaries are established, these data are plotted with respect to river miles. This information will be used in the preliminary water quality assessment to estimate the dilution and transport of the wastewaters which enter the stream.

For example, the hypothetical South River average monthly flows are plotted in Figure 2-8 for gaging stations at the upstream and the downstream end of the study area. The spatial increase in flow between the Route 80 Bridge gage and Little Falls gage is represented by the difference between the solid and dashed lines in the hydrograph. The freshwater flow originating in the Beaver River drainage basin is also shown in Figure 2-8.

The spatial distribution of the freshwater flow in the South River drainage basin is presented in Figure 2-9 . The average annual, summer and low flow profiles are also plotted in this figure. At Milepoints 5 and 10, the flow is incremented by the Beaver River and Black Creek, respectively. The linear increase in flow between Milepoints 0.0 and 33 is due to the surface and groundwater return flows along the length of the river. The above example illustrates the presentation and reduction of the hydrological data for 208 planning areas.

In addition to computing flow statistics and spatial flow distributions, the total stream flow can be broken down into its components which are the flow inputs from groundwater, tributaries, surface returns, continuous point sources such as wastewater treatment plants, and sewer wet and dry weather overflows. Tributary flow data will generally be available from the U.S.G.S. or it can be generated from local runoff rates and the tributary drainage area as subsequently discussed. Wastewater treatment plant flows and other continuous point sources are available from the treatment plant records, the 201 and 303(e) basin plans and the city drawings. An average estimate of the groundwater input can be obtained from the annual hydrograph. For example, Figure 2-10 shows the hydrograph over 4 years of record for Big Cottonwood Creek in Salt Lake County, Utah. On the hydrograph, the groundwater flow is estimated as the flow occurring during periods of little rainfall or snowmelt. The surface

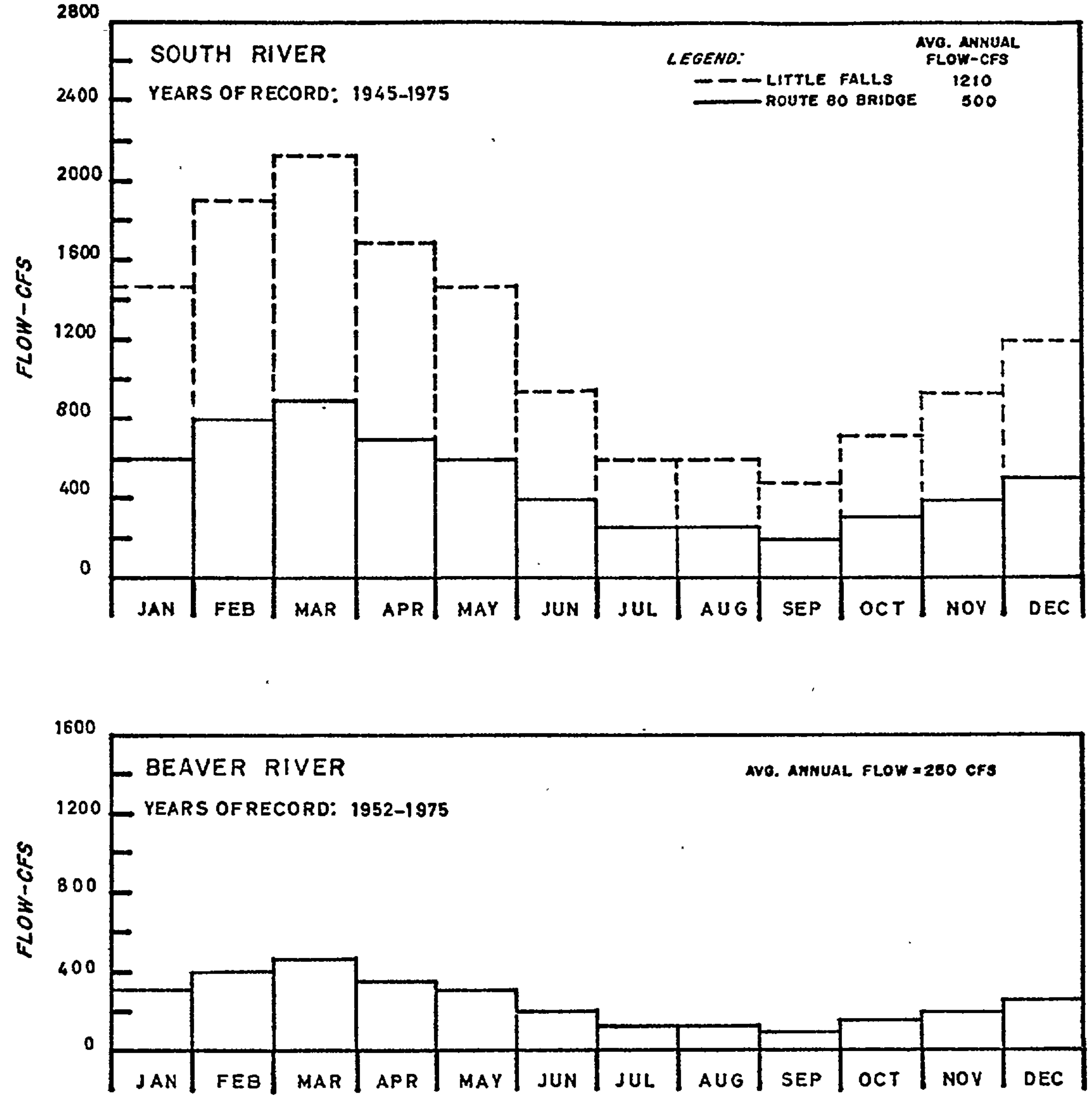


FIGURE 2-8
AVERAGE MONTHLY FLOW, DATA
(HYPOTHETICAL SOUTH AND BEAVER RIVER, U.S.A.)

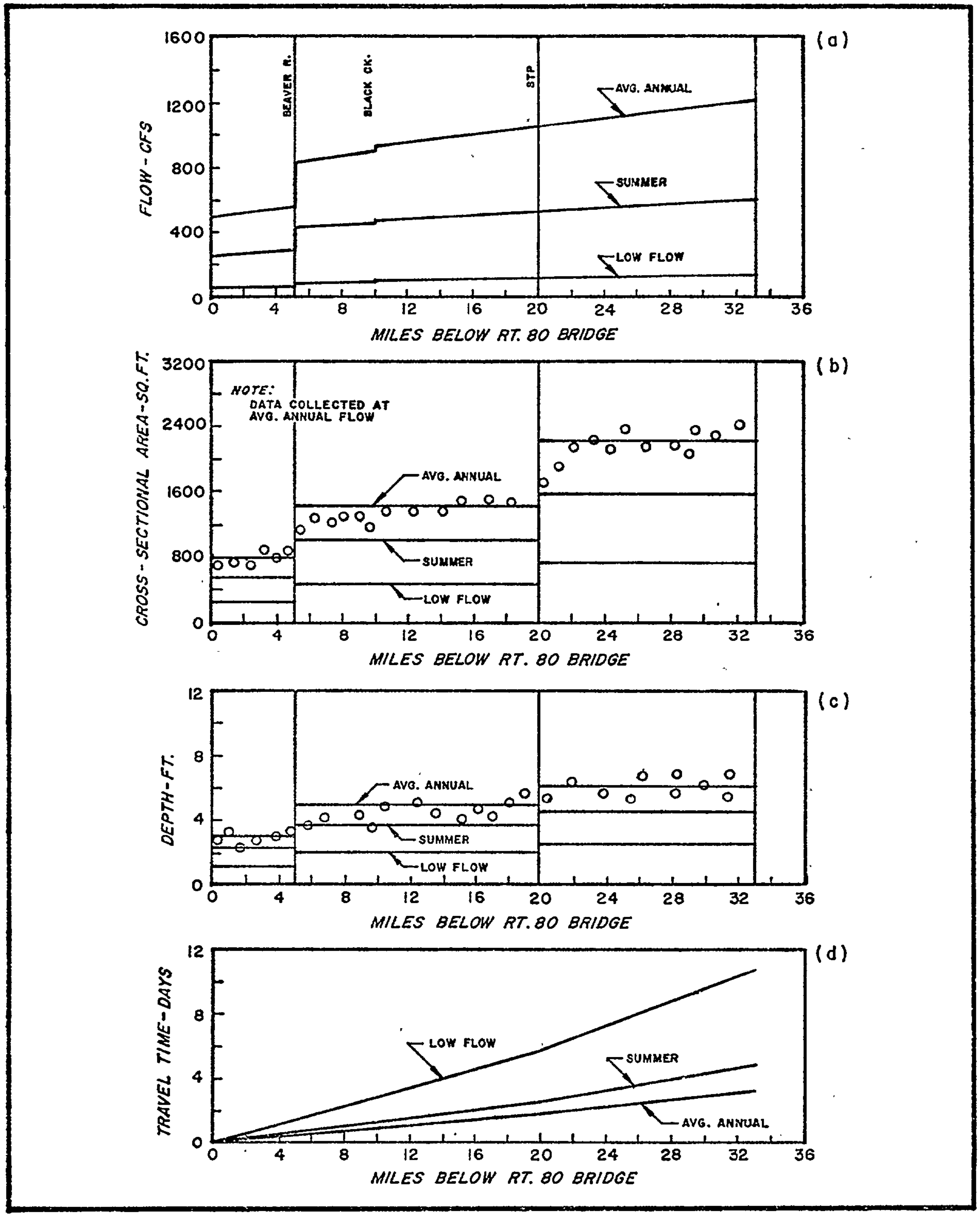
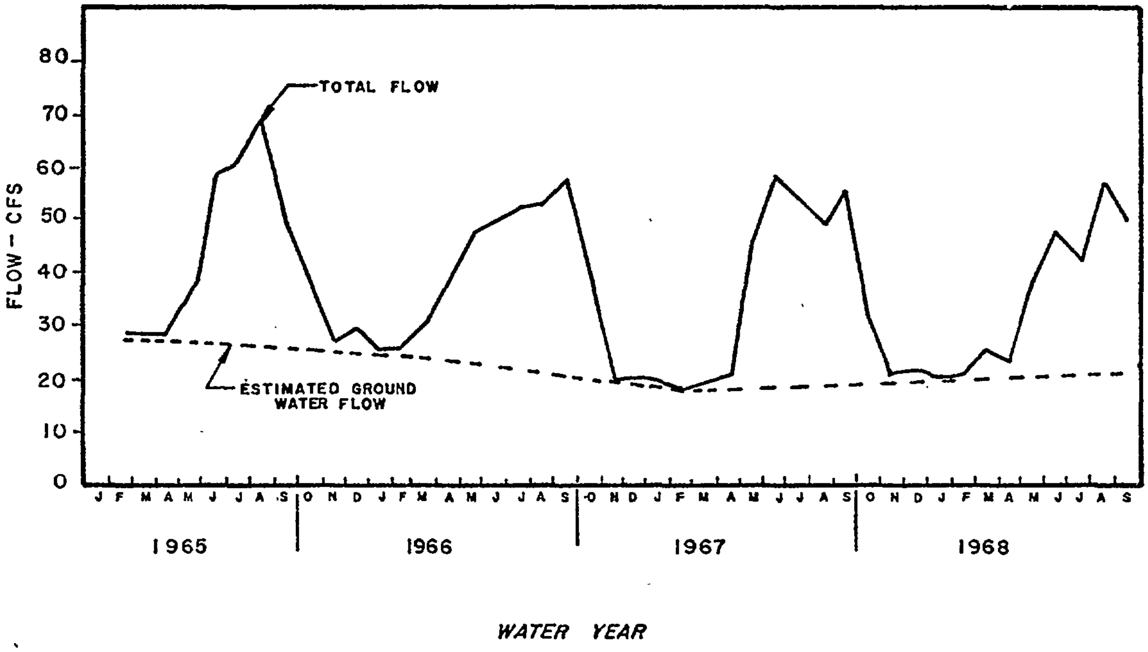


FIGURE 2-9

RIVER FLOW, GEOMETRY AND TIME OF TRAVEL DATA
(HYPOTHETICAL SOUTH RIVER, U.S.A.)



SOURCE: (2)

FIGURE 2-10
TOTAL FLOW AND ESTIMATED GROUND WATER FLOW
BIG COTTONWOOD CREEK, UTAH

runoff is the remaining unknown and it can be assigned as the difference between the total flow and the known components.

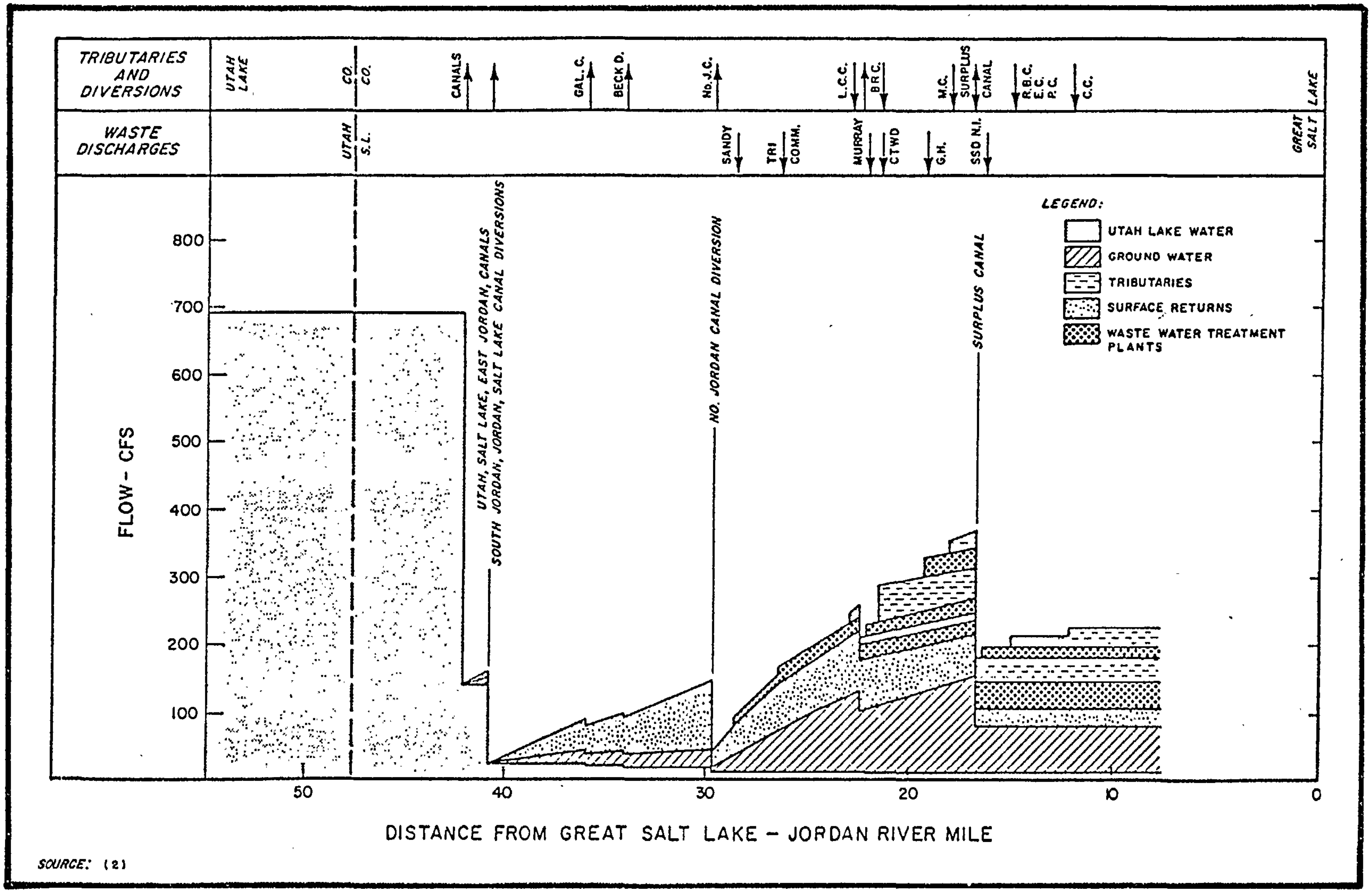
The components of the flow at many locations in the river can be assembled into a spatial flow distribution. In Figure 2-11, the component flows are developed for the Jordan River during a typical summer flow period. The flow in the Jordan River is a combination Utah Lake water, groundwater, tributary flow, agricultural surface returns, and wastewater flow.

For some streams U.S.G.S. gaging records do not exist. In order to determine the stream flow for a refined and definitive impact analysis, stream flow monitoring may have to be instituted. However, crude estimates of average stream flows can be made in order to continue the preliminary impact analysis. Average runoff yields can be used and applied to the specific drainage areas to estimate average stream flows. Figure 2-12(a) shows the average annual runoff yield distribution for the entire country. Preliminary high and low flow runoff estimates can be made with the use of the monthly runoff distribution presented in Figure 2-7 and Figure 2-12(a).

In tidal rivers and estuaries, the freshwater flow at any location can be estimated by adding the freshwater flows orginating from tributaries, municipal and industrial point sources, surface inflow and groundwater inflow. The freshwater flow in tidal rivers and estuaries produces a freshwater velocity as it does in streams. However, the freshwater velocity in tidal waters is generally small compared to the 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 will be important parts of the water quality impact analysis (Chapter 5).

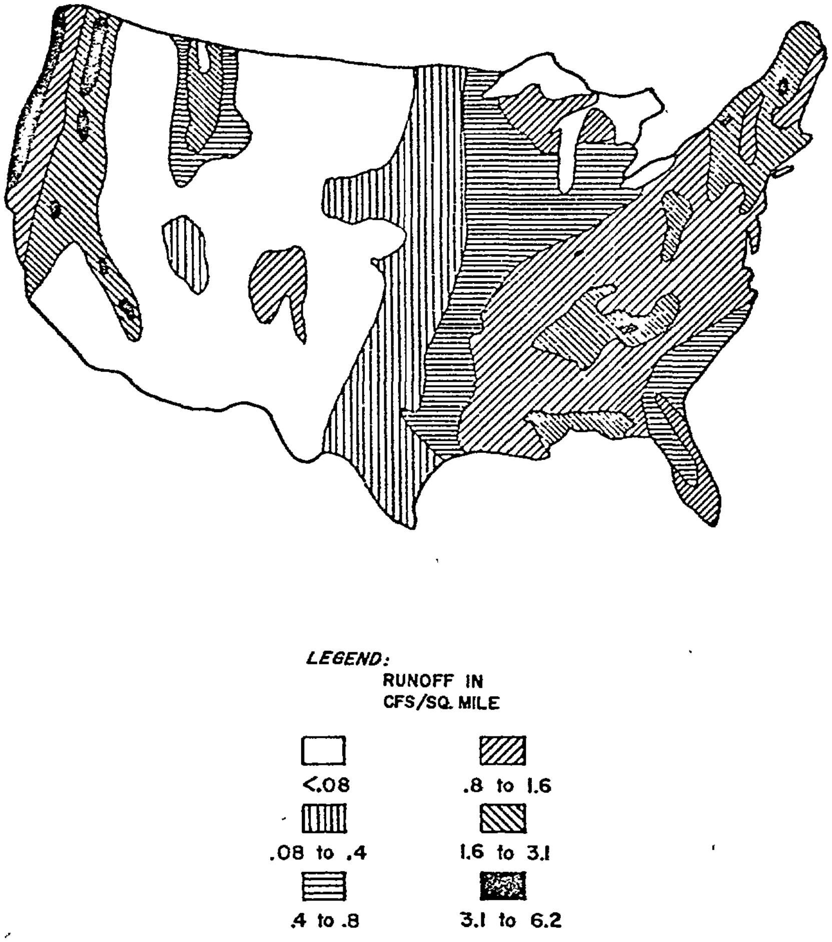
2.3.4 Topography

The topography of the study area is the slope and elevation of the land. This information is readily obtained from maps published by the U.S.G.S.. For local areas, city and state agencies can provide additional maps. In addition to providing an overall representation of the land contours within the study area, topographical maps may also be used to define the



SOURCE: (2)

FIGURE 2-11
COMPONENTS OF A TYPICAL SUMMER FLOW DISTRIBUTION
(JORDAN RIVER, UTAH)



REF. U.S.DEPT. OF INTERIOR

FIGURE 2-12(a)

ANNUAL RUNOFF YIELDS IN THE COTERMINOUS UNITED STATES drainage area of the entire river basin and also the drainage area of major tributaries. For many rivers, the U.S.G.S. reports the drainage area with their annual flow record publications. In the preliminary analysis, drainage area and runoff information can be used to estimate river flows as discussed subsequently.

Topographic data can also provide insight into the quantity and quality of surface runoff within the study area. Areas of high elevation generally have more rainfall than valley regions, due to orographic effects on the windward side. Hence, more rainfall can be expected from mountainous areas of the study area. Land slope also affects the relationship between runoff and rainfall. Steeper slopes produce more runoff per unit intensity of rainfall. Land areas with steeper slopes also have higher erosion rates and, consequently, higher suspended solids and associated pollutants in surface runoff. Appendix C provides a detailed discussion of the available maps and other information.

2.3.5 Geomorphological Information

Stream channel depth and cross-sectional area data are required for the main receiving water body. This information may be available from the U.S.G.S., State agencies, EPA, Coast and Geodetic Survey or previous studies. If no depth or cross-sectional area data are available, it is usually necessary to make in-stream measurements during the 208 study.

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 is used in the impact analysis for computing average velocity. The time of passage data and the channel cross-sectional area data should be related to the stream's flow. If enough data are not available to establish these relationships, then equations (2-1) to (2-4) can be used to estimate the changes in river characteristics for a different flow regime. They relate the changes in these characteristics to changes in the flow ratio to a power less than one:

Reference: (6)

$$\frac{A_2}{A_1} = (\frac{Q_2}{Q_1})^{0.5} \quad (2\text{-}1)$$

$$\frac{H_2}{H_1} = (\frac{Q_2}{Q_1})^{0.4} \quad (2\text{-}2)$$

$$\frac{V_2}{V_1} = (\frac{Q_2}{Q_1})^{0.5} \quad (2\text{-}3)$$

$$\frac{\text{Time of Travel}_2}{\text{Time of Travel}_1} = (\frac{Q_2}{Q_1})^{-0.5} \quad (2\text{-}4)$$

where Q is the flow rate, A is the cross-sectional area, H is the mean stream depth, and V is the average velocity.

It should be recognized that these relationships are true only for free flowing rivers and that the exponents may vary by 50% for any river. Therefore, if possible, the exponents should be established from available data. However, if there is not enough information available to establish the exponents, then equations (2-1) to (2-4) can be used with caution.

The hypothetical South River cross-sectional area and depth data are presented in Figures 2-9, plots (b) and (c). These data were collected at a river flow approximately equal to the annual average flow. Area and depth measurements for this flow regime are available for every mile along the watercourse.

For ease of calculations, the river was divided into 3 areas of approximate equal physical characteristics. Between Milepoints 0 and 5 the river was characterized as having a cross-sectional area of 800 sq. ft. and a depth of 3 ft. For Milepoints 5 to 20 and 20 to 33, the cross-sectional area averaged 1,400 sq. ft. and 2,200 sq. ft. while the depth average 5 ft. and 6 ft. respectively. Equations (2-1) and (2-2) were used to determine the cross-sectional areas and depths of the three river segments at summer flow and low flow. Table 2-3(b) summarizes the hypothetical geometry for the South River example.

Figure 2-9, plot (d), presents the time of travel information for the South River between the Route 80 Bridge and Little Falls. The time of passage through the system is calculated from the freshwater flow and the cross-sectional area in each river section: velocity = flow/cross-sectional area and Travel Time = distance/velocity.

2.3.6 Climatological Data

Climatological data is available for many locations throughout the country. The data is available from the National Oceanic and Atmospheric Administration which is a section of the U.S. Department of Commerce. Climatological data sheets contain information on air temperature, precipitation, wind, sunshine and sky cover, visibility and humidity. An example is shown in Figure 2-12(b). All data are daily data except precipitation which are hourly data. Data are compiled from records on file at the National Weather Records Center, Asheville, North Carolina and are available from the U.S. Printing Office, Washington, D.C. For the preliminary impact analysis, precipitation data (water equivalents) should be reduced to average monthly and annual average rainfall data. These data are used subsequently to provide preliminary estimates of the combined and separate sewer overflow mass discharge rates. Rainfall data are used in Chapters 3 and 4 to provide more refined mass discharge estimates.

2.4 Water Quality Data Base

River water quality data are necessary for a preliminary problem identification and also for the preliminary impact analysis. A review of existing river water quality data might reveal existing water quality problems. If existing water quality data are not sufficient in spatial and temporal detail, existing water quality problems may not be obvious. For example, dry weather water quality data does not identify the direct impact of stormwater runoff during the storm event.

TABLE 2-3(b)

HYPOTHETICAL SOUTH RIVER

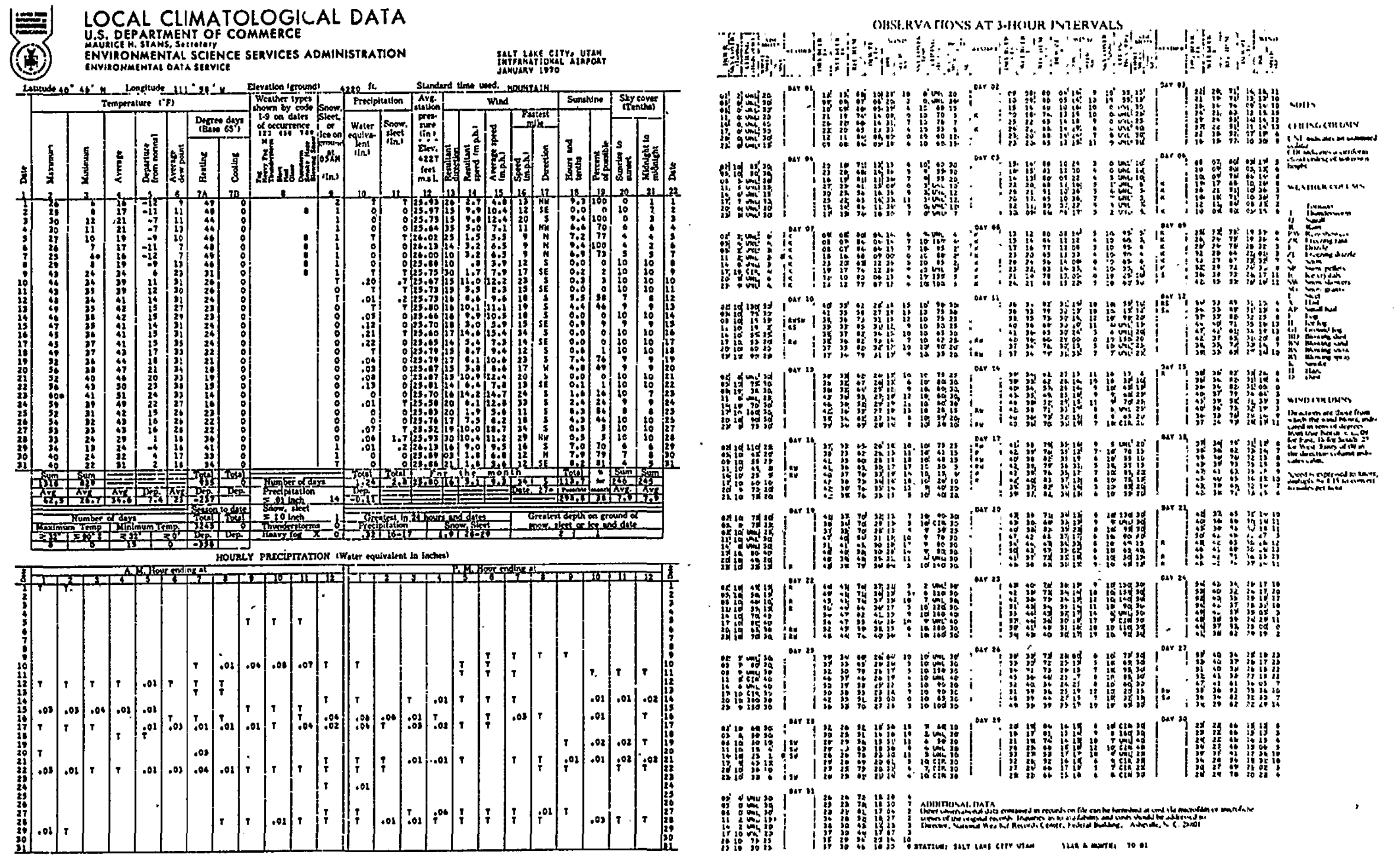
RIVER FLOW, GEOMETRY, AND TRAVEL TIME (AVERAGE ANNUAL, SUMMER, AND LOW FLOW)

Milepoint	Flow Condition	Flow cfs	Cross-Sectional[(a)] Area sq.ft.	Depth[(b)] ft.	Travel Time[(c)] days
0 - 5	Avg. Annual	535	800	3.0	0.46
	Summer	268	566	2.3	0.65
	Low Flow	54	253	1.2	1.45
5 - 20	Avg. Annual	943	1,400	5.0	1.39
	Summer	472	990	3.8	1.97
	Low Flow	95	443	2.0	4.40
20 - 33	Avg. Annual	1,158	2,200	6.0	1.56
	Summer	569	1,555	4.5	2.20
	Low Flow	114	696	2.4	4.93

(a) $A \alpha Q^{0.5}$

(b) $H \alpha Q^{0.4}$

(c) Travel time $= \frac{\text{dist.}}{\text{velocity}} = \frac{\text{dist.}}{\text{flow/X-Sect.A}}$



LOCAL CLIMATOLOGICAL DATA
U.S. DEPARTMENT OF COMMERCE
MAURICE H. STANS, Secretary
ENVIRONMENTAL SCIENCE SERVICES ADMINISTRATION
ENVIRONMENTAL DATA SERVICE

SALT LAKE CITY, UTAH
INTERNATIONAL AIRPORT
JANUARY 1970

Latitude 40° 46' N Longitude 111° 58' W Elevation (ground) 4220 ft. Standard time used. MOUNTAIN

OBSERVATIONS AT 3-HOUR INTERVALS

HOURLY PRECIPITATION (Water equivalent in inches)

SOURCE: (7)

FIGURE 2-12 (b)

SAMPLE LOCAL CLIMATOLOGICAL DATA SHEET

In order to establish the relationship between mass discharges and receiving water quality, a water quality model of the receiving water is needed. In addition, the model is used in the development of cost-effective pollution abatement alternatives through an understanding of the cause and effect relationship between the point and non-point source loads and the receiving water quality. Since the reliability of water quality models is directly dependent on the extent and reliability of the water quality data that are used for calibration, a sound water quality data base is essential for a successful 208 study.

A first step in assembling water quality data is to obtain a STORET retrieval of water quality at stations within the general study area limits. "STORET" is the acronym used to identify the computer oriented U.S.E.P.A. management information system for STOrage and RETrieval of water quality, streamflow, municipal waste facility inventory, fish kill, and other related data (Reference: 8). Water quality data may be retrieved from STORET in statistically analyzed form or in raw form and graphical displays.

The amount of data available from STORET may be quite sparse, or very extensive. Data handling and analysis is often more effectively accomplished if an initial step of identifying what is available is first performed. Next, the actual data is secured and compiled in an orderly fashion. Indiscriminate requests for data retrievals can literally inundate you with paper, and complicate the task of organizing and evaluating it. A more effective approach is to make a series of sequential retrievals and thus have much of the sorting and organizing accomplished by the computer system.

Data should provide an initial sense of <u>what</u> is going on -- the general levels of concentration of specific parameters for comparison with standards or objectives. Data should further indicate <u>where</u> things are happening, that is where in the stream changes occur, or whether problems tend to occur in certain locations. Since there are seasonal changes in stream flow, temperature, rainfall, and activity (construction, irrigation, recreation, for example) data should indicate <u>when</u> certain quality levels occur or problems manifest themselves.

To access stations and data, the analyst must know the station numbers or follow a program which searches selected geographical areas. A procedure is available to retrieve data for 208 areas using a keyword. EPA is presently involved in a program which will result in the publication of a simple STORET users manual. Generally, the 208 agencies can contact EPA and or the states to determine the availability of STORET information that can be retrieved. Table 2-4 lists the names, addresses and telephone numbers of the persons to contact for information and assistance concerning the STORET retrieval system.

Since the preliminary impact analysis is made on an average annual basis, an EPA STORET inventory retrieval is a valuable initial data base. An inventory provides the user with a statistical evaluation of all the available data for each station within the geographical search limits. A total inventory of all variables is not necessary, since many of the parameters listed in a total inventory will be of little use in the preliminary impact analysis. However, the water quality parameters are obtained at all stations on the main stem of the river and at all tributary stations.

In addition, raw data retrievals will be necessary to obtain water quality data during times of low river flows or during the rainy seasons if they exist. A raw data retrieval provides the user with the individual data that went into the statistical summary of the inventory. Special programs are available to plot stream data profiles, and to perform seasonal analysis and regressions.

As an aid in retrieving and assembling data, Table 2-5 presents a list of river water quality variables that are generally useful in analyzing dissolved oxygen, eutrophication and coliform problems. For river dissolved oxygen analyses, the minimum data requirements are dissolved oxygen, water temperature, and BOD_5. In saline waters, chloride or salinity measurements are required to determine the reduction in the dissolved oxygen saturation level associated with the salinity. It is recommended that pH data be reviewed to identify river segments with extreme pH values. Long-term BOD tests measure ultimate oxygen demand which is the driving force in the oxidation of BOD. Long-term BOD tests

TABLE 2-4

STORET POINTS OF CONTACT

	Point of Contact	User Assistance
Region I	Louis Gitto, Chief Systems Analysis Branch John F. Kennedy Federal Bldg. Boston, Massachusetts 02203	Louis Gitto
Region II	Herbert Barrack, Director Management Division 26 Federal Plaza New York, New York 10007 (212) 264-2520	Jack Sweeney (212) 264-4750
Region III	Larry Miller, Chief Water Quality Monitoring Office Surveillance & Analysis Division Curtis Building 6th & Walnut Streets Philadelphia, PA 19106 (215) 597-9823	Ted Standish (215) 597-8046
Region IV	John Marlar, Chief Technical Support Branch 1421 Peachtree Street N.E. Atlanta, Georgia 30309 (404) 526-3012	Dan Barber (404) 526-5989
Region V	Christopher Timm, Director Surveillance & Analysis Division 230 S. Dearborn Street Chicago, Illinois 60604 (312) 353-6738	Stu Ross (312) 353-2061
Region VI	David White, Chief Technical & Administration Systems Branch 1600 Patterson Street Suite 1100 Dallas, Texas 75201 (214) 749-1176	David White
Region VII	Walter Robohn, Federal Regional Council Representative 1735 Baltimore Avenue Kansas City, Missouri 64108 (816) 374-5495	Dennis Degner (816) 374-2018 758-2018

TABLE 2-4
(Continued)

STORET POINTS OF CONTACT

	Point of Contact	User Assistance
Region VIII	Keith Schwab, Director Surveillance & Analysis Division 1860 Lincoln Street Suite 900 Denver, Colorado 80203 (303) 837-4935	Tom Entzminger (303) 837-4985 FTS 327-4985
Region IX	Clyde Eller, Director Surveillance & Analysis Division 100 California Street San Francisco, California 94111 (415) 556-7858	William Lewis (415) 556-7550
Region X	Dr. Gary O'Neal, Director Surveillance & Analysis Division 1200 6th Avenue Seattle, Washington 98101	Claudia Rock (206) 399-1580

TABLE 2-5

STORET CODES FOR VARIABLES USED IN PRELIMINARY IMPACT ANALYSIS

		Problem			
STORET Code No.	Variable	BOD-Dissolved Oxygen	Eutrophication	Public Health	Other
00300	Dissolved Oyxgen	X	X		
00301	Percent D.O. Saturation	X	X		X
00010	Temperature	X	X	X	X
00400	pH		X		X
00525	SS				X
00530	TDS			X	
00310	BOD_5	X			
00319	Long-Term BOD	X			
00320	Long-Term BOD with Nitrification Inhibitor	X			
00605	Organic Nitrogen	X			
00610	NH_3-N (a)	X	X		
00615	NO_2-N	X	X		X
00620	NO_3-N	X	X		X
00665	Total Phosphorus		X		
00660	Ortho Phosphorus		X		
31507 31509	Total Coliform			X	
31515 31516	Fecal Coliform			X	
00060	Stream Flow (b)				

(a) Un-ionized ammonia can be computed from pH, temperature and NH_3-N

(b) If flow is retrieved, stream loadings can be calculated performed with and without a nitrification inhibitor measure the carbonaceous and total BOD respectively. The nitrogenous BOD is computed as the total BOD minus the carbonaceous BOD. Organic nitrogen, ammonia, nitrite, and nitrate serve as indicators of nitrification in a river because organic and ammonia nitrogen are oxidized to nitrite and nitrate.

Total coliform bacteria measurements are generally sufficient to identify contaminated sections of receiving waters. However, it is recommended that, when available, fecal coliform data also be analyzed to further define the fecal component of the coliform group.

Other potential water quality problems such as suspended solids, total dissolved solids, heavy metals, toxic organic compounds and pesticides should also be investigated by comparing existing water quality data to standards.

State agencies are required by Section 305 of Public Law 92-500 to conduct water quality surveys and to submit to the regional administrator a water quality inventory describing the water quality of all navigable waterways in the state on a yearly basis. Generally, the state monitoring programs vary from the measurement of flow and dissolved oxygen to the collection of the parameters shown in Table 2-6. Theoretically this data, along with U.S.G.S. data and other water quality data, is contained in STORET. However, normal time lags and general inefficiencies may prevent this from happening. Therefore, it is important to contact the individual states to obtain the most recent available water quality data. Table 2-7 lists an agency contact in each of 39 states. Additional sources of water quality data are listed in Table 2-8.

The first priority in reviewing the STORET data retrieval, or data from other sources, is to construct spatial water quality distributions for various time periods. For example, a review of STORET data for ten stations on a river might show seven of the stations were sampled on the same day, or during the same week or month. Extracting these data and plotting spatial distributions for as many constituents as possible is essential since these plots provide a "picture" of river water quality at a certain period in time.

TABLE 2-6

WATER QUALITY PARAMETERS COMMONLY MONITORED BY STATES*

Parameter	Number of States
Flow	47
Dissolved Oxygen	47
Coliform bacteria	45
Nitrogen (any form)	39
Phosphorus (any form)	35
pH	35
BOD/COD/TOC	27
Water temperature	29
Turbidity	26
Solids (any type)	27
Metals (any type)	17
Chlorides	19
Alkalinity	15
Conductivity	16
Color	11
Sulfate	14

*Only parameters specifically mentioned as being part of the State's monitoring program are counted. Only parameters listed by at least 10 States are included.

TABLE 2-7

STATE WATER QUALITY AGENCY

	Point of Contact
Alabama	Ann Cummings Water Improvement Commission State Office Building Montgomery, AL 36104 (205) 269-7971
Arizona	Phyllis Woolsey Water Quality Control Board 1740 West Adams Phoenix, AZ 85007 (602) 271-5453
Arkansas	R. C. Wilson Control and Ecology 8001 National Drive Little Rock AR 72209 (501) 371-1701
California	Phil Mendes State Water Res. Control Board 1416 9th Street Sacramento, CA 95814 (916) 322-2416
Colorado	John Hinton P.O. Box 138 Delta, CO 81416 (303) 874-4411
Connecticut	Charles Nula Department of Environmental Protection Water Compliance Unit State Office Building (Room 126) Hartford, CT 06115 (302) 678-4771
D.C.	James Otto Water Quality Control Division 614 H Street N.W. Washington, D.C. 20001 (202) 629-2538
Florida	H. Duane Mitchell Department of Pollution Control 2562 Executive Control Circle East Tallahasse, FL 32301 (904) 488-8626
Georgia	Michael Moss Department of Natural Resources 270 Washington Street Room 820 Atlanta, Georgia (404) 656 4988
Idaho	Gene Ralston Department of Health and Welfare State House Boise, ID 83720 (208) 964-2390
Illinois	Don Goodwin Illinois Environmental Protection Agency 2200 Church Hill Road Springfield, IL 62706 (217) 525-3362
Indiana	T.P. Chang Indiana Stream Pollution Control Board 1330 West Michigan Indianapolis, Indiana
Iowa	Jim Stricker Iowa Department of Environmental Quality 8920 Delaware Avenue P.O. Box 3326 Des Moines, IA 50316 (515) 265-8134
Kansas	Gerry Stoltenberg State Department of Health Water Quality and Point Source Data Division 740 Forbes AFB Topeka, Kansas (913) 296-3825
Kentucky	Douglas C. Griffin Division of Water Resources Department of Natural Resources and Environmental Protection 6th Floor-Capital Plaza Tower Room 626 Frankfort, KY 40601 (502) 564-3980
Louisiana	David Bruce Bureau of Environmental Health State Office Building P.O. Box 60630 New Orleans, LA 70160 (504) 527-5124
Maryland	Wayne Overman Tawes State Office Building Maryland Environmental Service Annapolis, MD 21401
Massachusetts	Russ Isaac Massachusetts Division of Water Control 100 Cambridge Street (Room 1901) Boston, Mass. (617) 727-3855
Michigan	Bruce Chaffin Michigan Water Resources Commission Steven T. Mason Building - 8th Floor Lansing, MI 48926 (517) 373-2867
Minnesota	Bob Popo Pollution Control Agency 1935 W. County Road Roseville, MN (612) 296-7222
Mississippi	Earl Lemaster Mississippi Air and Water Pollution Control P.O. Box 827 Jackson, Mississippi (601) 354-6783
Missouri	Maureen Mueller Department of Natural Resources Clean Water Commission P.O. Box 176 Jefferson City, MO 65101 (315) 751-3241
Nebraska	Judy Newkirk Department of Environmental Control Lincoln, NE 68509 (402) 471-2186

TABLE 2-7

STATE WATER QUALITY AGENCY

	Point of Contact
New Jersey	John Ruggero N. J. Dept. of Environmental Protection P.O. Box 2809 Trenton, NJ (609) 292-7493
New Mexico	Mike Snavely Water Quality Section Pere Building P.O. Box 2348 (505) 827-2948
New York	Phil Oby Department of Environmental Conservation 50 Wolf Road Albany, NY (518) 457-5734
North Carolina	Sue Gardner Dept. of National Economic Resources P.O. Box 27687 Raleigh, NC 27611 (919) 829-4740
North Dakota	Gerald Knudsen North Dakota State Department of Health State Capital Bismark, ND 58501 (701) 244-2375
Ohio	Diana Reed Environmental Protection Agency 361 East Broad Street Columbus, Ohio 43216 (615) 466-5760
Oklahoma	Jesse Strawbridge Water Quality Serv. P.O. Box 53551 Oklahoma City, Okla. (405) 271-5240
Oregon	Van Kollias Department of Environmental Quality Beau-Hill Portland, Oregon (503) 229-5983
Pennsyvania	John Kitch State of Penn. Management Services Division P.O. Box 2063 Fulton National Bldg. 3rd and Locust Streets Harrisburg, PA 17120 (717) 787-9640
South Carolina	Jay Sylvester South Carolina Health and Environmental Control (Room 488) Annex 2600 Bull Street Columbia, SC 29201 (303) 758-5165
Texas	Randy Merideth Water Quality Board P.O. Box 13246 Austin, TX (512) 475-5851
Vermont	Dick Cambio Vermont State Department of Water Resources Water Quality Division State Office Bldg. Montpelier, VT 05602 (802) 828-2763
Virginia	Clyde Goodin Virginia State Water Control Board 2111 N. Hamilton Street P.O. Box 1143 Richmond, VA 23230 (804) 770-2111
Washington	Robert James Dept. of Ecology State of Washington 7272 Clean Water Lane P.O. Box 829 Olympia, Washington 98501
West Virginia	Les Schulz Department of Natural Resources 1201 Grenbier Street Charleston, West Virginia (304) 348-2837
Wisconsin	Lyman Wible Wisconsin Dept. of Natural Resources P.O. Box 450 Madison, Wisconsin 53701 (608) 266-8107

TABLE 2-8

SUMMARY OF SOURCES OF WATER QUALITY DATA[a]

1. Bureau of Reclamation (Dept. of Interior)
2. U.S. Army Corps of Engineers
3. Environmental Protection Agency (Regional Offices)
4. U.S. Forest Service (Dept. of Agriculture)
5. Fish and Wildlife Service (Dept. of Interior)
6. U.S. Geological Survey (Dept. of Interior)
7. National Weather Service (Dept. of Commerce)
8. National Water Quality Surveillance System (EPA)
9. State, County and City Health Departments
10. Local University Biology, or Environmental Engineering Departments
11. Engineering Reports (such as 303-E Basin Plans and 201 Facilities Plans)

[a] See Appendix D for details for water quality data sources

Spatial water quality profiles should be developed for various river flow and seasonal conditions. Summer low flow conditions are generally critical with regard to dissolved oxygen. High river suspended solid levels occur during periods of peak surface runoff. Wet weather spatial surveys reveal stormwater runoff effects.

Water quality data with sufficient spatial detail is not likely to come from a STORET data retrieval. Detailed spatial water quality is generally the result of a special water quality study. In some instances the data collected during these studies are stored in STORET. Consequently, State governmental environmental agencies can be a source of extensive river quality data that is not a part of STORET. For example Figure 2-13, presents three spatial dissolved oxygen distributions in the Black River, New York measured between 1969 and 1973. Plots (a), (b), and (c) in Figure 2-13 represent data obtained from STORET, N.Y. State Department of Environmental Conservation, and an EPA sponsored study of the Black River respectively. A review of the STORET data does not reveal a dissolved oxygen problem. The State data shows two river dissolved oxygen measurements less than the standard. Finally, the data collected during the special study provides a detailed spatial dissolved oxygen distribution of the Black River which clearly shows a 25 mile reach of river where the standards were not met.

Although the primary goal of a preliminary water quality inventory is to produce spatial distributions of various constituents, it is also advisable to isolate some temporal water quality data. Temporal dissolved oxygen data may indicate that algal photosynthesis and respiration significantly affect river dissolved oxygen. Data for six stations on the Truckee River is shown in Figure 2-14. The diurnal dissolved oxygen data for Station 4 shows that the peak afternoon dissolved oxygen concentration of 10 mg/1 decreases to a minimum nighttime concentration of 4 mg/1, representing a dissolved oxygen change of 6 mg/1 over the day.

Should insufficient river water quality data exist, it is still appropriate to proceed with the preliminary analysis in order to locate regions of

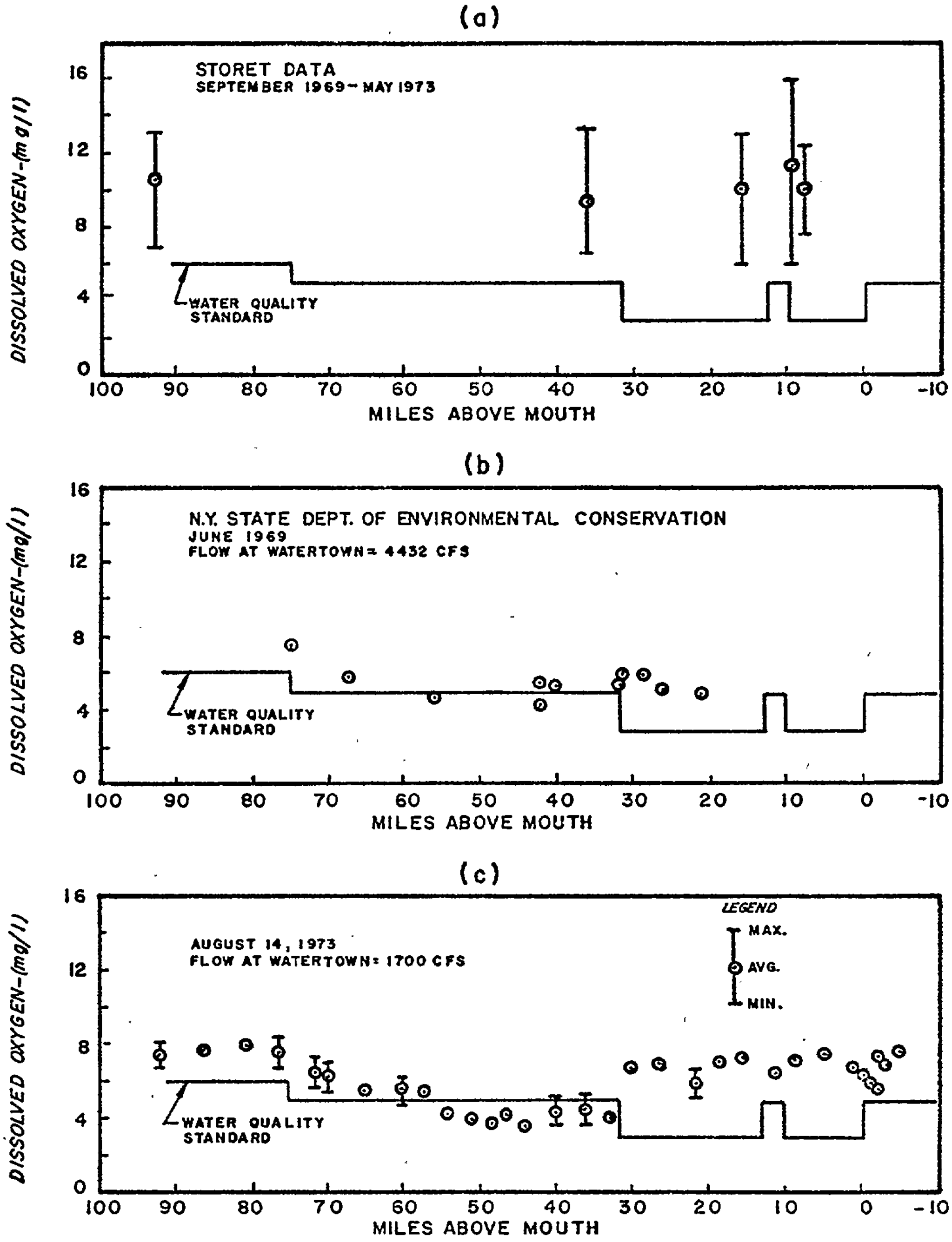


FIGURE 2-13
COMPARISON OF DISSOLVED OXYGEN DATA
(BLACK RIVER, N.Y.)

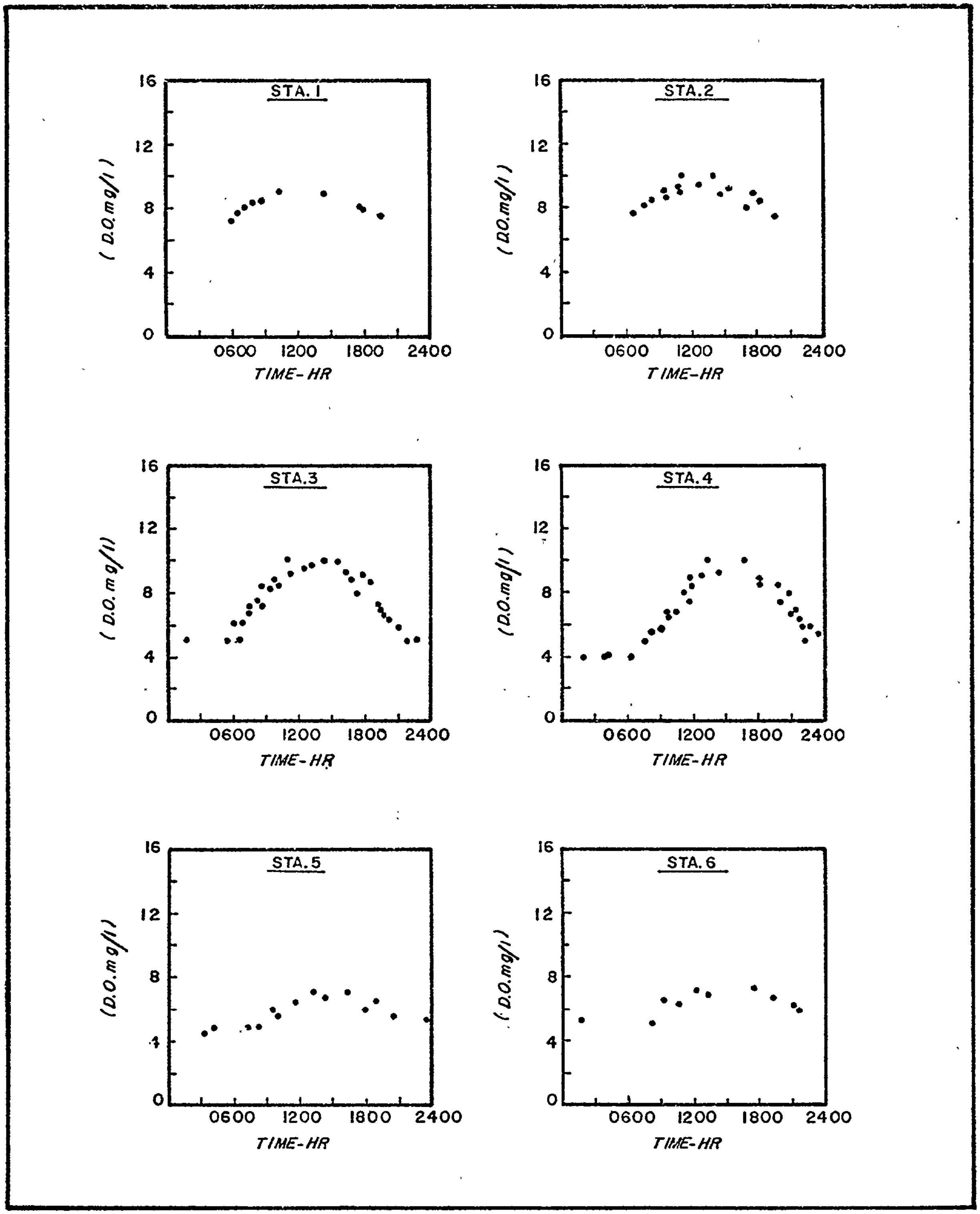


FIGURE 2-14
DIURNAL DISSOLVED OXYGEN VARIATIONS
(TRUCKEE RIVER, NEVADA)

probable water quality problems. The impact analysis can be performed using estimates and empirical relationships as discussed subsequently.

Visual observations of the study area during dry and wet weather is an additional source of data. Field trips can provide the analyst with increased understanding of the 208 study area and its complex problems. Inspections provide information on the exact location of point wastewater discharges, the location of NPS runoff inflow, areal extent of instream weed growths, and the extent of the lateral mixing zones below wastewater discharges or tributary inflows. In addition, field trips usually bring the analyst in contact with local residents who prove to be invaluable sources of qualitative data with respect to water use and misuse.

A field trip made during a storm event can provide data on the location of stormwater overflow points. Wet weather observations also lead to qualitative identification of instream water quality problems such as excess suspended solids or floatables. Finally, visual inspections of the 208 study area can pinpoint the location of NPS sediment sources. Such sources, as abandoned open cuts or construction activity, can be discovered on field trips.

2.5 Waste Source Identification and Evaluation

The procedures to be used for the preliminary problem assessment are based on a mass balance analysis of the receiving water. The need for reasonable estimates of the mass emission rates or loads is, therefore, apparent. The characterization of the various types of discharges is based on their origin and their variability. The minimal characterization is an estimate of the long term average mass discharge rate, typically in units of pounds/day. For certain classes of discharges, it may be necessary to have an estimate of the seasonal variation of the mass discharge rate over the year, if this variation is significant. For preliminary analysis, a significant variation is a monthly average variation factor of three times greater than the long term mean. This judgement is based on the probable accuracy of the estimating procedures used for other sources of mass for which measurements are usually not available. A three-fold variation appears to be the range of the uncertainty in these estimates and this uncertainty sets the relative accuracy of the entire analysis.

To understand this choice of criteria, it is necessary to realize that for a given planning area the estimates recommended for use in the preliminary assessment, in lieu of actual site-specific data, are the averages of the long term average concentrations from the runoff of many cities, agricultural lands, etc.. It is also necessary to consider these data as a statistical sampling problem from a set of random variables. As an example, for the urban combined sewer overflow concentration of suspended solids from many cities, an estimate is needed of how far in error the use of the mean suspended solids concentration from this set of random variables is from the actual concentrations. A guide is that for sets of random variables with a coefficient of variation of approximately one, a greater than three-fold variation has a probability of less than 10%. The 10% probability appears to be a reasonable bound and is the basis for the selection of a three-fold variation.

This probable variability sets the level of uncertainty in the overall preliminary assessment. If the seasonal variation of a source is less than this three-fold variation, its inclusion is not warranted since it is a refinement above the probable level of accuracy of the overall preliminary assessment.

2.5.1 Continuous Point Source Evaluation

Point sources effluent water quality data are available for both municipal and industrial sources of wastewater. Other point sources of pollution include continuous discharges from: faulty sewerage systems; inadequate or filled interceptors; faulty regulators; exfiltration; or continuous overflows at treatment works. It is likely that loading data will not be available for these other point sources. However, municipal wastewater effluent quality data can be obtained from EPA, state, and local regulatory agencies. One of the best sources of these data are 303(e) basin and 201 facility plans. Industrial point source effluent water quality data can be obtained from EPA, state, and local regulatory agencies and from the industries themselves. An additional source of effluent discharge data for the industry of interest is the NPDES and State permits. These are available either from the EPA region, or responsible state agency, or the industries themselves. The Surveillance and Analysis (S&A) division of the EPA region have measured effluent data for many industries. Effluent water quality data exists as flow and associated concentration.

From the flow and concentration data, average annual mass discharge rates for the continuous point sources are calculated for the variables in Table 2-9.

TABLE 2-9

VARIABLES USED IN PRELIMINARY PROBLEM ASSESSMENT

Problem Category	Variables
Dissolved Oxygen	BOD_5 and Total Kjeldahl Nitrogen
Public Health	Total Coliform Bacteria
Eutrophication	Total Nitrogen Total Phosphorus
Other Water Use Interferences	Total Suspended Solids

For total coliforms, the average of the logarithms of the counts is used to calculate the average. Since coliform data are generally log-normally distributed, the arithmetic average is not a suitable measure of the central tendency of the data. Arithmetic averages overweight the few very large measurements.

If adequate data is not available, municipal point source mass discharges can be calculated based on population (see Simplified Mathematical Modeling of Water Quality(6)) and industrial mass discharges can be estimated from the literature based on production rates (9).

The annual average point source loads are tabulated and located with respect to a river mileage coordinate system. Some of the sources can be eliminated from further analysis if the loads they discharge to the river are considered insignificant when compared to the other point source loads at or near that location. This may not be the case for the analysis of projected conditions where certain sources are controllable and others are less so.

The mass discharge rates for the point sources in the hypothetical South River 208 study area are presented in Table 2-10. Loading rates are presented for total nitrogen, total phosphorus, total suspended solids, BOD_5 and total coliform bacteria. Data are given for the upstream river quality, the Beaver River quality, Brown Pulp and Paper (industry), Jefferson Municipal Sewage Treatment Plant and the American Paper Company (industry).

In Table 2-10, the nitrogen, phosphorus and coliform loading rates from the two industries are considered insignificant and are not included in the preliminary impact analysis. This is a judgement based on the extremely small magnitude of the industrial loading rate when compared the other point sources.

2.5.2 Tributary Sources

Tributaries to the main receiving water and the quality of the farthest upstream point of the region being considered are also evaluated. These sources are treated as continuous point sources to the main reach of the receiving water. The evaluation of the magnitude of their contribution is also required for the analysis. In some cases, water quality data exists and the average concentrations can be obtained from measurements. This is assumed to be the case for the illustrative problem. If this is not the case, preliminary analyses of the tributary and upstream drainage basins are required. The methods to be used are identical to those to be presented for the main reach. The analysis is done for the portions of the drainage area for which there are insufficient measurements.

Seasonal variations are considered if the measured concentrations show a regular annual trend and the magnitude of the seasonal variation from the annual mean is three-fold or greater.

The variation of concentration with river flow is also of interest. Substances associated with point sources show a dilution effect whereas

TABLE 2-10

SUMMARY OF POINT LOADS
FOR HYPOTHETICAL SOUTH RIVER EXAMPLE

Source	Flow (cfs)	Total Nitrogen		Total Phosphorus	
		mg/l	lbs/day	mg/l	lbs/day
Upstream Q.	500(a)	0.2	540(b)	0.05	135(b)
Beaver R.	250(a)	0.5	675(b)	0.10	135(b)
Brown P & P	1.6	0	0	0	0
Jeff. STP	19.8	20	2138	10	1069
Amer. Pap.	0.8	0	0	0	0

Source	Flow (cfs)	Total SS		BOD_5		Total Coliform	
		mg/l	lbs/day	mg/l	lbs/day	No. 100/ml	No./Day
Upstream Q.	500(a)	10	27,000(b)	1.0	2,700(b)	500	$6.1x10^{12}$(b)
Beaver R.	250(a)	20	27,000(b)	1.5	2,025(b)	1,000	$6.1x10^{12}$(b)
Brown P & P	1.6	50	432	50	432	0	0
Jeff. STP	19.8	30	3,208	30	3,208	10,000	$4.8x10^{12}$
Amer. Pap.	0.8	200	864	500	2,160	0	0

(a) Average annual flow

(b) Loads at other flow regimes are computed proportional to flow sources associated with surface runoff and groundwater flow interact in more complex ways. This issue is beyond the scope of a preliminary analysis but is addressed in subsequent chapters.

2.5.3 Intermittent Urban Point Sources

The principal concern is the overflows and bypasses of the sewerage system during rainfall events. The precise and detailed evaluation of the mass discharges from a complex metropolitan region is a task which can occupy the entire efforts of a planning study. It is of interest, therefore, to estimate the importance of these mass discharges in comparison to all other mass sources. A method is required that can give approximate results which are suitable for comparative purposes.

Methods for estimating sewer overflow quantity and quality have been developed in recent years. The result is a series of estimating techniques and computer-based models which span a level of complexity from minute-to-minute simulations that consider the detailed hydraulics and mass transport in the urban watershed and sewerage system, to less complex models that process hourly rainfall data and associated pollutant concentrations in the resulting runoff. Appendix A identifies and describes a number of such models. From the point of view of a preliminary analysis, it appears that many of these methods are too detailed and cumbersome if what is required is an estimate of the significance of urban sewer overflow relative to the other sources of mass being considered. Examples of these methodologies are given in References 10 and 11. In this Manual, a simple and direct method is suggested which is based on estimating the average mass discharge rate as the product of the average concentration in the runoff, and the average quantity of runoff. That is, if $\overline{W}_o$ is the long term average mass discharge rate from an overflow point, then

$$\overline{W}_o = \bar{c}\ \bar{C}_V\ \bar{I}_T\ A \qquad (2\text{-}5)$$

where:

$\bar{c}$ is the average concentration (mg/l)

$\bar{C}_V$ is the average runoff coefficient (unitless)

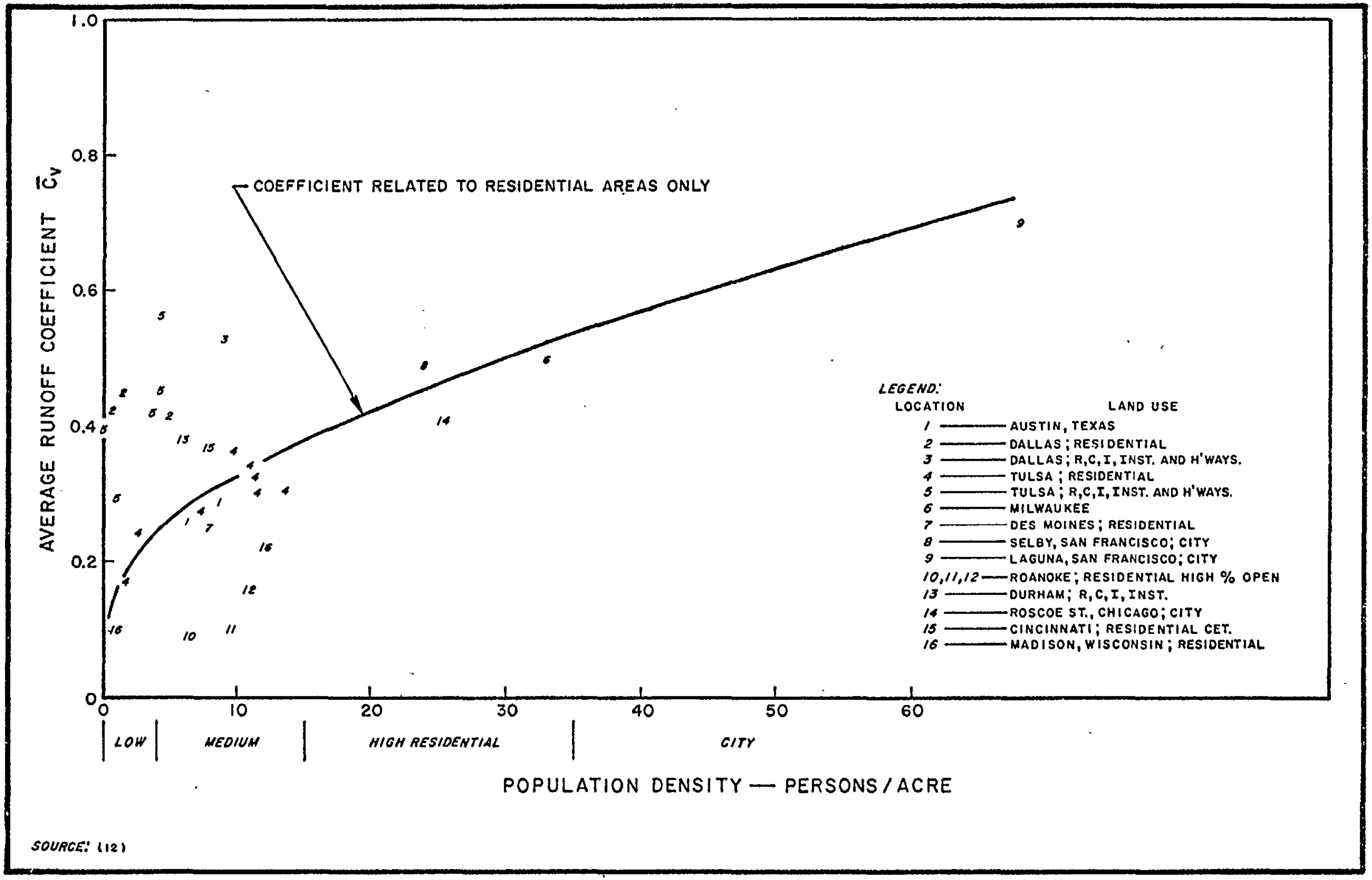
$\bar{I}_T$ is the average rainfall divided by the period of averaging, T (in/hr).

A is the drainage area (acres).

This equation assumes that the random variables $\bar{c}$, $\bar{C}_V$, and $\bar{I}_T$, are uncorrelated. Recent analyses of available data (10) indicate that this assumption is reasonable, especially if a large drainage area is served by a single rain gage.

Treating the runoff coefficient as a random variable appears, at first glance, to be unnecessary since, in principle, it is deterministically related to the processes of rainfall and flow over catchments of definable properties. However, the rainfall itself has properties which are not definable from normally available rainfall records. The critical element is the spatial extent of the rainfall represented by the measurement at a point rain gage. It is this component of the actual runoff which is behaving randomly. Therefore, it appears reasonable to treat the overall runoff coefficient, as a random variable. An analysis of runoff coefficients given in Figure 2-15 indicates that a relationship exists using population density as a measure of the degree of imperviousness. For low and medium population densities, a mean of 0.3 ± 0.15 seems reasonable, and results in approximately a two-fold uncertainty.

Similiar arguments can be made for treating the runoff concentration as a random variable. Although, in principle, the mass in the runoff is related to the accumulations of mass in the catchment and sewerage system, it is a difficult task to formulate the relationships in a way that is broadly applicable. All models that attempt this calculation require extensive calibration data on a number of events in order to give reasonable results. In lieu of such an effort, it appears justified



SOURCE: (12)

FIGURE 2-15
RUNOFF COEFFICIENT RELATED TO POPULATION DENSITY

to regard the concentration as varying randomly and require only that its statistical properties be estimated.

For the preliminary analysis the average concentration of the constituents of interest are required. Table 2-11 summarizes the average and standard deviation of the long term average concentrations for the parameters of interest for selected U.S. cities. For BOD and suspended solids, the upper 90th percentile is approximately three times the mean so that for a given city these concentrations can be regarded as being known to within a three-fold uncertainty.

The method is applied in the following way: available data for both quality and quantity of urban stormwater are used if available. Site specific data are aggregated into annual average mass discharge rates (lbs/day) for the preliminary analysis. If available, mass discharge rates are tabulated for total nitrogen, total phosphorus, BOD (5 day), total coliform bacteria and total suspended solids. In addition, the major sewer overflow discharges are located with respect to the river mileage coordinate system. Minor overflow discharge rates should be combined if they are closely spaced. For example, five 0.05 MGD combined sewer overflows within 1 mile of each other may be added together to form a 0.25 MGD overflow at one location.

If site specific separate and combined sewer quality and quantity data are not available, then steps 1 through 9 give a method to estimate the mass discharge rates for the preliminary analysis.

1. Locate major overflows and determine if they are combined or separate sewers
2. Locate and group minor overflows
3. Associate a drainage area (A) (acres) with each overflow
4. From the demographic inventory, estimate a population density (persons/acre) for each urban drainage area. Estimate a mean runoff coefficient $\bar{C}_V$ for each urban drainage area. Figure 2-15 or alternate techniques may be used.
5. Obtain the average annual rainfall from the weather bureau

TABLE 2-11

SUMMARY OF STORMWATER POLLUTANT CONCENTRATIONS FOR SELECTED U.S. CITIES$^{(a)}$

	Stormwater Overflow Concentrations$^{(b)}$					
	Separate Sewers$^{(c)}$			Combined Sewers$^{(d)}$		
Pollutant	Mean	Standard Deviation	$\nu^{(e)}$	Mean	Standard Deviation	$\nu^{(e)}$
BOD_5	27	25	0.9	108	36	0.3
Suspended Solids	608	616	1.0	372	275	0.7
Total Coliforms	3×10^5	-		6×10^6	-	
Total Nitrogen (as N)	2.3	1.4	0.6	9	6	0.7
Total Phosphorus (as P)	0.5	0.4	0.8	2.8	2.9	1.0

(a) Reference (12)

(b) All units mg/l, except coliforms, MPN/100 ml

(c) Summary of the averages of twenty cities

(d) Summary of the averages of twenty-five cities

(e) ν = coefficient of variation = Standard Deviation ÷ Mean

ν describes the relative variability of the average concentration of pollutants in runoff; as ν increases, this indicates that the pollutant concentration is becoming more variable. E.g., $\nu > 1$ highly variable, $\nu < 0.2$ not very variable.

6. Convert average annual rainfall, v, to average hourly intensity, $\bar{I}_T$. $\bar{I}_T$ (inches/hour) = V (inches annual rainfall) ÷ T (hours in averaging period), i.e., $\bar{I}_T = V/T$.
7. Calculate average annual runoff, $\bar{Q}_o$ (cfs) = $\bar{I}_T$ (in/hr) $\bar{C}_v$A (acres).
8. Using the average concentrations given in Table 2-11 for combined and separate sewer overflow, estimate the average annual mass discharge rate, $\bar{W}_o$, for each discharge. $\bar{W}_o$ (lbs/day) = 5.4 $\bar{C}$ (mg/l) $\bar{Q}_o$ (cfs).

The average annual effect of the urban stormwater runoff on the water quality of the hypothetical South River is estimated using this procedure. The annual rainfall in the South River Basin is 40 inches/year so that $\bar{I}_T = 0.00457$ inches/hr. Approximately ten square miles (6400 acres) of the total 20 square mile area of Jefferson City is sewered with combined sewers. The remaining area is unsewered. The example further makes the simplifying assumption that in Jefferson City there is no provision at the municipal sewage treatment plant for the acceptance of any stormwater. Therefore, the total untreated load flows directly to the South River.

The population density of the combined sewer area is 20 persons per acre. From Figure 2-15 the runoff coefficient at 20 persons per acre is $\overline{C}_v = 0.42$. The average intensity and average runoff coefficient gives an average stormwater overflow from Jefferson City of $\overline{Q}_o = 12.3$ cfs.

Since the combined sewers in Jefferson City are evenly spaced over the entire 5 miles of city, the loads are expressed as lbs. per day of a constituent per river mile. The sewer overflows are combined with the pollutant concentrations in Table 2-11 to yield non-point source mass discharge rates. These rates are included in the summary table of non-point sources in Section 2.5.2.2.

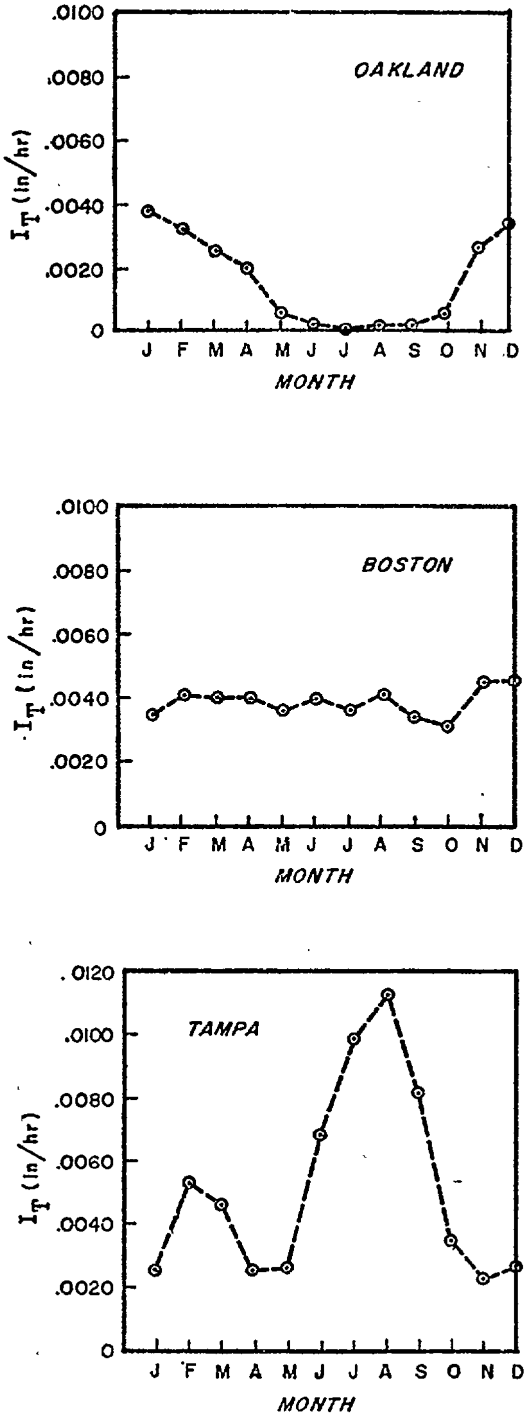
The major difficulty with a long term average analysis of transient stormwater discharges is that the discharge occurs only during a rainfall and this can cause water quality problems during the transient discharge. Even if the average mass discharge rate is small, it is necessary to perform an approximate analysis of the probable effect during such an event. Consider the impact of a continuous rainfall and runoff lasting 3 to 5 days. During this type of storm the rainfall intensity can be 5 to 10 times greater than the average annual intensity. In addition, the base river flow can increase 2 to 5 times the average flow preceeding the storm. This combination of rainfall and runoff can be caused by the slowly moving frontal storms which take 3 to 5 days to pass over the drainage basin. Although it is difficult to assess in a preliminary way the probability of occurrence of such an event, this type of storm provides a rough basis for analysis of a critical but not very improbable event. An alternate possibility is a localized storm which does not appreciably change stream flow.

For the South River Basin, rainfall records indicated that a 3-day storm intensity of 7 times the annual average rainfall intensity is not uncommon in the summer. In addition, the base river flow increases to 3.2 times the annual average flow during these storms, as shown by an inspection of the stream hydrograph. Therefore, for the preliminary impact analysis, the storm load is assumed to be 7 times the annual average mass discharge rate.

The seasonal variation of the average urban stormwater mass discharge is due primarily to the seasonal variation in rainfall volume and storm frequency. Seasonal rainfall variations can be quite substantial, in some cases exceeding a three-fold change. Some typical seasonal rainfall variations in intensity are illustrated in Figure 2-16.

2.5.4 Non-Point Sources (NPS)

Non-point source pollution is defined as pollution which enters a water body from diffuse origins on the watershed and does not result from discernible, confined or discrete conveyances. The contribution of non-point sources can be a substantial and significant portion of the total sources that impact the receiving waters being considered. As a consequence, an estimate of their magnitude and receiving water impact is required for rational preliminary analysis. The available methodology for making quantitative estimates of the magnitude of non-point source loading rates is analogous to that available for stormwater related urban point



SOURCE: (12)

FIGURE 2-16
AVERAGE MONTHLY RAINFALL INTENSITY

sources. Complex computer models which attempt a deterministic and detailed calculation on short time scales are being developed but, as in the case of the urban stormwater models, their application, if warranted, requires extensive field data and detailed verification analysis. The models cannot be applied without such an effort since they are not predictive without suitable calibrations.

A class of methods being developed are based on soil erosion and sediment transport as the principal source of the non-point source mass reaching the receiving water. The erosion rate is computed using the Universal Soil Loss equation which relates the sediment yield of an area to a rainfall factor (rainfall erosion potential), soil erodibility factor (topographic slope and steepness factor), cropping management factor (related to extent of vegetation cover), and erosion control practice factor. The ratio of sediment generated in the region of analysis to that reaching the receiving water is used to account for the sediment transport. In order to compute the mass discharge rate of nutrients and BOD, it is further assumed that these constituents are a fixed multiple of the sediment mass reaching the receiving water. Since several of these factors can be uncertain up to an order of magnitude, the estimate of sediment loading to the receiving water can have a substantial uncertainty associated with it. A more detailed exposition and evaluation of this method is contained in Chapter 4.

Therefore, the method that is recommended for the preliminary analysis is, as in the case of the stormwater evaluation, a simple estimate based on the average annual yields of drainage basins of various categories. For the purposes of the preliminary analysis, sources of non-point loads are separated into the following land use types: agriculture, feedlots, forest, and non-sewered urban. The choice of the long term scale (annual) and large spatial scale (drainage basin wide) is consistent with the temporal and spatial scale of the preliminary impact analysis. Although seasonal effects are addressed in an approximate way, the uncertainty of the annual estimates may exceed the variations due to seasonal effects.

The available literature on non-point source nutrient yield, which are presented in units of pounds/square mile/day, are based on yearly averages reported in the literature. A wide variability in the data is encountered within specific land use patterns. Significantly different nutrient loads are also observed according to the monitoring procedures used to obtain the data. Therefore, the data are chosen in terms of the spatial and temporal scales appropriate for the application. Non-point source load estimates are segregated according to the three basic monitoring procedures generally used to obtain the data. These three procedures are:

1. Seepage Study - These include lysimeter studies and sampling of tile drainage effluent. The water being analyzed has percolated through the soil profile and, thus, may contain significant quantities of leached nutrients.

2. Runoff Study - These studies typically involve very small tracts of land devoted to a single and specific land use. Samples are collected only during runoff events. Water sampled in these studies includes significant quantities of particulate matter with which most of the nutrients are associated.

3. Drainage Area Study - These studies involve continuously flowing streams which drain a particular land type. Flow is usually monitored continuously and samples are periodically collected for nutrient analysis.

Only the results of drainage area study are used for the preliminary analysis since the spatial scale is most appropriate. In addition, it is quite difficult to translate small temporal and spatial scale mass loadings to the quantity which eventually enters the receiving water. For example, if agricultural lands are considered, a large range in yield is possible for both total and inorganic nitrogen and phosphorus depending on the spatial scale to which the data apply. Table 2-12 presents a summary of available data for the three spatial scales. Seepage and runoff studies exhibit a range of at least two orders of magnitude for all nutrient forms reported. The range in the results of

TABLE 2-12

AGRICULTURAL NUTRIENT YIELDS, POUNDS/SQUARE MILE/DAY

1. Seepage Studies - Tile Drainage or Lysimeter Studies

	Nitrogen Yield			Phosphorus Yield		
	Number[(a)]	Mean	Range	Number[(a)]	Mean	Range
Total	15	44.0	0.5-172.	9	1.82	0.08-12.1
Inorganic	28	30.4	0.5-128.	6	0.83	0.02- 3.9

2. Runoff Studies- Surface Runoff From Small Test Plots

	Nitrogen Yield			Phosphorus Yield		
	Number[(a)]	Mean	Range	Number[(a)]	Mean	Range
Total	25	48.2	1.41-414.	16	9.23	0.17-47.0
Inorganic	14	1.23	0.02-7.2	-	-	-

3. Drainage Area Studies - Stream Sampling

	Nitrogen Yield			Phosphorus Yield		
	Number[(a)]	Mean	Range	Number[(a)]	Mean	Range
Total	23	15.	1.9 -58.0	35	0.73	0.05-3.9
Inorganic	17	12.	1.72-32.8	9	0.23	0.05-0.91

4. Comparison of Mean Yields

	Nitrogen		Phosphorus	
	Total	Inorganic	Total	Inorganic
Seepage Studies	44	30.4	1.82	0.83
Runoff Studies	48	1.2	9.23	-
Drainage Area	15	12.	0.73	0.23

(a) Indicates number of studies.

Source: Reference (13)

drainage area studies is somewhat smaller, especially for total nitrogen where the minimum and maximum value differ by about an order of magnitude.

As shown at the bottom of Table 2-12, the different monitoring procedures yield significantly different results. The seepage studies indicate very high nitrogen yields with a high percentage of inorganic or soluble nitrogen being leached from the soil. Similarly, runoff studies indicate similarly high yields for total nitrogen; however, most of this is in the particulate form. Phosphorus yields are higher in the runoff studies than in seepage studies, reflecting the ability of many soils to retain this element. Nutrient yields reported from drainage area studies are significantly lower than seepage or runoff studies. Furthermore, the nitrogen measured in drainage area studies is predominantly in the inorganic form. Apparently, much of the particulate matter observed in runoff events from small plots of land is not transported to perennial stream channels.

Thus, drainage area studies appear to have the widest and most direct application for estimation of nutrient loads to receiving waters. Runoff studies would have application in evaluation of the impact of specific land management practices, e.g., plowing techniques, crop rotations, and fertilizer applications. However, these runoff studies would have to be conducted in conjunction with drainage area studies, in order to quantitatively establish the link between the two. The physical transport and the chemical mechanisms involved in this link may be quite complex, and a substantial technical and financial commitment would be required to establish, calibrate, and verify a quantitative framework for the analysis.

A summary of the drainage basin yields for the various categories of land use are shown in Table 2-13. This table is used for the preliminary assessment of non-point source loading.

Seasonal variations of non-point source loads can be substantial, particularly if the sources are from agricultural lands. It is likely that the variability of the seasonal distribution is a cause of the

TABLE 2-13

RUNOFF AREAL LOADING RATE - POUNDS/SQUARE MILE/DAY(a)
(Average Range)

Land Use	Total Nitrogen	Total Phosphorus	BOD_5	TSS	Total Coliform
Agriculture	15 (1.9-58)	1.0 (0.05-3.9)	40 (6.3-57)	2,500 (449-6,594)	-
Forest	4 (1.3-16)	0.25 (0.01-1.4)	8 (6.3-11)	400 (71-620)	-
Pasture	8 (3.9-13.3)	0.5 (0.4-1.0)	17 (9.4-27)	670 (19-1,320)	-
Feedlots	1,700 (1,080-2,290)	370 (200-610)	-	-	-
Landfill	1,250(b) (50-2,500)	-	15,000 (80-33,100)	-	-
Urban	8 (3.3-28)	1.3 (0.4-7.9)	70 (20-129)	3,400 (306-7,526)	1,000(c) (1,000-24,000)

(a) References 14-38

(b) Runoff concentration in mg/l

(c) Runoff concentration in numbers/100 ml variability of the estimates in Table 2-13. A method for estimating this variability is discussed in Chapter 4.

2.5.4.1 Urban Non-Point Runoff - Application

The sources of urban runoff considered in this analysis are areas of the city which are not sewered by either combined or separate storm sewers and the runoff from sanitary landfill areas. The non-sewered urban area should be located with respect to the river and its drainage area (square miles) determined. If mass loading rates are available from previous studies, they are used in the impact analysis. Generally, site specific loading rates will not be available and the literature values presented in Table 2-13 should be used to estimate the mass loading rates for total nitrogen, total phosphorus, BOD_5, total coliform bacteria, and total suspended solids.

The sewered urban area of the hypothetical example city, Jefferson City, is 10 square miles. The non-sewered runoff loads, as calculated from the information in Table 2-13, are presented along with the other non-point source loads in Table 2-15. As with the combined sewer overflows, the non-sewered urban runoff occurs only between Milepoints 15 and 20 and is treated as a linearly distributed mass discharge.

2.5.4.2 Rural NPS Runoff - Application

For the preliminary analysis, rural NPS runoff is assumed to originate from forests, agricultural areas, feedlots, pastures, and undisturbed natural areas. If site specific mass discharge rates for total nitrogen, total phosphorus, BOD_5, total coliforms, and total suspended solids are available, they are used in the impact analysis.

If mass discharge data is not available, estimates are made based on land use and total land area. To do this, it is necessary to sub-divide the drainage basin into major land use types. The major land uses are aggregated along the length of the river. For example, agricultural areas may represent ten square miles per linear mile of river in a certain river basin. The mass discharge rate for land uses other than the undisturbed lands is estimated using the drainage areas and the areal loading rates shown in Table 2-13.

Background water quality concentrations are usually available for most drainage basins. In general, background water quality is affected by land uses and water use practices in drainage basins upstream of 208 drainage areas. Therefore, water quality variables will be at concentrations in excess of undisturbed background water quality. If no data is available, background water originating from relatively undisturbed drainage areas can be estimated using the values in Table 2-14.

The cumulative drainage area distribution associated with each land use in the hypothetical South River drainage basin is presented in Table 2-2 and Figure 2-6. In summary, the drainage areas are: upstream, 500 square miles; tributary, 250 square miles; agriculture, 280 square miles; forest, 150 square miles; urban, 40 square miles. In each river reach, the change in drainage area for the agricultural areas and forest are determined and associated with an areal loading rate from Table 2-13. These loading rates are presented in Table 2-15 as linear mass discharge rates (lbs/mi/day). The upstream and tributary mass discharge rates are assumed to be based on observed data and are not calculated from the data in the loading Tables.

2.5.5 Summary Analysis of Mass Discharges

The results of the preliminary loading estimates for each reach of the hypothetical South River are shown in Figure 2-17(a). The distributed sources (lbs/mi/day) are aggregated into the total mass discharge entering the reach of river being considered in order that a comparison to the point loading can be made. Such a comparison can be quite useful in assessing the relative importance of each type of loading. However, until some form of receiving water analysis is performed it is unclear if these loads are producing significant water quality problems.

For the hypothetical South River, the agricultural loads for nitrogen predominate in the upstream reaches; point and combined sewer overflow loads predominate in the downstream reach. The point source for phosphorus

TABLE 2-14

SUMMARY OF BACKGROUND CONCENTRATIONS(a) FROM VIRGIN LAND

Parameter	Concentration Range (mg/1)	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 (39) for iso-concentration maps of virgin land runoff concentrations

(b) Number/100 ml

TABLE 2-15

SUMMARY OF NON-POINT SOURCE AND COMBINED SEWER LOADS FOR HYPOTHETICAL SOUTH RIVER BASIN

River Segment (Mile-Points)	Pollutant Source	Total Nitrogen-N lbs/mi/day	Total Phosphorus lbs/mi/day	TSS lbs/mi/day	BOD_5 lbs/mi/day	Total Coliform No./day/mi x 10^{10}
0 - 5	Agriculture	90	6	15,000	240	37
	Forest	32	2	3,200	64	10
	Urban Runoff	-	-	-	-	-
	Comb. Sewer	-	-	-	-	-
5 - 15	Agriculture	195	13	32,500	520	79
	Forest	10	0.6	1,000	20	3
	Urban Runoff	-	-	-	-	-
	Comb. Sewer	-	-	-	-	-
15 - 20	Agriculture	150	10	25,000	400	61
	Forest	8	0.5	800	16	2
	Urban Runoff	16	2.6	6,800	140	49
	Comb. Sewer	120	38	4,940	1,435	36,000
20 - 33	Agriculture	81	5.4	13,460	215	33
	Forest	23	1.4	2,300	46	7
	Urban Runoff	-	-	-	-	-
	Comb. Sewer	-	-	-	-	-

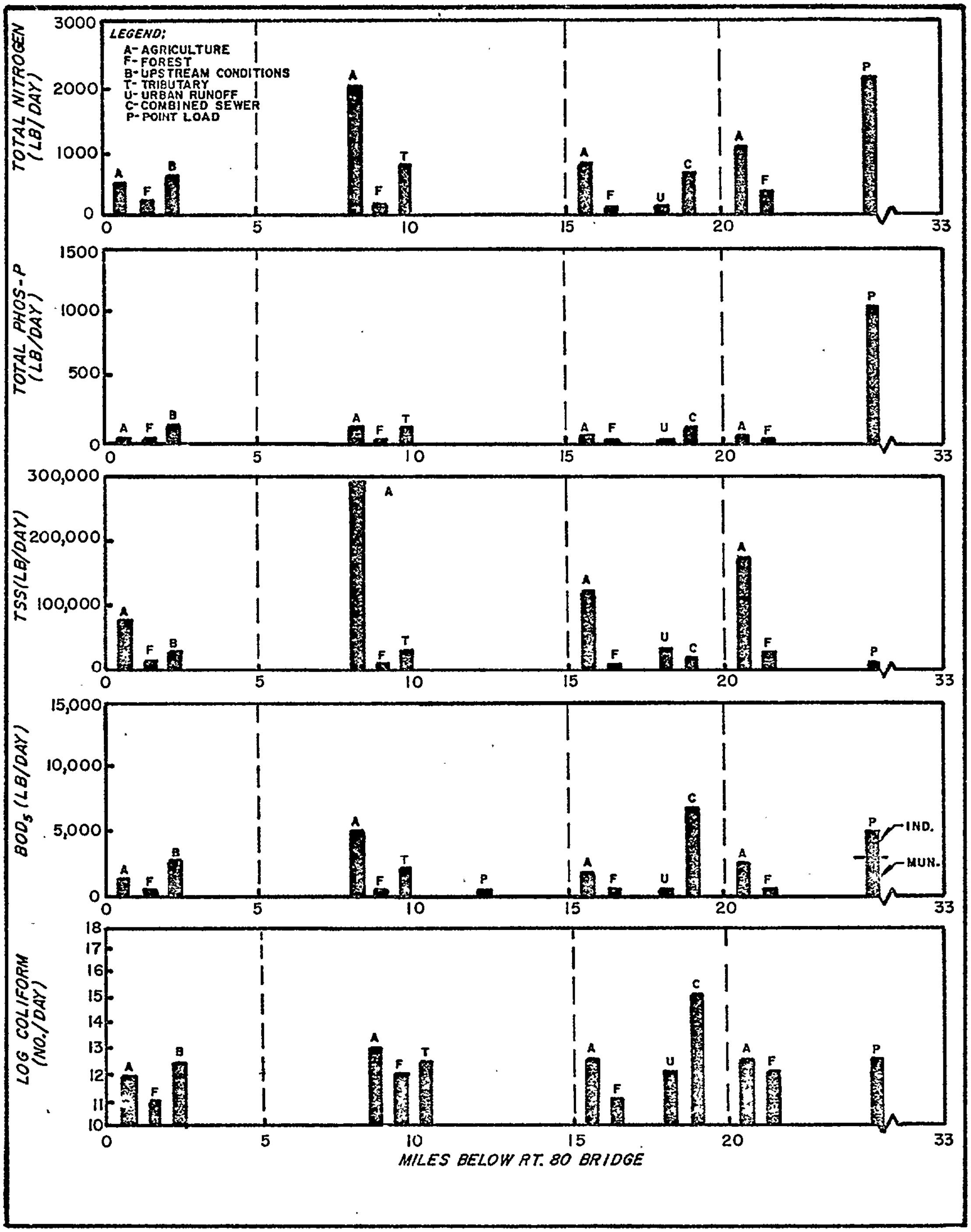


FIGURE 2-17 (a)
COMPARISON OF POINT SOURCE AND NONPOINT SOURCE
LOADS PER RIVER SEGMENT
(ANNUAL AVERAGE)
HYPOTHETICAL SOUTH RIVER EXAMPLE

predominates over all other sources, whereas the agricultural source for suspended solids predominates. Substantial quantities of BOD are derived from agricultural, point, and combined sewer overflow, whereas combined overflow dominates the coliform discharge by more than two orders of magnitude.

It is important to realize that the estimates used in this presentation can have a probable three-fold uncertainty due to the uncertainty in the quantities used in the estimates. However, most conclusions reached from an inspection of this figure are not substantially changed. Agricultural, point and combined sewer sources contribute the majority of the load for some or all constituents, whereas forest, upstream inflows, and tributary sources are small in comparison.

In summary, Figure 2-17(a) allows the analyst to put in perspective the relative importance of the point and non-point source pollutional loadings. At this point, the analyst might make a preliminary estimate as to which of the sources of pollution can be omitted from further analysis. What the analysis does not illustrate is the affect that each of the loads has on the instream water quality. Therefore, it is necessary to continue the preliminary analysis in order to determine the impact of these loads on receiving water quality.

2.6 Receiving Water Analysis

The analysis of the water quality of receiving waters is an integral part of a preliminary assessment for a planning study. By establishing the cause and effect relationship between the mass discharges and the resulting concentrations, a rational assessment can be made of the importance of the various sources being considered in terms of their affect on receiving water.

From the point of view of water quality control and management, it is desirable to examine water quality problems in terms of specific constituents, or groups of constituents, which are discharged as a result of man's activities and natural phenomena. One of the initial steps in the planning effort is the identification of the water quality problems presently observed and those projected under future conditions of population growth and development. Having identified the significant present and future water quality problems, it is then necessary to select the constituents which are discharged to the environment, from natural and man-made activities, that are responsible for water quality problems. It is then appropriate to consider a meaningful engineering framework, usually a mathematical model, for analysis of the cause and effect relationships and the methods available for improving and managing the system. The factors which are included in the mathematical analysis are the hydrology and the climatology of the area. From this data, the water balances, the hydraulic circulation, the temperature structures, the assimilation mechanisms and reactions that are involved in the specific water quality problem are developed. Within this framework, each specific water quality problem may be viewed from a characteristic time and space scale which sets the degree of simplicity or complexity of the required mathematical model.

What follows is a discussion of the general principles of water quality analysis in receiving waters. Although the preliminary assessment methodology is restricted to the impact on streams and rivers, the general principles and discussions will form the basis for the subsequent discussions in Chapter 5 that apply to more complex situations and put into perspective the data previously assembled.

2.6.1.1 Time and Space Scales

Certain problems can be attacked relatively quickly, employing the simpler conceptual hydraulic and quality models associated with analysis of long-term phenomena, that is, phenomena associated with a large time scale. The type of problem which is properly addressed in this context is related to the long-term patterns of substances which are conservative, such as dissolved solids or those substances which change at such slow rates that they may be regarded as conservative.

A second scale of time which is appropriate in the investigation of water quality problems is the annual cycle in which the time unit is a week, month, or season. At this intermediate time scale, it may be necessary to account for lateral and vertical spatial variation in water quality. The eutrophication problem is amenable to analysis utilizing this intermediate time scale.

A third time scale is one in which the time unit is hours extending over an interval of one day to possibly one week's period. This time scale establishes a comparable spatial dimension. The spatial scale may, therefore, involve two and possibly three dimensional analyses. Typical problems addressed in this respect would be transient algae blooms, unexpected spills or discharges of pollutant mass from combined sewer overflows.

A wide variety of planning problems can be analyzed using steady state mathematical models which can provide the necessary spatial detail for important water quality variables. Certain phenomena can achieve steady state conditions within a short time interval and as such, can be modeled with relative ease. Examples of the phenomena which can be modeled on a steady state basis are the distribution of bacteria, dissolved oxygen concentrations, and nutrient distributions. These steady state representations are particularly useful because of the ease of model operation and ability to respond rapidly and relatively inexpensively to specific planning questions.

2.6.1.2 Hydrology and Climatology

The hydrology of the basin or metropolitan region, in particular the freshwater flow, is of considerable importance in mathematical modeling. This parameter determines not only the dilution which the sources of mass receive, but also the velocity at which they move downstream. The flow also affects some of the reaction coefficients.

The determination of the water temperature characteristics of the river sets the level of the reaction coefficients in any model related to bacterial or higher order biological activity and the saturation concentration of dissolved oxygen.

2.6.1.3 Hydrodynamics

The hydrodynamic properties of a body of water, for example, velocity, tidal characteristics, and turbulent diffusion, form the basic transport mechanisms which classify the body of water into one of several generic categories to be discussed below. The degree of detailed hydrodynamic information that is required is strongly dependent on the time and space scale of the problem under consideration.

River velocities can often be related to river flows by an exponential relationship (see section 2.3.5). If information is available which correlates velocity with flow (or depth with flow), this information can form a basis for predicting the velocity (or depth profile) regime in a river under different flow conditions.

Tidal velocities can often be obtained from the U.S. Coast and Geodetic Survey Tide and Current Tables, or from direct measurement. The net river flow in estuarine analysis also forms an important input into the mathematical model of estuarine systems. Flow records are often available for estuarine tributaries that would allow one to construct the net river flow regime at the head end of an estuary and downstream along its length. Flows due to incremental drainage area accretions can be readily estimated with data from upstream reaches, using the approach discussed in Section 2.3.3.

For large lakes and coastal waters, the hydrodynamic situation becomes increasingly more complex. Density stratification further adds to the difficulty of specifying the hydrodynamic circulation. For large and complex circulation patterns as in lakes, the hydrodynamic equations must be considered in determining water movements and subsequent pollutant distributions. However, some simplified techniques are available and are discussed in Chapter 5.

2.6.2 Model Classification of Natural Systems

The classification of natural water systems for water quality analysis is based primarily on the number of spatial dimensions which must be considered and on the mixing characteristics of the body of water.

2.6.2.1 Streams and Rivers

The simplest situation 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. The fundamental equation that governs the transport of material in a non-dispersive system for steady state and constant parameters is:

$$\frac{Q}{A}\frac{dc}{dx} = -KC + \frac{w}{A} \qquad (2\text{-}6)$$

where:

c = concentration of substance of interest (mg/1)
t = time (days)
x = distance downstream (miles)
A = cross-sectional area (sq. ft.)
Q = river flow (cfs)
K = first-order decay coefficient (1/day)
w = distributed source of mass (lbs/day/mile)

For a complete specification, 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. For more complex reactants three, four or more equations may be required, all of which interact through reaction kinetics.

2.6.2.2 Estuaries

An estuary is defined here as 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. The primary difference between estuaries and the one-dimensional river flow situation is the dispersive mass transport due to the tidal mixing occasioned by tidal flow reversals. This forms an important transport phenomena in addition to the net freshwater flow through the estuary and, as such, must be included specifically in the analysis.

Several methods are available to directly evaluate the dispersion coefficient (see Chapter 5).

2.6.2.3 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 is usually no dominant mechanisms which determines 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.

A number of attempts have been made to define the hydrodynamic regime associated with lakes, reservoirs and impoundments. In general, the mixing, turbulence and advection are due to winds, seiches, and density differences. From a practical planning standpoint, two options are open to modeling lakes and impoundments. It may be possible to apply some of the refined mathematical techniques which have been developed to evaluate the hydraulic regime as discussed in Chapter 5 and Appendix A. Alternatively, it may be possible and practical, depending on the water quality problem being addressed, to employ observed data and field measurements as an adequate assessment for the hydrodynamic circulations. As an example, it is possible to obtain data on the thermal stratification within the lake or impoundment and accept this as the basis for segmentation of a model of the lake. In addition, it is possible to inject dye into various areas of the lake and determine dispersion, mixing, and circulation patterns from an observation of the transport of dye within the lake.

2.6.2.4 Coastal Waters

Coastal waters encompassing tidal embayments and near shore coastal waters can require two or three-dimensional analyses. The techniques available for evaluation of the hydraulic regime in terms of circulation pattern, dispersional coefficients, etc., are essentially similar to those available for evaluation of these phenomena in lakes and in estuaries. Once again, the particular water quality problem being addressed will dictate the most effective method of developing an adequate understanding of the hydraulic circulation and mixing patterns.

For the preliminary impact analysis which follows, one-dimensional streams are used for illustration purposes. Streams are discussed in the following sections because of their ease of analysis. The calculations for a simple stream can be performed on a desk top calculator or a slide rule. It is understood that many of the 208 study areas include estuaries, lakes and coastal waters. For these more complex water bodies, the analyst will have to rely in the loading estimates and the modeling techniques presented and discussed in Chapter 5 and Appendix A. The solutions of the equations in estuary, lake and coastal modeling are much more complex and generally require computer solution techniques.

2.6.3. Stream Impact Analysis

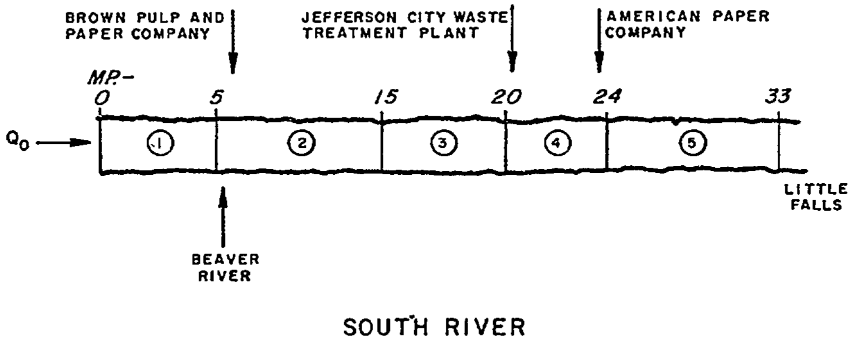
The least complex receiving water body in terms of calculating the impact of wastewater discharges is a stream or river. The essentially one-dimensional character of the transport, together with the characteristically short time to reach equilibrium make possible the simplifying assumptions of one important spatial dimension and temporal steady state. The additional complexity of spatially varying hydraulic and geometric characteristics are approximated by analyzing the receiving water as a sequence of segments within which it is assumed that the hydraulic and geometric parameters are constants.

In a preliminary analysis, it is recommended that the stream be segmented into a maximum of five reaches. The purpose of segmenting 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 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.

For the hypothetical example, the South River is segmented into five stream segments as shown in Figure 2-17(b). The reaches are between Milepoints 0 and 5, 5 and 15, 15 and 20, 20 and 24, 24 and 33. Stream divisions are made at Milepoints 5 and 15 for geometry and flow changes. The divisions at Milepoint 20 and 24 are created for the municipal point source load input and the industrial point source load input.

The average geometry is previously summarized in Table 2-3 and the average flows are summarized in Table 2-16. In order to simplify the equations used in the preliminary stream impact analysis which follows, the average stream flow within the segment is used. This is an approximation of the actual flow which increases linearly from the beginning to the end of the stream reach. The average stream flow is defined as the flow at the beginning of the section plus the flow at the end of the section divided by two.

It should be noted that for the example summer storm analysis, the flow was defined as being constant throughout the study area. This simplification was made because of the large base flow and the variability of the rainfall and runoff coefficient during a short duration storm. Chapters 3 and 4 will provide details necessary to increment the stream flow runoff during a storm event.



LEGEND:
① – MODEL SEGMENT NUMBER

FIGURE 2-17(b)

MATHEMATICAL MODEL SEGMENTATION FOR THE HYPOTHETICAL SOUTH RIVER

TABLE 2-16

HYPOTHETICAL EXAMPLE
SUMMARY OF SOUTH RIVER FLOWS BY MODEL SEGMENT

Segment	Milepoint	Annual Average Flow (cfs)	Summer Flow (cfs)	Summer Low Flow (cfs)	Summer Storm Flow (cfs)
1	0	500 (535)	250 (268)	50 (54)	(1625)
	4.99	570	285	57	
2	5.0	820 (903)	410 (451)	82 (91)	(1625)
	14.99	985	492	99	
3	15.0	985 (1025)	492 (513)	99 (103)	(1625)
	19.99	1065	533	107	
4	20.0	1065 (1088)	533 (544)	107 (109)	(1625)
	23.99	1110	555	111	
5	24.0	1110 (1160)	555 (580)	111 (116)	(1625)
	33.0	1210	605	121	

( ) = average flow in segment

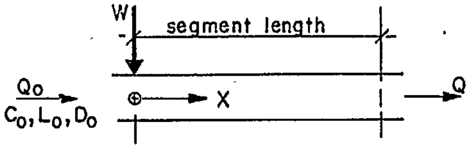
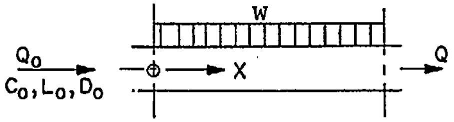
2.6.3.1. Method of Analysis

For the preliminary analysis, all in-stream concentrations are calculated from equations based on the principle of conservation of mass under steady state conditions. Critical seasonal effects are estimated by assuming constant waste and stream characteristics for the particular season. Concentrations will be 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 (point, agricultural, forest, etc.) can be calculated separately and, at a given location, added together to give the total in-stream concentration. In summary, a spatial one-dimensional steady state analysis is performed in order to calculate the distributions of a receiving water constituent throughout the length of the stream in the 208 study area. Constituents to be analyzed are grouped into three categories: conservative, single reactant and coupled sequential reactants.

2.6.3.2. Conservative Constituents

Conservative constituents are those that are not subject to reactive change and remain dissolved or suspended in the stream. For the present, it is assumed in subsequent analyses that total nitrogen, total phosphorus and total suspended solids fall into this category on an annual average basis.

The solutions to the governing linear differential equation for both point and distributed sources are shown in Table 2-17. Note that the spatial coordinate x is in the direction of flow and that it is reset to $x = 0$ at the upstream end of each segment. The constant Co is the in-stream concentration at $x = 0$ due to waste sources entering segments of the river upstream of the segment under consideration. It is calculated by summing up all upstream waste inputs and dividing the sum by the flow at $x = 0$. Incremental increases in concentrations due to wastes entering the segment being analyzed are evaluated by the term W/Q for the point sources and wx/Q for the distributed sources where W is the point source mass discharge rate (pounds/day) and w is the distributed mass discharge

	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:

O = 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 COEFFICIENT
K_d = BOD OXIDATION COEFFICIENT
K_a = D.O. REAERATION COEFFICIENT
C_o, L_o, D_o = CONCENTRATION AT X = 0
W = POINT SOURCE LOADING RATE
w = NON-POINT SOURCE LOADING RATE

TABLE 2-17

SUMMARY OF SOLUTIONS FOR POLLUTANT CONCENTRATIONS IN THE RECEIVING WATERS rate (pounds/mile/day) and x is distance (miles). Concentrations within the segment increase linearly due to distributed sources.

Since there are no removal or growth mechanisms involved, the analysis of conservative substances reduces to a simple additive calculation of accumulating waste loads, by source (point, agriculture, etc.) and dividing the cumulative source total by the appropriate flow. This procedure is illustrated in Table 2-18 for calculating total nitrogen concentrations due to agricultural sources. Repeating this procedure for all sources results in the estimate of the annual average concentrations of a conservative substance in the river. The technique for calculating the total nitrogen concentrations for all sources is illustrated in Figure 2-18. Annual flow is plotted. Then, cumulative waste loadings by source are generated and receiving water concentrations are determined as the quotient of the load and flow. For example, the cumulative total nitrogen loading entering model segment 4 at Milepoint 20 is approximately 7,500 lbs/day. Dividing this by the flow of 1,065 cfs and a conversion factor of 5.4 results in a concentration of 1.35 mg/l, as plotted in the Figure 2-18. Of this total, agricultural sources contribute 0.57 mg/l due to cumulative loads of 3,150 lbs/day and the effluent from the municipal sewage treatment plant (2,135 lbs/day) contributes 0.38 mg/l.

2.6.3.3. Reactive Constituents

Reactive constituents are subject to change within the receiving water due to physical, chemical and biological reactions. The variables included in the preliminary analysis framework that fall into this category are BOD, coliform bacteria and nutrients. For reactive substances, the critical season is usually the low flow, high temperature period of the year. Therefore, the analysis is performed for that period. Although total nitrogen and phosphorus are treated as conservative on an annual average basis, they are considered reactive during the summer low flow period due to algal uptake of the nutrients and subsequent removal by settling.

TABLE 2-18

HYPOTHETICAL EXAMPLE

EXAMPLE CONSERVATIVE SUBSTANCE IMPACT CALCULATION TABLE AGRICULTURAL NON-POINT SOURCE LOADS

Section Number	River Mile-Point	T-N[(a)] added (lbs/day)	T-N Cummulative (lbs/day)	Q[(b)] Cummulative (cfs)	Total Nitrogen Concentration (mg/l)
1	0	0	0	500	0
	4.99	450	450	570	0.15
2	5.0	0	450	820	0.10
	14.99	1950	2400	985	0.46
3	15.0	0	2400	985	0.46
	19.99	750	3150	1065	0.57
4	20.00	0	3150	1065	0.57
	23.99	324	3474	1110	0.59
5	24.0	0	3474	1110	0.59
	33.0	729	4203	1210	0.65

(a)T-N added = T-N (lbs/day/mi) x length(mi)

(b)Annual average flows

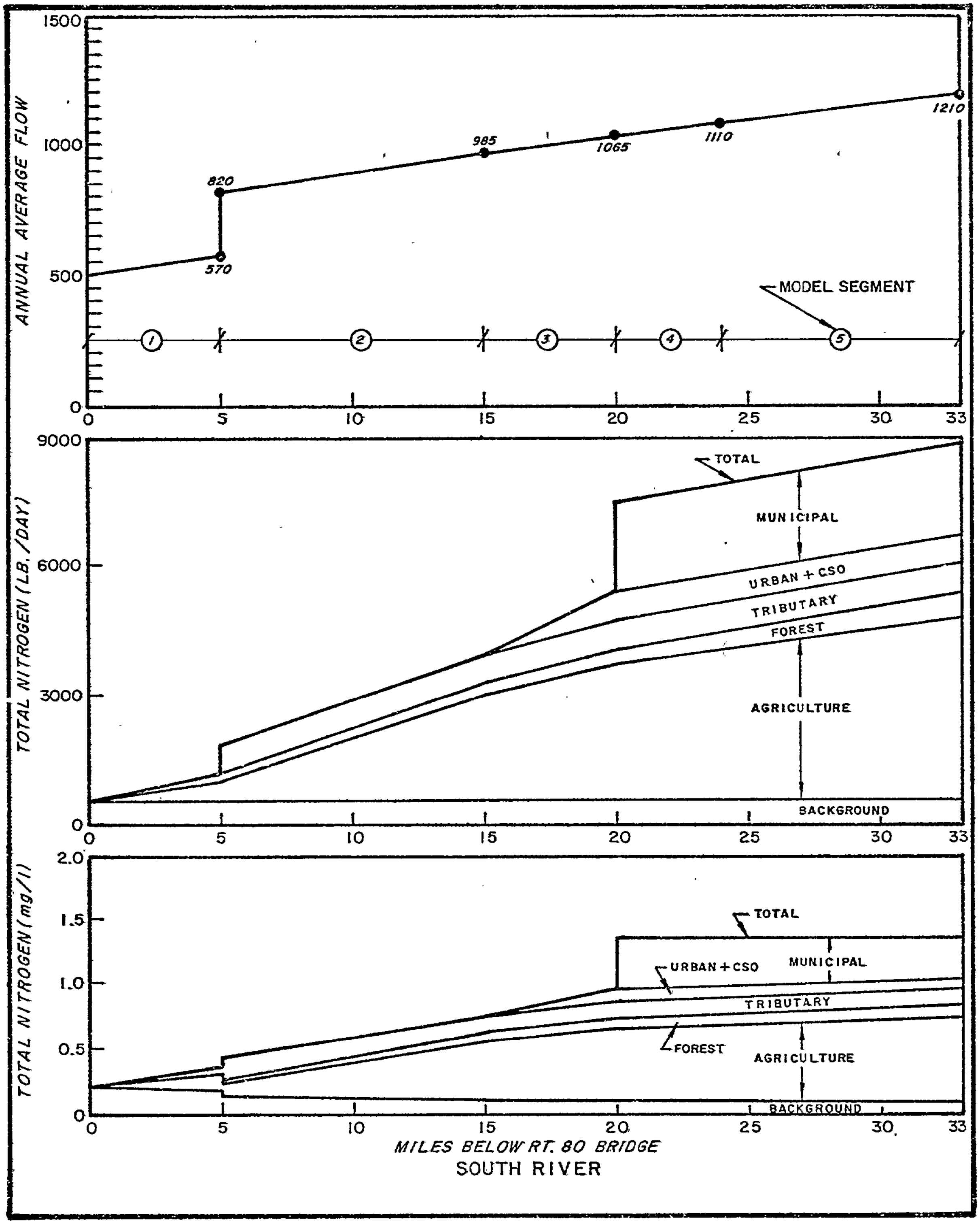


FIGURE 2-18
INSTREAM TOTAL NITROGEN CALCULATION
(ANNUAL AVERAGE FLOW)
HYPOTHETICAL EXAMPLE

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 2-19. The BOD reaction rates are particularly applicable to the carbonaceous fraction but, in the preliminary analysis, the rate is also considered appropriate for the nitrogenous oxygen demand. The nutrient removal rates are generally applicable to conversion to other nutrient forms, but the lower range also applies to estimated first order algal settling.

TABLE 2-19

RANGE OF VALUES OF REACTION COEFFICIENTS IN STREAMS
(REF. 6)

Substance	K_r (per day)[(a)]
Coliform Bacteria	1 - 3
BOD_5	0.2 - 2.0
Nutrients	0.1 - 1.0

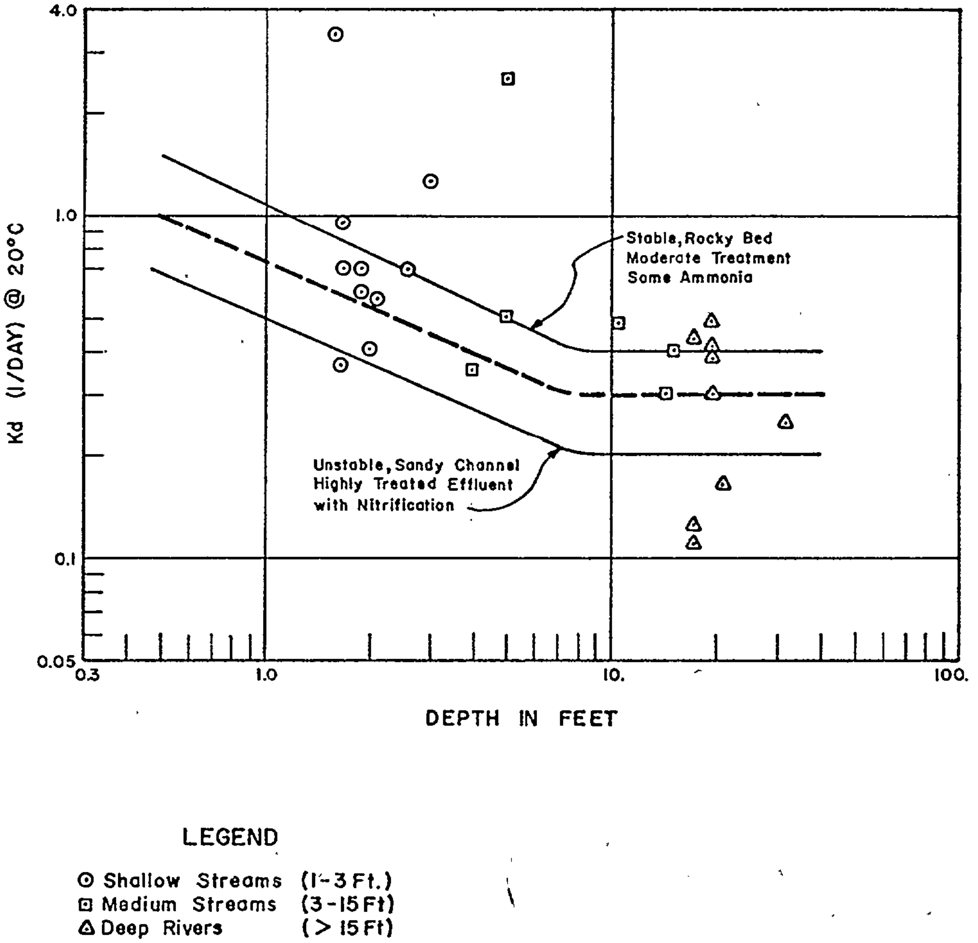
(a) Base e, 20°C

The coefficients in Table 2-19 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:

$$K_r(T) = K_r(20)(1.047)^{T - 20} \qquad (2\text{-}7)$$

where $K_r(T)$ is the reaction coefficient at temperature, T(°C), and $K_r(20)$ is the reaction coefficient at 20°C.

In this preliminary analysis, the BOD_5 reaction coefficient (K_r) accounts for both the oxidation and the settling of BOD_5. In the dissolved oxygen analysis which follows, the BOD_5 oxidation rate is assumed as being equal to the BOD_5 removal rate. Therefore, there is no settling of BOD_5. An estimate of the BOD_5 oxidation rate can be made using the information in Figure 2-19(a). This Figure relates the oxidation rate to the stream depth and is based on data collected during many stream studies.



SOURCE: (6)

FIGURE 2-19 (a)

OXIDATION COEFFICIENT (Kd) AS A FUNCTION OF DEPTH

Receiving water concentrations caused by point and distributed sources are calculated from the solutions cited in Table 2-17 for reactive substances. The form of the solution is similar to that for conservative constituents where the first term represents the effects of upstream loads and the second term represents the impact of point or distributed sources entering the segment being analyzed. In the first term, the constant L_o represents the residual concentration at $x = 0$ due to all upstream sources and the exponential accounts for decay of L_o throughout the segment being analyzed. The symbol e is the base of natural logarithms (2.718). K_r is the decay rate at the summer water temperatures and U is the average low flow water velocity in the segment ($U=Q/A$ = average flow/ cross-sectional area). In some cases, time of travel data will be available and should be used for the calculation of freshwater velocity. Measured time of travel data provides the analyst with accurate stream transport information and should be used instead of the calculated freshwater velocity (Q/A). In this preliminary analysis, the reaction rate for each constituent is assumed constant for all waste sources and for all segments. Refinements to this procedure are discussed in subsequent chapters. The second term of the reactive point source solution is similar in behavior to the first term, with W/Q analogous to L_o. The maximum effect occurs at $x = 0$ and then decreases in the downstream direction. For distributed reactive waste inputs, there is no stream impact at $x = 0$ and a build-up of concentration occurs downstream, with a maximum possible value to $w/(A.K_r)$, as x approaches infinity.

Sample calculations for five-day BOD are contained in Table 2-20 for the summer low flow period. As indicated in the Table, the average flow, cross-sectional area and stream velocity are first determined together with the appropriate temperature-adjusted reaction rate. The initial concentration due to upstream sources (L_o) is then calculated from a mass balance at $x = 0$. Aggregated point and distributed sources are then entered. Substitution of the above data into the solutions presented in Table 2-17, results in a spatial distribution of five-day BOD within the segment being analyzed. Similar computations follow for subsequent segments.

TABLE 2-20

HYPOTHETICAL EXAMPLE
REACTIVE CONSTITUENT (BOD_5) SAMPLE IMPACT CALCULATION

SUMMER LOW FLOW ANALYSIS

SEGMENT 1 - MILEPOINTS 0.0 to 5.0

Q	=	268 cfs	L_o	=	UPSTR. BOD_5 = 1.0 mg/1
A	=	566 sq. ft.	L_o	=	$(L_o) . (Q_o/Q_1)$ = (1.0) . (250/268) = 0.933mg/1
U	=	(Q/A) . (16.36) = 7.75 mi/day	W	=	0
K_r	=	0.3 @ 20°C & 0.377/day @ 25°C	w	=	304 lb/mi-day (AG + FOR)

Rewriting Equations of Table 2-17 with conversion factors for units:

$$L(X) = L_o e^{-K_r x/U} + (W/5.39Q)\ e^{-K_r x/U} + (3.04\ .\ w/(A.K_r))\ .\ (1 - e^{-K_r x/U})$$

for x = 1 mi,

$$L(1) = 0.933\ e^{-0.377(1)/7.75} + 0 + (\ 304\ \ .\ \ 3.04/(566\ \ .\ \ 0.377)).(1 - e^{-0.377(1)/7.75})$$

$$L(1) = 0.889 + 0 + 0.201 = 1.09 \text{ mg/1}$$

for $x = 5^-$ mi,

$$L(4.99) = 1.667 \text{ mg/1}$$

SEGMENT 2 - MILEPOINTS 5.0 to 15.0

Q	=	451 cfs	L_o	=	(1.667) . (268/451) = 0.991 mg/1
A	=	990 sq. ft.	W	=	1444 lb/day (trib. & indust)
U	=	7.45 mi/day	w	=	540 lb/mi-day (AG & FOR)
K_r	=	0.377/day			

for MP 5, x = 0; $L(0) = 0.991\ e^{-0.377(0)/7.45} + (1444/(451)\ .\ (5.39))\ e^{-0.377(0)/7.45}$

$$+ (540)\ .\ (3.04/(990\ x0.377))(1-e^{-0.377(0)/7.45})$$

$$L(0) = 0.991 + 0.594 + 0.00 = 1.59 \text{ mg/1}$$

$$L(14.99) = 2.703 \text{ mg/1}$$

Procedure repeated for segments 3, 4, and 5.

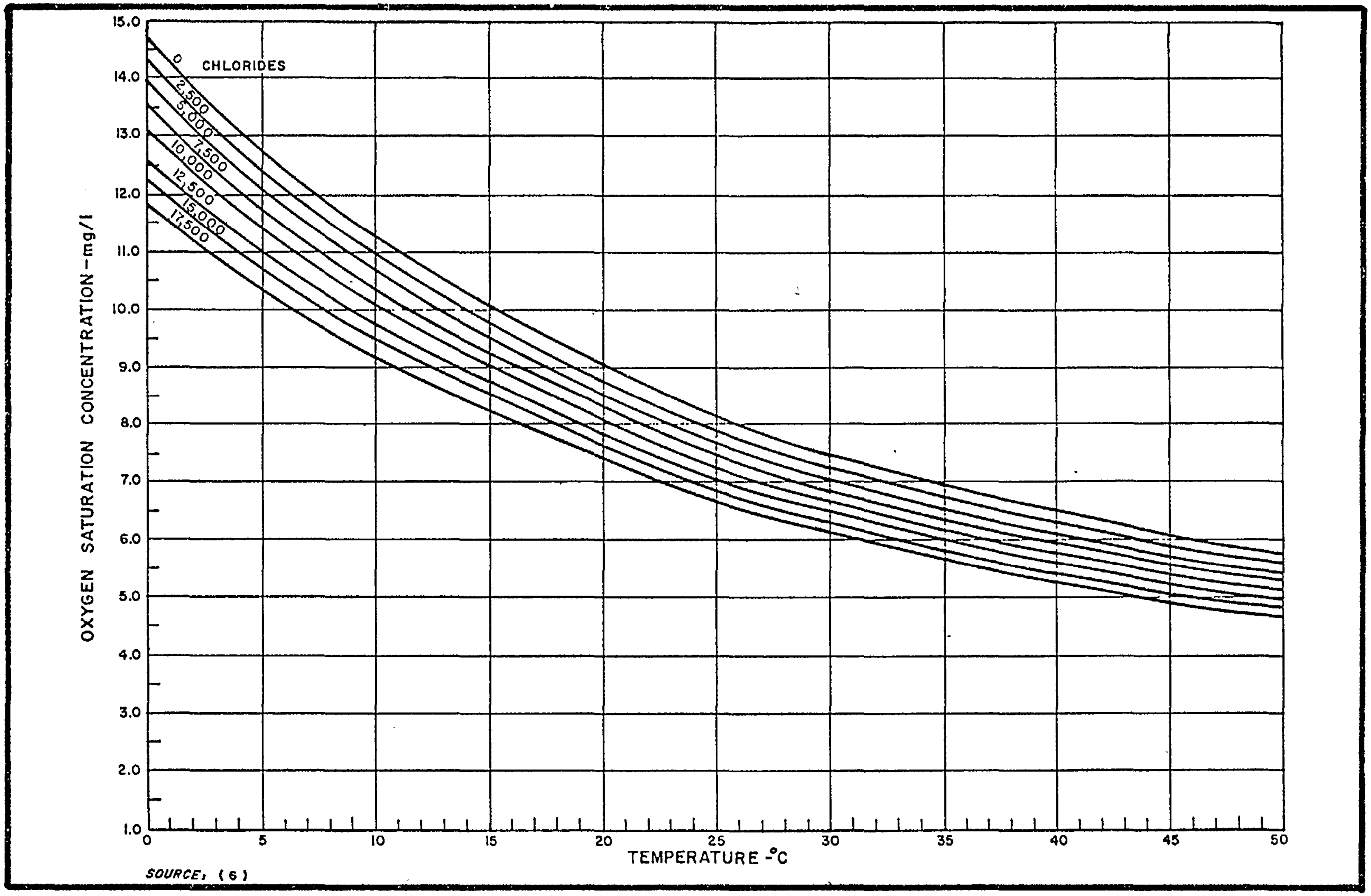
The procedure is suitable for an ultimate BOD analysis (carbonaceous and nitrogenous) as well as for coliform bacteria and summer nutrient analyses. Relative effects of each point source and each distributed source (by land use type) are easily determined by substitution of the single source of interest in the appropriate segment(s). The procedure can be tedious and use of a programmable calculator is recommended.

2.6.3.4 Sequentially Reacting Constituents

Sequential reactants occur if the growth or removal of the initial constituent causes changes in a second constituent. For the preliminary analysis, the initial substance being considered is ultimate oxygen demand (UOD) and dissolved oxygen deficit is the second substance. Thus, the removal of UOD causes an uptake of oxygen and an increase in the DO deficit of the stream. The deficit itself is reduced through reaeration. Stream dissolved oxygen concentrations are calculated by deducting the DO deficits from the temperature-dependent saturation concentration. Since saturation levels are lowest during the summer high temperature periods, and reaction rates are highest, analyses for dissolved oxygen will be carried out for summer low flow, drought flow and storm flow conditions. Curves of dissolved oxygen saturation concentrations versus temperature are shown in Figure 2-19(b).

As mentioned above, the ultimate oxygen demand from waste sources (carbonaceous and nitrogenous BOD) will be considered as the source of DO deficit. A single reaction rate will be used for both components and confirmatory analyses to determine whether nitrification occurs will be discussed in Chapter 5. For the preliminary analysis, the removal rate of UOD (K_r) will be set equal to the uptake rate of oxygen (K_d), which assumes that settling of the BOD is insignificant. Reaeration coefficients (K_a) will be calculated from the O'Connor-Dobbins formulation (Reference 40)

$$K_a = 12.96\ U^{1/2} \Big/ H^{3/2} \qquad @\ 20^{\circ}C \tag{2-8}$$



SOURCE: (6)

FIGURE 2-19 (b)

D.O. SATURATION - TEMPERATURE - CHLORIDE RELATIONSHIP where 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:

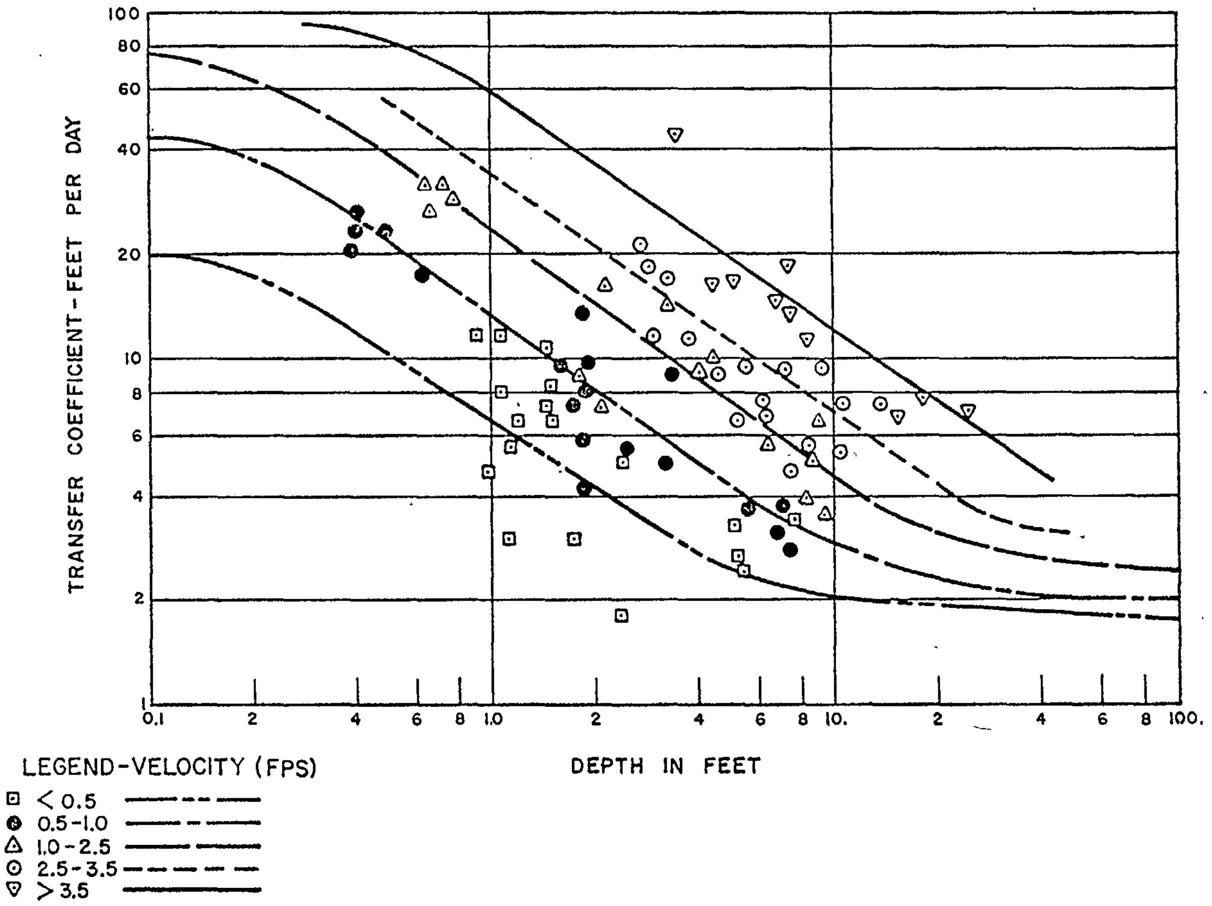
$$K_a(T) = K_a(20)\ (1.024)^{T-20} \tag{2-9}$$

Figure 2-20 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.

The effect of benthal demands and the daily averaged algal effect are not included and should be included in a more refined analysis if appropriate.

Dissolved oxygen deficits in the stream are calculated from the equations in Table 2-17 which contain the reaction rates K_a, K_r and K_d discussed above. The symbol D represents the deficit, D_o is the deficit at x = 0 due to all upstream sources of UOD, and L_o is the residual concentration of UOD at x = 0 due to the upstream sources. The third terms of the equation, beginning with W/Q and $w/A.K_r$, represent the effects of the point and distributed sources of UOD entering the segment under analysis.

Sample calculations for the dissolved oxygen analysis under summer low flow conditions are contained in Table 2-21. For the first segment, average geometry and flow information (Q, A, H) for the segment is entered, and the average velocity is calculated. The assumed reaction rate of the UOD is temperature corrected to 25°C. The reaeration rate is then calculated for the velocity and depth of the segment and adjusted for the temperature of 25°C. Initial deficit from upstream sources is flow-adjusted to D_o = 0.93 mg/1. For carbonaceous BOD, the initial in-stream concentration (L_o) is calculated and the magnitudes of the BOD_5 point and distributed loads entered. Similar entries are made for the nitrogenous BOD constituent. Note that the agricultural and forest nitrogen loads are considered to be non-reactive since much of this input is either slowly reacting or in the inorganic (nitrate) form. The initial BOD_5 concentration and loadings are then expressed as UOD by scaling the BOD_5 by 1.5 (an estimate of the ratio of ultimate carbonaceous



SOURCE: (6)

FIGURE 2-20

TRANSFER COEFFICIENT (K_L) AS A FUNCTION OF DEPTH

TABLE 2-21

COUPLED (BOD - DO) SAMPLE IMPACT CALCULATION

SUMMER FLOW ANALYSIS

HYPOTHETICAL SOUTH RIVER EXAMPLE

SEGMENT 1 - MILEPOINTS 0.0 to 5.0

Q = 268 cfs
A = 566 sq. ft.
H = 2.3 ft.
U = (Q/A) . 16.36 = 7.75 mi/day
K_r = 0.3 @ 20°C & 0.377 @ 25°C

K_d = K_r = 0.377 @ 25°C
K_a = $12.96\ u^{\frac{1}{2}}(ft/sec)/H^{3/2}(ft)$ @ 20°C
K_a = $12.96(0.473^{\frac{1}{2}}/2.3^{3/2}) = 2.555$ 1/day
K_a = $2.555\ (\ 1.024^{25-20})$ @ 25°C = 2.877 1/day

Def : $D_o = (D_o)\ .\ Q_o/Q_1 = (1.0)\ .\ 250/268 = 0.932$ mg/l

CBOD : $L_o = (L_o)\ .\ Q_o/Q_1 = (1.0)\ .\ 250/268 = 0.932$ mg/l
W = 0
w = 304 lb/mi-day (AG & FOR)

NBOD : $L_o = 0$
W = 0
w = 0 (AG & FOR assuming non-reactive)

UOD : $L_o = (0.932)\ .\ 1.5 + (0)\ .\ 4.57 = 1.40$ mg/l
W = 0
w = (304) . 1.5 + (0) . 4.57 = 456 lb/mi-day

Rewriting equations of Table 2-17 with conversion factors for units:

$$D(x) = Do\ e^{-K_aX/U} + L_o \cdot \frac{K_d}{K_a-K_r}\ (e^{-K_rx/u} - e^{-K_ax/U}) + (W/Q\ .\ 5.39) \cdot \frac{K_d}{K_a-K_r} \cdot (e^{-K_rx/U} - e^{-K_ax/U})$$

$$+ \frac{w}{A.K_r} \cdot \frac{K_d}{K_a-K_r}\ .\ 3.04\ .\ (\frac{K_r}{K_a}\ e^{-K_ax/U} - e^{-K_rx/U} + \frac{K_a - K_r}{K_a})$$

for x = 1 mi,

$$D(1) = 0.932\ e^{-2.8(1)/7.75} + (1.40) \cdot \frac{0.377}{2.877-0.377}\ (e^{-0.377(1)/7.75} - e^{-2.877(1)/7.75}) + 0$$

$$+ (\frac{456}{566\ .\ 0.377} \cdot (\frac{0.377}{2.877-0.377})\ .\ 304\ .\ (\frac{0.377}{2.877}\ e^{-2.877(1)/7.75} - e^{-0.377(1)/7.75}$$

$$+ \frac{2.877 - 0.377}{2.877}$$

D(1) = 0.643 + 0.056 + 0 + .003 = 0.702 mg/l
DO = C_s - D(1) = 8.17 - 0.702 = 7.15 mg/l
:
D(4.99) = 0.38 mg/l
DO = 7.79 mg/l

TABLE 2-21
(Continued)
COUPLED (BOD - DO) SAMPLE IMPACT CALCULATION
SUMMER FLOW ANALYSIS

SEGMENT 2 - MILEPOINTS 5.0 - 15.0

Procedure repeated for Segment 2

D(5.00) = 0.50 mg/;

DO = 7.67 mg/l

Q_2 = 451 cfs

:

D(14.99) = 0.92 mg/l

L(14.99) = 2.70 mg/l

DO = 7.24 mg/l

SEGMENT 3 - MILEPOINTS 15.0 - 20.0

Q = 513 cfs

A = 990 cfs

H = 3.8 ft

U = (Q/A) . 16.35 = 8.48 mi/day

K_r = K_d = 0.3 @ 20°C = 0.377 @ 25°C

K_a = 12.96 $(.518)^{½}/(3.8)^{3/2}$ = 1.26 @ 20°C

K_a = 1.40 1/day @ 25°C

CBOD: L_o = upstream BOD = (2.70) . $\frac{451}{513}$ = 2.97 mg/l

W = 0

w = 1991 lbs/mi-day

NBOD: L_o = upstream oxidizable nitrogen = 0

W = 0

w = 136 lbs/mi-day

UOD:

L_o = (2.37) . 1.5 = 3.56 mg/l

w = (1991) . 1.5 + (136) . 4.57 = 3608 lbs/mi-day

W = 0

DEF: D_o = (0.92) . $\frac{451}{513}$ = 0.81 mg/l

At mile 15 x = 0, D_0 = 0.81

At mile 17 x = 2, $D(2) = 0.81\ e^{-1.42(2)/8.48} + (3.56)\ .\ \frac{0.377}{1.42-0.377}\ (e^{-0.377(2)/8.48} - e^{-1.42(2)/8.48}) + 0$

$$+ (\frac{3608}{990\ .\ 0.377})\ .\ (\frac{0.377}{1.42-0.377})\ .\ (3.04)\ .\ (\frac{0.377}{1.42}\ e^{-1.42(2)/8.48} - e^{-0.377(2)/8.48}$$

$$+ \frac{1.42-0.377}{1.42})$$

D(2) = 0.59 + 0.25 + 0 + 0.11 = 0.95 mg/l

DO = 8.17 - 0.95 = 7.22 mg/l

D(19.99) = 1.35 mg/l, D.O = 6.82 mg/l

Procedure repeated for Segment 4, and 5

BOD to BOD_5). The ratio of ultimate CBOD to BOD_5 is not always equal to 1.5 and the 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 preliminary 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.

Substitution of the above data into the equation for coupled constituents (Table 2-17), allows calculation of dissolved oxygen deficits at any location in segment 1. Subtraction of the deficits from the saturation concentration of 8.17 mg/l at 25°C (Figure 2-21) results in the predicted dissolved oxygen concentrations. This procedure is then repeated for downstream segments, as illustrated for segment 3 in Table 2-21.

Effects of individual waste sources on the dissolved oxygen deficit are then determined by inputting each source into the equations and calculating resulting deficit concentration profiles. Programmable calculators are quite helpful to reduce the time required for these calculations.

2.7 Illustration and Interpretation of Preliminary Impact Analysis

The simple methodology presented in the impact analysis section of this chapter is applied to total nitrogen, total phosphorus, dissolved oxygen, total coliform bacteria and total suspended solids. Spatial distributions of these constituents are calculated using the available data and the cited literature estimates. Since the preliminary impact analysis is performed using estimated loading information, it should be recognized that the analysis is as accurate as the estimated numbers used in the analysis. Three-fold or more changes in computed concentrations in the receiving water can result if either significant loads or receiving water characteristics (e.g., reaction rates) are in error. However, this uncertainty is inherent in any preliminary assessment which relies

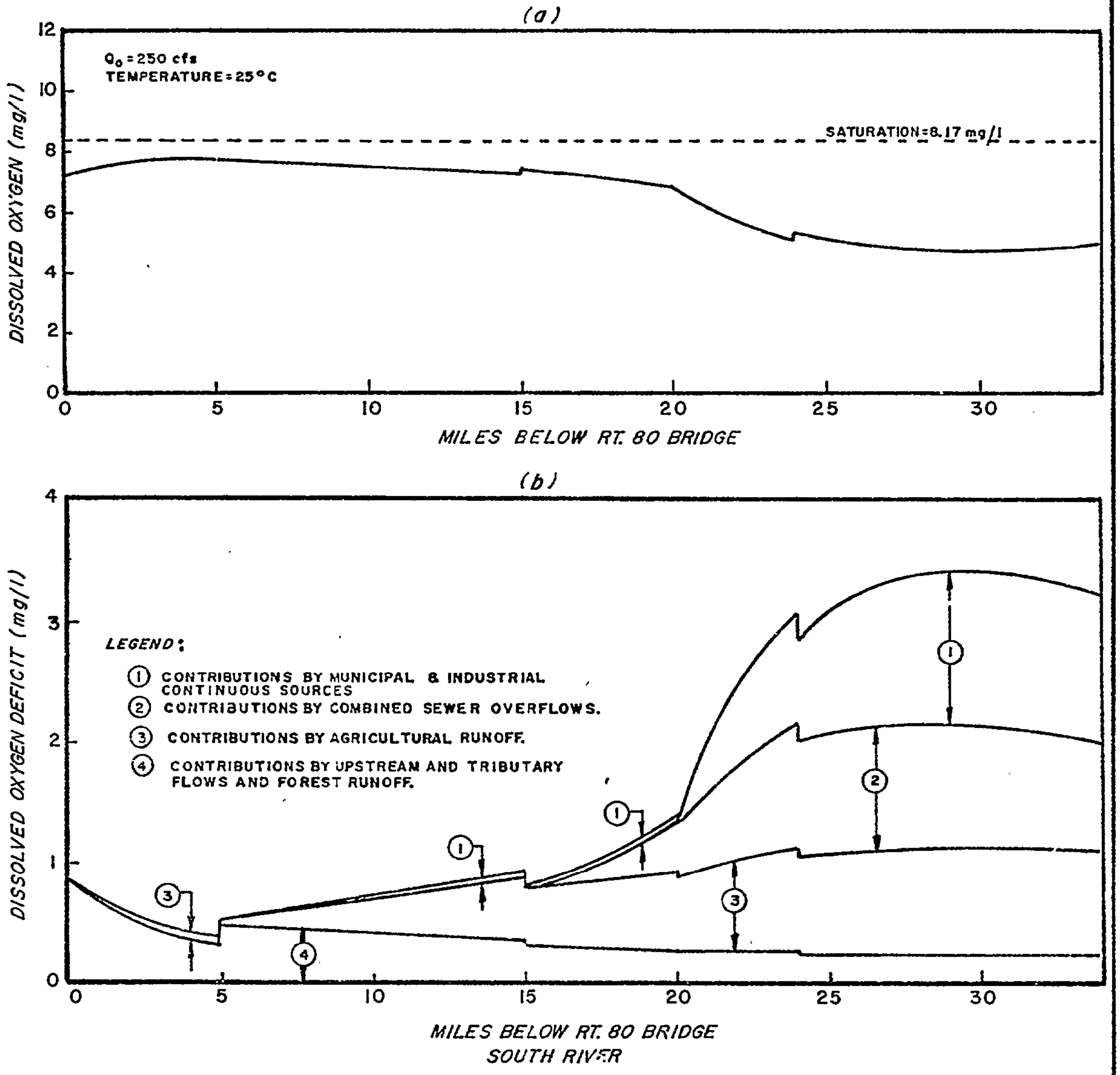


FIGURE 2-21
COMPUTED ESTIMATES OF DISSOLVED OXYGEN DISTRIBUTIONS (SUMMER AVG. FLOW) AND COMPONENT D.O. DEFICIT DISTRIBUTIONS HYPOTHETICAL EXAMPLE

on average conditions and rule-of-thumb estimates. Its utility is not diminished by these uncertainties, rather the results of the calculations should be interpreted with the limitation clearly in mind. For each type of analysis, a suggested set of rules is given in order to place the calculations in proper perspective. Available water quality data can also be used to assess the accuracy of the analysis and to correct gross inconsistencies.

2.7.1 Dissolved Oxygen Analysis

2.7.1.1 Summer Average Flow

Summer is generally the critical period for stream dissolved oxygen levels. Low river flows reduce the dilution of point and non-point source loads and high river temperatures increase reaction rates and reduce the dissolved oxygen saturation level thereby lessening the assimilation capacity of the river. Impact analyses are presented for average summer flow conditions and the minimum average 7-consecutive-day, one-in-ten-year, low flow.

First, if there is sufficient data, the observed dissolved oxygen data is compared with the stream standards to determine if there is a problem at summer flows and drought flows. This preliminary comparison is performed before the impact analysis. If the observed dissolved oxygen levels are well above the standards (no violations), then the impact analysis for dissolved oxygen is not performed. However, if the observed data is marginal with respect to the standards or violates the standards or if no pertinent data is available, then the impact analysis is necessary.

The interpretation of the preliminary analysis is based on an assessment of the reliability of the calculation and is shown in Table 2-22.

TABLE 2-22

DISSOLVED OXYGEN ANALYSIS

If Calculated Dissolved Oxygen Deficit is:	Probability of a D.O. Problem
Less than 0.2	Improbable
0.2 to 2.0	Possible
2.0 to 10.0	Probable
greater than 10.0	Highly Probable

For the latter categories further analysis is required.

Figure 2-21(a) shows the calculated dissolved oxygen distribution for the hypothetical South River during the month of July 1975. The solid line in this figure is the calculated dissolved oxygen profile. During July 1975, the flow at the Route 80 Bridge was 250 cfs and the average stream temperature was 25°C. The UOD oxidation rate of 0.3/day is temperature corrected to 25°C and used along with the annual average point and non-point source loads to calculate the dissolved oxygen profile. Figure 2-21(b) shows the component dissolved oxygen deficit responses to all point and non-point source loads. The point sources loads (municipal and industrial), the combined sewer overflows and the agricultural runoff loads each produce about 1.0 mg/l of dissolved oxygen deficit between Milepoints 25 and 33. The contribution of the upstream deficit, tributary deficit, and forest runoff at the point of maximum deficit (Milepoints 25 to 33) is 0.25 mg/l. Based on this analysis, the net effect of all load sources is in the order of 3 mg/l dissolved oxygen deficit. It is, therefore, probable that a DO problem exists in the South River. Data collection efforts and further analysis should be directed towards definition of the oxygen demanding materials originating from the municipal and industrial loads, the combined sewer overflows, and the agricultural runoff. Instream, water quality data collection should be instituted in the region of calculated low dissolved oxygen concentration.

2.7.1.2 Summer Storm Flow

In the preliminary summer storm flow analysis, the combined sewer overflows, urban runoff, and municipal waste treatment plant loads are the major loads. Any increase in loads during storm events, such as a change in upstream water quality and agricultural runoff, cause only minor dissolved oxygen depressions because of the high stream flow rate.

For the hypothetical South River drainage basin, the review of the hydrological data, as discussed in Section 2.3.3, shows the long duration storm selected raised the stream flow by a factor of 3.2 times the annual average flow. This results from a 3 to 5-day storm event with rainfall of 7 times the annual average rainfall. Therefore, the annual average combined sewer loads are increased by a factor of 7 to reflect the increase in intensity when compared to $\bar{I}_t$. The dissolved oxygen deficit response to the combined sewer overflows, urban runoff and waste treatment plant is presented in Figure 2-22.

The maximum deficit produced by the combined sewer overflows and urban runoff is 3.8 mg/l which indicates a probable problem. The combined overflows are the largest contributors of oxygen demanding material to the South River, therefore, additional efforts should be directed towards the collection of site specific data and refining the loading contribution of the combined sewer overflows during this type of storm event.

2.7.1.3 Drought Flow

The drought flow dissolved oxygen profile can be computed as well. Since surface runoff is responsible for the transport of the non-point source loads and since surface runoff is minor during a drought period, only the point sources are used in the impact analysis.

For the hypothetical South River 208 study, the point source loads, both municipal and industrial and the upstream BOD and oxygen deficit, are the only oxygen demanding materials reaching the South River. The summer drought flow is 50 cfs at the Rt. 80 bridge and the stream temperature is 25°C. The deficit response (Figure 2-23(b)shows that oxidation of the UOD

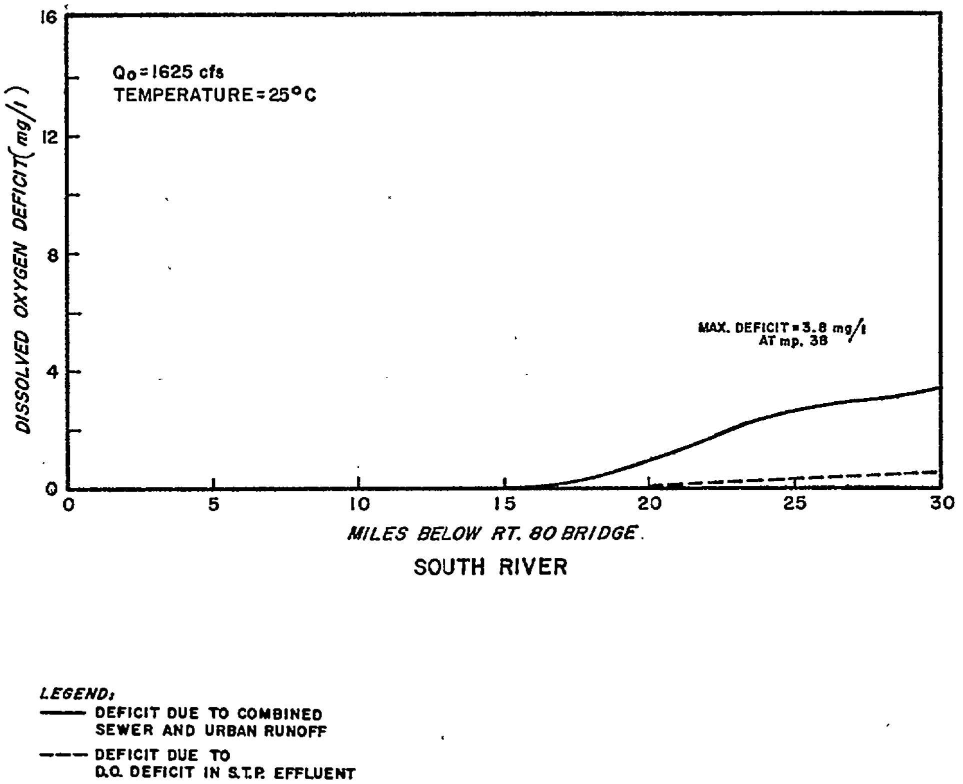


FIGURE 2-22
COMPONENT D.O. DEFICIT DISTRIBUTIONS (SUMMER STORM FLOW)
HYPOTHETICAL EXAMPLE

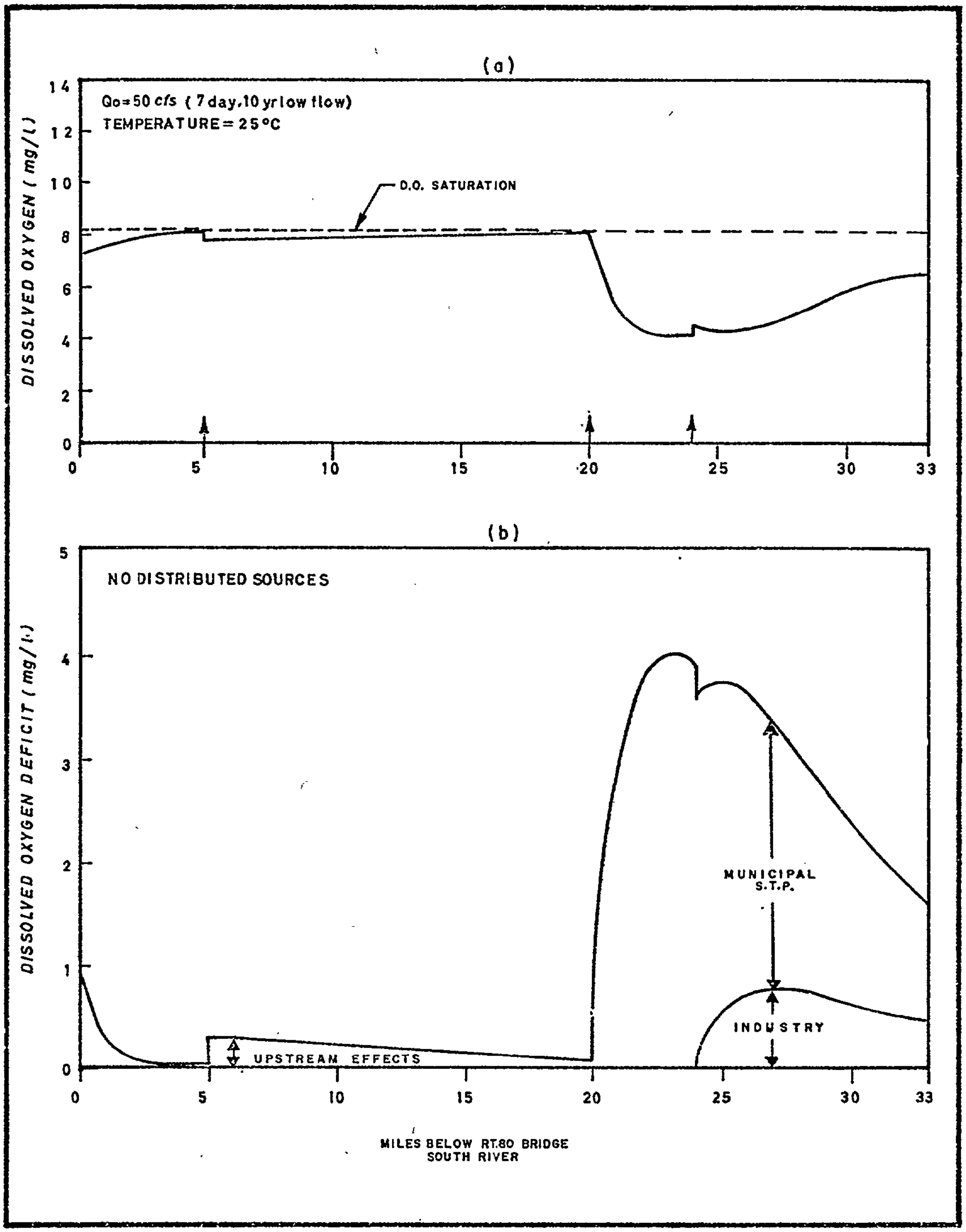


FIGURE 2-23
COMPUTED OXYGEN (DROUGHT FLOW) AND COMPONENT D.O. DEFICIT DISTRIBUTIONS HYPOTHETICAL EXAMPLE

discharge from the municipal Jefferson City STP is responsible for all of the deficit at the point of maximum deficit.

Therefore, based on a preliminary analysis, there is a probable dissolved oxygen violation in the South River at the 7-day, 1-in-10-year low flow.

Because of the probable problem, it is necessary to refine the analysis to increase the confidence level. Recommended areas of refinement are definition of the BOD reaction rate, the NBOD reaction rate, the river geometry at low flow, the benthic oxygen demand and any photosynthetic oxygen production or utilization. These refinements are discussed in Chapter 5.

2.7.2 Public Health

In a preliminary impact analysis, total coliform bacteria are considered as an indicator of the potential for health problems which arise from direct contact with the water. Since direct human contact with water usually occurs during the summer months, an annual average impact analysis is improper in most areas and only summer average on storm event analyses are performed.

For the interpretation of the results of a preliminary coliform analysis Table 2-23 can be used for guidance. In addition, a recent study by Johns Hopkins University (41), which relates total coliform bacteria to water-borne disease causing organisms, may aid in the interpretation of results.

TABLE 2-23

TOTAL COLIFORM ANALYSIS

If the Calculated Concentration Is:	Probability of a Coliform Problem
less than 100/100 ml	Improbable
less than 1000/100 ml	Possible
more than 1000/100 ml	Probable
more than 10,000/100 ml	Highly Probable

2.7.2.1 Summer Average Flows

The hypothetical South River coliform loading data has been presented in Table 2-15. In the impact analysis, the coliform decay rate is 1.0/day at $20^\circ C$. Treating the coliform bacteria decay as a first order reactive substance, using a summer average flow of 250 cfs, and the average annual loads; the profile shown in Figure 2-24(a) is computed. As shown in this figure, the computed profile greatly exceeds the total coliform levels indicative of a highly probable problem. The combined sewer overflows are indicated to exert the predominant influence on the instream coliform concentrations. The other sources of pollution are calculated to contribute less than 700/100 ml of coliform bacteria. The conclusion to be drawn from this preliminary impact analysis is that any future efforts with respect to public health problems in the hypothetical 208 study should be directed to further quantification of the combined sewer overflow coliform loads.

2.7.2.2 Summer Storm Flow

An estimate of the instream coliform concentrations during a typical summer storm is presented in Figure 2-24(b) for the hypothetical example. A typical summer storm for the South River study area is characterized by an increase in stream flow to 3.2 times the annual average flow which results from 2 to 5 day steady rainfall of 7 times the annual average rainfall. As described in Section 2.5.3 the sewer overflow loading rates increase to 7 times the annual average loading rates during this storm period.

The instream coliform concentrations resulting from the combined sewer overflows during the storm is the major source of coliform pollution as is shown in this Figure. The preliminary analysis calculates instream coliform levels to reach a maximum of 250,000/100 ml. This concentration and the concentration for the entire river downstream of the city further substantiates the highly probable existence of a problem.

The direction to be followed for further coliform analysis should provide more information on the quantity and quality of the combined sewer

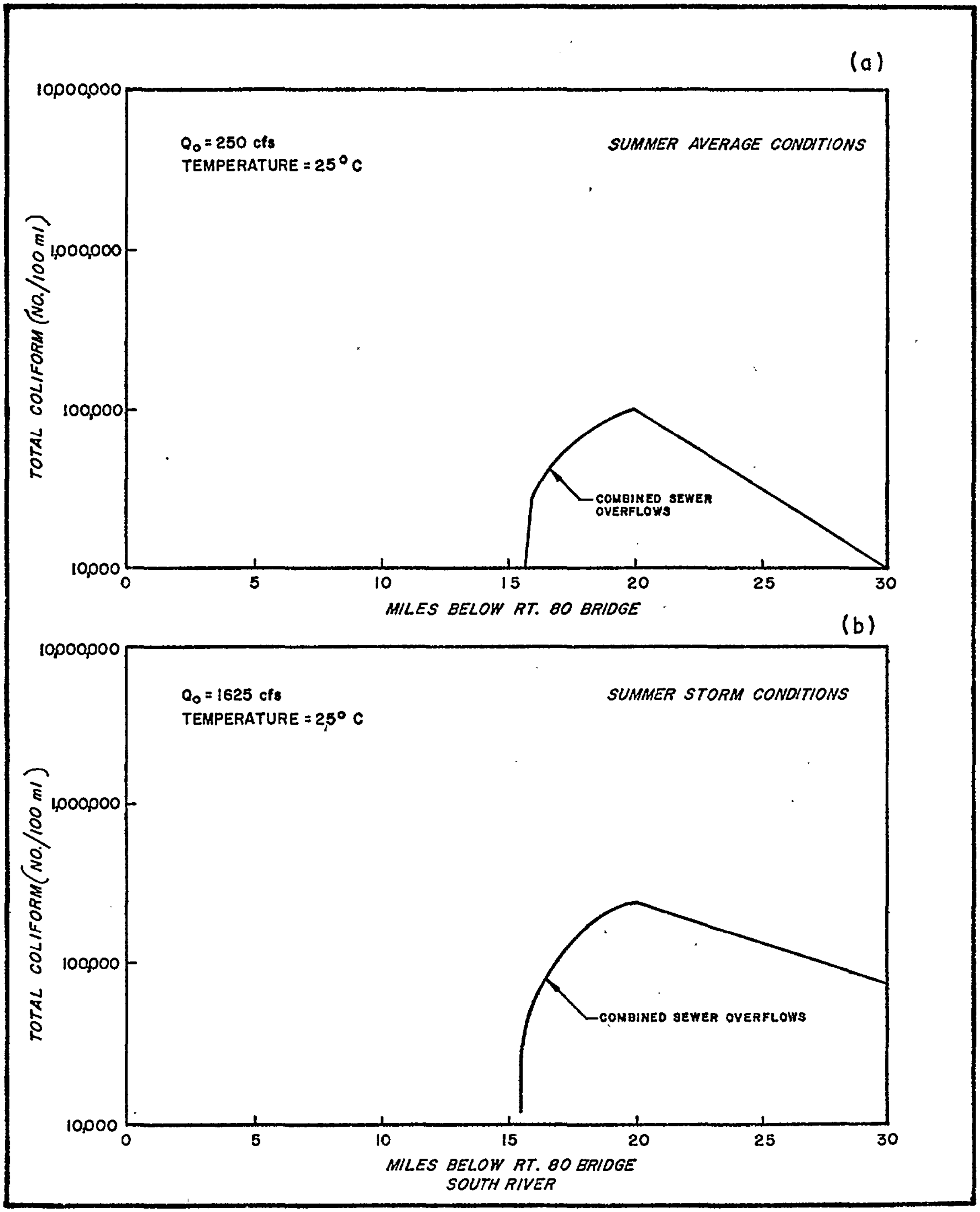


FIGURE 2-24
TOTAL COLIFORM DISTRIBUTIONS
(SUMMER AVERAGE FLOWS, SUMMER STORM FLOWS)
HYPOTHETICAL SOUTH RIVER EXAMPLE

overflow. This would require that instream coliform data and combined sewer overflow data be collected during both average conditions and during a long duration summer storm.

2.7.3 Eutrophication

The preliminary analysis of eutrophication is based on the following reasoning. If excessive quantities of both nitrogen and phosphorus exist, it is likely that, given the proper environment, algae will grow. Since the precise determination of these conditions is beyond the scope of a preliminary analysis, it is assumed that, in the absence of any contradictory data, conditions will be favorable for growth. Therefore, if the concentrations of both nutrients are in excess of growth requirements, a potential eutrophication problem exists. The interpretation of the calculated concentration of total nitrogen (TN), and total phosphorus (TP) nutrients in the receiving water is shown in Table 2-24.

TABLE 2-24

EUTROPHICATION ANALYSIS

If the Calculated Concentration is:	Probability of a Problem
TN less than 0.01 mg/l or TP less than 0.001 mg/l	Improbable
TN more than 0.1 mg/l and TP more than 0.01 mg/l	Potential
TN more than 1.0 and TP more than 0.10 mg/l	Probable

It is recognized that these concentrations are quite low and there are situations for which these concentrations are exceeded and no substantial biomass develops. Environmental factors, such as climate, geomorphology of the receiving water, turbidity, etc., are as important in determining whether a problem will develop as are the concentrations of available nutrients. Thus, these factors must be considered in modifying the conclusions drawn from the preliminary analysis.

The impact analysis for total nitrogen and total phosphorus is performed for an annual average and summer flow. For the summer or warm weather periods when plant growth is occurring, there is uptake of the nutrients by rooted plants or an uptake by planktonic forms which results in removal of the total nutrients by the algal synthesis and subsequent settling. In addition, phosphorus can be adsorbed to particles which settle. Thus, a removal rate is specified for the summer average condition. On an average annual basis, the nutrients that settled out during the summer can be returned to the flow during the scouring that occurs at high flows. If this occurs the annual average behavior is that of a conservative substance.

2.7.3.1 Annual Average Flow

The methodology presented in Section 2.6.3 is used to calculate preliminary nutrient concentrations in the sample drainage basin. The average annual stream flow of 500 cfs provides dilution of the total nitrogen and total phosphorus loads which are presented in Table 2-15. The calculated total nitrogen profile is presented in Figure 2-25(a). The total nitrogen concentration downstream of Milepoint 20 is 1.3 mg/l. It should also be noted that on a preliminary basis the municipal wastewater treatment plant and agricultural runoff are responsible for the majority of the nitrogen discharged to the South River.

Figure 2-25(b) presents the calculated total phosphorus profile. The municipal wastewater discharge provides about 70% of the instream phosphorus.

2.7.3.2 Summer Average Flow

Total nutrients are estimated by assuming nutrient removal is occurring during the algal growth period. For a preliminary analysis, total nutrients are removed at a rate of 0.1/day. When the dilution (upstream) flow is reduced to 250 cfs, and the point and NPS loading rates remain the same, the profiles presented in Figure 2-26 are computed. In general, the

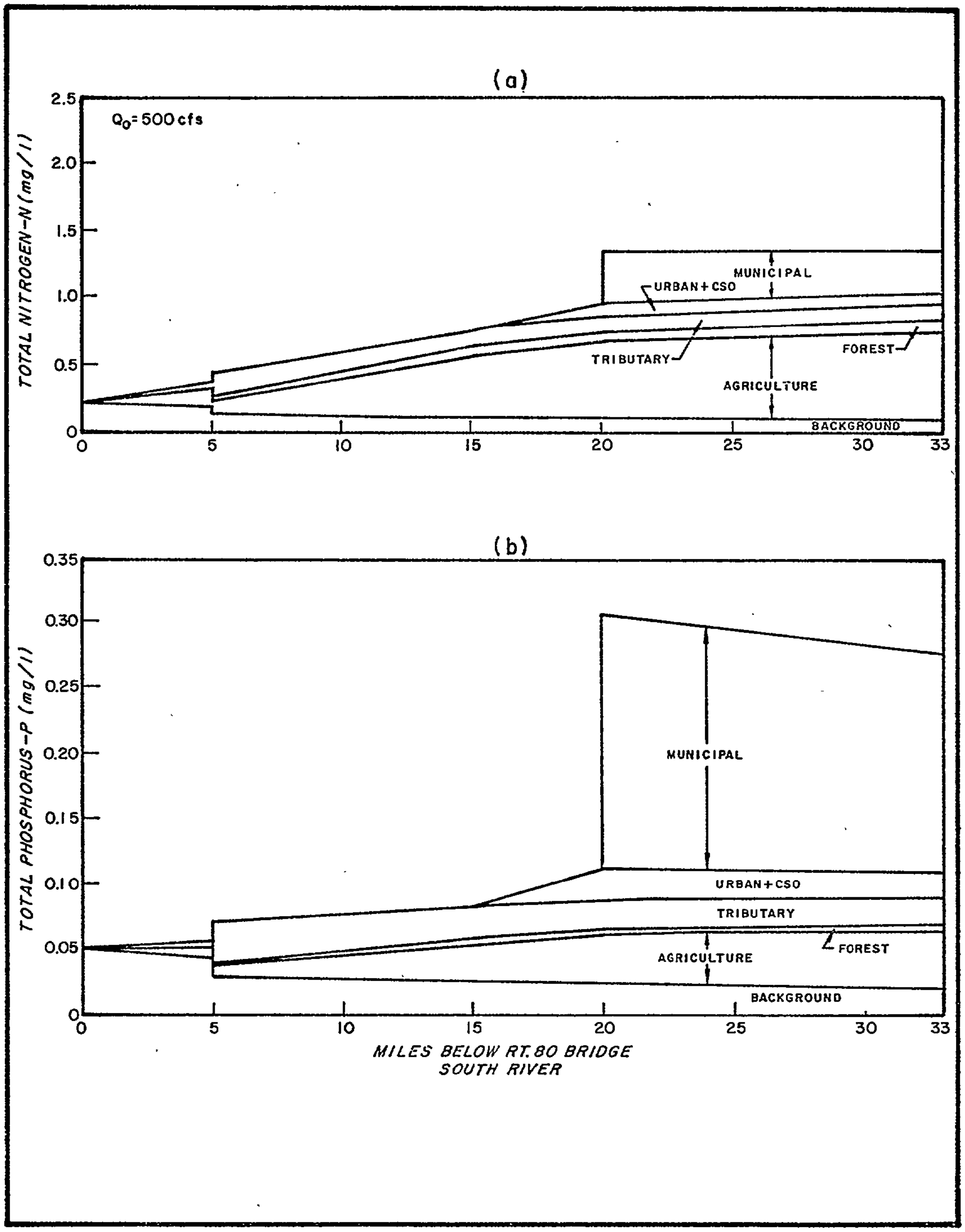


FIGURE 2-25
TOTAL NUTRIENT DISTRIBUTIONS
(ANNUAL AVERAGE FLOW)
HYPOTHETICAL EXAMPLE

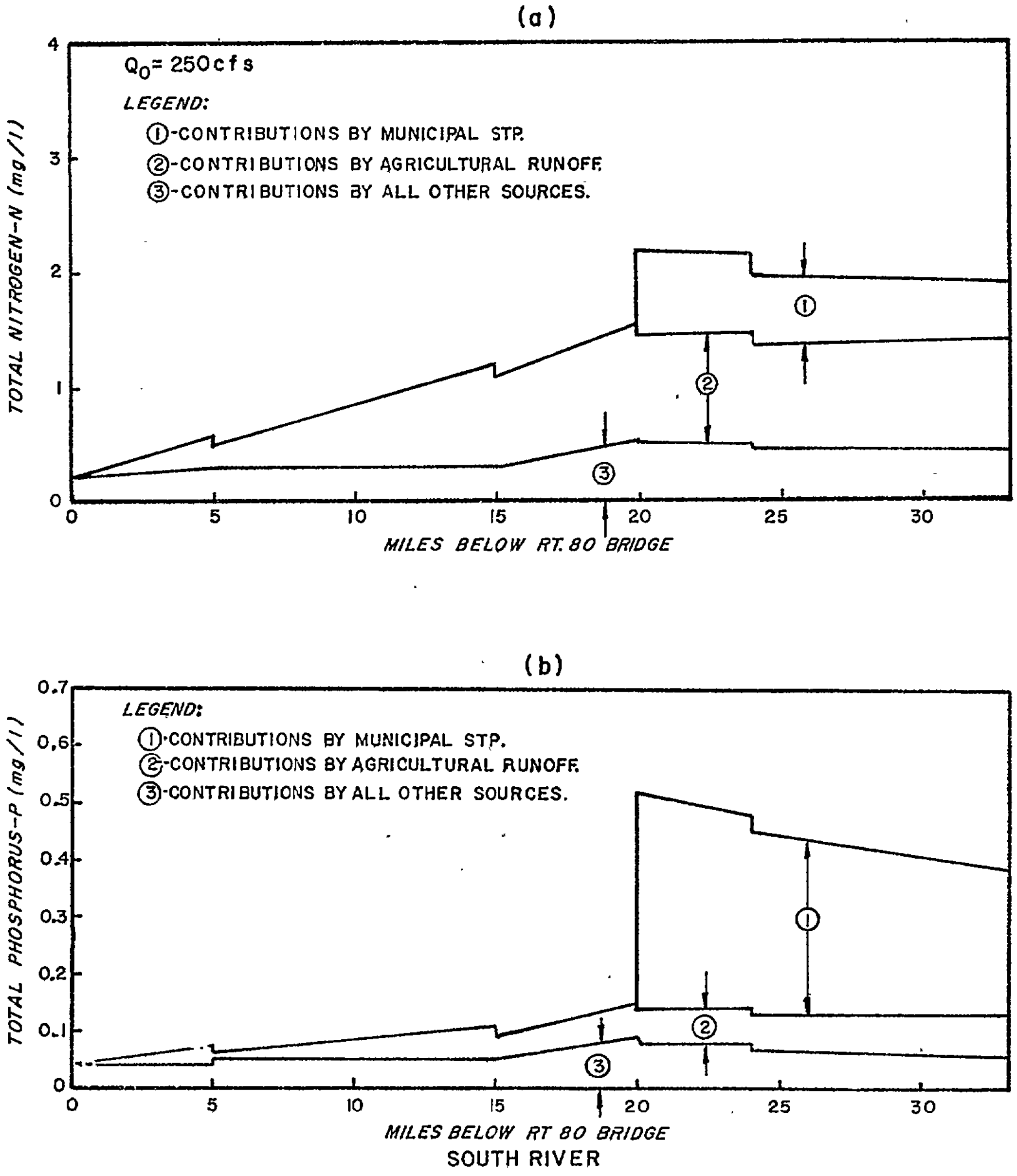


FIGURE 2-26
TOTAL NUTRIENT DISTRIBUTIONS
(SUMMER AVERAGE FLOW)
HYPOTHETICAL EXAMPLE

stream concentrations of total nutrients approximately double. Also, as shown for the annual average flow, the agricultural NPS nitrogen and municipal nitrogen loads are responsible for the majority of the calculated instream nitrogen. The municipal wastewater treatment plant effluent contributes by far the majority of the phosphorus to the South River.

Based on a preliminary analysis, the potential for a eutrophication problem is indicated for the South River. Nutrient concentrations are greatly in excess of that required for substantial algal biomass. However, two factors may mitigate against its development. The travel time through the stream is ten days or less (Figure 2-9) and relatively high suspended solids concentration are calculated in Section 2.7.4, which would limit light penetration. Hence, it is clear that further analysis is necessary before a definitive assessment can be made.

2.7.4 Other Water Quality Interferences

The other water quality interferences can be caused by substances that are either conservative, reactive or sequentially reactive. For example purposes, total suspended solids are evaluated for the hypothetical South River 208 study. For an approximate analysis, the total suspended solids are considered to be conservative.

The interpretation of total suspended solids concentrations, TSS, are tentatively based on the levels indicated in Table 2-25.

TABLE 2-25

TOTAL SUSPENDED SOLIDS

If Calculated Concentration is:	Probability of a Problem
Less than 10 mg/1	Improbable
Less than 100 mg/1	Potential
More than 100 mg/1	Probable

2.7.4.1 Annual Average Flow

The estimated TSS NPS loads and known point source loads previously discussed are used as the input data for the impact analysis. The calculated instream TSS concentrations are shown in Figure 2-27(a) using the annual average stream flow for dilution of the loads. The computed unit responses for each land use show the agricultural NPS TSS loads account for over 90% of the instream suspended solids.

2.7.4.2 Summer Storm Flow

As discussed in Section 2.5.3, a summer storm in the sample South River Basin has been defined as having 7 times the average annual rainfall intensity over a 2 to 5 day period. This increased rainfall results in an increase in the stream flow to 3.2 times the average flow. During this type of storm, the impact of urban sources are evaluated. The calculated total suspended solids profile for these conditions is presented in Figure 2-27(b).

Total suspended solids increase over existing conditions is 50 mg/l. Therefore, the urban point and intermittent point sources create a potential total suspended solids problem in the South River during a long duration summer storm. Hence, a more detailed analysis is required together with a data gathering program.

2.7.5 Summary of Results of the Impact Analysis

The preliminary assessment of the hypothetical South River planning area indicates that problems in all the categories analyzed are either probable or highly probable. However, the causes of these problems have been restricted to only certain of the sources of mass within the drainage area. Table 2-26 presents the results in a compact form. It is clear from this compilation that agriculture, and point sources are each important in assessing dissolved oxygen and eutrophication problems. Further, coliforms are dominated by urban combined sewers while suspended solids are dominated by agriculture and urban combined sewers.

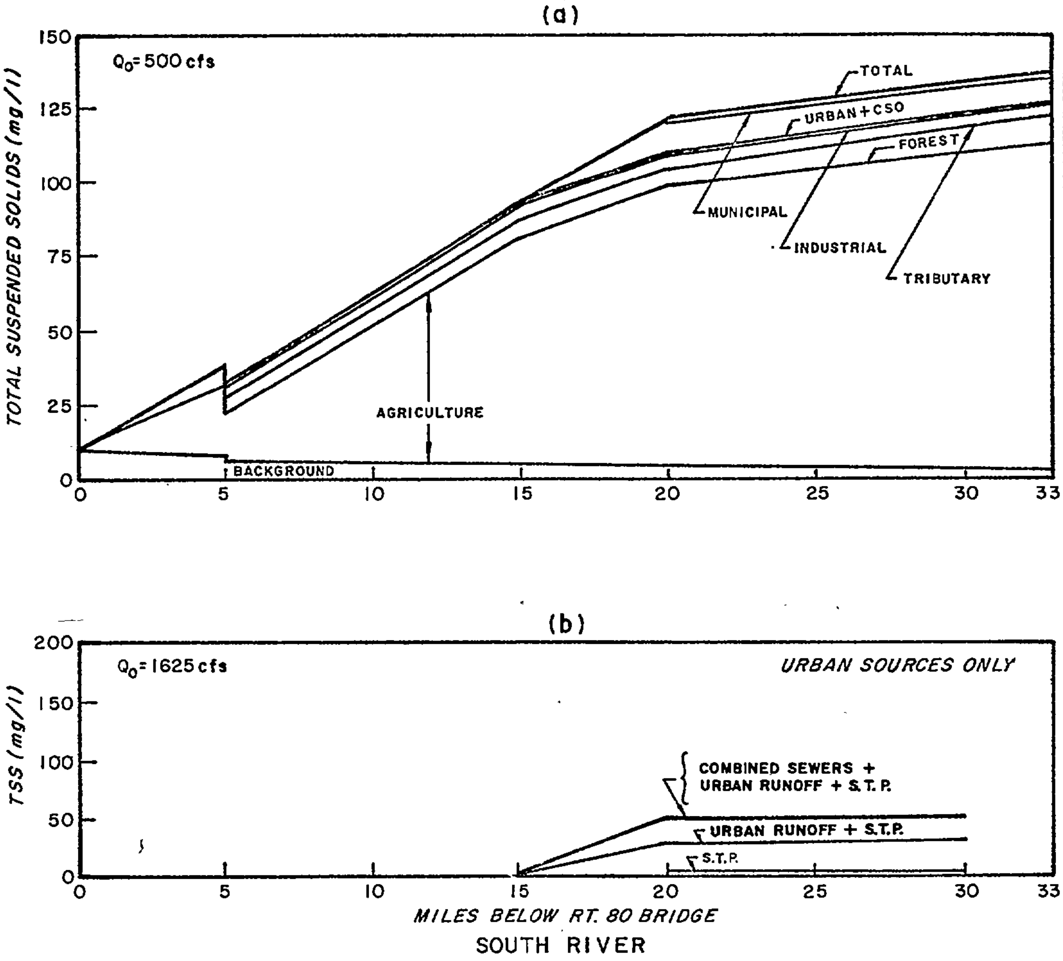


FIGURE 2-27
TOTAL SUSPENDED SOLIDS DISTRIBUTIONS
(ANNUAL AVERAGE FLOW, SUMMER STORM FLOW)
HYPOTHETICAL EXAMPLE

TABLE 2-26

SIGNIFICANT SOURCES OF SOUTH RIVER POTENTIAL WATER QUALITY PROBLEMS
(HYPOTHETICAL EXAMPLE)

Sources	Dissolved Oxygen	Public Health	Eutrophication	Suspended Solids
Agriculture	significant contributor to probable problem at summer flow	-	significant contributor to public porblem at annual and summer flow	predominate contributor to potential problem at annual and summer flow
Forest	Not likely to be a significant contributor to any of the listed problems			
Upstream conditions	-	-	Further analysis required	-
Tributaries	-	-	Further analysis required	-
Urban NPS	Not likely to be a significant contributor to any of the listed problems			
Urban combined sewers	significant contributor to probable problem at summer flow and predominate contributor at stormflow	Predominate contributor to a highly probable problem at summer and storm flow	Further analysis required	Predominate contributor to probable problem at storm flow
Point Sources	Significant contributor to probable problem at summer flow and predominate contributor at drought flow	-	Significant contributor to probable problem at annual and summer flow	-

Two sources, the runoff from the forested areas and urban runoff from non-sewered areas, can be eliminated from further considerations.

In summary, the preliminary impact analysis methodology and the load estimate generation procedure presented previously is given as a suggested procedure. It is realized that there are and will be some areas of the technique which are subject to question. However, the procedure presented can, in many cases, put a proper perspective on the problem. The preliminary analysis might identify specific areas for additional analysis or it might identify areas which warrant additional data collection. In some instances the analysis will account for the total extent of the mathematical modeling analysis which is needed for the 208 project.

If pollution sources, such as combined sewer overflows, are identified as being responsible for affecting water quality then further analysis might be in order. Chapter 3 and Chapter 4 can aid the analyst in a proper direction to take for the additional analysis. If the water body is more complicated than a stream, then both the preliminary data collection and the load generation should help the analyst put a better perspective on the sources of pollution. In addition, Chapter 5 will help by introducing higher level modeling techniques which are available for water quality impact analyses.

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CHAPTER 3

PROCEDURES FOR ASSESSMENT OF URBAN POLLUTANT SOURCES AND LOADINGS

3.1 Introduction

This chapter discusses procedures for the assessment of waste loads generated in urban areas at a higher level of refinement and detail than employed in the preliminary analysis described in Chapter 2. Since the assessment methodology for continuous point sources, at the higher level of refinement, is very similar to procedures presented in Chapter 2, Chapter 3 emphasizes procedures for those urban loads which enter receiving waters as intermittent point sources, and provides a limited discussion of the applicability of these procedures for estimating the impact of non-point source urban loads. A more complete discussion of the assessment methodologies for nonpoint source non-urban loads is presented in detail in Chapter 4.

3.2 Identification of Level of Sophistication Required

The output from a preliminary assessment as discussed in Chapter 2, is a broad identification of major pollutant sources within an urban area along with a perspective on the relative magnitude of the pollutant loads generated, the temporal and spatial distribution of these loads, and a preliminary assessment of the impact of these loads on water quality problems within the planning area.

Once the major pollutant loads have been identified and can be attributed to sources for which control measures are possible, assessment procedures with higher levels of sophistication should be employed to refine the previous loading and impact estimates. At higher levels of refinement, it is appropriate to consider the changes in the temporal and spatial distribution and magnitude of loads from sewered and non-sewered urban areas. This allows the planners to further differentiate mass pollutant loadings as modified by the nature and extent of storm and combined sewer conveyence, retention and treatment facilities.

Although predictive methodologies and analytical approaches with a high level of sophistication and detail are described in Appendix A of this manual, the procedures recommended for the level of assessment in this chapter are the simplest of the available techniques that will allow estimation of pollutant loadings and impacts necessary to support the decisions that must be made. While it is generally accepted that higher levels of sophistication will permit more accurate or reliable pollutant load quantification and impact analyses, the decision to use highly sophisticated procedures for problem assessment must be tempered with an understanding of the inherent weaknesses and limitations of the overall analysis. These limitations stem from measurement errors in rainfall related pollutant loadings, from the complex nature of pollutant transport mechanisms within urban drainage areas and from the wide range of physical and chemical transformations that occur in the receiving waters.

Another point that must be considered in this second level of refinement in the analysis is the choice between very precise and accurate estimates of the loads from a small number of events or a less refined analysis of longer term loading histories. This choice often depends on such factors as the availability of historical rainfall and stream flow data, the physical characteristics of the drainage area and the water quality standards of the receiving water.

The justification for detailed analysis at higher levels of sophistication is related to the environmental and economic risks involved in decisions made from analyses with lower levels of precision and refinement. These risks should be balanced against the cost for data collection and analysis associated with the sophisticated procedures. Balanced analytical frameworks and data collection activities should be developed for the major loading components and associated receiving water responses.

3.2.1 Identification of Appropriate Time Scale

Pollutant loadings from urban areas may be separated into two categories: those which are related to storm events and those which are not. Storm events are highly variable in magnitude and transient in nature, occurring randomly in time. Combined and separate storm sewer overflows and surface runoff through natural channels are generated by storm events, and therefore have similar characteristics, though different in magnitude. The component of rainfall which is lost to soil infiltration may also generate loads, via the transport or leaching of soluble pollutants. The characteristics of loads entering a stream from groundwater are quite different from the loads associated with surface runoff. All or part of these percolating loads may remain as groundwater, or infiltrate sewer lines. That portion which does enter the receiving water could tend to do so over an extended period of time and over an extended area. Thus, storm events can generate highly-variable transient loads which are discharged directly to the receiving water as well as diffuse indirect loads which enter the receiving water through groundwater inflow attenuated beyond the actual rain event.

Urban loadings not associated with storm events include the typical point source discharges from municipal and industrial treatment plants, continuous discharges of raw sewage from faulty regulators, inadequate or partially filled interceptors, treatment plant bypasses, and groundwater inflow pollutant loadings from sources such as improperly functioning septic tank leach fields and improperly designed or operated landfills. Such loadings are more or less continuous, except for the diurnal activity of municipal and industrial sources and seasonal activities from specific industries. Urban pollutant loadings, therefore, are time dependent to varying degrees.

Waste loads with a low order of time dependence may be characterized on a yearly basis (pounds/year), and prorated linearly down to some time interval of interest (pounds/month, pounds/day). Except for cases where significant seasonal changes occur, such as population movement in vacation areas or seasonal industrial activity, such prorated daily or monthly loading rates will be fairly representative of the actual loading rates from the source during these periods.

For transient storm generated loads, this is not the case. A yearly stormwater loading rate represents the sum of all individual storm loads which have occurred throughout the year. If such annual loads are prorated down to monthly or daily loading rates, the resulting loads are artificial, since the distribution of rainfall is not uniform. In the characterization of storm loads, an average annual loading rate may be used as a measure of the relative impact of storm runoff. For example in a case where sediment deposits in the receiving water are a primary concern, the cumulative effect of all storm overflows is more significant than that of an individual event or a specific period within an event. For this case, it is unimportant whether a prorated daily or monthly loading rate reflects the instantaneous rate during that period, as long as the cumulative load calculated by using these rates is representative of the long term loading to the receiving water. Long term loading characterizations are appropriate where the impacts of storm overflows are not confined to occurrences during individual storm events. Impacts may be observed at any time, and possibly become most severe only during non-storm periods.

Where water quality impacts from stormwater overflows are as transient as the occurrence of the overflows, the long term average loading becomes much less important, and represents only the first step in the characterization of storm runoff. Coliform organisms, which are present in storm overflows and have relatively rapid die-off rates in natural waters, may be used to illustrate this point. If a coliform load enters a receiving water during a 6-hour storm event, the peak concentration observed in the receiving water would generally be restricted to that order of time. Based on the magnitude of the load, the transport characteristics of the drainage area and receiving water, and the die-off rate of the coliforms, the total impact will often be dissipated before the next storm occurs. In this case, yearly or monthly average loading rates do not adequately indicate the actual load during a storm. Further, they do not provide information on whether the resulting receiving water concentrations are objectionable, or how often concentrations in excess of stream standards or water quality objectives may be observed. These average loads for pollutants with short-term transient impacts can be useful for identifying the significance of a particular source or the magnitude of a problem. Reliable assessment, however, requires a greater level of detail in the definition of such loads.

Figure 3-1 illustrates various levels of refinement which might be employed in the definition of transient stormwater loads. It represents the transition from a relatively simple average yearly loading to a quite detailed representation of storm runoff accomplished by introducing additional temporal detail. The level of refinement proceeds as follows:

Level 1. Average yearly storm load
Level 2. Actual event distribution
Level 3. Variation within events

Level 1 uses the average annual stormwater loading rate, W_o, as was used in Chapter 2. W_o is calculated by determining the total storm load during the year (pounds), and assuming that it occurs continuously (during both rain and non-rain periods). Confidence in the value of W_o would require the monitoring of all storm loads over a period of several years. This is impractical within the 208 planning and assessment time frame. Most storm analysis techniques must rely, at least initially, on estimates of such values extrapolated from other studies or projected from data secured from limited areas and limited time periods.

Level 2 considers the actual temporal distribution of stormwater events. It indicates the variability of the pollutant load from event to event. It does not, however, indicate how the load varies within each event. The variation of runoff load per event is due primarily to the amount of rainfall occurring during each event. It is also due to the variation in time between storms which affect surface debris inventory, and to seasonal changes in debris accumulation in the drainage basin (e.g., leaf fall, spring fertilization, etc.).

Level 3 describes the actual runoff loading rate for all storm and for any time within each storm, whether it be hour by hour or minute by minute

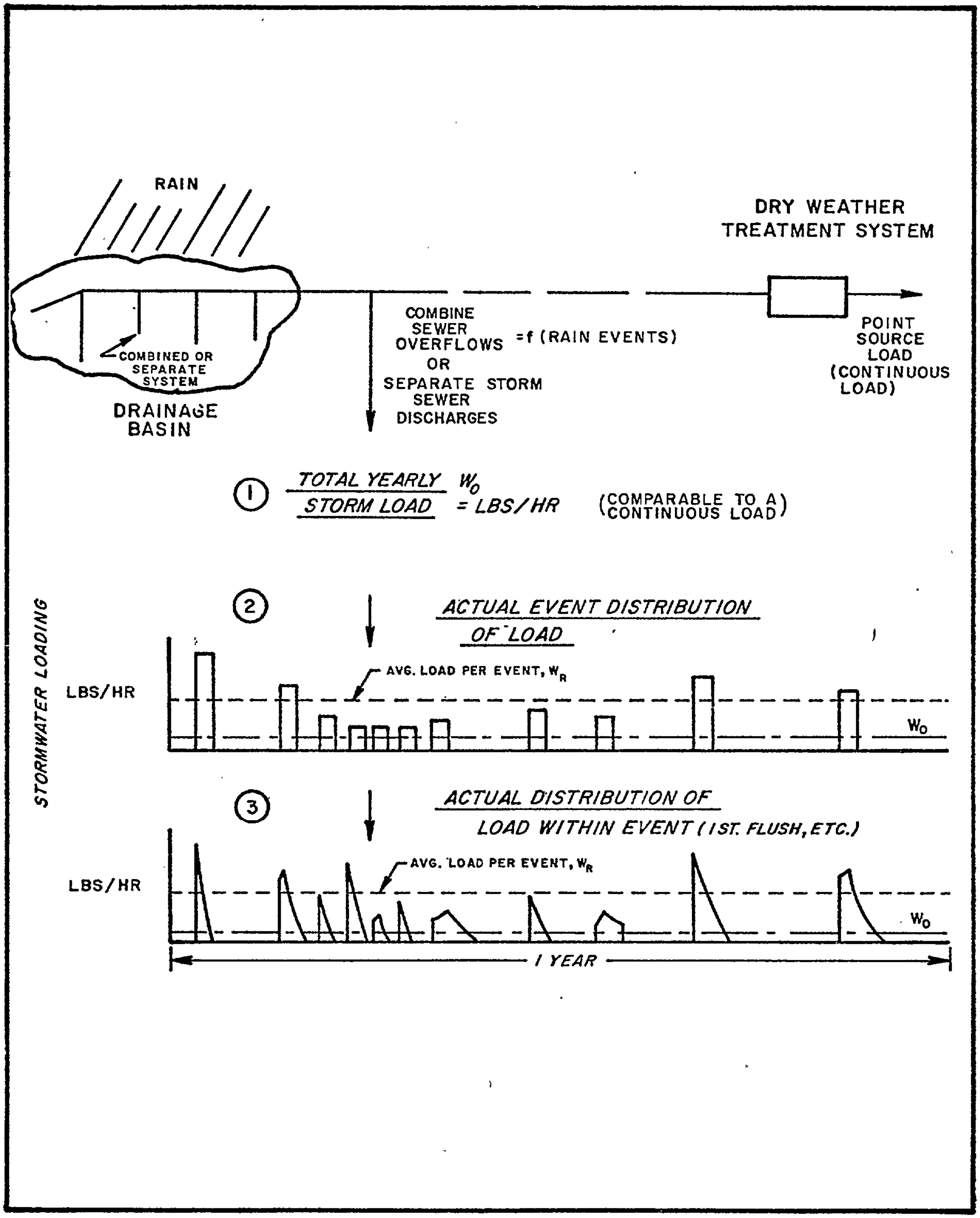


FIGURE 3-1
VARIOUS LEVELS OF DETAIL IN
STORMWATER LOAD CHARACTERIZATION

characterization. This level indicates the "first flush" effects, varying storm patterns, and varying intensity within the storms.

When it becomes inappropriate to use the long term average pollutant loading rate, W_o, in an impact analysis, then some higher level of stormwater loading definition must be provided. Selection of an appropriate level of detail in definition of storm loads is best dictated in assessment studies by receiving water impacts. Pollutants discharged to receiving waters have characteristic time and space scales associated with the impacts they cause. These scales are illustrated by Figures 3-2 and 3-3, and can be used to provide guidance in determining the time scale of the averaging which is appropriate.

Thus, while suspended solids loads may, in most cases, be characterized on an annual basis, more reactive contaminants (coliform organisms, oxygen consuming materials) will usually require definition on a scale in the range of hours. Note that waste load definition on a scale finer in detail than one to several hours (approximately the scale of storm events) is not necessary for the evaluation of transient water quality impacts. Assessment studies will, therefore, not normally require load definition in greater detail than that represented by Level 2 in Figure 3-1.

Urban areas generate both continuous and intermittent point source wastewater loadings. The continuous point source loads will now be discussed. The intermittent point source loads are discussed in Section 3.4.

3.3 Characterization of Continuous Urban Point Source Loads

The methodology for characterizing continuous point source loads described in Chapter 2 remains appropriate at the higher levels of refinement discussed in this chapter. The data should be sufficient to allow for a reliable estimate of continuous point source loads. These loads are the most obvious in an urban area, they are usually important, and information will usually be available for adequate characterization.

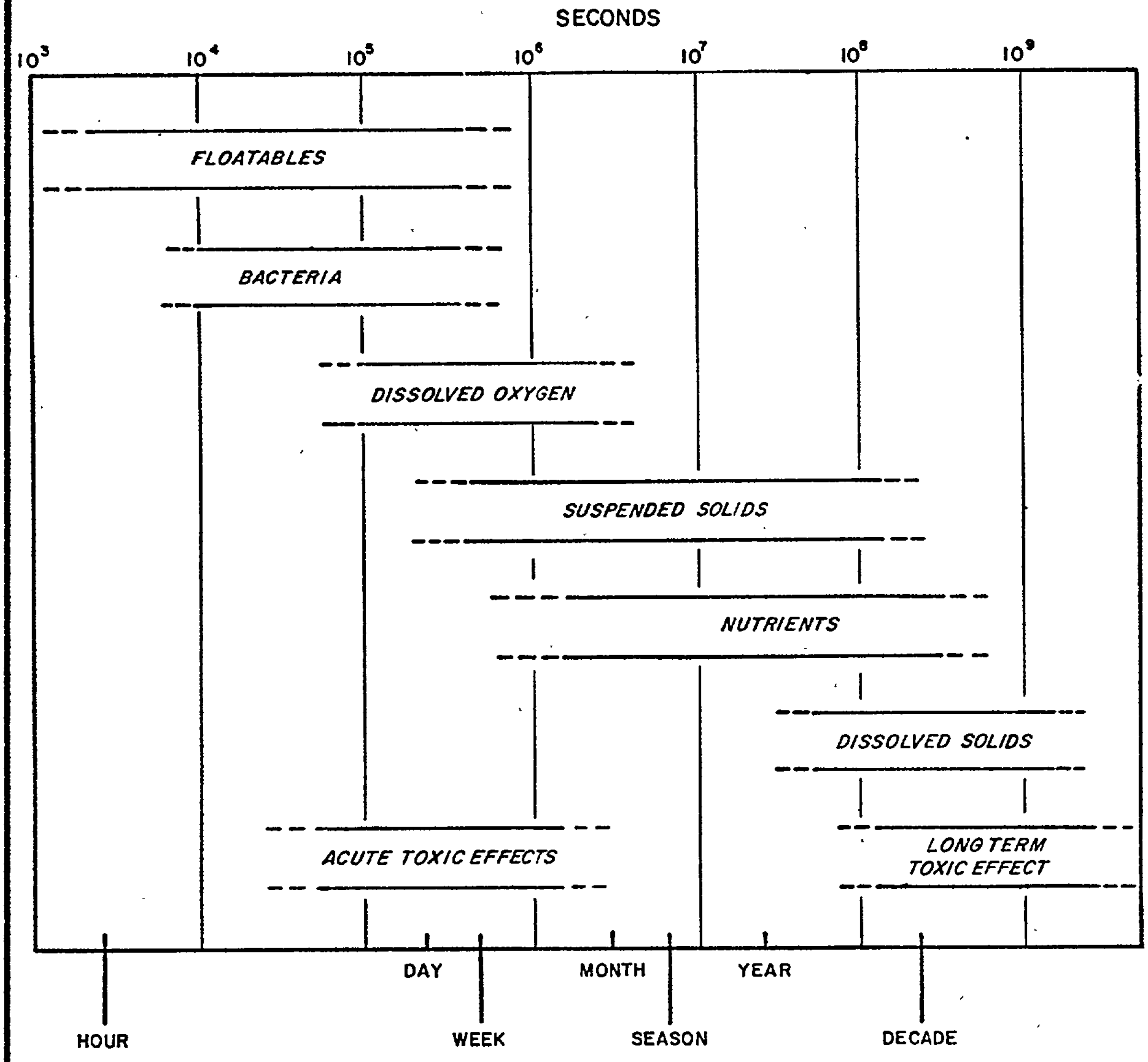


FIGURE 3-2
TIME SCALES
STORM RUNOFF WATER QUALITY PROBLEMS

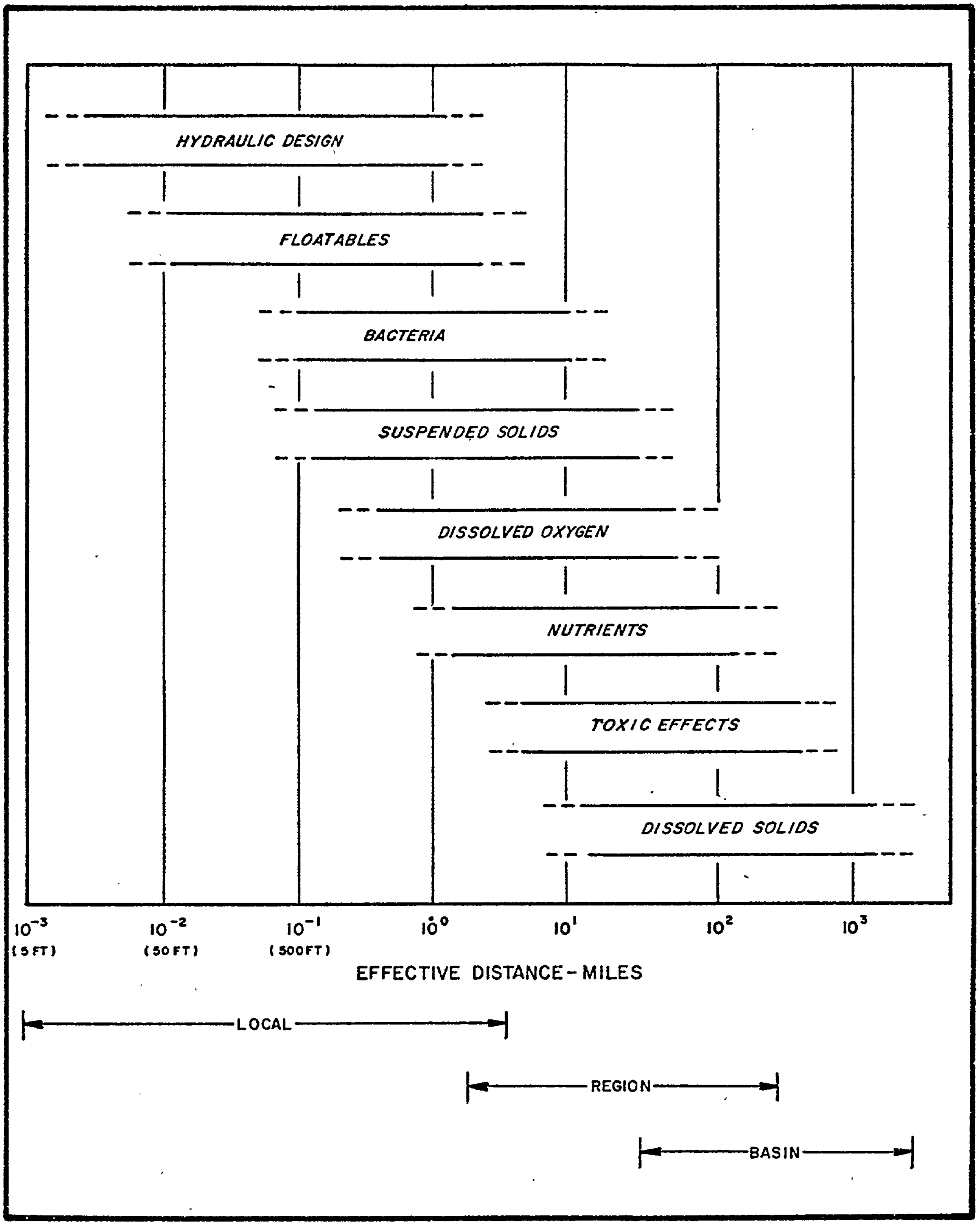


FIGURE 3-3
SPACE SCALES
STORM RUNOFF WATER QUALITY PROBLEMS

The assessment of waste loads from an urban area seeks to identify the relative significance of loads from different sources. Since many of the urban loads (intermittent point sources, non-point loads) will require characterization by estimating techniques, the overall assessment will be strengthened by accurately characterizing those loads for which an adequate data base exists. Municipal and industrial point source discharges usually fall into this category.

3.3.1 Municipal Continuous Point Loads

As described in Chapter 2, waste loads from municipal treatment facilities should be well quantified and documented. Sources of such data accessible to the 208 Agency include treatment plant records, state or local regulatory agency records, EPA NPDES (National Pollution Discharge Elimination System) permit data or monitoring reports. An additional source of data on municipal and other point source loads may be found in study reports for 201 Facilities Plans and for 303-(e) Basin Plans, where these have been completed in the area. The Regional EPA offices maintain lists of completed basin plans for their region. These plans will normally contain, in addition to data on point source loads, information on water quality standards and objectives and on allocations of waste loads from point sources.

There is a class of municipal continuous waste loads which will not normally be documented or easily identified, but which may be significant sources of load in some areas. Many cities with combined sewer systems discharge unmonitored waste loads continuously as a result of defective or improperly operated regulators and bypasses, partially filled interceptors, or interceptors whose capacity is exceeded by normal dry weather flows during some period of the day. Dry weather overflow data for these discharges will not normally be available but it may be possible to estimate them. Public works or sewer maintenance departments or regulator and interceptor maintenance crews are sources of information. Urban areas with separate sewer systems may also have significant discharges from the sanitary sewer systems. During periods when the groundwater tables are high, significant groundwater infiltration often occurs and excess flow from the sanitary sewers may be bypassed at the plant or at upstream locations, thereby producing an often overlooked pollutant load to the receiving stream. In addition, separate sewered areas may have continuous discharges from storm sewers caused by unauthorized connections.

3.3.2 Industrial Continuous Point Loads

With the enactment of the Water Pollution Control Act Amendments of 1972 (PL 92-500) the NPDES program was established (1). With this program, the Federal Government established long term goals for industry to achieve increased abatement of pollutants discharged to the nation's waterways. As discussed in Chapter 2, all industries are required by law to obtain a discharge permit, and to continually report to the state and/or federal agency the quality of the discharge to the receiving water. The permit indicates the effluent limit for discharges to the receiving water. The compliance monitoring programs mandated by NPDES report regularly on the actual discharges.

As with the municipal point source loads, there should be a sufficient amount of data available to allow for a determination of existing industrial loadings. Discharge permit requirements or results of monitoring programs can be obtained from the EPA Regional Office, the state regulatory agency, or possibly from the specific industrial dischargers. Most of the present effluent limits are interim in nature, and the Federal Government has set a specific time table during which certain effluent requirements must be met. Current regulations require that all industries achieve Best Practicable Treatment (BPT) by 1977 and Best Available Treatment (BAT) by 1983. To provide the 208 planner with a basis for making estimates of these future industrial loadings, effluent concentrations for some typical industries are provided.

Each industry is assigned a code number by the Department of Commerce, depending on the specific products which it produces. The first two digits of the Standard Industrial Classification (SIC) Codes for a few sample industries are as follows:

SIC	Type Industry
20	Food and Kindered Products
22	Textile Mill Products
26	Paper and Allied Products
28	Chemical and Allied Products
29	Petroleum and Coal Products
33	Primary Metal Products

The effluent pollutant concentrations to be expected under the different levels of removal for each SIC Industry type for both BPT and BAT are presented in Tables 3-1 and 3-2, respectively. It must be emphasized that the mean effluent characteristics shown in these tables represent only approximately the average discharges for each industry. The actual pollutant discharges will vary widely from one sub-category to another within a given industry. There can be a number of different types of plants within a specific industry with differing manufacturing processes and products. An additional factor is the varying water usage per unit of production from plant to plant. Therefore, these estimates should not be taken as characteristic of an entire industry. They provide only a rough estimate of the mean effluent pollutant concentration. Additional guidance for estimating such effluent concentration can be obtained from the EPA Development Document for a specific industry which describes the basis on which the discharge limitations were established (2 thru 14).

It should be noted that all of the effluent guidelines are not finalized and that many industries are in negotiations with EPA concerning the validity of the current effluent limits. Therefore, the effluent concentrations in Tables 3-1 and 3-2 should only be used if no other information is available.

In instances where the receiving stream which receives the industrial discharge is classified as "water quality limited", the EPA must make effluent allocations for each industry on a case by case basis. Generally, such allocations will differ from the values discussed above, and will be written into the permit along with a compliance schedule for meeting the allocation.

TABLE 3-1

TYPICAL INDUSTRIAL EFFLUENT CONCENTRATIONS
BPT - "1977"
APPROXIMATE MEAN EFFLUENT CHARACTERISTICS(a)
(in mg/1)

Parameter	20 Food	22 Textiles	26 Paper	28 Chemicals	29 Petroleum	33 Metals
TSS	40(b)	49(b)	58(b)	40(b)	10(b)	20(b)
BOD$_5$	29(b)	22(b)	39(b)	30(b)	15(b)	38Ub)
COD	135	225(b)	156	1400(b)	102(b)	-
TDS	-	700	3785	4350	-	-
Cl	565	25	135	-		-
Total-P	17	2	-	5		.50
Total-N	50	2	-	20	70	62(b)
Lead	-	.03	-	2(b)		.25(b)
Zinc	-	5	-	0.25(b)		.25(b)
Cadmium	-	.005	-	0.225(b)		.15(b)
Oil	10(b)	10	-	15(b)	5(b)	5(b)

(a) Represents an approximate estimate of the mean effluent concentrations for each industry. The pollutant concentration could vary widely within an industry as a result of varying water usages.

(b) Concentrations developed from effluent guidelines which exist for these parameters, other concentrations were obtained from any existing treatment plant data found in the Development Documents (2-14).

TABLE 3-2

TYPICAL INDUSTRIAL EFFLUENT CONCENTRATIONS
BAT - "1983"
APPROXIMATE MEAN EFFLUENT CHARACTERISTICS(a)
(in mg/l)

Parameter	20 Food	22 Textiles	26 Paper	28 Chemicals	29 Petroleum	33 Metals
TSS	10(b)	8(b)	23(b)	13(b)	5(b)	5(b)
BOD_5	7(b)	11(b)	22(b)	15(b)	5(b)	4(b)
COD	48	72(b)	88	460(b)	26(b)	-
TDS	-	700	3785	4350	-	-
Cl	565	25	135	-	-	-
Total-P	1.2	2	-	3	-	0
Total-N	9(b)	2	-	3	28	5(b)
Lead	-	.03	-	1(b)	-	0(b)
Zinc	-	5	-	.25(b)	-	0(b)
Cadmium	-	.005	-	.05(b)	-	0(b)
Oil	10(b)	10	-	3	1.35(b)	5(b)

(a) Represents an approximate estimate of the mean effluent concentrations for each industry. The pollutant concentration could vary widely within an industry as a result of varying water usages.

(b) Concentrations developed from effluent guidelines which exist for these parameters, other concentrations were obtained from any existing treatment plant data found in the Development Documents (2-14).

3.3.3 Need for Monitoring

Specific monitoring efforts for point source continuous discharges may be adviseable or necessary during an assessment study. Although, as indicated earlier, it is likely that a fairly comprehensive data base on such loads is available, efforts to provide supplementary data by means of a monitoring program may be indicated for a number of reasons. Some of the more often encountered reasons are:

1. Need to verify data reported from municipal and industrial sources
2. Need to develop temporal and special data for complex situations where ongoing stream surveys are being performed.
3. Need to quantify impacts from previously unidentified sources, such as overflows caused by severe infiltration of separate sanitary collection systems, faulty regulator operations in combined sewer systems, partially filled interceptors, or unauthorized connections to storm sewers
4. Need to obtain specific contaminant information for cases where appropriate sources were monitored but contaminants of interest were not included.

3.4 Characterization of Intermittent Point Source Loads

Because of their variability, intermittent point source loads require the use of unique estimating procedures for adequate quantification. Extensive data on storm runoff loadings have been obtained in recent years and have been reported in the literature (15-19). These provide an immediate source of information, but are not a substitute for local data and analysis. Unless reported data have been collected in the study area, local data must be collected and used in developing estimates of storm loads. The need for adequate local data is based upon the following factors:

1. Differences in rainfall frequency, intensity, and duration along with drainage area characteristics significantly influence storm loads.

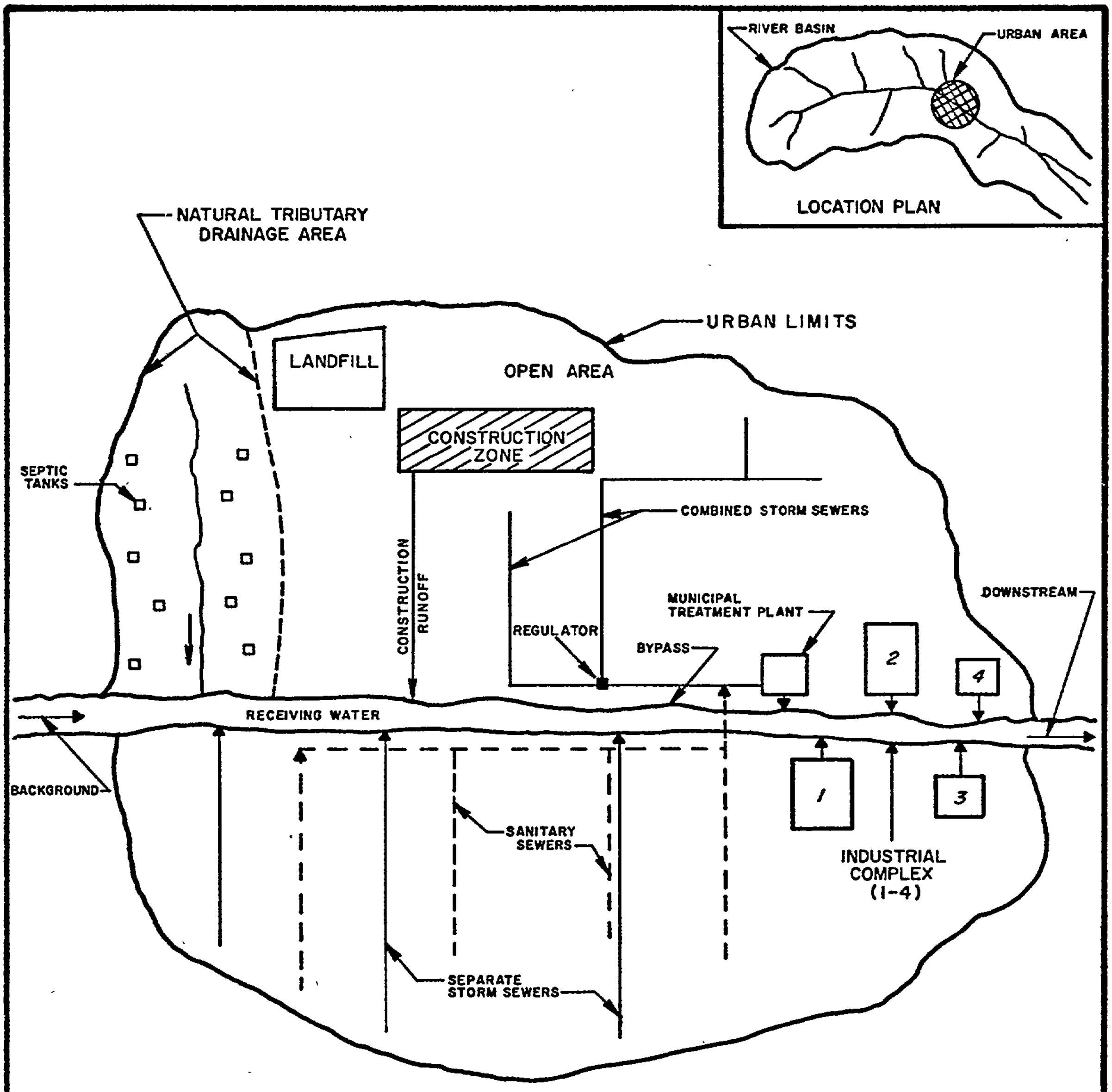
2. Significant variations in loads from individual storms are observed in the same area.
3. Studies which develop loading data generally cover relatively short periods of time compared with the long term patterns of rainfall in any area.

The most important factor in the generation of intermittent (storm) loads is rainfall. Rainfall data for an area should be among the first set of local data obtained and analyzed. All of the load estimating methods employ rainfall as the fundamental data input.

Hourly rainfall records for U.S. Weather Bureau stations in the study area may be obtained from the National Weather Records Center, Asheville, North Carolina, either on magnetic tape or on punched cards. For a long term period, the cost of such records is approximately $100 per gage.

In addition, data from local rain gages may be obtained from sources within the study area, reduced to an hourly record and utilized in a similar fashion. Data from the U.S. Weather Bureau stations should always be analyzed, because the data are reliable and will generally cover the longest historical record. Data from local gages are often an important source of more area-specific information and should be analyzed whenever available. The actual use of the data depends upon the methodology employed, and is described further in Sections 3.4.3 and 3.4.4.

Other data inputs required are the drainage basin and land use characteristics of the study area. For urban areas, this can be initally summarized (as illustrated in Figure 3-4) by the relative distribution of land use classifications (residential, industrial, commercial, open, etc.) served by basic conveyance systems (combined sewers, separate storm sewers, natural conveyances). Appendix C and the *Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Vol. II: Cost Assessment and Impacts* (16) describe in detail the available sources of this information and the procedures for its use.



LAND USE BREAKDOWN	% OF AREA		
	COMBINED SEWERS	SEPARATE STORM SEWERS	NATURAL CONVEYANCE
RESIDENTIAL LO. DENS. MED. DENS. HI. DENS. COMMERICAL INDUSTRIAL RESIDUALS DISPOSAL OPEN			

TOTAL AREA ____________________.

FIGURE 3-4

DRAINAGE BASIN CHARACTERISTICS

The amount of runoff from storms and the pollutant loads associated with this runoff are determined by three basic elements:

1. Rainfall
2. Land use
3. The extent and type of stormwater collection and conveyance systems.

Each methodology described in this chapter employs a procedure to calculate flow and loads based on these inputs. The more sophisticated the methodology, the greater the detail required for the characterization of the drainage area. Any methodology may use initial estimates for relationships which convert rainfall and land use data into runoff flows and loads. Specific local data are needed to confirm or refine these relationships.

3.4.1 Identification of Appropriate Methodology

In general, there is no single methodology which is uniquely appropriate for the assessment of urban storm pollutant loadings in all 208 areas. The utility of each method is dictated by many factors, some of which include:

1. Local conditions
    a. land use
    b. conveyance system
    c. seasonal variation of rainfall
2. Availability of local data
    a. water quality data
    b. hydraulic data
    c. raingage data
    d. demographic data
3. Type of receiving water
    a. stream
    b. estuary
    c. lake
    d. coastal water

4. Water quality problems
    a. dissolved oxygen
    b. public health
    c. eutrophication
    d. other (TDS, TSS, etc.)
5. Time scale of water quality problems
    a. transient (hour, days)
    b. longer term (month, season, year)
6. Decisions involved
    a. feasibility of control
    b. economic risk
    c. environmental risk
7. Study constraints
    a. time
    b. cost

Specific methodologies which employ various levels of detail are used as part of the framework in which an urban loading assessment is performed. Examples of appropriate methodologies for storm loads are presented and described later in this chapter. The transient storm loads generated by these methodologies, together with the continuous point loads, are then used to assess receiving water impacts, as described in Chapter 5.

3.4.1.1 Selection of Suitable Estimation Procedures

The methods which are available to characterize and quantify stormwater loads all include the following essential elements:

1. Rainfall
2. Drainage basin characterization
3. Runoff quality and flow characterization

A major difference between the available methods lies in the detail in which the elements are represented. High levels of detail in the analysis require more definition of the drainage basin along with an associated increase in input data requirements. All of the methods attempt to describe the relationship between the runoff load and the various elements which influence it. A partial list of these elements includes:

1. Separate or combined sewer system
2. Collection system characteristics
3. Areal variability of rainfall
4. Rainfall intensity
5. Rainfall duration
6. Time between storms
7. Degree of imperviousness of drainage area
8. Drainage basin slope
9. Soil type, vegetative cover, erosion control practices
10. Land use: commercial, industrial, residential, open space, etc.
11. Population density
12. Surface pollutant accumulation and decay rate
13. Street cleaning frequency

Some of the simpler methods use empirical coefficients which aggregate many of the above influences. Other more sophisticated methods attempt to provide refined loading estimates by incorporating cause-effect relationships for the above elements.

Examples of more sophisticated simulation models which have been applied to estimate stormwater loads include: NPS Model, STORM, SWMM (continuous version). These and other applicable models are included in a discussion presented in Appendix A (NPS Model is discussed in Chapter 4) which describes the salient features of each model and its applicability for estimation of storm loads and receiving water impact.

The notable aspect of each of the models listed above is that they are continuous simulators. That is, they re-enact the stormwater process (the sequence of storm events over some extended period of time). They do not concentrate on single storm events. It is a knowledge of this event sequence and its impact, which often provides the best information and perspective for assessment studies.

Any method requires calibration before it can be used reliably. That is, samples of the runoff must be collected for a sufficient number of runoff events covering a variety of conditions so that consistent cause-effect relationships can be established. Once calibrated and verified, the methods can be used to make estimates of loads beyond the limited data set used in the calibration-verification procedure.

Where local data needed to calibrate the above models is lacking, estimates or default values for essential parameters can be used for the characterization of the cause-effect relationships in the models. It is well to recognize, however, that the default values were developed based on experience in a particular study area and are not universally applicable. Local effects and conditions have a significant influence, and without at least some local data for comparison, high degrees of confidence in calculated results may not be warranted.

Two simplified approaches to the characterization of storm loads are described in detail in this chapter. One is a simulation approach which represents loadings by storm events. The other estimating procedure presents a purely statistical method for the assessment of runoff and treatment. Each incorporates a level of detail which is appropriate for assessment studies, and permits the examination of long term characteristics. While both permit estimates to be made without local data, the use of such data for calibration to local rainfall, runoff and quality characteristics will produce load estimates with a higher degree of confidence.

3.4.1.2 Levels of Accuracy Required in Storm Load Characterization

At each level of detail in the stormwater load characterization (e.g. yearly, seasonally, monthly, event by event, etc.) described in Figure 3-1, there is a certain level of accuracy provided by the various calculation techniques. Two types of accuracy can be considered in applying available techniques: (a) event accuracy and (b) statistical accuracy for a long term sequence of events.

Many simulator models attempt to correctly relate phenomena in the drainage area to their associated effect on stormwater loads. They require calibration to define a consistent set of coefficients which establish the magnitude of the parameters in the cause-effect relationships. In order to estimate these parameters, a sufficient number of stormwater overflow events are monitored. Then the model coefficients are estimated and adjusted until there is a good agreement between the predicted load distributions within individual events and the actual observed values. The simulator is then considered calibrated. If other events with different rainfall properties agree equally well with observed data, the simulator can be considered to be verified. A good calibration requires comparison with a number of significantly different storm events, such that there is a reasonable agreement between actual and calculated distributions of loads for events which cover a wide range of relevant magnitudes. This calibration may be quite difficult in that it requires substantial amounts of data on drainage area characteristics, storm flow, and quality; all for a sufficiently large number of events. Where the calibration is accomplished for only a few events, the confidence in the prediction would be high only for conditions similar to those used in the calibration. For significantly different event conditions, such as more or less intense storms, differing durations, longer or shorter periods between storms, etc., there would be less confidence in the prediction.

Event accuracy reflects a very high level of definition of virtually all of the significant cause and effect relationships which influence stormwater loads. Both data requirements and the analytical effort required to achieve a satisfactory degree of event accuracy are high.

Statistical process accuracy is directed toward the accurate statistical representation of stormwater loads over extended time periods. Individual storm events may be either underestimated or overestimated, however when the statistical summary of the long term sequence of events can be appropriately characterized for a period of interest, the model output can be considered to have statistical accuracy.

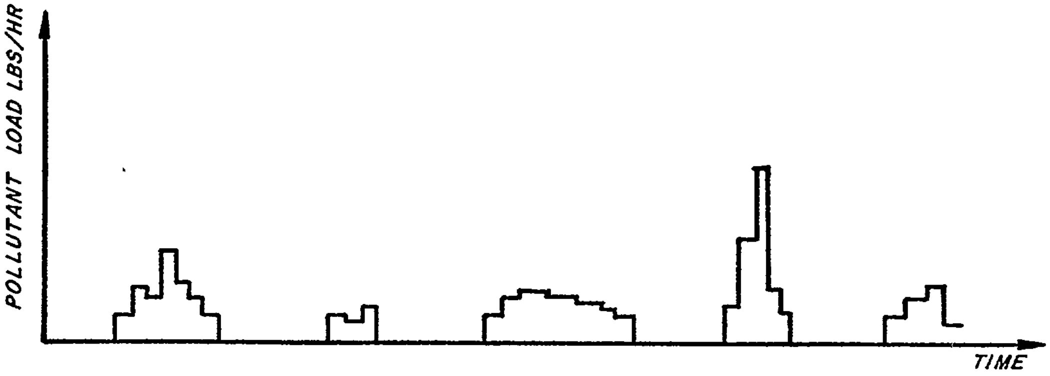
Load characterizations of this type are often appropriate in the second level of problem assessment since at this level of analysis it is not always necessary to have an accurate and complete understanding of the mechanisms which actually determine storm loads. Statistical summaries, which include the mean, the coefficient of variation, and the frequency distribution of the load are sufficient information for a reasonable and complete assessment of present conditions, and also for an estimate of the effect of certain control measures.

In some cases, the areal variability of rainfall and the insufficient number and distribution of rain gages introduce errors in storm event calibration. Such errors become less important, however, when the models are directed toward reproduction of the longer term process statistics rather than toward individual events. The "continuous" versions of some of the available simulator models are intended to stress the longer term effects rather than the detail of single events. Figure 3-5 illustrates a manner in which loads from a long term sequence of storm events may be analyzed to summarize those features of the storm process which are significant for an assessment study. Pollutant loads generated by individual storm events are estimated on the basis of the properties of the storm event causing the runoff. The magnitude and time of occurrence are determined as illustrated by Figure 3-5(a). Statistical procedures may be used to develop a probability density function representing the frequency of occurrence of loads of various magnitude, as illustrated by Figure 3-5(b). Providing the period of record used for analysis is long enough to yield statistically significant results, this analysis may be made for annual periods, specific seasons of the year or individual months of interest. Additional discussion on the development of these probability density functions is provided in Sections 3.4.3 and 3.4.4.

3.4.1.3 Levels of Spatial Detail Required in Storm Load Characterization

Another important aspect of stormwater load characterization is the spatial detail with which loads are defined. There are situations where either the event accuracy or the statistical process accuracy will be reduced because the load estimating methodology does not adequately describe the spatial distribution of stormwater loads from an area. Included among the conditions which will require increased levels of detail in the definition of waste load input locations are the following:

(a) SINGLE EVENT DISTRIBUTION



(b) STATISTICAL REPRESENTATION OF LOADING FROM SEVERAL EVENTS

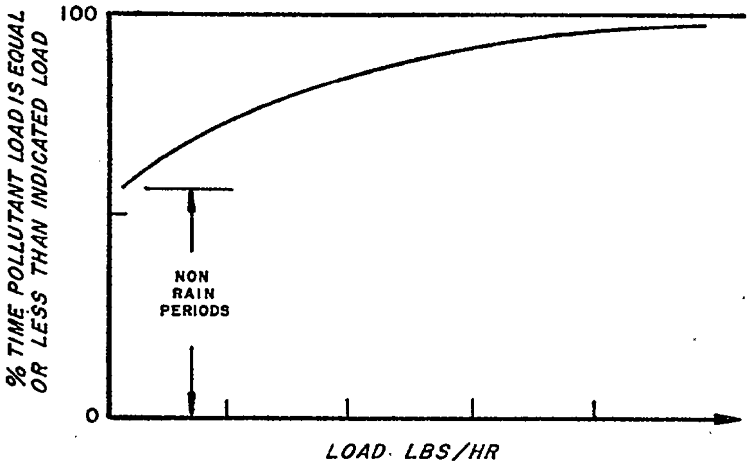


FIGURE 3-5

REPRESENTATIONS OF A LONG TERM SEQUENCE
OF STORM POLLUTANT LOADS

1. Aggregating smaller sub-catchments into a single large drainage area may be difficult for a combined sewer collection system with multiple overflow locations. The estimation of an "average" interceptor capacity representative of the entire drainage area will introduce error in the load estimates for some areas.
2. Storms in an area may be highly localized and result in significant areal variability of rainfall. The delineation of smaller sub-catchments will be necessary to increase the accuracy of load estimates in some cases.
3. For some receiving waters, it will be necessary to characterize storm loads according to their actual point of discharge, in order to make an accurate assessment of the impact of these multiple loads on receiving water quality.

There are, however, conditions under which it is appropriate to ignore such spatial detail without introducing unacceptable error into the estimation of storm loadings. This is particularly true for analyses being performed at an assessment level of detail and, for water quality analyses where longer term statistical process accuracy is the objective and where the assumption that the rainfall occurs uniformly throughout the entire study area is reasonable.

Due to the areal distribution of rainfall, different sub-catchments of a drainage basin may have considerably different runoff loads during a particular storm. The longer term frequency distributions of the rainfall on each of the subcatchments, however, are more likely to be similar, and a larger area may be aggregated when determining long term runoff flows and loads. Caution must be used, however, when aggregating different subcatchments for the purpose of determining runoff flows and loads during storms. The assumption that the rainfall is falling uniformly and simultaneously throughout the area may cause a large overestimation of the runoff flow and load that occurs during storms.

For impacts which are long term in nature, or for receiving waters (tidal estuaries, for example) where significant mixing takes place, it is often adequate to ignore some of the spatial detail, even for analyses with higher levels of refinement.

Figure 3-6 illustrates schematically the type of spatial aggregation of individual loads which can often be made in assessment studies utilizing the level of refinement discussed in this chapter. The loads from subcatchment sections one through five may be aggregated and assumed to enter the receiving water at a single location.

3.4.2 Characterization of Rainfall-Runoff Relationship and Runoff Quality

The key element in making reliable storm load estimates with any of the available methodologies is the determination of appropriate relationships for the study area which describe the amount of rainfall which will leave the area as runoff and the pollutant concentrations associated with the runoff.

Both the statistical method and the simulator method later described in this chapter require estimates of runoff quantity and quality. Techniques for determining both an average runoff coefficient and representative pollutant concentrations in storm flows for use in both these methods are presented in this section.

3.4.2.1 Estimation of Runoff Coefficient

The volumetric runoff coefficient, C_v, measures the fraction of the storm volume which reaches the conveyance system as runoff. Because the use of a volumetric runoff coefficient aggregates the combined effects of variable storm properties and drainage basin characteristics, it is not treated as a constant, but as a random variable. The purpose of this section is to determine an average value, C_V, which may be employed in storm load assessments.

It is preferable to estimate C_V by comparing raingage data to runoff monitored during corresponding storms. Flow measurements should be taken within the sewer system and/or the natural conveyance channels. Flow data that are developed can include faulty data because of measuring

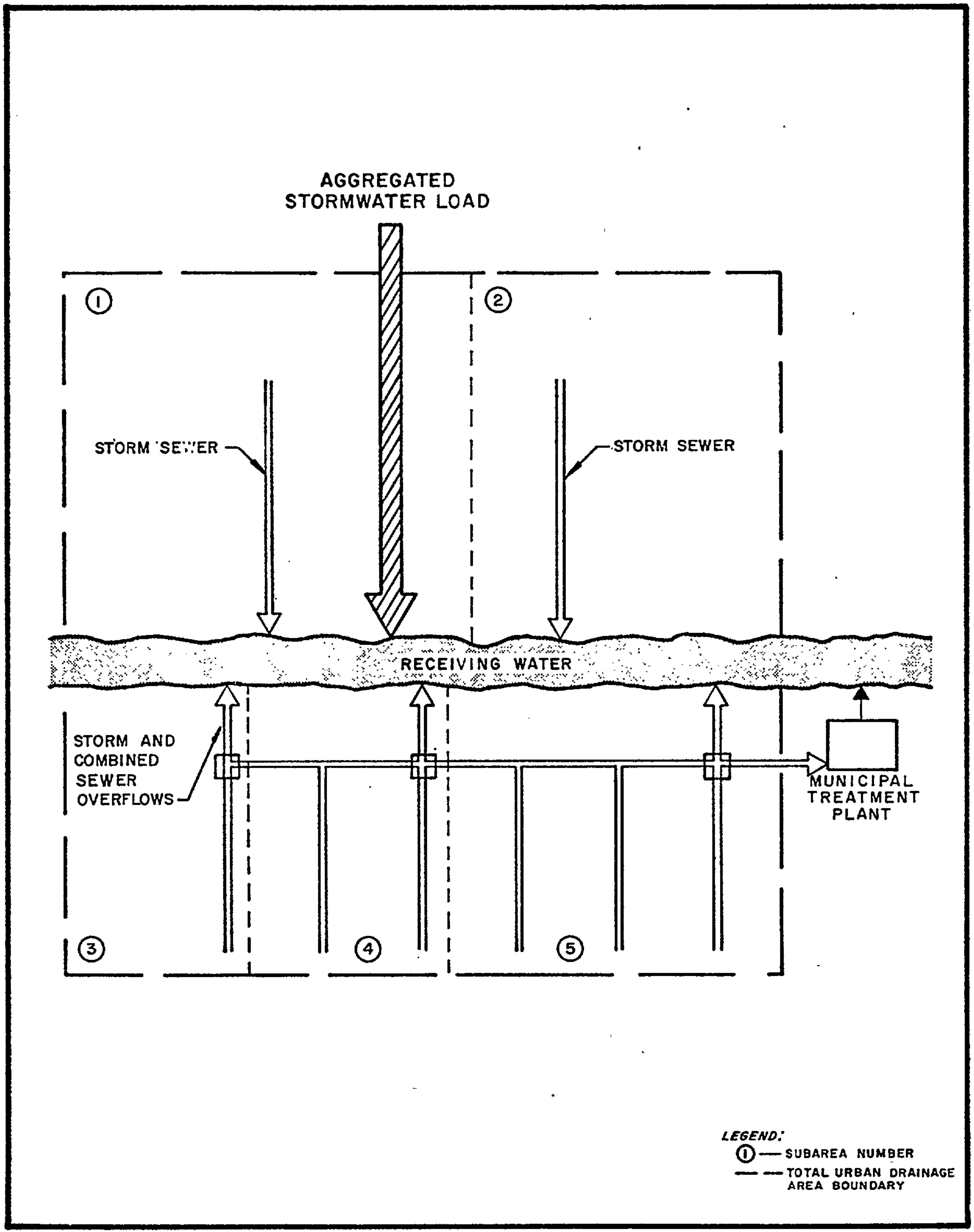


FIGURE 3-6
COMPARISON OF SPATIAL DETAIL
IN STORMWATER LOADING CHARACTERIZATION

equipment and maintenance weaknesses, therefore data and maintenance records should be screened carefully to identify such occurences. It is incorrect to assume that, because two significantly different runoff volumes are recorded for similar rainfall events, the equipment is necessarily faulty. The two storms may have occured with different antecedent soil conditions or with a different areal extent and spatial distribution of rainfall with respect to the raingage recording the events.

A sufficiently large number of observations of runoff quantity for at least one representative location for which drainage area characteristics are well known would be required to develop a representative value for the average runoff coefficient, C_V. In a statistical evaluation of rain events performed by the National Oceanographic and Atmospheric Administration (NOAA), one of the findings was 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 (20). 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.

From statistical sampling theory, the Central Limit Theorem states that the distribution of sampled averages tends to become normal as the number of samples increases, no matter what the distribution of the variable being sampled (21). This property may be used to estimate the number of storms necessary for an adequate estimate. This number is dependent upon the variation of the parameter being measured (in this case the runoff coefficient), 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 3-7 shows the relationship between the number of storms monitored and the resulting level of accuracy. The curves presented are based on a coefficient of variation of 0.75 for the runoff coefficient, which appears to be a reasonably conservative estimate based on observed data (22). In general, 20 to 40 storms will provide adequate representation.

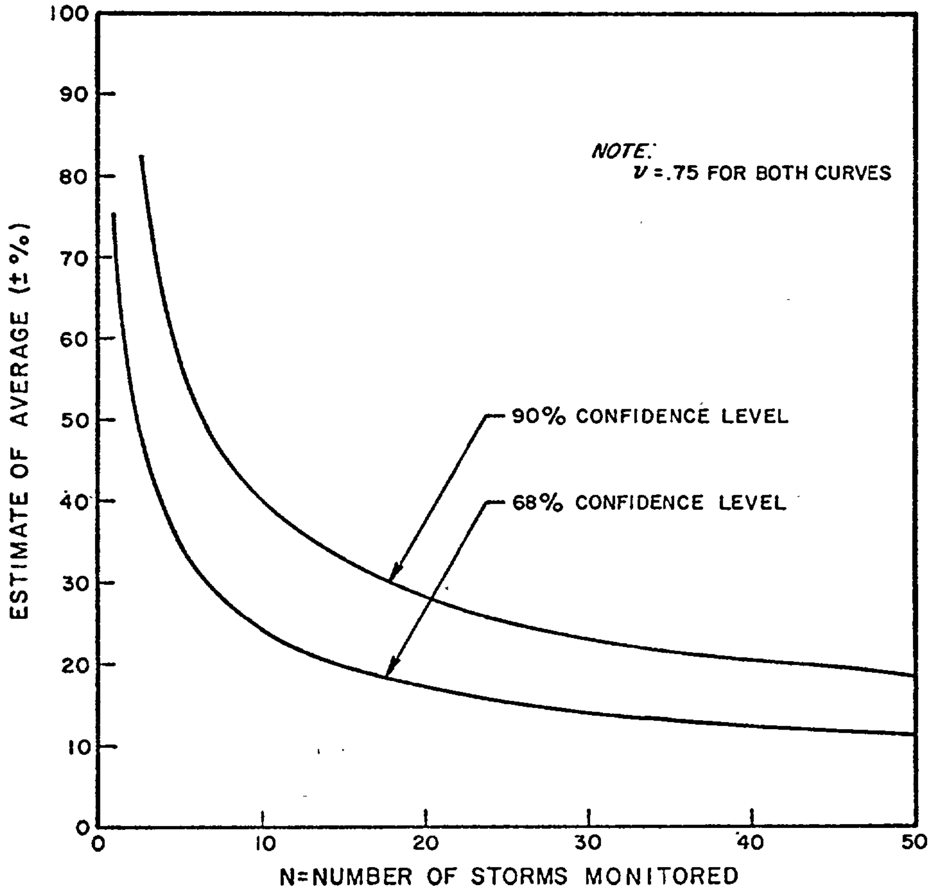


FIGURE 3-7
ERROR IN ESTIMATE OF AVERAGE
VERSUS NUMBER OF STORMS MONITORED

When extensive monitoring is not feasible for a study data is not available at the time an analysis is made, estimates may be made from land use and drainage basin characteristics. The greater the percentage of impervious surfaces in a watershed, the greater will be the runoff for a given size storm. This is due to the reduction in the amount of rainfall lost to infiltration. Relationships between the percent impervious area and the runoff coefficient have been developed from the analysis of published data and are shown in Figure 3-8. These include results averaged from eight cities (22) (the solid curves in Figure 3-8) and the equation developed for use in the STORM simulation model (24,25). The volumetric runoff coefficient, C_v, is the ratio of the runoff volume to the volume of the rainfall, and therefore accounts for depression storage as well as infiltration. The equation in the STORM simulation model, uses an average instantaneous runoff coefficient and does not account for depression storage. In this model, a separate adjustment is made to account for depression storage, as follows (15):

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:

$$DS = 0.25 - 0.1875 \text{ (Percent Impervious Area/100)} \qquad (3\text{-}1)$$

When using the STORM equation, the depression storage should be subtracted from the rainfall volume before multiplying by the runoff coefficient. However, the depression storage correction may be too refined at this level of analysis due to the variation in the data upon which both the estimates of the volumetric and the instantaneous runoff coefficients are based.

The percentage of impervious area in a drainage basin may be determined by examining aerial photgraphs or detailed land use plans. This may be a long and tedious task, however, particularly for large drainage areas. Estimates may also be made on the basis of the general fraction of land in different land use categories. Specific land use classifications are

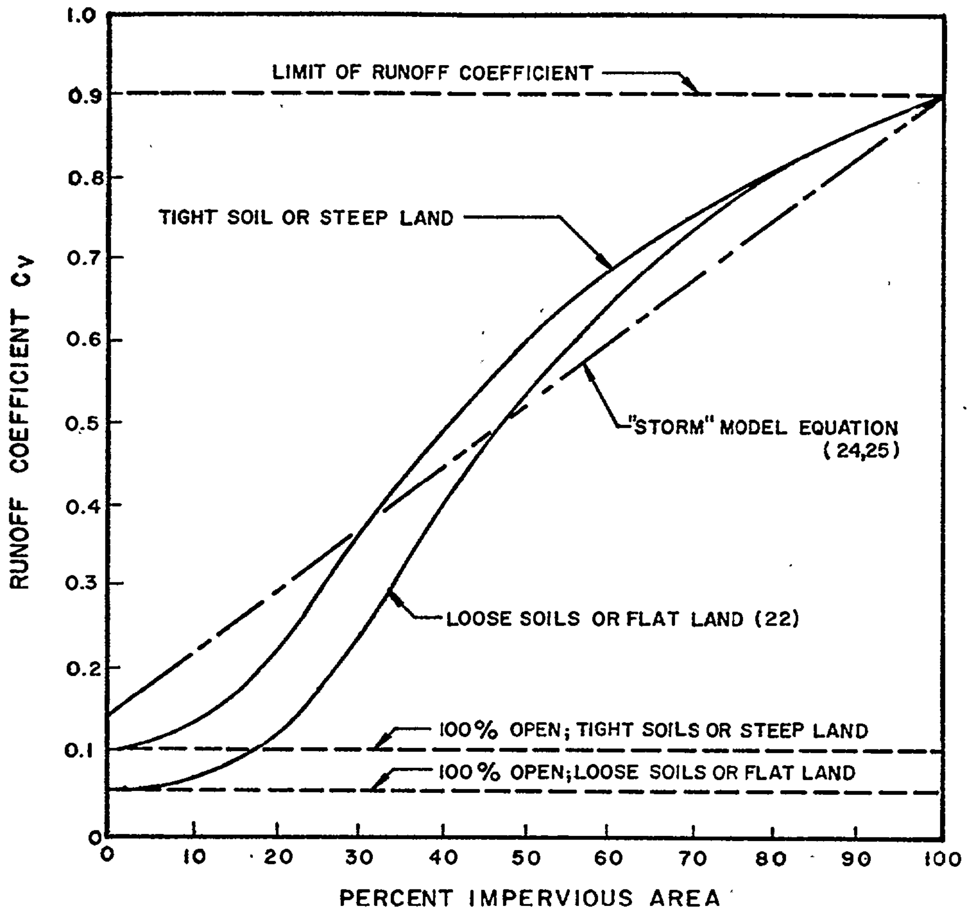


FIGURE 3-8

RUNOFF COEFFICIENT DETERMINATION FROM LAND COVER INFORMATION assigned an average percent impervious area as shown below. The indicated values or other handbook estimates (26) 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
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%
Comercial	10%
Industrial	10%
Institutional, Public	5%
Open, Undeveloped	25%
Total	100%

The overall percent impervious area would be:

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

This value would be used in Figure 3-8 to determine the average runoff coefficient, C_V.

If information on the land use categories cannot be obtained, the percent imperviousness in developed areas may be estimated from the population density. Graham et al. (Washington, DC) (23), the American Public Works Association (18) and Stankowski (New Jersey) (27) have developed equations to predict imperviousness as a function of population density. The imperviousness is to be estimated for the developed portion of the urbanized area only. The weighted average imperviousness and population density were also calculated for nine Ontario cities (28). These results are plotted on Figure 3-9 along with the three estimating curves (15). If the New Jersey data, which is based on 567 municipalities, is selected as a reasonable guideline, the equation used to estimate imperviousness is:

$$\text{Percent Impervious Area} = 9.6\ PD_d^{\ (0.573-0.0391\ \log_{10} PD_d)} \qquad (3\text{-}2)$$

where:

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

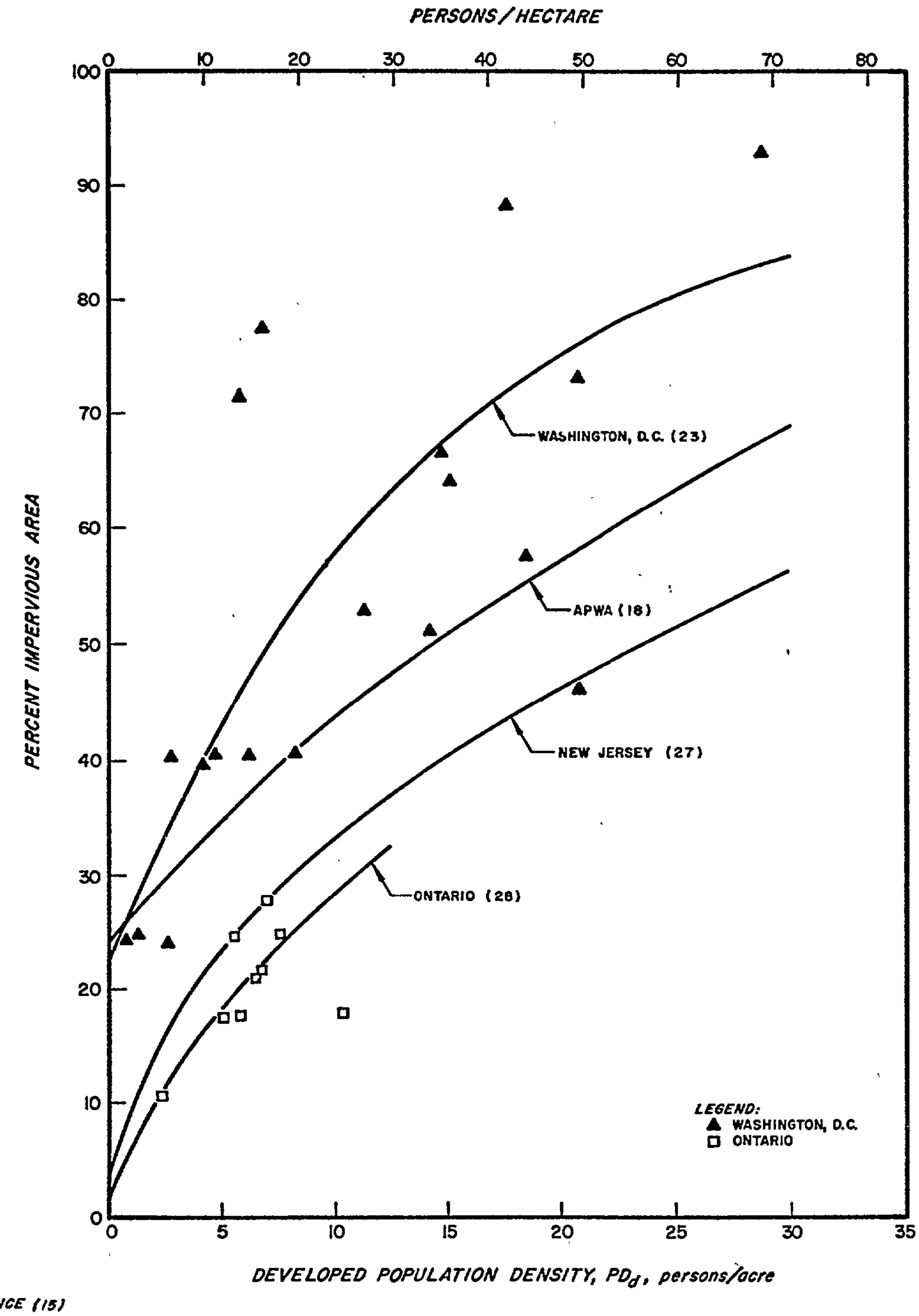
The percent impervious area is used to obtain the runoff coefficient, C_V, from Figure 3-8.

Note that individual estimates of the runoff coefficient should be made for areas served by combined sewers, separate sewers or natural conveyance. This is because significant differences in pollutant concentrations are found in the storm flows which enter receiving waters from such areas.

The final component of flow which may be of interest in combined sewer systems is the dry weather flow. This may be estimated as 100 gallons per person-day, when more area specific data is not available.

3.4.2.2 Estimation of Runoff Pollutant Concentrations

The most reliable estimates of runoff pollutant concentrations are obtained from extensive local monitoring programs. The most comprehensive quality data from a monitoring program would reflect the changes in concentration during various storm events for each overflow location.



REFERENCE (15)

FIGURE 3-9
IMPERVIOUSNESS AS A FUNCTION OF
DEVELOPED POPULATION DENSITY

Information on variations in quality within a storm event will help to identify the magnitude of the first-flush phenomenon. The measurement of quality for each separate area would reflect the impact of the mix of various land uses on the wastewater loads discharged.

The use of composite or grab samples from overflows may be substituted in an assessment study for the complete time-history measurements which would be costly and time consuming. This may cause a distortion in the results, because the first-flush phenomenon, if it occurs, is not acknowledged. The occurrence of a first-flush phenomenon is dependent upon the size and characteristics of a catchment area and storm characteristics. Grab or composite sampling will provide an insight into the quality of the overflow only on an event averaged basis.

A sufficient number of storms must be monitored to adequately characterize pollutant concentrations. The guidelines presented in Section 3.4.2.1 on the runoff coefficient may be used to determine the number of storms to be monitored for quality estimates for specific catchments. Since observed data (22) suggests that a reasonable estimate of the variation of runoff concentrations between storms in a specific catchment is also on the order of 0.75, Figure 3-7 can be used to estimate a general requirement for sampling 20 to 40 storms.

A sufficiently large series of measurements at a few overflow locations, representative of different land uses or degrees of imperviousness, can then be used to synthesize results for the remaining catchments.

When time and budget constraints make an extensive monitoring program infeasible, estimating procedures may be used to establish representative concentrations of pollutants of interest. The most direct estimates of the average runoff concentration, $\bar{c}$, may be made from Table 3-3 (previously presented in Chapter 2). Runoff concentrations of six major pollutants were monitored for a number of cities with either separate or combined sewers, and the averages of these concentrations reported for each city (22). The mean and standard deviation of these site-specific average concentrations are found in Table 3-3. The standard deviation listed in Table 3-3 reflects differences between cities and should not be used to

TABLE 3-3

SUMMARY OF STORMWATER POLLUTANT CONCENTRATIONS(22)

	STORMWATER OVERFLOW CONCENTRATIONS			
	SEPARATE DRAINAGE AREAS (a)		COMBINED AREAS (b)	
POLLUTANT (c)	MEAN	STANDARD DEVIATION	MEAN	STANDARD DEVIATION
BOD_5	27	25	108	36
COD	205	118	284	110
S.S.	608	616	372	275
Total Coliforms (d)	$3x10^5$	-	$6x10^6$	-
Total Nitrogen (as N)	2.3	1.4	9	6
Total Phosphorus (as P)	0.5	0.4	2.8	2.9

(a) Summary of 20 cities, storm sewers and unsewered areas

(b) Summary of 25 cities, combined sewer areas

(c) All unites mg/1 except coliforms, MPN/100 ml

(d) Geometric mean

estimate ν_c, the variation of the pollutant concentrations in the runoff from different storms in a given study area. It should be used only as a guide for estimating the mean runoff concentration, $\bar{c}$. One may choose $\bar{c}$ equal to the mean for the many cities, or for a more conservative estimate of $\bar{c}$, the mean plus one standard deviation. For example, a value for Total Nitrogen for a combined sewered area may be chosen as $\bar{c}$ = 9 mg/l (as N), or more conservatively as $\bar{c}$ = 15 mg/l (as N).

Many attempts have been made to relate stormwater loads to land use as an alternative or possible refinement to the basis for estimating runoff quality summarized by Table 3-3. These have been somewhat successful, but there is usually a very high average error of estimate in the prediction; that is, for a given land use pattern, the runoff quality still varies greatly. Despite this limitation, the ability to relate runoff pollutant concentrations to land use provides the ability to predict the magnitude of future runoff load changes due to changes in land use. A general procedure for predicting runoff quality from land use and population density, developed on a nationwide basis, is outlined as follows. This procedure is based on load estimates developed in reports for the Environmental Protection Agency (15,16). In these studies, loads were determined as follows:

$$M_s = \alpha(i,j) \cdot R \cdot p_1 (PD_d) \cdot \gamma \tag{3-3}$$

$$M_c = \beta(i,j) \cdot R \cdot p_1 (PD_d) \cdot \gamma \tag{3-4}$$

where:

M_s = pound of pollutant (j) from land use (i) with separate and unsewered conveyance (lbs/acre-yr)

M_c = pound of pollutant (j) from land use (i) with combined sewer conveyance (lbs/acre-yr)

$\alpha(i,j)$ = constant for pollutant (j) and land use (i) with separate and unsewered conveyance (lbs/acre-in)

$\beta(i,j)$ = constant for pollutant (j) and land use (i) with combined sewer conveyance (lbs/acre-in)

R = annual precipitation (in/yr)

$p_1(PD_d)$ = population function

PD_d = population density (persons/acre)
γ = street sweeping effectiveness factor

Equations (3-3) and (3-4) permit an estimate of BOD_5, SS, VS, PO_4, and N loads as a function of land use, type of sewer system, population density, and street sweeping frequency.

Table 3-4 shows the land use categories and pollutants.

TABLE 3-4

LAND USE AND POLLUTANTS

Land Uses		Pollutants	
1 = 1	Residential	j = 1	BOD_5, Total
i = 2	Commercial	j = 2	Suspended Solids (SS)
i = 3	Industrial	j = 3	Volatile Solids, Total (VS)
i = 4	Other Developed Areas	j = 4	Total PO_4 (as PO_4)
		j = 5	Total N (as N)

The runoff load from residential areas will increase with increasing population density. The population function is shown below.

for i = 1 (Residential): $p_1(PD_d) = 0.142 + 0.218\ (PD_d)^{0.54}$ (3-5)

for i = 2 or 3 (Commercial or Industrial): $p_1(PD_d) = 1.0$ (3-6)

for i = 4 (Other Developed Areas): $p_1(PD_d) = 0.142$ (3-7)

The intercept for residential areas (0.142) was determined based on data for open space. The exponent (0.54) is based on the exponent of the STORM imperviousness equation at a population density of 8 persons per acre. Lastly, the coefficient (0.218) is based on an average of data points with PD_d ranging from 5 to 15 persons per acre to yield a $p_1(PD_d)$ of 0.895 at 10 persons per acre.

The street sweeping effectiveness factor, γ, was derived by making numerous runs of STORM with varying street sweeping frequencies (15). The factor is a function of the street sweeping interval, N_s (days) and is defined by equation (3-8).

$$\gamma = \frac{N_s}{20} \text{ if } 0 \leq N_s \leq 20 \text{ days}$$
$$\gamma = 1.0 \text{ if } N_s \geqq 20 \text{ days} \qquad (3\text{-}8)$$

Table 3-5 lists α and β factors that are used in quations (3-3) and (3-4) for the loading analysis. The methodology thus far presented is for the determination of yearly stormwater loads. Both estimating procedures presented in this Chapter require average runoff concentrations. To provide this Equations (3-3) and (3-4) for the runoff load were converted to estimate the runoff concentration.

The resulting equations are:

$$\bar{c}_s = a(i,j) \cdot p_1(PD_d) \cdot \gamma \qquad (3\text{-}9)$$

$$\bar{c}_c = b(i,j) \cdot p_1(PD_d) \cdot \gamma \qquad (3\text{-}10)$$

where:

$\bar{c}_s$ = concentration of pollutant (j) from land use (i) with separate and unsewered conveyance (mg/1)

$\bar{c}_c$ = concentration of pollutant (j) from land use (i) with combined sewer conveyance (mg/1)

$a(i,j)$ = constant for pollutant (j) and land use (i) with separate and unsewered conveyance (mg/1)

$b(i,j)$ = constant for pollutant (j) and land use (i) with combined sewer conveyance (mg/1)

The factors a and b were determined from the following conversion:

$$a(i,j) = (\alpha(i,j) \cdot F)/C_v(i) \qquad (3\text{-}11)$$

$$b(i,j) = (\beta(i,j) \cdot F)/C_v(i) \qquad (3\text{-}12)$$

where:

F = 4.42, a constant (mg/1)/(1b/acre-in)

$C_v(i)$= runoff coefficient for land use (i)

$C_v(i)$ was estimated from the percent imperviousness generally associated with each land use, as described in Section 3.4.2.

for i = 1 (Residential) $C_v(1) = 0.30$

for i = 2 (Commercial) $C_v(2) = 0.70$

TABLE 3-5

$\alpha^{(a)}$ AND $\beta^{(b)}$ FACTORS$^{(c)}$

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

(a) Applies to separate and unsewered drainage areas

(b) Applied to combined drainage areas

(c) Units are lb/acre-in

for i = 3 (Industrial) $C_v(3) = 0.60$
for i = 4 (Other Developed) $C_v(4) = 0.10$

Table 3-6 lists the converted a and b factors that are used in equations (3-9) and (3-10) for the determination of runoff concentrations. The a and b factors of Table 3-6 were calculated with the assumed runoff coefficient for each land use, $C_v(i)$ and may be adjusted according to equations (3-11) and (3-12) if it is felt that a different runoff coefficient is appropriate for the particular land use in the given study area.

To illustrate the use of this methodology, assume that an area has combined sewers and portions of the area are residential (with $PD_d = 20$), commercial, and industrial. If the street sweeping interval is greater than 20 days, the BOD_5 pollutant concentration for each area may be calculated as follows:

Residential: $p_1(PD_d) = 0.142 + 0.218\ (20)^{0.54}$

$p_1(PD_d) = 1.24$

$\bar{c}_c = (48.6) \cdot (1.24) = 60.3$ mg/1 BOD_5

Commercial: $\bar{c}_c = 83.2 \cdot (1) = 83.2$ mg/1 BOD_5

Industrial: $\bar{c}_c = 36.7 \cdot (1) = 36.7$ mg/1 BOD_5

Other: $\bar{c}_c = 20.6 \cdot (.142) = 2.9$ mg/1 BOD_5

The concentration calculated for each land use category may then be assigned to the corresponding runoff flow from that area.

Note that the values obtained from this methodology represent averages of widely scattered data, and one should not be surprised if the actual monitored runoff concentrations for a particular area differ noticeably from those calculated. The concentrations calculated with the EPA land use-population density approach differ from the results summarized in Table 3-3. For example, in the land use approach all combined sewer concentrations are assumed to be 4.12 times greater than the separate sewer concentrations, while Table 3-3 indicates higher SS concentrations in separate sewer areas.

TABLE 3-6

a(a) AND b(b) FACTORS(c) FOR GIVEN $C_v(i)$

			Pollutant				
	Land Use	$C_v(i)$	BOD_5 (j=1)	SS (j=2)	VS (j=3)	PO_4 (j=4)	N (j=5)
i=1	Residential(a)	0.30	11.8	240	139	0.50	1.9
i=2	Commerical(a)	0.70	20.2	140	88	0.48	1.9
i=3	Industrial(a)	0.60	8.9	214	105	0.52	2.0
i=4	Other Developed Areas(a)	0.10	5.0	119	115	0.44	2.7
i=1	Residential(b)	0.30	48.6	989	574	2.04	8.0
i=2	Commerical(b)	0.70	83.2	578	364	1.97	7.7
i=3	Industrial(b)	0.60	36.7	883	434	2.14	8.4
i=4	Other Developed Areas(b)	0.10	20.6	492	473	1.81	11.0

(a) Applies to separate and unsewered drainage areas

(b) Applied to combined drainage areas

(c) Units are mg/l

The method presented for relating runoff concentrations to land use and drainage basin characteristics was developed with correlations to nationwide data. These estimates may be further refined with local runoff quality data. Regression technqiues have been used to relate runoff concentrations to local land use characteristics and storm parameters with varying degrees of success (29,30,31). Note that unless statistically significant correlations are obtained, regression equations will provide no more accuracy than the simpler techniques presented.

The estimations thus far introduced are for the mean runoff concentration, $\bar{c}$. The other value of interest is the variation of the runoff concentration (between storms), ν_c. Preliminary analyses have indicated ν_c ranging from 0.50 to 1.00. For an initial estimation, a conservative value of $\nu_c = 1.00$ may be selected (22).

Once representative estimates of the runoff coefficient and the pollutant concentrations have been determined, the storm loading assessment may proceed. Two particular approaches, a statistical method and a simulator method will now be presented.

3.4.3 A Statistical Method for the Assessment of Storm Loads

A simplified statistical approach to stormwater loading problems is described in this section. The statistical method for the assessment of runoff and treatment (22) is a flexible tool for the initial analysis of stormwater loads, their impacts, and alternative control strategies. It is a methodology rather than a specific computer model with fixed inputs, algorithms, and results. The basic statistical attributes of the rainfall-runoff process are used to provide simple initial estimates of the quantities pertinent to stormwater assessments. The statistical method may be applied initially without extensive data requirements and sophisticated urban runoff models. More complex models may be subsequently incorporated if more refined estimates are needed.

The statistical method begins with an analysis of rainfall, the basic driving force in the generation of stormwater loads. Raingage data is analyzed to provide a statistical summary of the rainfall process. The characteristics of the drainage area are then used to determine the runoff flow and associated pollutant load. This load may be modified by treatment or storage; either by the existing conveyance system, or by special stormwater control facilities. The stormwater loads thus developed are subsequently applied to the receiving water to determine the severity of their impacts (Chapter 5).

Initial storm load assessments require a broad summary of the rainfall-runoff process. The statistical method performs this summary by determining the statistical properties of the relevant storm characteristics. The simulator method presented in Section 3.4.4 operates directly on the actual rainfall record to perform the assessment. Both methods determine the expected frequency of various storm load magnitudes. If the probability distribution functions of the storm characteristics are well represented by the theory of the statistical method, the results of the statistical and simulator methods will be similar. The statistical method provides a more general initial summary, while the simulator method allows for more specific analyses of temporal and spatial detail. The statistical method is outlined in Figure 3-10, and specific procedures for employing the methodology will now be presented.

3.4.3.1 Statistical Rainfall Characterization

Hourly rainfall data is obtained for a minimum of five years of record to provide sufficient confidence in the rainfall characterization. Longer term records covering many years are preferable and are usually available for U.S. Weather Bureau Stations.

The hourly data are then summarized into storm events, each with an associated unit volume (v, in), duration (d, hr), average intensity ($i = v/d$, in/hr), and time since the previous storm (δ, hr), measured from the midpoint of the successive storms. A storm definition must be established to determine when in the hourly record a storm begins and ends. Additional refinements may be added to include such things as snowfall, trace storms, or whatever appears appropriate for the study area (22). A computer program is available which reads hourly rainfall records, defines the end of a storm by a fixed number of consecutive

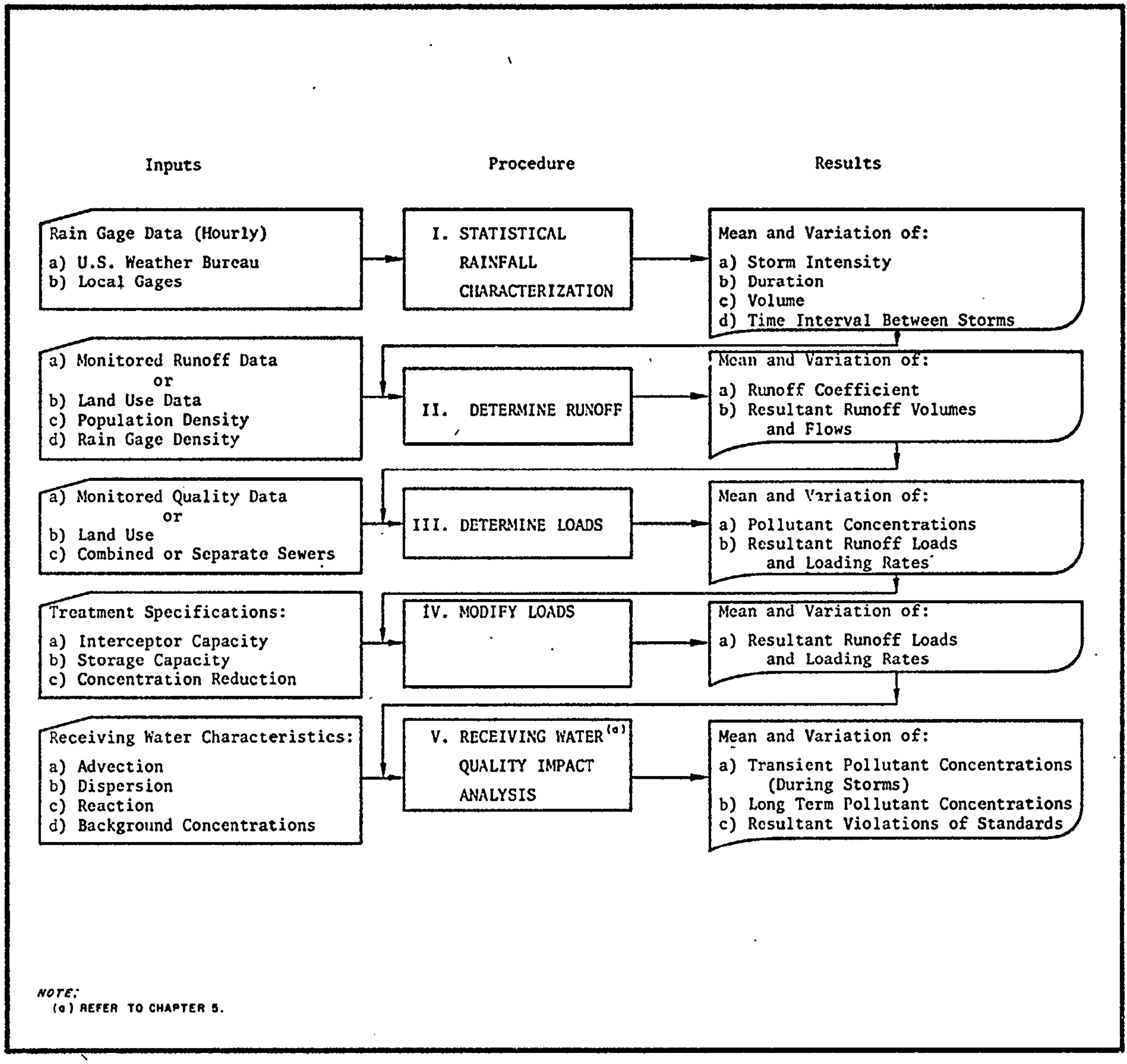


FIGURE 3-10
STATISTICAL METHOD FOR THE ASSESSMENT OF R·JNOFF

hours without precipitation, includes all forms of precipitation and determines the relevant individual storm characteristics. This program is outlined in Appendix E which will be included in this manual at a later date.

The statistics of the storm parameters are then computed. The mean and the coefficient of variation of each parameter are determined: the mean is the arithmetic average; the coefficient of variation is defined as the standard deviation divided by the mean. The required statistics are summarized below.

Parameter	For Each Storm	Mean	Coefficient of Variation
Storm Intensity	i	I	ν_i
Duration	d	D	ν_d
Unit Volume	v	V	ν_v
Time Between Storms	δ	Δ	ν_δ

Note that if storm intensities and durations are independent, the mean storm volume, V, will equal ID. In many areas, they are not independent; for example, long less-intense storms tend to occur in the winter, and short more-intense storms tend to occur in the summer. In such cases, V will not equal ID. To avoid this potential error, the rainfall analysis program determines V for the individual storm volumes. If a particular season or period is considered critical due to adverse receiving water characteristics or greater pollutant accumulation rates, the representative summary may simply be made on the long term record of storms occurring during the selected season. The rainfall analysis program described in Appendix E provides monthly summaries which can be used to indicate and analyze seasonal characteristics.

Once the mean and the coefficient of variation of the rainfall characteristics have been determined, the cumulative density function may be developed. This is done by assuming that the rainfall parameters, storm intensity, i, duration, d, and time between storms, δ, are gamma distributed. Storm volume, v, would then have a distribution determined by the product of two gamma distributed random variables (i and d).

In practice, storm volumes have also been found to be fairly well represented by a gamma distribution.

The cumulative density function for a gamma distribution is shown in Figure 3-11. A number of graphs have been used because of the confusing manner in which the curves intersect. For example, if the variation of storm intensities determined from the statistical rainfall analysis is: $\nu_i = 1.00$, from Figure 3-11(a) the 90th percentile intensity would then be 2.3 times the mean intensity, I. In other words, ten percent of the storms have average intensities greater than 2.3(I). Interpolation may be used for intermediate values of variation.

The expected number of storms greater than a given value may then be calculated. The average number of storms occuring during a given period is first calculated:

$$\text{Average number of storms} = \frac{\text{Length of Period}}{\Delta} \qquad (3\text{-}13)$$

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:

$$\text{Average Number of Storms} = \frac{1 \text{ year} \;.\; 8760 \text{ hr/year}}{70 \text{ hr}} = 125$$

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 intensities greater than 2.3(I).

3.4.3.2 Determination of Runoff

The rainfall parameters are next converted into runoff. The approach for initial assessments uses the average volumetric runoff coefficient, C_V. The runoff coefficient indicates the fraction of the storm volume which reaches the conveyance system:

$$V_R = 3630 \;.\; C_V VA \qquad (3\text{-}14)$$

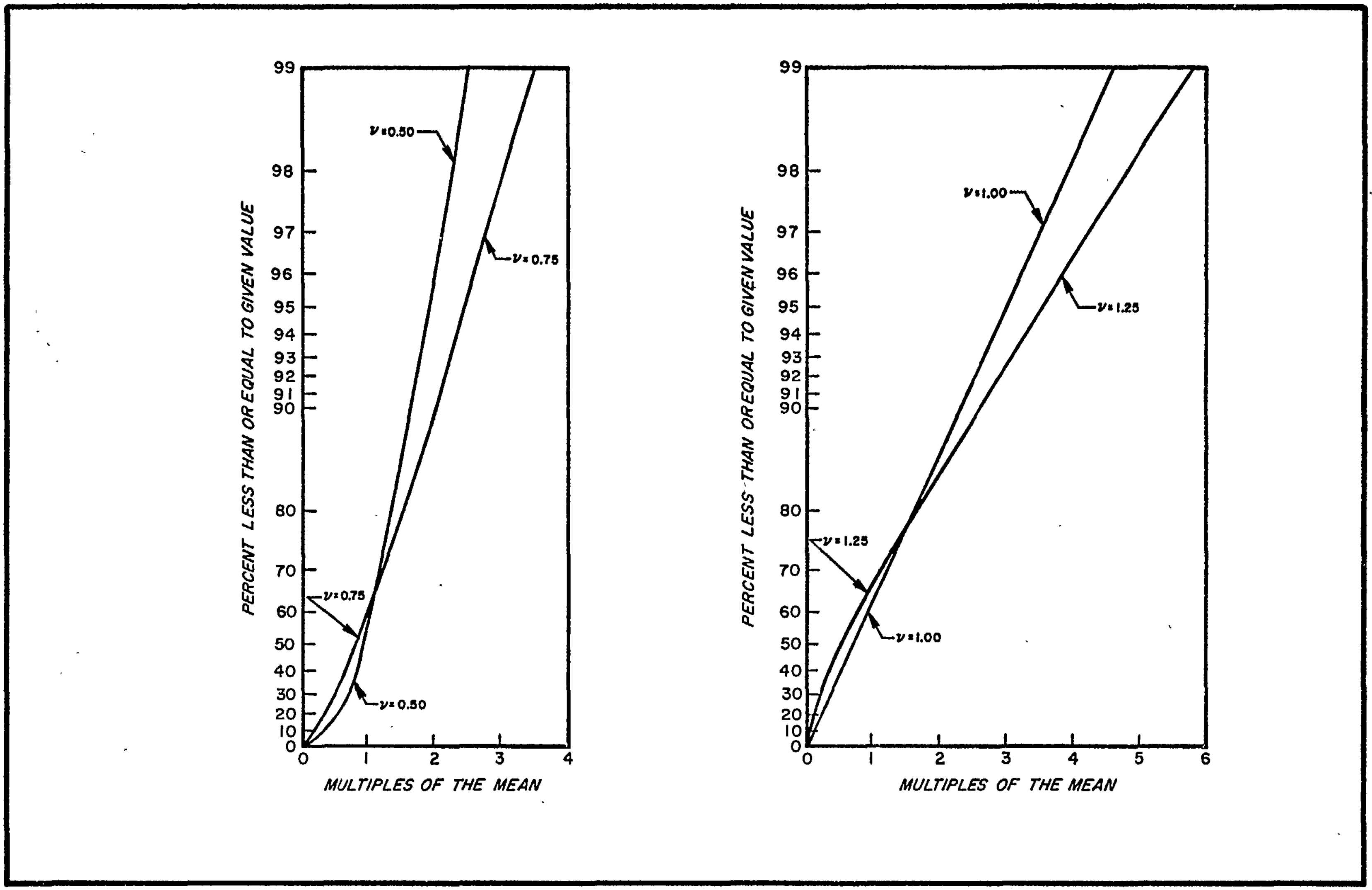


FIGURE 3-11(a)
CUMULATIVE DENSITY FUNCTION FOR
GAMMA DISTRIBUTION

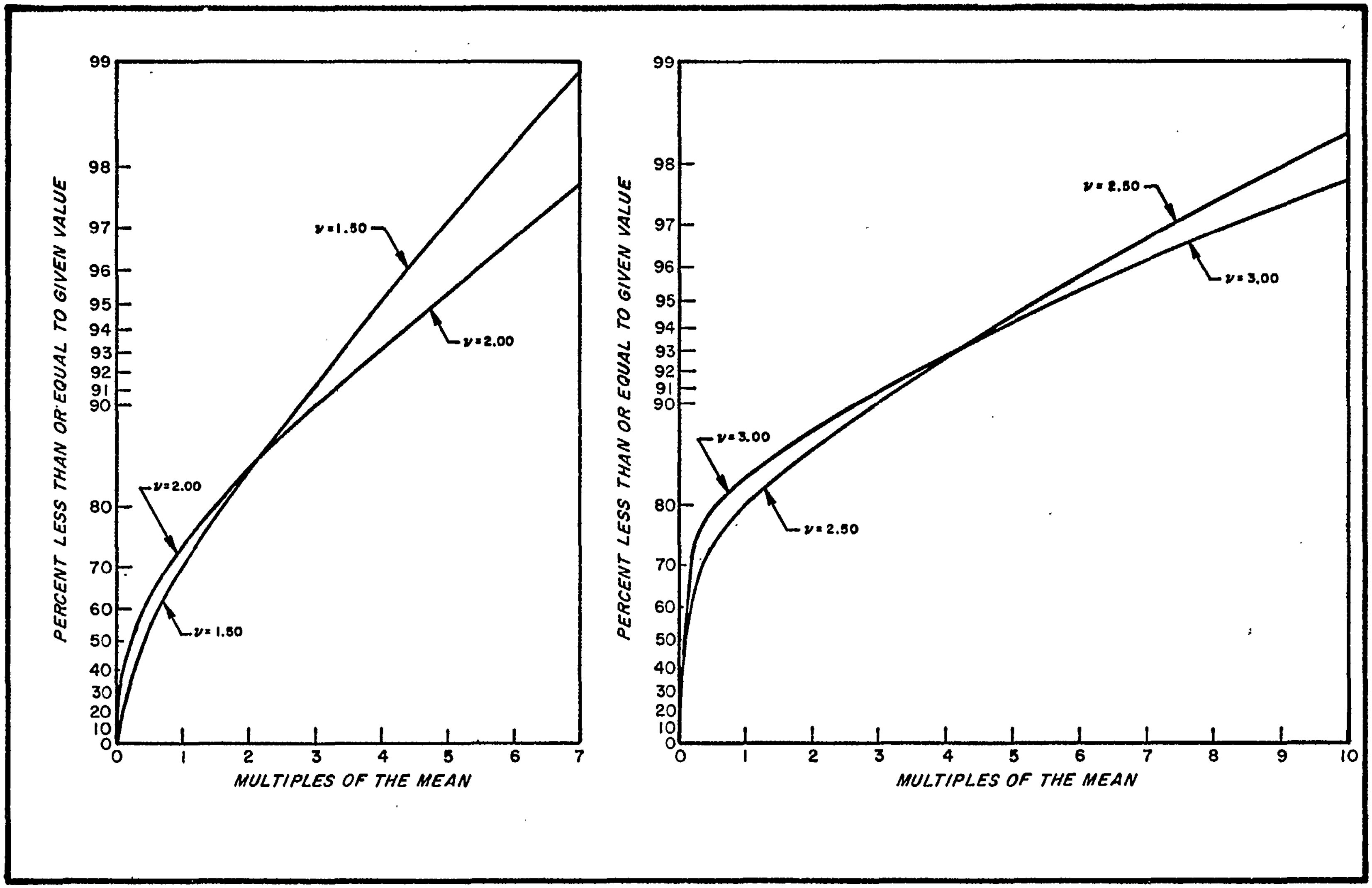


FIGURE 3-11(b)
CUMULATIVE DENSITY FUNCTION FOR GAMMA DISTRIBUTION

where:

V_R = mean runoff volume (ft^3)

V = mean unit rainfall volume (in)

A = drainage area (acres)

3630 = conversion factor to make units consistent (ft^3/acre-in).

The runoff coefficient may also be used to estimate the flow rate:

$$Q_R = C_V IA \tag{3-15}$$

where:

Q_R = mean runoff flow (cfs)

I = mean rainfall intensity (in/hr).

Depending upon the size and characteristics of the drainage basin, this may not acurately represent the attenuation of runoff beyond the end of a rain event. The value of Q_R will be overestimated for a large catchment area with a long time of concentration (the time it takes runoff from the farthest portion of the drainage area to reach a particular point in the receiving water). The estimate of Q_R may be tood conservative even for an initial assessment. Unit hydrograph analysis (32) may provide guidance for evaluating or correcting the overestimation of Q_R. In some cases, however, more sophisticated models employing the time routing of flows may be required.

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 is 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. Procedures for estimating the average runoff coefficient, C_V, are presented in Section 3.4.2.1..

The fraction of the storm volume that is measured as runoff is not fixed. There is some variation in the runoff coefficient due to differences in soil moisture, soil infiltration rates or the available depression storage due to the influence of individual storm conditions and the length of time since the previous storm. There is also a variation in the measure of the runoff coefficient resulting from the variation from storm to storm in the amount of rainfall recorded at a point (raingage) versus the amount that actually falls over the entire catchment area.

Studies dealing with the relationship between point rainfall data and the areal distribution of rainfall (33-38) indicate that for many areas, point rain data will be representative of very large areas, when evaluated on a long term basis (e.g. annual rainfall volume). Characterizing runoff during individual storm events from a large drainage area with a limited number of raingages, however, is a separate consideration. Significant variation in the measured runoff coefficient has been observed. Studies are now underway (22) to quantify a relationship between the raingage density, the type of storm patterns typical for an area, and the variation in the runoff coefficient. Preliminary results suggest that the variation in volumetric runoff coefficient can become substantial if the raingage density is less than one per square mile (22). When completed, the information gained from these studies will permit the variation of the runoff coefficient to be incorporated into the estimate of the variation of the runoff volume, ν_{vR} and the variation of the runoff flow, ν_q. Until then, the estimate of runoff variation may be based solely upon the variation of the measured rainfall parameters:

$$\nu_{vR} = \nu_v \qquad (3\text{-}16)$$

$$\nu_q = \nu_i \qquad (3\text{-}17)$$

Runoff flows and volumes have also been observed to be well represented by a gamma distribution. The cumulative density functions of Figure 3-11 may then be used to predict the fraction of storms and the expected number of times per year, month, or season that a given flow or volume is exceeded.

3.4.3.3 Determination of Loads

Runoff flows may be translated into stormwater loads by multiplying the runoff flows by the appropriate pollutant concentration, c (mg/l). If storm runoff flows and concentrations are 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).

$$W_R = 5.4 \cdot \bar{c}\, Q_R \tag{3-18}$$

It is best to begin by assuming $\bar{c}$ and Q_R are independent. If data collected for specific pollutants indicates they are not, the following refinement may be employed:

$$W_R = 5.4 \cdot \bar{c} Q_R \, (1 + \nu_c \nu_q \rho_{cq}) \tag{3-19}$$

where:

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

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

ρ_{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. Note that the correlation between flow and concentration is not the first flush effect, which relates the concentration within storms to the time or volume since the beginning of the storm (to be dealt with in the section on storage/treatment in Chapter 6). The correlation dealt with here relates to storm event averages of the flow and concentration. Procedures for estimating pollutant concentrations in stormwater are presented in Section 3.4.2.2.

The variation of the runoff loading rate is due to the variation of the concentration and the flow. Assuming the flow and concentration are independent, the variation of the runoff loading rate, ν_w, may be determined as follows:

$$\nu_w = \nu_q \nu_c \sqrt{1 + 1/\nu_q^2 + 1/\nu_c^2} \tag{3-20}$$

The calculation of the variation of the runoff loading rate, ν_w, when flow and concentration are not independent has not yet been fully developed. Until the method (22) for determining the variation of the runoff loading rate, ν_W, is developed for the case where flows and concentrations are not independent, the assumption that flows and concentrations are independent must be made, and Equation (3-20) used for a first estimate.

The actual cumulative density function for storm loading rates has not yet been determined. It is formed by the product of two gamma distributed random variables (c and q). As a first estimate, the loading rate is assumed to be gamma distributed as well, as is the case when a constant "typical" value is assigned to the runoff concentration. The cumulative density function of Figure 3-11 may again be used to predict the fraction of storms and the expected number of times per year, month, or season that a given storm loading rate is exceeded, as outlined in the section on rainfall characterization.

The average loading rate, W_R, is representative of stormwater 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 non-rain 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 may also be calculated from the storm statistics for the year, month, or season of interest as follows:

$$W_o = W_R \, D/\Delta \tag{3-21}$$

where:

D = average storm duration (hr)

Δ = average time between storms (hr)

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. As suggested in Section 3.4.3.1, 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 occurrance 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 loading rate during storms, W_R, and the variation, ν_W, become the important indicators of stormwater loadings. The relative impact of long term and transient storm loads on receiving water quality is discussed in Chapter 5.

3.4.3.4 Modification of Loads by Existing System

Stormwater loads may be reduced by employing end of pipe control techniques, including interception and storage for eventual treatment (to be analyzed in detail in Chapter 6). Control measures may also include management practices, such as street sweeping. Where appropriate, the effect of these practices can be incorporated in the determination of the mean runoff concentration, $\bar{c}$, as discussed in Section 3.4.2.2.

If existing systems already contain some level of treatment or control, this must be incorporated in the determination of current stormwater loads. For urban areas with combined sewer systems, the interceptor will prevent a portion of the runoff from reaching receiving waters as storm overflows. A basis is available with the statistical methodology for analyzing the effect of combined sewer interceptors which have a capacity in excess of dry weather flows (DWF).

The conveyance system may be characterized by an excess interceptor capacity, Q_I, which is the available capacity in the interceptor for stormwater runoff. In a combined sewer system, Q_I is the total capacity minus the dry weather flow (determined by regulator operation). The fraction of the long term runoff load, f_I, which is captured by an interceptor with excess capacity, Q_I, has been calculated and is shown in Figure 3-12. The performance is a function of the ratio of the excess capacity to the mean runoff flow, Q_R, and the variation of the runoff flow, ν_q. Note that the greater the variation in runoff flow, the more poorly the interceptor will perform on average. The reduction of the long term runoff load corresponds to a reduction in the mean runoff loading rate, W_R, for storm events. The fraction not captured is $f_I = \bar{W}/W_R$ (where $\bar{W}$ is the load bypassed or overflowed).

Conveyance systems also provide a degree of in-system storage, V_E, both in the pipe themselves and due to the layout of diversion structures. The storage capacity effectively retains runoff until the storm subsides, when the stored stormwater may then be fed to the treatment system or directly to the stream. The long term performance of a storage device is a function of the ratio of the effective storage capacity to the mean runoff volume, V_R, and the variation of the runoff volume, ν_{vR}. Note that the volume utilized by the dry weather flow should be subtracted from the total internal storage to determine the effective storage capacity. The long term performance curves are shown in Figure 3-13. The fraction of the load not captured is $f_V = \bar{W}/W_R$. These curves were drawn assuming there is no first flush effect. When a first flush effect does exist, a disproportionately high fraction of the runoff <u>load</u> will be captured by the storage device. The improvement in long term storage device performance due to the first flush effect is depicted in Figure 3-14. When the first flush exists, Figure 3-14 may be used to redraw the performance curve obtained from Figure 3-13, with the corresponding new values of f_V. The fraction of the load not captured by the combined effect of interception and storage, f_{IV}, may be approximated by the product of the individual fractions not captured: $f_{IV} = f_I f_V$.

Assuming that the load captured by the conveyance system receives treatment such that there is a fractional removal, r, the resultant mean runoff loading rate, $W_R{}^*$, will be:

$$W_R{}^* = f_{IV} W_R + (1-f_{IV})\, W_R(1-r) \qquad (3\text{-}22)$$

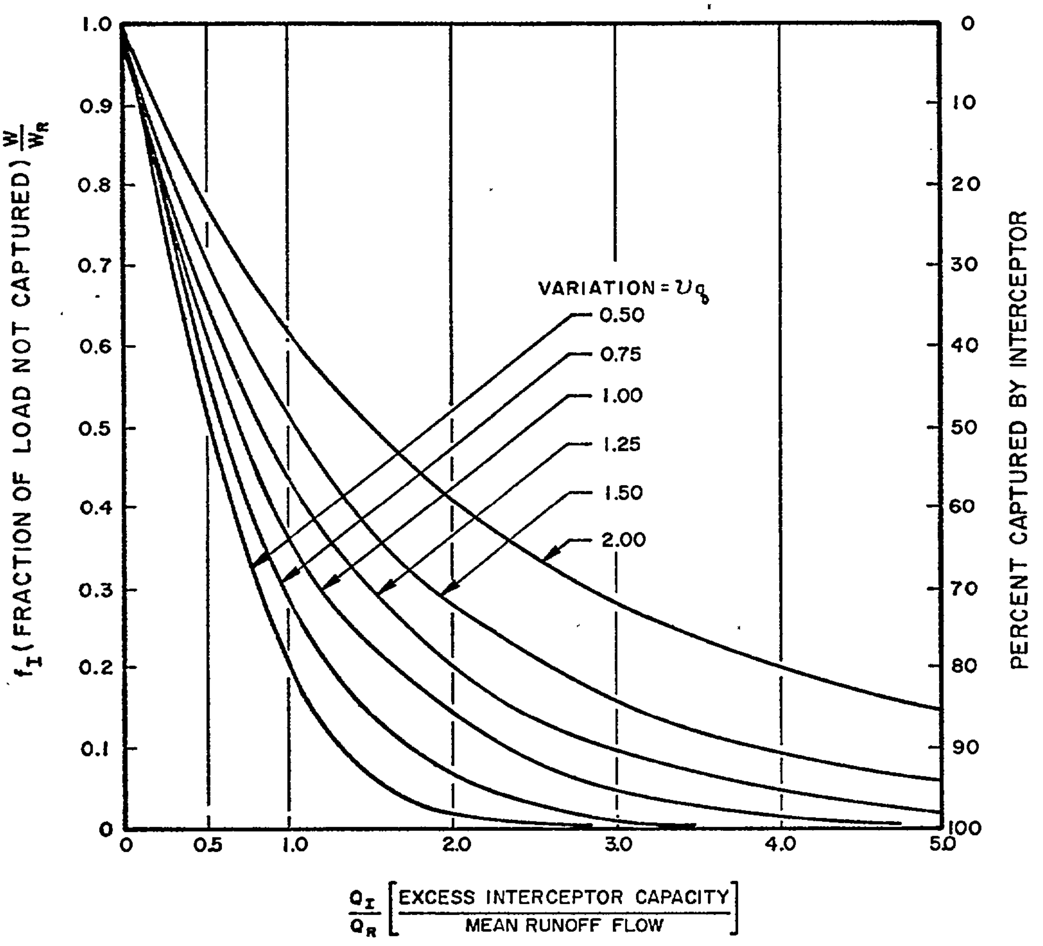


FIGURE 3-12
DETERMINATION OF LONG TERM INTERCEPTOR PERFORMANCE

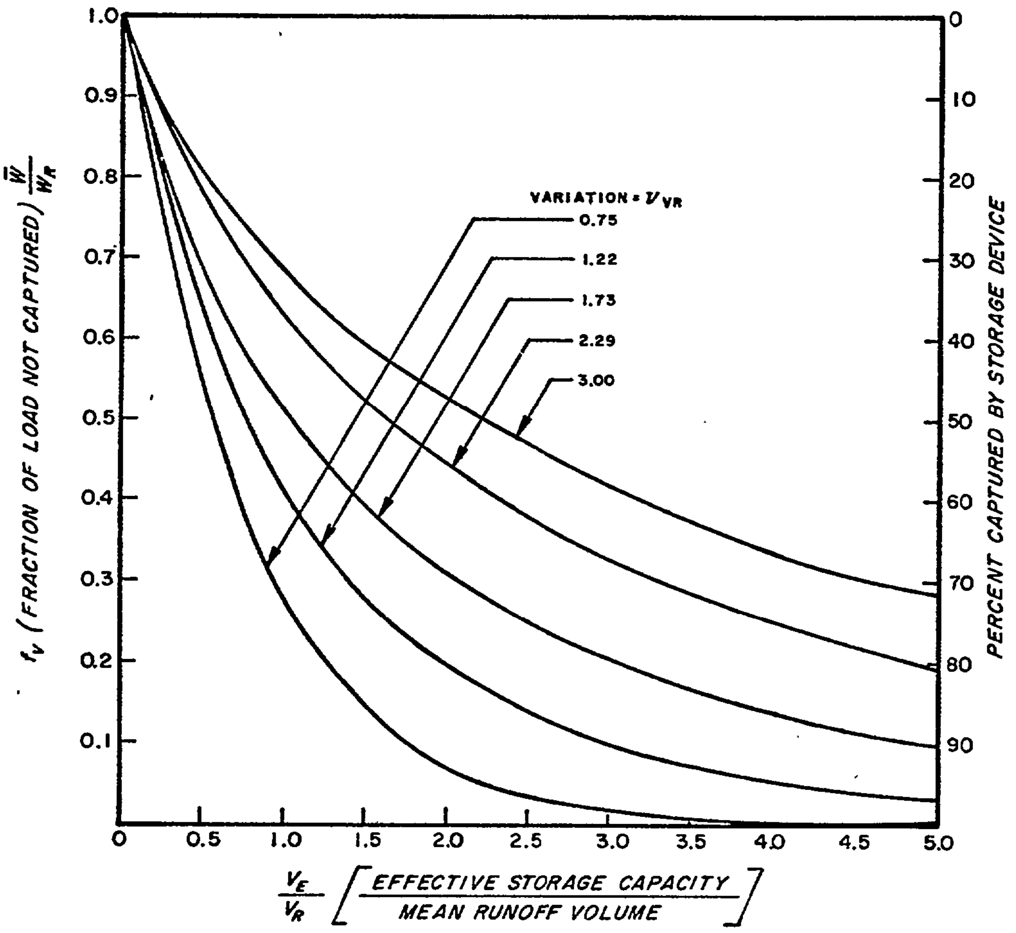


FIGURE 3-13

DETERMINATION OF LONG TERM STORAGE DEVICE PERFORMANCE

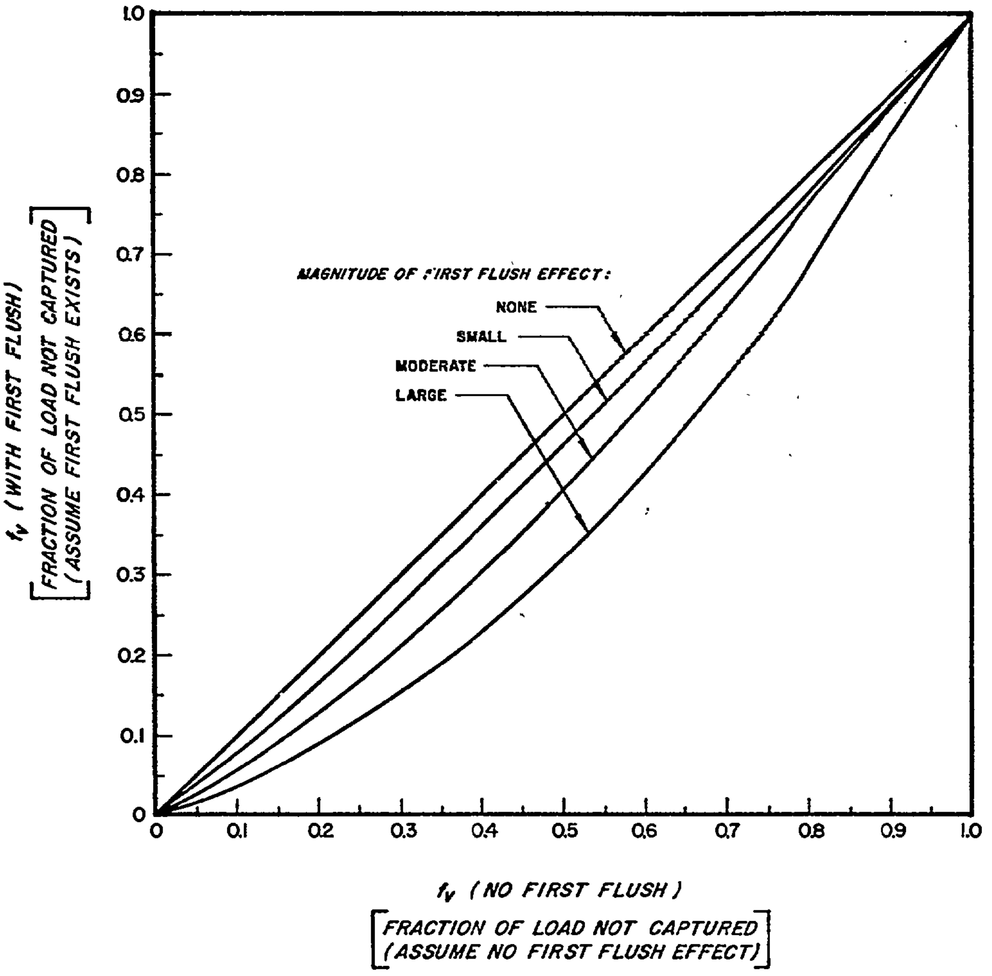


FIGURE 3-14
IMPROVEMENT IN LONG TERM STORAGE DEVICE PERFOMANCE
DUE TO FIRST FLUSH EFFECT

In a combined sewer system, r would represent the treatment received at the municipal treatment plant. The curves relating the impact of interception and storage on the variation of the runoff load are currently being developed (22).

3.4.3.5 Receiving Water Quality Impact Analysis

Once the stormwater loads have been determined, they are applied to the receiving water. The statistical nature of the resulting loads, together with the receiving water characteristics, determine the statistical properties of pollutant concentrations in the receiving water. Factors such as advection, dispersion, reaction, and background concentrations in the receiving water will determine the resulting water quality impact. Statistical analyses may be necessary for both transient and long term concentrations. Methods for performing these analyses will be presented in Chapter 5. The final output of the methodology is the predicted frequency with which relevent water quality standards and guidelines are violated.

3.4.3.6 Example Application of the Statistical Method

To illustrate the use of the statistical method, an example will be presented for a hypothetical drainage area, using rainfall records from the City of Denver, Colorado.

Statistical Rainfall Characterization

Twenty-five years (1949 through 1973) of hourly rainfall data for U.S. Weather Bureau Station 052220 were analyzed. The resulting storm statistics were calculated for each month and are shown in Figure 3-15. Seasonal patterns are evident in the data, with shorter, more frequent and more intense storms occurring in the summer. Assuming that critical water quality conditions occur during the summer, from June through September, the relevant storm characteristics should be taken from this period. These are summarized below.

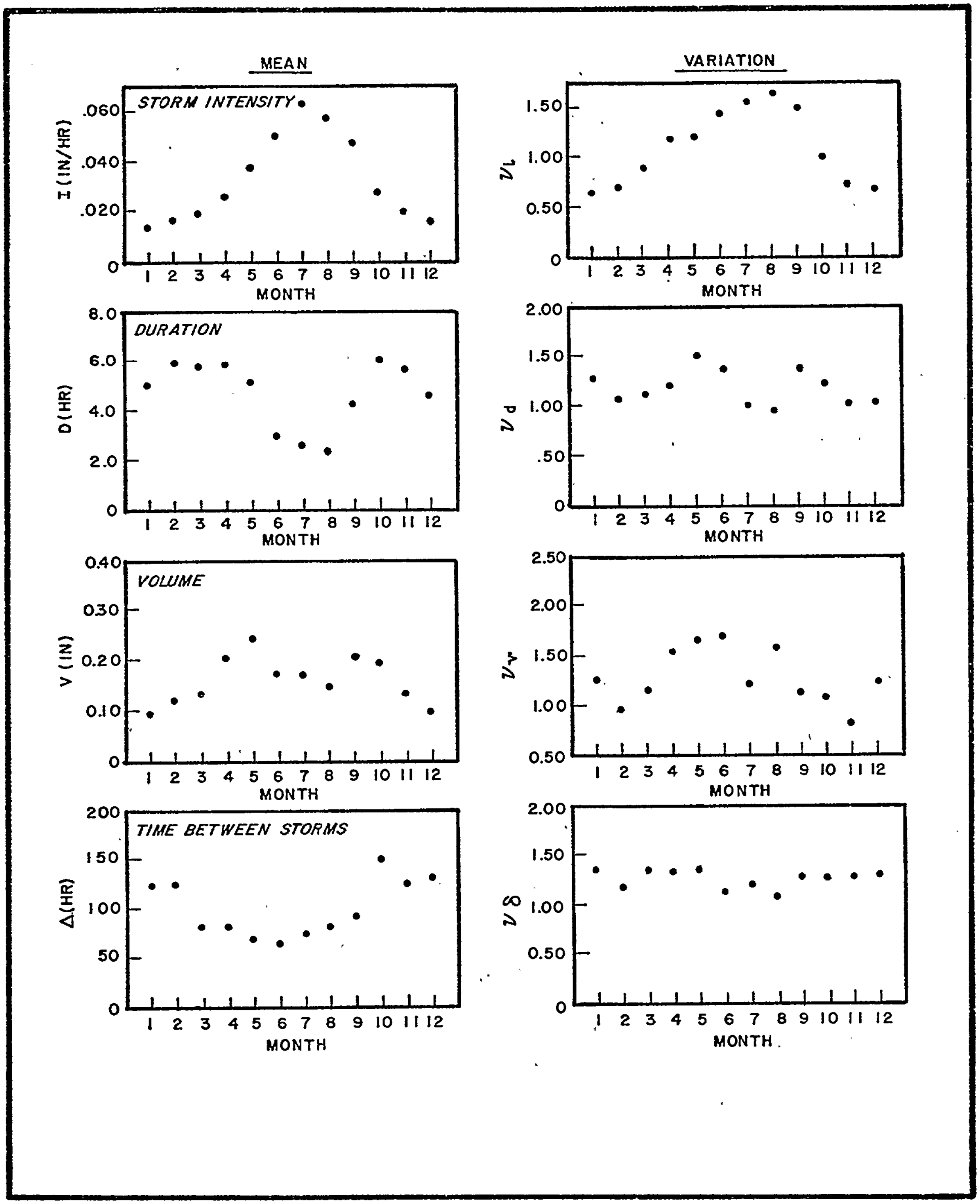


FIGURE 3-15

MONTHLY STATISTICAL RAINFALL CHARACTERIZATION
DENVER, COLORADO
STATION 052220

APPROXIMATE SUMMER STORM CHARACTERISTICS FOR DENVER
(JUNE THROUGH SEPTEMBER)

Characteristics	Mean	Variation
Storm Intensity	$I = 0.055$ in/hr	$\nu_i = 1.55$
Duration	$D = 3.0$ hr	$\nu_d = 1.15$
Volume	$V = 0.18$ in	$\nu_v = 1.90$
Time Between Storms	$\Delta = 80$ hr	$\nu_\delta = 1.15$

Determination of Runoff

The hypothetical drainage area is estimated as 1875 acres with an imperviousness of a little over 30 percent. Assume that 70 percent of the area is served by separate sewers or natural conveyance, and 30 percent is served by combined sewers. Given this assumption, the separate or unsewered area is 1310 acres, and the combined sewer area is 565 acres. In the example calculation it is assumed that the percent imperviousness is the same in both subareas. The runoff coefficient, C_V, for both subareas, is estimated from Figure 3-8, to be 0.35. One should note, however, that the population density in combined sewered areas is generally greater than that of separate sewered areas. Thus the percent imperviousness as shown in Equation 3.2 and consequently the runoff coefficient is usually higher in combined sewered areas.

The mean runoff flow and volume during summer months is calculated from the approximate summer storm characteristics, the runoff coefficient, and the drainage areas.

Separate or Unsewered Area

$$V_R = 3630 \cdot C_V VA$$

$$= 3630(\text{ft}^3/\text{acre-in}) \cdot (0.35) \cdot (0.18 \text{ in}) \cdot (1310 \text{ acre})$$

$$= 3 \times 10^5 \text{ ft}^3$$

$$Q_R = C_V IA$$

$$= (0.35) \cdot (0.055 \text{ in/hr}) \cdot (1310 \text{ acre}) \cdot (1 \text{ cfs}/(\text{acre-in/hr}))$$

$$= 25 \text{ cfs}$$

Combined Sewer Area

V_R = 3630 (ft^3/acre-in) . (0.35) . (0.18 in) . (565 acre)

= 1.3×10^5 ft^3

Q_R = (0.35) . (0.055 in/hr) . (565 acre) . (1 cfs/(acre-in/hr))

= 11 cfs

The variation of the runoff volume and flow in both subareas may be estimated from the variations calculated for the corresponding rainfall values:

$\nu_{vR} = \nu_v = 1.90$

$\nu_q = \nu_i = 1.55$

Determination of Loads

For this example, BOD_5 will be used as the variable of interest. From Table 3-3, the BOD_5 concentration of the runoff from the separate and unsewered area, $\bar{c}_s$, is estimated to be 27 mg/l, and the BOD_5 concentration of the runoff from the combined sewer area, $\bar{c}_c$, is estimated to be 108 mg/l. The resulting loading rate, W_R, during summer storms is calculated as follows:

Separate or Unsewered Area

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

= (5.4 lb/day/cfs-mg/l) . (27 mg/l) . (25 cfs)

= 3600 lb/day BOD_5

Combined Sewer Area

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

= (5.4 lb/day/cfs-mg/l) . (108 mg/l) . (11 cfs)

= 6400 lb/day BOD_5

The variation of the BOD_5 loading rate for each subarea, ν_w, is estimated using Equation (3-20). The variation of the flows, ν_q, has been estimated as 1.55, and the variation of the BOD_5 concentration, ν_c, may be conservatively estimated as 1.00, because local data is not available. The calculation of ν_w follows:

$$\nu_w = \nu_q \nu_c \sqrt{1 + \frac{1}{\nu_c^2} + \frac{1}{\nu_q^2}}$$

$$= (1.55) \cdot (1.00) \quad \sqrt{1 + \frac{1}{(1.00)^2} + \frac{1}{(1.55)^2}}$$

$$= 2.41$$

The long term summer loading rate, including the non storm periods, will be:

Separate or Unsewered Area

$W_o = W_R D/_\Delta$ = (3600 lb/day) . (3.0 hr/80.0 hr) = 135 lb/day BOD_5

Combined Sewer Area

W_o = (6400 lb/day) . (3.0 hr/80.0 hr) = 240 lb/day BOD_5

The total average loading rate from both the separate or unsewered and combined sewer areas may be calculated by adding the loads from each:

Total Drainage Area (Hypothetical)

W_R = (3600 + 6400) = 10,000 lb/day BOD_5

W_o = (135 + 240) = 375 lb/day BOD_5

The simplified assumption that storm loads are gamma distributed may now be used to estimate the frequency of occurrence of different loading rates from the total drainage area. Given that ν_w = 2.41, Figure 3-11(b) indicates the cumulative density function for storm loads in multiples of the average load, W_R = 10,000 lb/day BOD_5. The average number of storms per summer will be:

$$\text{Average number of storms} = \frac{\text{Length of Period}}{\Delta}$$

$$= \frac{122 \text{ day (June-Sept.)} \cdot (24 \text{ hr/day})}{80 \text{ hr}}$$

$$= 36.6$$

The number of storms per summer exceeding a given average loading rate is then the fraction exceeding that rate times the average number of storms. The calculations for various loading rates are summarized below:

FREQUENCY OF STORM LOADS FOR HYPOTHETICAL DRAINAGE AREA

W (lb/day BOD_5)	W/W_R	Percent less than or Equal to W	Fraction Greater than W	Number of Storms Per Summer Greater than W
10,000	1.0	79	0.21	7.7
20,000	2.0	86	0.14	5.1
30,000	3.0	89	0.11	4.0
40,000	4.0	92.5	0.075	2.7
50,000	5.0	94.5	0.055	2.0

W_R = Avg. Summer Runoff Load = 10,000 lb/day BOD_5

Modification of Loads By Existing System

The stormwater loads for the hypothetical drainage area have been calculated by assuming 70 percent of the area is served by separate or unsewered conveyances, and 30 percent of the area is served by combined sewers. Existing separate sewer systems generally convey runoff loads to the receiving water without modification. The combined sewer load, however, will be modified by the interceptor system. Assume that the combined sewer area interceptors have an excess capacity of 11 cfs, equal to the mean runoff flow ($Q_I/Q_R = 1.0$), and an internal storage of 0.65 X 10^5 ft^3, or one half the mean runoff volume ($V_E/V_R = 0.50$). For Q_I/Q_R of 1.0, and ν_q of 1.55, Figure 3-12 indicates the fraction not captured by the interceptor, f_I is 0.53. For V_E/V_R of 0.50, ν_{vR} of 1.90, and a moderate first flush effect, Figure 3-13 and 3-14 are used to determine the fraction not captured by storage, f_V of 0.68. Therefore, the fraction of runoff not captured due to both interception and storage, f_{IV} is (0.53). (0.68) or 0.36. Assuming the captured runoff is treated with forty percent removal (r = 0.40), the modified average summer storm load, W_R*, from the combined sewer area is:

Combined Sewer Area

$$W_R^* = f_{IV}W_R + (1-f_{IV})W_R(1-r)$$

$$= (0.36) \, . \, (6400) + (0.64)\ (6400)\ (0.60)$$

$$= 4760 \text{ lb/day BOD}_5$$

with a long term modified summer mass discharge rate, W_o*:

$$W_o^* = 180 \text{ lb/day BOD}_5$$

Therefore the total modified storm load from the drainage area, including both the spearate or unsewered area, and the combined sewer area is:

Total Drainage Area

$$W_R^* = (3600 + 4760) = 8360 \text{ lb/day BOD}_5$$

$$W_o^* = (135 + 180) = 315 \text{ lb/day BOD}_5$$

As the purpose of this chapter is to demonstrate methods for stormwater load estimation, example receiving water calculations will not be shown here, but will appear in Chapter 5.

3.4.4 A Simulation Method for the Assessment of Storm Loads

In addition to the statistical method, storm loads in urban areas may also be estimated by the use of simulation techniques. Simulators can be particularly useful in the examination of storm loads, the problems they cause, and their control measures, because of the detailed representation of the sequence of individual events, both in space and time, which they provide.

A review of Appendix A will indicate that available simulators represent a wide range of sophistication and level of detail. Most operate from an input of hourly rainfall data, and, based on physical characteristics and properties assigned to the drainage area, they calculate the loads generated on the same time scale as the rainfall input. The simulators which can be utilized to the best advantage in the assessment stage in 208 planning, are those which at the sacrifice of detail for individual storm events, are able to process relatively long periods of rainfall records and thus simulate a broad range of individual events.

A simulator which is considered to be particularly suitable for use in assessment studies is described below. The description is intended to provide information on the basic methodology employed by simulators in general, and to illustrate how a simulator can be utilized in the estimation of urban stormwater loads. The model which will be discussed is a simplified stormwater management simulator, developed by Metcalf and Eddy and used during a stormwater assessment study for Rochester, New York (29). It was developed as a screening technique for the planning and preliminary sizing of control facilities, and is more suitable for use in assessment studies than the more complex simulators for which it was substituted. The simulator can be used for the estimation of loads from combined, separate or unsewered systems.

Use of the simplified stormwater simulator for estimating storm loads includes the following tasks:

1. Rainfall Characterization
2. Data Preparation
3. Storage - Treatment Balance
4. Overflow-Quality Assessment

These are a series of interrelated tasks that can be performed either individually or together, and are composed of small computer programs and hand computations. The storage-treatment balance is the component which uses the computer simulator. The simulator generates a continuous record of overflows from a stormwater collection system by reproducing the drainage area characteristics and calculating responses to rainfall.

The other tasks listed are either required as support for the simulator (data preparation and overflow-quality assessment) or provide additional input in the overall stormwater assessment study (rainfall characterization). It will become apparent in discussing the tasks how each fits into the assessment.

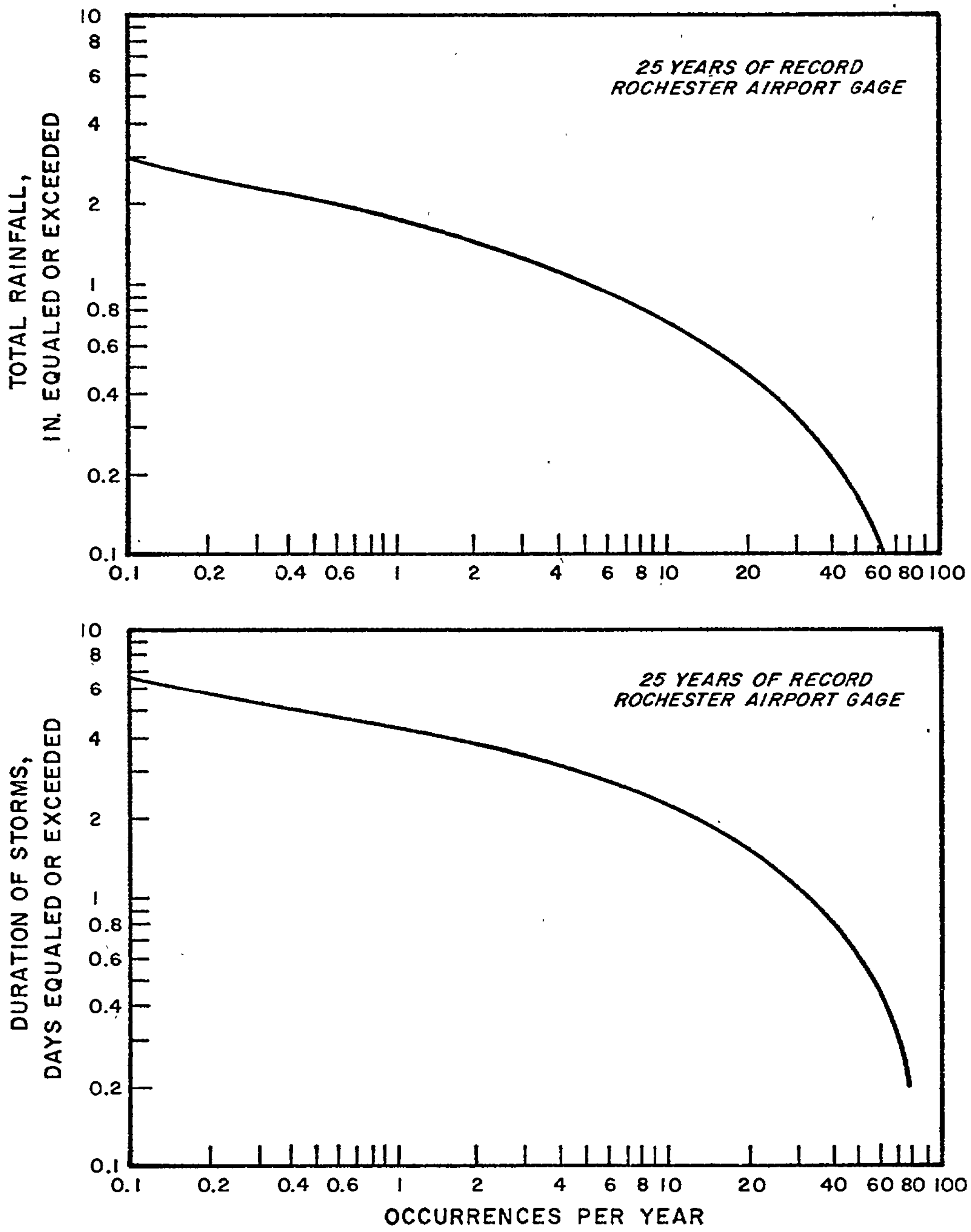
Rainfall characterization is a part of the approach incorporated into this simplified simulator to provide insight into the characteristics of rainfall and therefore runoff, occurring in an area. This characterization is similar to that employed in the statistical method previously described in that it characterizes the relative frequency of rain events or the properties of rain events, e.g. duration, total rainfall per storm, maximum intensity, etc. Examples of frequency curves presented in Figure 3-16 indicate the number of occurrences per year for storm volume and storm duration. Procedures for generating these characteristics from rainfall records are provided in the referenced report (29).

The methodology provided in this report will give information on the following items:

1. The total number of storms.
2. The number of storms having a total volume of less than 0.1 inch (approximate depression storage value).
3. The number of storms having durations greater than 24 hours.
4. The average number of days between storms.

The characteristics defined by this rainfall analysis are not used directly in the simulator, but as background information to aid in the evaluation and interpretation of results.

Data preparation is an important step in the modeling process. In addition to rainfall data, information on drainage area and collection system characteristics must be secured in order to provide the required input for the operation of the simplified simulator.



REFERENCE (29)

FIGURE 3-16
FREQUENCY OF STORM OCCURRENCES

Simplified simulator operation is illustrated conceptually by Figure 3-17. In the program, rainfall is converted into runoff, by using a K factor which is a volumetric runoff coefficient similar to the C_v that is used in the statistical method presented earlier. The runoff is stored in a specific storage volume, which represents the volumetric capacity of the storm and combined sewer system. Runoff which enters the system is removed by a specific "treatment rate" which represents the hydraulic capacity of the interceptor downstream of the overflow point. For combined sewers, both the storage volume and the interceptor capacity ("treatment rate") used in the simulator are previously calculated "net" values, which account for volume and flow capacity utilized by dry weather sanitary sewage flow (DWF). The schematic thus illustrates a combined sewer system. Separate storm sewer systems or natural conveyances in unsewered areas would be accomodated simply by equating DWF to zero, and the interceptor capacity to zero. Thus, the only flow in the system is storm runoff, and all of it "overflows".

When runoff exceeds the storage capacity with a continuous flow greater than the "treatment flow rate" during the time interval analyzed, an overflow occurs. The simulator can function on either a daily or hourly time step. A daily time step is suggested for analysis initially in order to make analysis of an entire period of record (often 20 years or more) practical. Examination of this output is used to identify critical periods for further examination. For specific periods of interest, including critical storms, the analysis may be performed on the hourly time step.

The simplified simulator calculates runoff from a drainage area and the net amount which enters the receiving water through overflows from the collection system. When suitable pollutant concentations are assigned to the runoff or overflow volumes, storm generated pollutant loads are calculated as the product of volume and concentration.

The critical elements which determine the accuracy of the waste loads calculated by the simulator are the relationships established for:

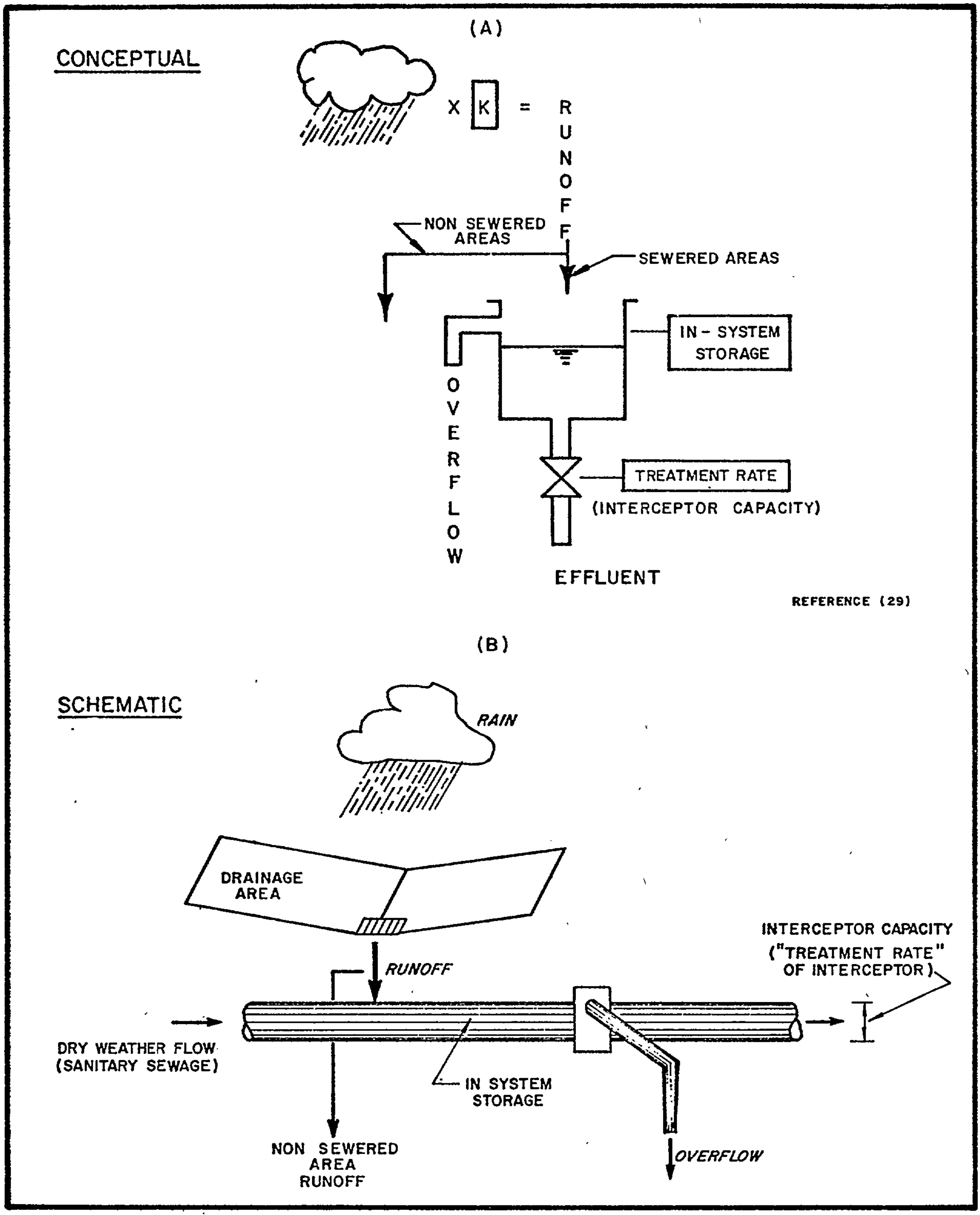


FIGURE 3-17
CONCEPT OF STORAGE-TREATMENT PROGRAM

1. Defining the component of rainfall which will leave the drainage area as runoff. This is the volumetric runoff coefficient, C_v, (or K, depending on the reference used).
2. Defining the pollutant concentrations associated with the runoff.

The output from the simulator is a record of the time and volume of runoff and overflows, and the waste loads associated with them, together with a summation of these parameters. The summation is terminated at the end of each year for the daily analysis, and at the end of each month for the hourly analysis.

The simulator operates on actual rainfall records, and therefore it internally accounts for the synergistic effect of storms coming close together with overlapping demands on storage capacities. If the rainfall record covers many years, then the runoff, overflow volumes and durations can be filed and ranked, and a statistically significant frequency of occurence curves can be generated.

3.4.4.1 Example Application of the Simplified Simulator

The simulator can be applied, utilizing a range of levels of spatial detail. In its simplest form, it would operate on a single aggregated drainage area in the same manner the statistical method was applied in the previous example. The addition of spatial detail may be employed when appropriate, while still utilizing the basic rain-runoff-quality calculations which are employed at the simplest level.

For the simplest case, use of the simulator would involve the following steps:

1. Secure rainfall records. They may be analyzed to determine the statistical properties for aid in evaluating simulator output. Hourly rainfall data is used in the simulator.
2. Characterize the Drainage Area

   a. Determine total area, area served by combined sewers, and separate and unsewered area.

b. Determine percent impervious area, using guidelines or estimating relationships previously presented.

c. Estimate runoff coefficient (called K factor in this reference), using either available data or basing estimate on imperviousness.

d. Estimate or measure average dry weather sewage flow.

3. Estimate or determine internal storage in sewer system, and a "typical" interceptor capacity for combined sewers which is representative of sewers in the area. Separate storm sewers or natural conveyances would have both internal storage and "interceptor capacity" set at zero, since all storm runoff reaching such systems will "overflow". For combined sewers, net interceptor capcity is established as the difference between the hydraulic capacity of the sewer line and that part of the capacity utilized by dry weather flow.

To operate the simulator, each of the above parameters which are characteristic of the study area are incorporated into the program as constants. A record of hourly rainfall data is then read as program input. Output will consist of a tabulation of the runoff and overflow volumes to the receiving water for the period of record analyzed.

This tabulation may then be summarized, averaged or analyzed statistically to characterize the volumetric storm overflows.

Storm loads may be determined by assigning a pollutant concentration to the simulated volumes and calculating a load. In the absence of local data, relationships previously presented may be employed for estimates of typical concentrations.

Additional insight into the use of the simulator at any level of spatial detail will be provided by examination of the subsequent example application.

The example presented below illustrates a more detailed application of the simplified simulator. It describes the estimation of storm runoff loads from the Rochester, N.Y. urban area (29), which has both separate and combined sewer systems.

System schematic diagrams which show the overflows, drainage areas associated with the overflows and the pertinent interceptor capacities for the combined sewers are required to identify the characteristics of the sewer system and its existing overflow points. An essential first step in developing these data is to acquire the best and most recent sewer and storm drainage maps for the region under investigation.

Overflows are defined as any point on the collection and interceptor system specifically designed to permit excess flows to bypass the routing to the treatment plant. Some of the important characteristics of the overflows which should be identified in the system schematic are:

1. Location of the overflows on the interceptor system
2. The hydraulic capacity of the overflows and/or regulating structures that control the overflows
3. The capacity of any restrictions within the interceptor system that restrict flow to the overflow
4. The drainage area served by the overflow point.

Drainage areas or subareas are defined by delineating the sewered area that is tributary to a particular overflow structure (one overflow for each subarea). These drainage subareas subdivide the entire sewered area. The signficant characteristics of each drainage subarea are:

1. The total surface area
2. Percent of the subarea that is impervious
3. Percent distribution of the industrial, commercial, and residential (single-family and multifamily) and other significant land uses
4. Average slope of the ground
5. Average dry-weather flow.

The interceptor system described in the schematic should include the following information:

1. The components that connect each subarea to the treatment plant
2. The maximum capacity of these components

3. The capacity of components that are particularly restrictive in the system near an overflow
4. The available in-system storage.

The maximum capacities of the interceptor system are often calculated using Manning's equation assuming unsurcharged open channel flow. If the system can surcharge, significantly higher flow rates can occur and appropriate values for maximum interceptor capacity should be calculated.

In-system storage should be identified where it provides significant volumes in trunk lines or in interceptors. The effort required to define both existing and any unrealized potential for in-system storage is worthwhile, since in some cases storage volume in such existing lines may be increased dramatically by low cost modifications (e.g. weirs, dams), and provide a cost effective control technique.

An example of a system schematic prepared for the Rochester, New York study is shown by Figures 3-18, 3-19 and 3-20. These figures illustrate the sequential development and consolidation of pertinent data utilized in the operation of the simplified simulator.

On a map of the urban area, all significant overflows are located. Then, using sewer and storm drainage system maps, the drainage sub-catchment area which contributes flow to the system at each overflow point is delineated (Figure 3-18).

A schematic of the collection system is then prepared (Figure 3-19) which indicates clearly the routing, interconnections and other features of the system. The location of the input to the system from each of the sub-areas is shown. The hydraulic capacity of the lines between each of the sub-area inflow points and each overflow point are determined and recorded.

Figure 3-20 represents a final condensation of the salient features of the collection-overflow system. It summarizes and illustrates the physical and spatial characteristics of the drainage area which will be structured into the simplified simulator. It defines the routing of the

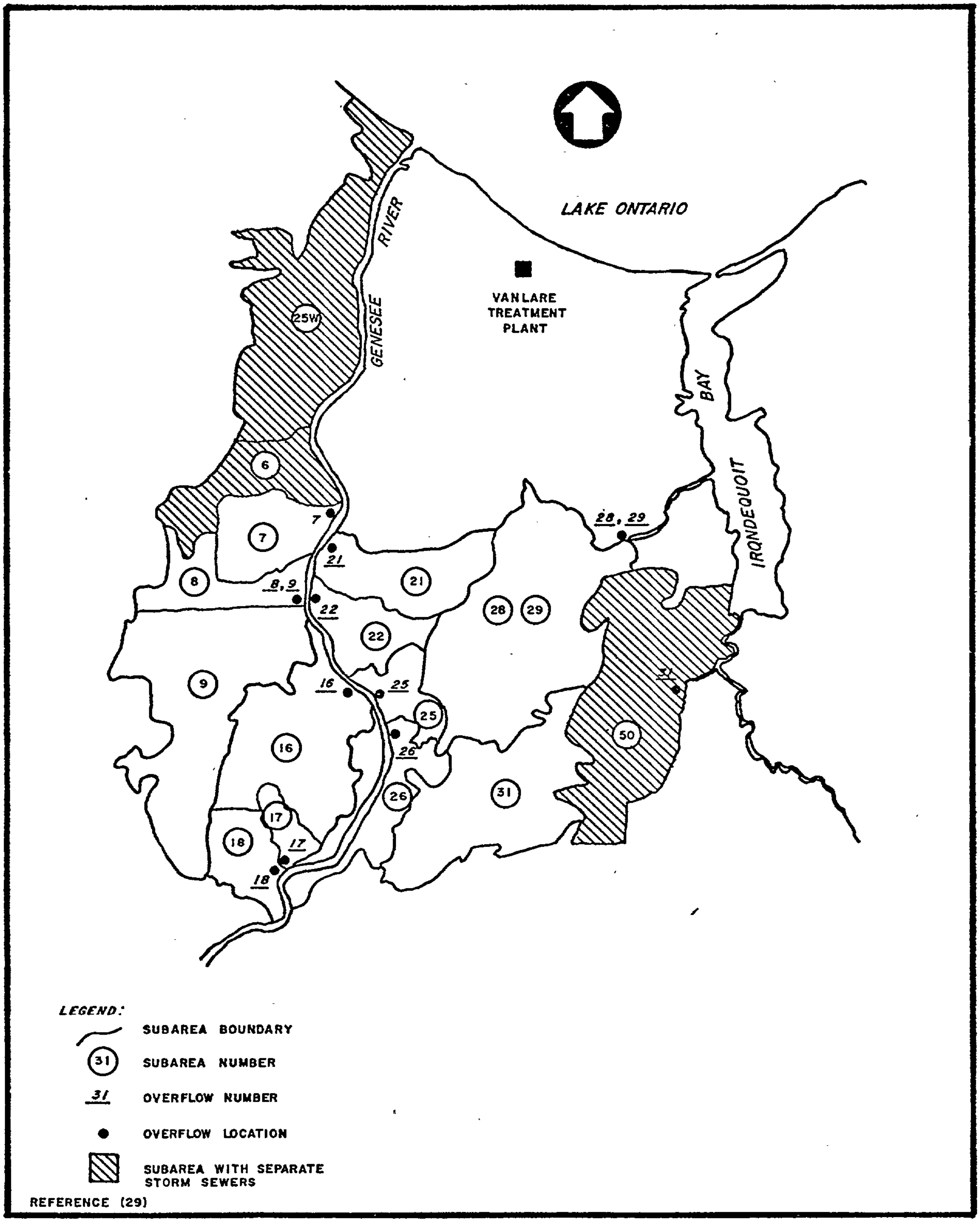


FIGURE 3-18

ROCHESTER SUB-DRAINAGE AREA AND OVERFLOW LOCATIONS

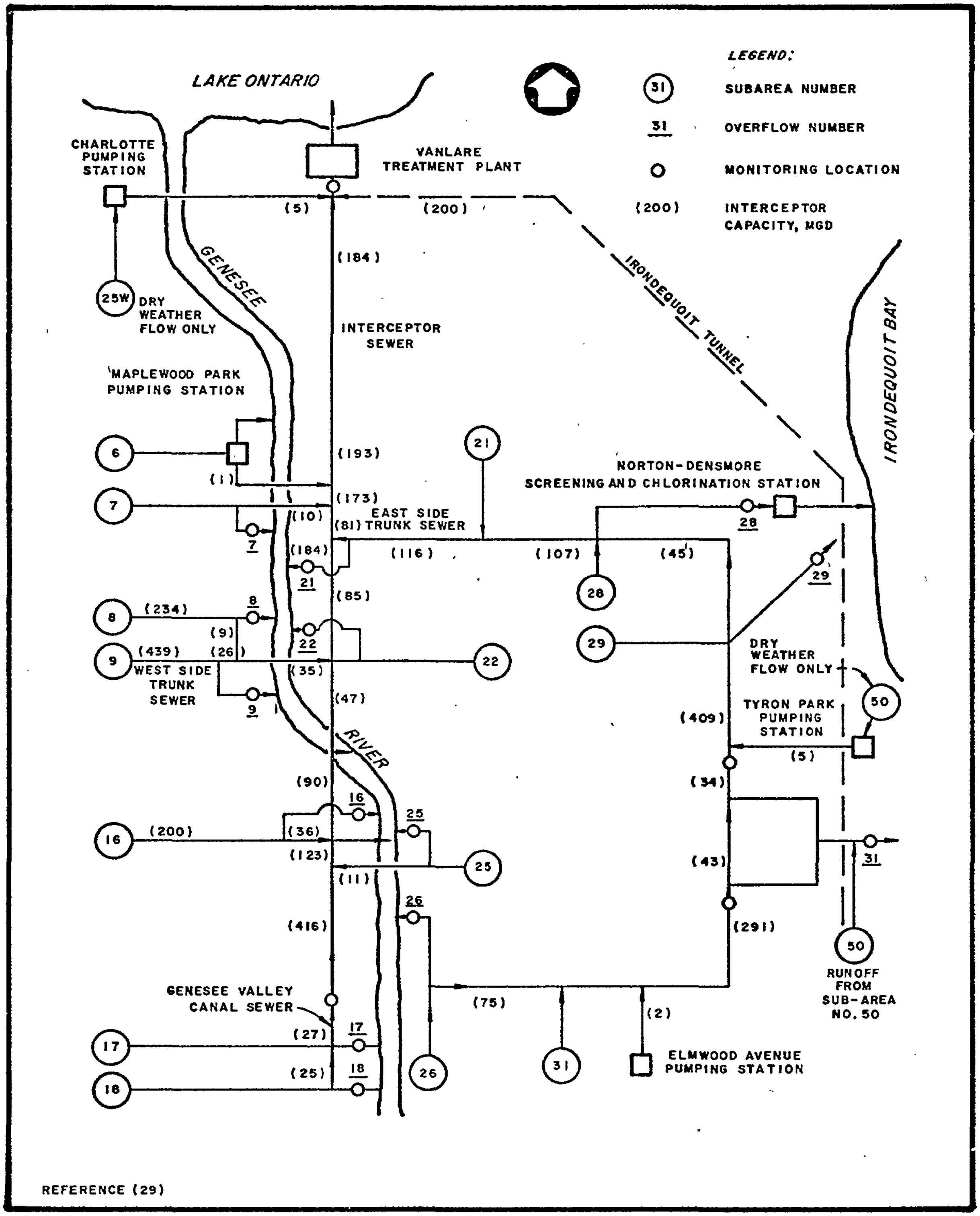


FIGURE 3-19
COLLECTION SYSTEM LAYOUT

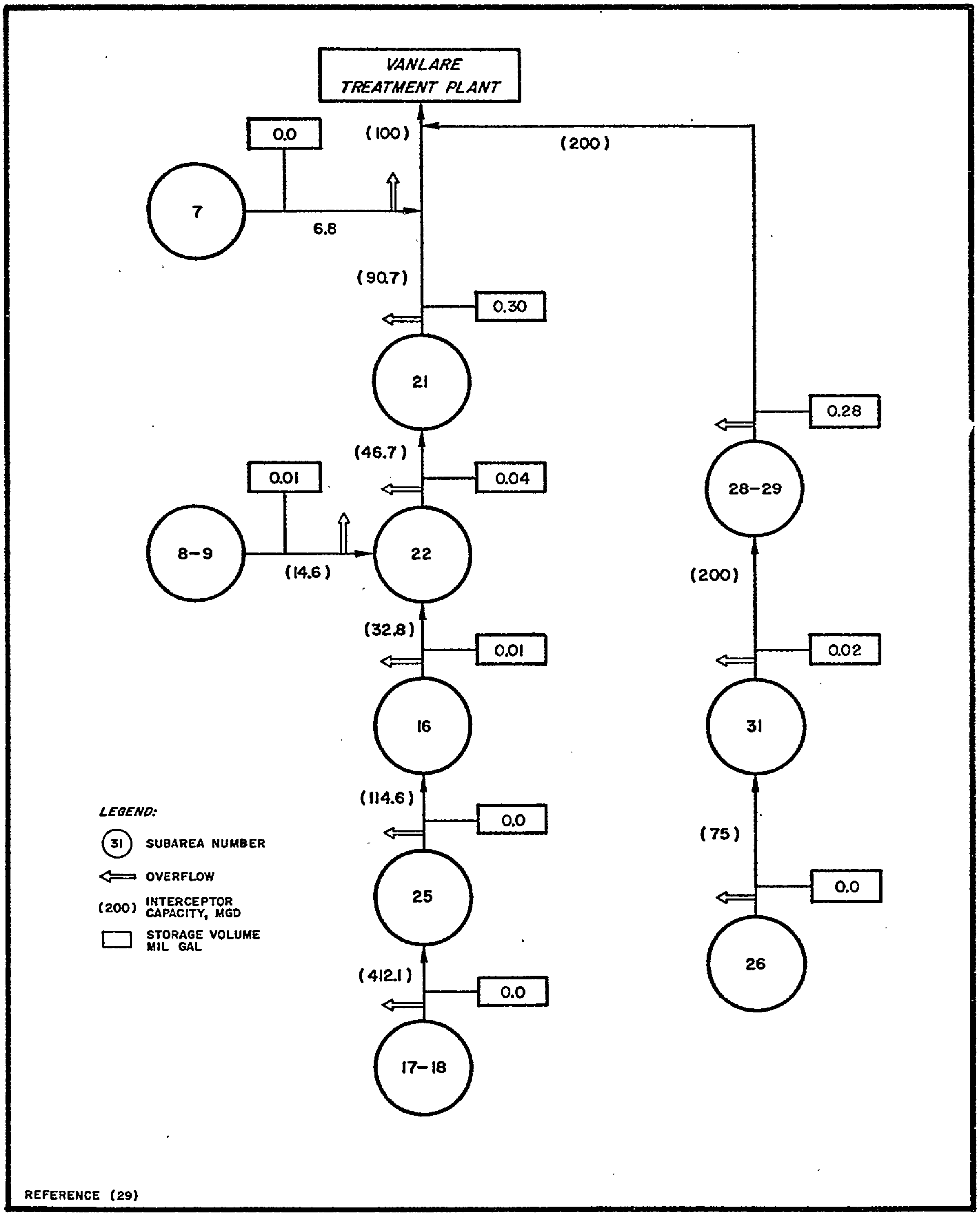


FIGURE 3-20
SUMMARY OF COLLECTION SYSTEM

storm flows and loads which will be generated for each of the indicated sub-areas by the load generating methodology employed in the simulator.

Storm loads are generated by the simplified simulator on the basis of rainfall and the characteristics of each of the sub-catchments. The significant features of the drainage area are determined, and summarized as illustrated in Table 3-7. Procedures for developing land use data bases are described in Appendix C. For the Rochester, New York study illustrated by this example, aerial photographs supported by field observations were used to define the distribution of the total area into the various land use categories. Area determinations were made by planimeter measurements from maps and photographs. Slopes were determined by a field survey, and the imperviousness was estimated on the basis of aerial photographs supported by field observations. The average dry weather sewage flow (DWF) in the combined sewers was estimated using the contributing population for each sub-area factored by a per-capita rate of flow.

Table 3-8 illustrates the procedure used to calculate the available wet weather capacity of the interceptor system. This is determined for each limiting segment downstream of an overflow point, as the difference between (a) hydraulic capacity of the interceptor which is based on diameter and slope, and (b) the cumulative dry weather sewage flow in the line at that point.

Figure 3-21 illustrates the results of a statistical analysis performed on the simulator output for long term rainfall records for the Rochester, New York example. Both runoff volume, and overflow from the combined sewer collection system were analyzed and the plot indicates graphically the estimated amount of total storm runoff retained by the existing system of interceptors. As a long term average, the data may be interpreted to indicate that approximately 70 percent of runoff is intercepted and contained by the existing system. For the larger storm events, which occur less frequently, this retention efficiency can be expected to be less than 50 percent. Waste loads would be estimated by assigning an appropriate concentration, obtained from Section 3.4.2.2, to these volumes.

TABLE 3-7

EXAMPLE OF DRAINAGE SUBAREA CHARACTERISTICS

Sub-area No.	Total area, acres	Land Use, %					Average slope, ft/ft	Impervious Area, %	DWF (maximum avg), MGD
		Residential							
		Single-family	Multi-family	Commercial	Industrial	Open			
6(a)	1,277	19.3	1.3	1.9	65.8	11.8	0.0074	55.0	7.06
7	715	83.9	1.0	7.3	0.2	5.5	0.0118	50.0	3.21
8	984	34.5	2.2	47.0	3.2	13.2	0.0066	45.0	6.36
9	2,603	52.5	0	4.1	37.1	6.4	0.0060	50.0	14.00
16	826	50.0	9.4	33.8	1.1	5.7	0.0070	55.0	5.78
17	235	83.8	3.8	2.1	0	10.2	0.0067	40.0	1.33
18	541	93.7	0.6	3.8	0	2.2	0.0073	40.0	2.60
21	821	79.4	0	9.0	6.8	4.9	0.0065	35.0	4.60
22	569	59.8	25.3	6.7	4.9	3.3	0.0070	50.0	3.41
25	348	30.0	9.9	44.9	5.0	10.2	0.0080	80.0	4.50
25W(a)	1,390	50.0	10.0	20.0	10.0	10.0	0.0150	35.0	6.01
26	554	30.0	9.9	44.9	5.2	9.9	0.0100	65.0	5.91
28	778	65.0	10.0	10.0	4.9	10.0	0.0100	50.0	4.36
29	1,430	65.0	10.0	10.0	5.0	10.0	0.0100	55.0	7.86
31	1,592	50.0	10.0	20.0	15.0	5.0	0.0100	47.0	10.13
50(a)	1,720	65.0	20.0	5.0	5.0	5.0	0.0150	40.0	11.90

(a) Serviced by separate storm sewers

Reference (29)

TABLE 3-8

EXAMPLE OF CALCULATION OF WET-WEATHER FLOW CAPACITY, MGD

Subarea Number	DWF[a] maximum average (1)	Sum of DWF (2)	Maximum interceptor capacity (3)	Available wet-weather capacity (4)
West Side System				
17 and 18	3.9	3.9	416	412.1
25	4.5	8.4	123	114.6
16	5.8	14.2	47	32.8
8 and 9	20.4	34.6	35	14.6[b]
22	3.4	38.0	84.7	46.7
21	44.7[c]	82.7	173.4	90.7
7	3.2	85.9	10.0	6.8[b]
6	7.0	92.9	184	100.0
East Side System				
26	5.9	5.9	....	....
31	22.0	27.9[d]	200	200
28 and 29	12.2	40.1[d]	200	200

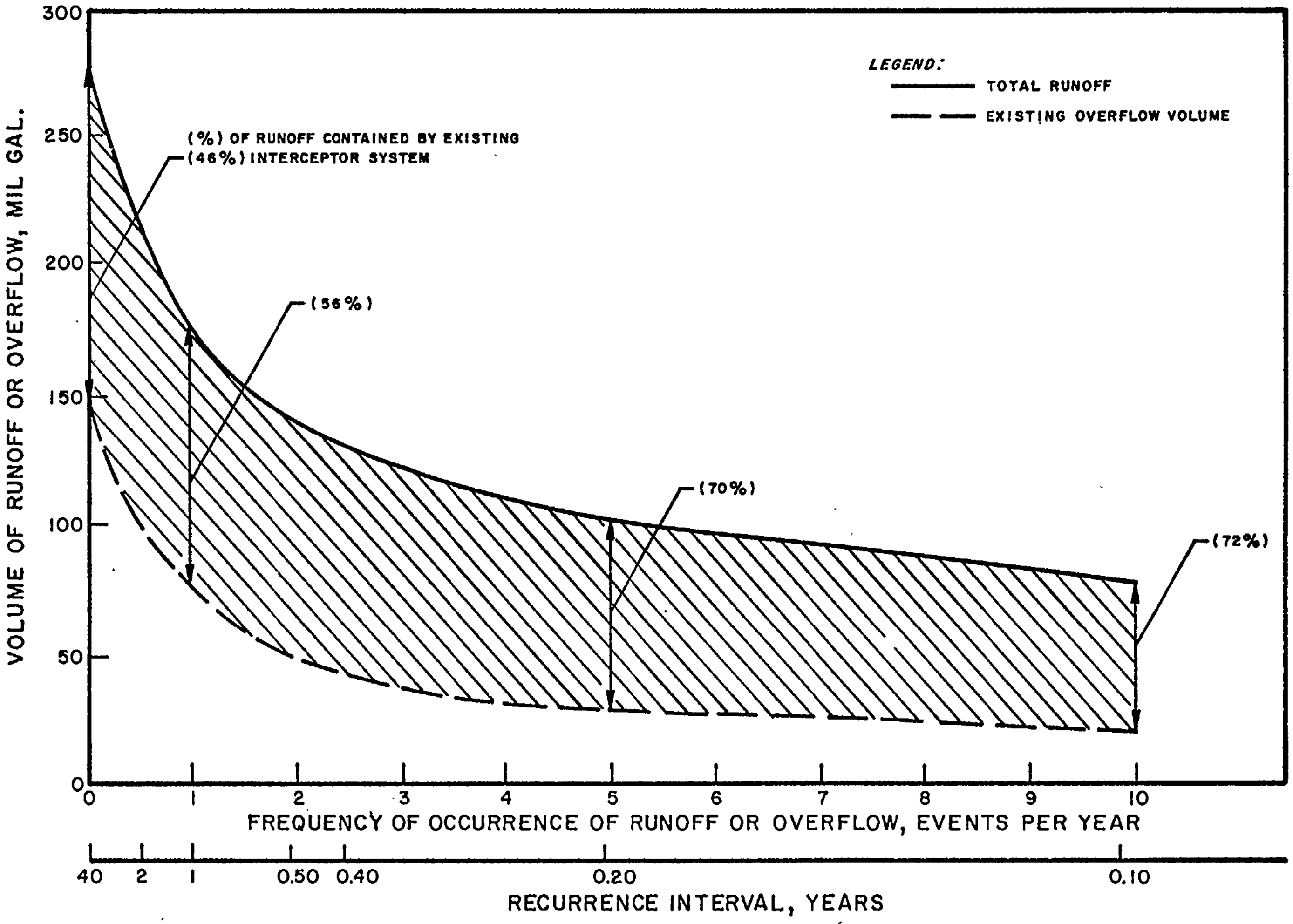
[a] DWF = (Average) Dry Weather Flow

[b] The limiting segment is not on the main interceptor

[c] Of this amount, 4.6 MGD is from Subarea 21; 40.1 MGD is from the East Side trunk sewer

[d] The equivalent of DWF is carried by the east side trunk sewer

Reference (29)



REFERENCE (29)

FIGURE 3-21
FREQUENCY OF OCCURRENCE OF RUNOFF AND OVERFLOW

3.4.5 Alternate Source Generation and Transport Prediction Methods

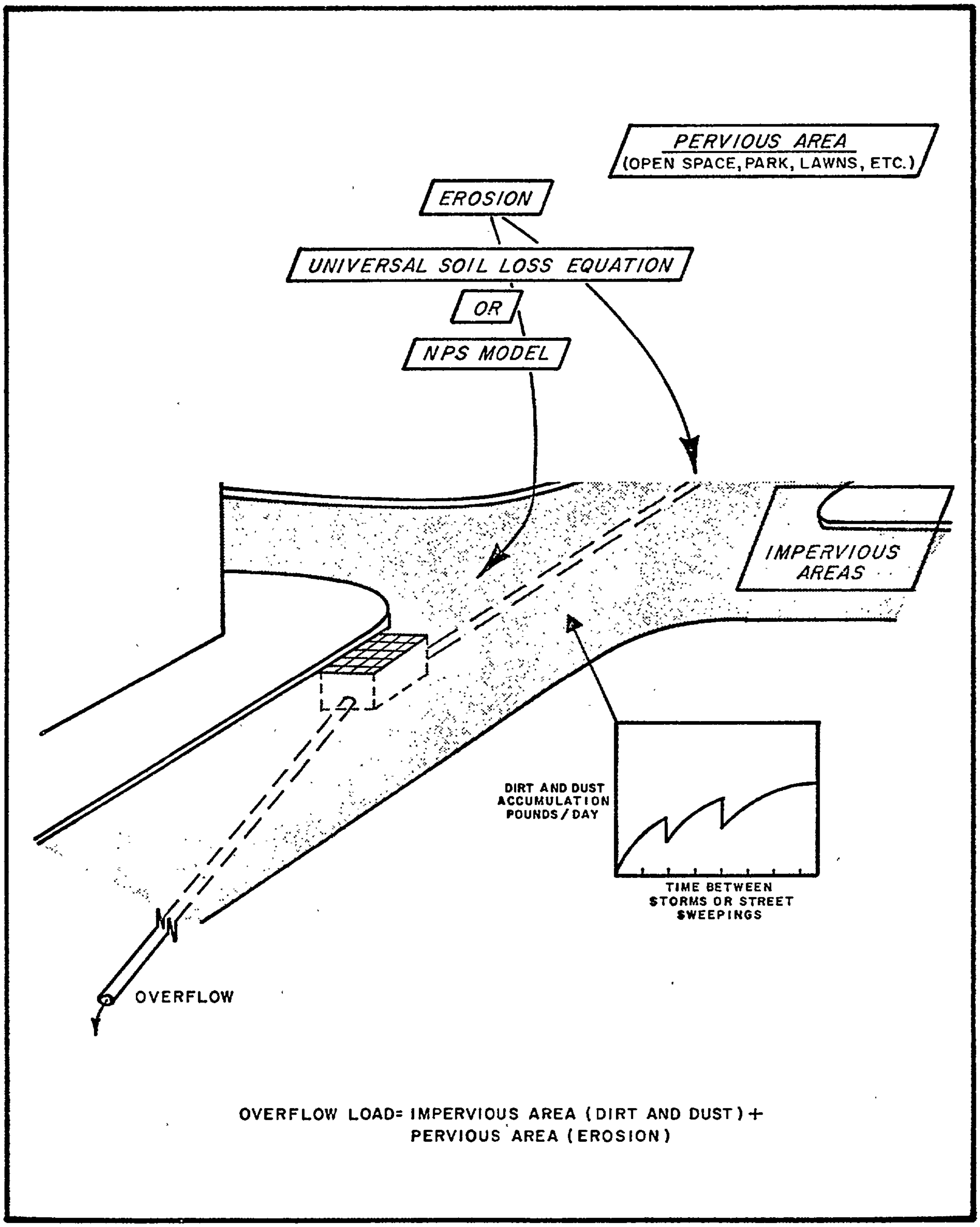
Each of the methods to estimate stormwater loads, discussed previously in this chapter, characterize the concentration of various pollutants in stormwater runoff. Estimates of concentration are empirical and are based on one or more of the following:

1. Collection system type (combined or separate)
2. Land use
3. Rainfall characterisitcs (intensity, duration, interval between storms).

Appropriate concentrations are assigned to runoff or overflow volumes for the calculation of storm loads. These stormwater load characterizations estimate "end-of-pipe" loads which will discharge either to receiving waters or to control devices. Chapter 5 describes procedures for estimating water quality impacts in receiving waters for the storm loads. Chapter 6 describes procedures for assessing the effect of various control measures on these loads. The characterization of end-of-pipe storm loads, developed by the methodologies described in this chapter, provide information in a form which can be utilized directly in these subsequent analyses.

Other techniques for estimating storm loads have been developed and are employed in some of the methodologies presented in Appendix A. Instead of utilizing empirically determined values or relationships for pollutant concentrations, descriptive models of the mechanisms by which pollutant loads are generated on land surfaces and transported to receiving waters are formulated and incorporated into the load estimating procedure.

Figure 3-22 illustrates the characterization of a drainage basin for use with models which employ source generation and transport. A drainage area is composed of pervious and impervious surfaces, and a distinct mechanism of pollutant generation and transport by storms is used for each land surface type. Sediment and sediment-like material is used as the indicator for pollutants because it is considered the major constituent of pollution from the land surface.



OVERFLOW LOAD= IMPERVIOUS AREA (DIRT AND DUST) +
PERVIOUS AREA (EROSION)

FIGURE 3-22

CHARACTERIZATION OF URBAN STORMWATER LOADS ACCORDING TO THEIR SOURCE (INSITU) GENERATION

It has been stated that the most important contributor of pollutants observed in urban runoff-overflows is the debris on the land surface (17,39). This occurs primarily as deposits in streets, gutters and other impervious areas draining to the street or storm sewers. Pollutants tend to accumulate on the land surface in many ways. Some of the most common accumulations occur as debris dropped or scattered by individuals, sidewalk sweepings, wastes and dirt from construction or renovation, remnants of household refuse dropped during collection or scattered by animals or winds, transportation residuals, and the fallout of particulate matter from the air. Regardless of the way in which pollutants tend to accumulate on the urban watershed, they can be generally classified into one of the following categories of street litter: rags, paper, dust and dirt, vegetation and inorganics. Based on street litter samples taken during a study in Chicago (17), the most significant category is dust and dirt except during the fall of the year when vegetation becomes the dominant component. It has been supposed that nearly all of the pollutants found in urban runoff can be associated with the dirt and dust component of street litter. However, the direct link between street dust and dirt accumulations and urban runoff quality is controversial (40). Competing contaminant sources contributing to runoff loads not accounted for by dirt and dust include:

1. Illicit and cross connections
2. Residuals scoured from pipe and channel networks
3. Neighborhood refuse and refuse management practices
4. Construction and erosion related activities
5. Air carried and deposited pollutants
6. "Natural" background loadings.

In addition, for combined sewer systems, storm overflows would carry contaminants contributed by raw sewage.

Significant monitoring efforts would be required to calibrate internal source generation and transport processes, so that they accurately represent local conditions. It has been suggested that 3 to 5 years of runoff data would be optimal in order to evaluate parameters under a variety of climatic, soil moisture, seasonal, and water quality conditions

(41). This is a significantly higher degree of effort than required in either the statistical method or the simplified stormwater simulator. Although estimates or default values are also available for use with these source generation techniques, they are much less readily checked and adjusted for local conditions by limited monitoring programs. In addition, there is a marked scarcity of data for use with these methods when compared with the amount of data available for the estimation of pollutant concentrations. For these reasons, methodologies which utilize source generation and transport mechanisms have not been included in the procedures recommended in this chapter for the estimation of urban storm loads. A suitable estimation of loads from non-urban areas, which provides a more detailed analysis than that outlined in Chapter 2, does require the use of these techniques. They are accordingly discussed further in Chapter 4 of this manual which addresses the estimation of non-urban loads.

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CHAPTER 4

ASSESSMENT OF NONURBAN, NONPOINT POLLUTANT SOURCES AND LOADINGS

4.1 Introduction

Areas requiring comprehensive planning for management of water quality almost always consist of a series of complex watersheds having a variety of land uses and hydrologic configurations. In many cases a relatively small percentage of the total land area is urban with the remaining nonurban areas a mixture of forested, agricultural, mining, or open lands. Assessment of nonpoint source pollutants (type and loadings) generated from urban areas was discussed in Chapter 3 of this manual and included consideration of both intermittent point sources (stormwater sewers) and diffuse sources (watershed drainage to surface waters). The following sections are intended to provide guidance in determining the type and magnitude of pollutants generated from various nonurban land uses.

Quantitative evaluation of the magnitude and impact of nonurban, nonpoint source pollutants is currently more art than science. Recently developed techniques, largely computer simulation models, describing watershed processes generating stream flow and associated water quality, are briefly described in Appendix A and will be slightly embellished in a later section but will not be discussed in detail. Application of any assessment methodology, simple or complex, should be attempted only with understanding of the important features of hydrology, soils, sediment transport, and land use. Equally important are the data bases required to evaluate the problem at various levels of resolution. The following sections will describe the general nature of nonpoint pollutant source, types, and loadings; and discuss the tools available to assess the magnitude and timing of loadings, and present in detail (with examples) a simplified approach for estimating loads based on the Universal Soil Loss Equation (1). Finally a concise summary of currently available specific information for each nonpoint source is presented.

4.2 General Characteristics of Nonpoint Source Loads

Essentially all nonurban, nonpoint source loads enter surface or groundwater through the overland or subsurface flow paths of the hydrologic cycle. Notable exceptions include man-made diversions and sewers for highway drainage or other specific hydraulic structures. NPS problems must be evaluated with these facts in mind. Stated simply, nonpoint source pollutants result from the interactions of the hydrologic cycle and land use. Land use and the associated environmental conditions determine the type, form, concentration, location, quantity, and time distribution of pollutants within a given watershed. These factors in turn determine the availability of each pollutant for transport to surface or groundwater via the hydrologic cycle. Finally, the energy and space-time distribution of each flow component (surface and subsurface) determine the amount of available pollutants reaching areas where water quality impacts are important.

Before numerical estimates of NPS loads are attempted or representative data presented, two qualitative relationships must be established: (1) the impact of land use on pollutant type and (2) the impact of the hydrologic cycle on pollutant transport.

4.2.1 Qualitative Relationship Between Land Use and Potential Pollutants

Land use can be conveniently defined at two levels for the purpose of relating man's activities to pollutants. The broad categories of agriculture, forests, mines, construction sites, waste disposal sites, and hydrologic modification areas define general land uses from which specific pollutants are emitted. Each of these land uses may have a wide array of specific activities of interest. For example, row-crop agriculture undergoes tillage, chemical application, harvest, and fallow periods during which specific pollutant loadings may occur.

The quality of water draining nonurban areas is also influenced by watershed properties that are independent of land use. The geological formations of an area influence the ionic constituents of both surface and groundwater. Similarly, untouched "wilderness" areas are subject to the same erosive and leaching processes acting on intensively managed areas. Usually, water coming from these areas is of high quality and need only be considered in estimating total loads to the system. Exceptions, no doubt, exist so hard and fast rules cannot be established.

4.2.1.1 Land Use Category - Pollutant Matrix

Most planning areas have ready access to broadly-defined land use data for initial analysis of NPS problems. For purposes of this manual, land use categories are defined as follows:

Construction--lands used for the construction of temporary or permanent facilities which are not directly linked to the watershed hydraulic network (see hydrologic modification).

Agriculture---lands used for production of crops or livestock in areas where water is supplied by rainfall.

Silviculture---lands used for production of timber or other forest products.

Residuals management---lands used for utilization or disposal of waste residuals from either public of private sources.

Hydrologic modification---lands used as sites for operations which modify the hydraulic network of the watershed.

Mining---lands used for the extraction of minerals from the earth and for on-site materials-handling roadway network.

In addition to these categories, others have been defined but have not been included here. The most notable is irrigated agriculture. The major problem associated with irrigated agriculture is quality of the return flow. However, in areas where pollution problems result from the practice of irrigated agriculture, other nonurban NPS problems are usually of little concern. Also, management options available for control of return flows are unlike those proposed for other sources.

The relationship of the above land use categories to potential pollutants is given in Table 4-1. The noted relationships do not imply that water quality problems automatically follow - it only shows those pollutants which have a known potential for becoming a water quality problem as a result of the land use.

TABLE 4-1

NONURBAN NONPOINT SOURCE POLLUTION MATRIX

	Pollutants						
Source	Sediment	Nutrients	Pesticides	Salinity	Organic matter	Micro-organisms	Trace metals
Construction	X	X	X		X	X	X
Agriculture	X	X	X	X	X	X	X
Silviculture	X	X	X		X		X
Residuals management	X	X			X	X	X
Hydrologic modification	X	X			X		X
Mining	X	X	X				X

4.2.1.2 Land Use Activity - Pollutant Matrix

Within each land use category, an array of human activity can be defined and presented as a matrix showing relationships between those activities and their potential for pollution. While a complete review of activity for each land use would be too lengthy for presentation here, it is useful to separate each land use category into the next level of resolution. Therefore, an activity-pollutant matrix for each category is provided for easy reference.

Construction: Usually, at any one time, only a small percentage of a watershed is experiencing construction activity. Because many construction activities are locally intensive, their sites relative to surface waters become very important, and the pollutant generating potential is best determined by site-specific analysis. If water quality impacts are to be either monitored or predicted, however, the scheduling of such activities must anticipate and reflect the short-term duration of active construction activities. For purposes of this manual, construction activities are elaborated as follows:

Clearing, grubbing, pest control---initial activities associated with site surveys, equipment and materials transport, removal of undesired vegetation, etc.

Rough grading---preparation of land surface for location and desired elevations of planned facilities.

Facility construction---actual construction.

Site restoration---final landscaping, clean-up, excess material removal, etc.

The relationship of these activities to potential pollutants is given in Table 4-2.

Agriculture: Agriculture is one of the two major land uses in the United States (forestry is the other). Nationally, over half of the total land area is classified as agricultural and is grossly divided into cropland, pastures, and open rangelands. Activity within this land use category includes the infinite array of operations performed during intense management of each agricultural enterprise. The major activities and associated pollutants resulting from crop and animal production are shown in Table 4-3.

TABLE 4-2

SUMMARY OF CONSTRUCTION ACTIVITIES AND ASSOCIATED POLLUTANTS

	Pollutant								
Activity	Sediment	Nutrients	Pesticides	Sanitary wastes	Petroleum products	Other chemicals	Trash	Cement	Metals, trace
Clearing, grubbing, pest control	X	X	X						X
Rough grading	X		X		X	X			
Facility construction			X	X	X	X	X	X	X
Site restoration	X	X							X

TABLE 4-3

SUMMARY OF AGRICULTURAL ACTIVITIES AND ASSOCIATED POLLUTANTS

	Pollutant					
Activity	Sediment	Nutrients	Pesticides	Organic material	Micro-organisms	Salts
Crop Production						
Seed bed preparation	X					
Chemical application	X	X	X			
Cultivation	X					
Harvesting	X			X		
Animal Production						
Concentrated feeding	X	X		X	X	X
Grazing - normal	X		X			
- overgrazing	X		X			
- along streams		X		X	X	

These activities remain quite broad and can be divided into considerably more operations. For example, there are a number of ways to prepare a seedbed which, in turn, may impact the quantity of pollutants available for movement by runoff or percolation. The kinds of pollutants should remain essentially the same, however. A complete discussion of various management systems within these categories is given in a recently published EPA-USDA report (2).

The common practice of land-spreading animal wastes has been omitted because it is included in the Residuals Management section.

Silviculture: Silviculture is defined as the cultivation of trees. For purposes of nonpoint source planning and control (and this manual), the definition is broadened to include all operations associated with the production, harvesting, and regeneration of timber. These operations are defined as follows:

Access---those activities required to access standing timber and transport harvested products (roads and trails).

Harvesting---those activities required to cut, transport, and collect logs for removal via the access system.

Reforestation---those activities required to prepare sites for reseeding or species conversion.

Intermediate growing practices---those activities required to control undesirable species, prevent fires, or otherwise promote growth.

Silvicultural operations are different from agricultural operations in two key ways: (1) rotations occur over 20 to 60 years, during which many of the above operations occur only for short time intervals, and (2) during any one time interval, only a portion of the total forested area is subject to the activities. These facts tend to mitigate nonpoint source pollutant loads, but significant problems may exist in some cases.

The relationship of silvicultural activities to potential pollutants is given in Table 4-4.

TABLE 4-4

SUMMARY OF SILVICULTURE ACTIVITIES AND ASSOCIATED POLLUTANTS

	Pollutants			
Activity	Sediments (Organic, inorganic)	Nutrients (Fertilizers, fire retardants)	Pesticide	Thermal pollution
Access	X			
Harvesting	X	X	X	X
Reforestation	X	X	X	
Intermediate Growing Practices		X	X	

Residuals Management: Residuals management practices are usually part of a complete waste management system. However, the primary concern of removing pollutants from waste streams often fosters neglect of the problems associated with disposal of the resulting residuals. Residuals include water and wastewater treatment sludges, septage effluent, municipal refuse, industrial wastewater treatment sludges, combustion and air pollution control residuals, dredge spoils, mining spoils, and animal wastes from confined feeding.

General statements about the nature and magnitude of the nonpoint source problems are difficult because the design and maintenance of each system varies significantly. For example, the practice of temporary dairy manure storage followed by land spreading on snow or frozen ground results in vastly different nonpoint loads than year-round spreading in the warmer areas of the country. Usually, however, two different problems arise - leaching of pollutants from buried or injected wastes and runoff or surface applied or incorporated wastes.

Residuals and their associated potential pollutants are given in Table 4-5.

Hydrologic Modifications: Hydrologic modifications in the truest sense would include all activities that alter the pathways of the hydrologic cycle. All construction activities and most other agricultural or silvicultural operations modify the hydrologic system in some way. For purposes of this manual, a more narrow definition is proposed: modifications occurring "in-stream" or "near-stream" such that there are direct links between the activity and water bodies. These activities include construction of dams and impoundments, channelization, dredging, and other in-water construction (bridges, docks, etc.).

The construction phases of hydrologic modifications result in essentially the same nonpoint source problems as other construction activities. Post-construction and maintenance may be more significant, however, because of the direct contact of the facility with the water body. For example, the impact of boat docking facility construction may be relatively short-term but subsequent waste oil, refuse, etc., from its use may be a continuous long-term source of pollutants.

TABLE 4-5

SUMMARY OF RESIDUAL WASTES AND ASSOCIATED POLLUTANTS

Activity	Nutrients	Organic material	Heavy metals	Micro-organisms	TDS	Suspended solids	Alkalinity	Acidity	Fly ash	Other	Odors
Wastewater sludge	X		X	X							X
Septage residual	X	X	X	X		X					
Water treatment sludge		X	X	X						X	
Municipal refuse		X	X	X	X		X	X			
Combustion and air pollution control residual							X		X	X	
Industrial waste sludge					X					X	
Feedlot manure	X	X	X	X		X					X
Mining waste								X		X	
Dredge soil		X				X				X	

The major hydrologic modification activities and their potential pollutants are given in Table 4-6.

Mining: Mining operations in certain regions of the country are the most significant watershed activity. The current and projected energy and mineral resource demands suggest a rapid growth of new mining and intensification of existing activities. The nonpoint source loads can be significant because of the dramatic change in the landscape and the characteristics of the newly exposed soils now subjected to erosion and leaching. Above and below ground mining result in somewhat different problems but both have certain activities in common that can be conveniently separated by their potential to yield nonpoint source pollutants. These activities are shown in Table 4-7.

Mine drainage is considered the most significant problem. In addition to sediments transported to streams by runoff, mineral constituents like acids, heavy metals, nutrients, and radionuclides have been measured in drainage water. Many of these pollutants result in acute toxicity problems for receiving waters as well as the common problems of nutrient enrichment, sedimentation, dissolved oxygen, etc.

4.2.2 Qualitative Relationship Between the Hydrologic Cycle and Pollutant Transport

The hydrologic cycle in large part determines the timing, volume, frequency, and quality of nonpoint source loadings. The land use activities described in previous sections determine the location and form of the various pollutants but any assessment or estimate of actual loadings must be made with proper recognition of the role of the watershed hydrologic response.

The watershed is best viewed as a system which yields outputs (including nonpoint source pollutants) in response to a series of inputs. Yevjevich (3) described this concept nicely when he wrote "Continental surfaces, underground acquifers, inland bodies of water, plants, and soils are environments with complex water inputs, environmental compositions, responses, and outputs. This environmental trinity, input-response-output, in combinations, mutual dependences, and feedbacks is defined as the hydrologic system." A systems description of agricultural watersheds is given by Stewart,

TABLE 4-6

SUMMARY OF HYDROLOGIC MODIFICATIONS AND ASSOCIATED POLLUTANTS

	Pollutant						
Activity	Sediment	Nutrients	Pesticides	Organic compounds	Trace metals	Thermal pollution	Other chemicals (silica, sulfide)
Channel modification	X					X	
Impoundments	X			X	X	X	X
Dredging	X	X		X	X		X
Maintenance facilities		X		X			

TABLE 4-7

SUMMARY OF MINING ACTIVITIES AND ASSOCIATED POLLUTANTS

	Pollutant							
Activity	Sediment	Nutrients (fertilizers)	Pesticides	TDS	Sanitary wastes	Heavy metals	Acid wastes	Radioactive materials
Exploration	X							
Access and support facility construction	X				X			
Mineral extraction	X			X	X	X	X	X
Mineral processing	X			X	X	X	X	X
Mine closure	X	X	X	X	X	X	X	X

et al. (4) which can be generalized to describe any nonurban system. Figure 4-1 demonstrates the idea.

The nature of the inputs and outputs of the system in Figure 4-1 have important characteristics that must be understood before a complete assessment of nonpoint loadings can be made. Indeed, successful NPS control can only be achieved by knowing where the system is amenable to treatment and the magnitude and frequency of specific system inputs, properties, and outputs. Successful monitoring to determine the magnitude of NPS problems or the effectiveness of in-place controls is also keyed to the factors shown in Figure 4-1.

Precipitation inputs drive the system and, in large part, determine the total volume and time-distribution of runoff. In addition to their uncontrollable nature, precipitation and solar radiation are stochastic and spatially variable. The impact of these features on calibrating certain rainfall-runoff models was briefly discussed in Chapter 3. Precipitation measurement via raingage networks is a well-established science and historical records are available for many areas of the country. Statistical and mathematical techniques referred to earlier in Chapter 3 are also available to areally distribute measured point-rainfall data (5, 6, 7, 8, 9). Significant quantities of pollutants can enter the watershed via solution-stripping by precipitation. Typical pollutant types and loadings are given in section 4.2.3.

System inputs classified in Figure 4-1 as controllable are those substances or activities introduced by man. The land use activities described in the previous sections summarize these inputs. Location of inputs are variable but can be part of the controls introduced for reduction of nonpoint source loads. Indeed, it is only through this set of inputs that water quality improvements can be made.

As watershed system inputs are transmuted to system outputs, the system properties described in Figure 4-1 modify their behavior. Some of these properties, like soil type and topography, are for the most part fixed in both time and space. Others, like vegetative cover and drainage networks, are subject to change by the set of controllable inputs. Techniques to measure

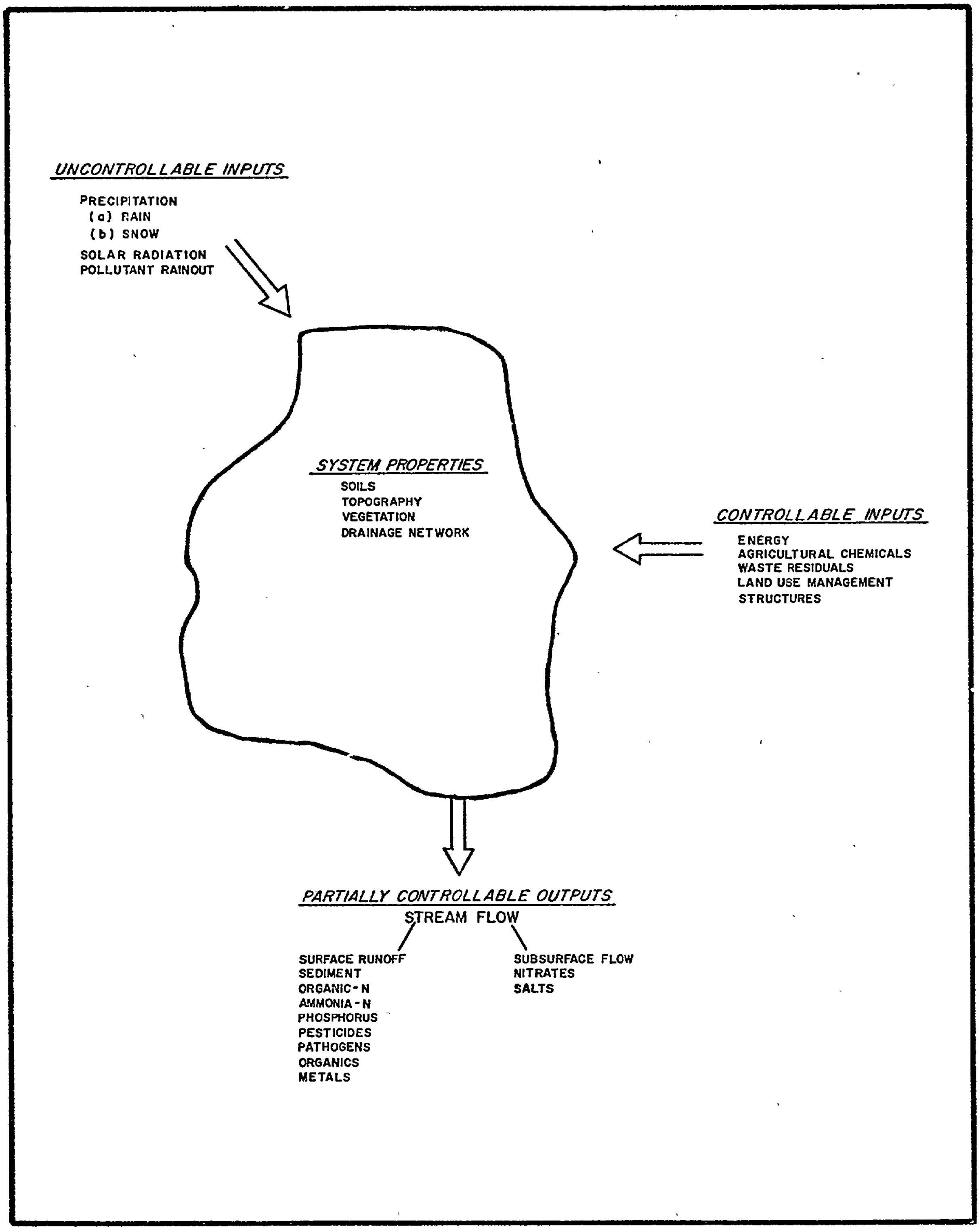


FIGURE 4-1
WATERSHED SYSTEM NON-POINT SOURCE POLLUTANT
LOAD RESPONSE TO HYDROLOGY AND LAND USE

and specify these properties include interpretation of aerial photos, soils maps, topographical maps, and other data bases as described in Appendix C.

System outputs are described as partially controllable. This is an important point to consider when a control program is contemplated. Quantification of the degree of controlability possible is no trivial task and is directly related to the uncontrollable and stochastic nature of the system inputs. Obviously, an absolute standard or goal is impossible to achieve without violations for certain time periods, however small.

Another important feature of the system outputs of Figure 4-1 is the division between surface runoff and subsurface flow. The relative distribution of these flow components varies as a function of surface conditions, watershed size, and geological formations. In general, as a watershed increases in size, a greater proportion of the streamflow is determined by subsurface sources. Estimates of relative magnitudes are important to the correct interpretation of measured water quality data and the allocation of measured loads to their respective sources. Some of the NPS models described in section 4.3.4 are capable of predicting this relative distribution (10, 11). Other empirical hydrograph analysis techniques for this purpose are also available as described by Chow (12).

Resolution of watershed drainage into surface runoff or subsurface flow is important because most pollutants are transported in much greater quantities in one component of flow than in the other. The outputs shown in Figure 4-1 classify the major pollutants by their major modes of transport. There are, as always, exceptions to these rules as in those areas where extremely permeable (e.g., sandy) soil profiles exist or where large areas are impermeable. NPS controls also must be planned in recognition of flow distribution because many candidate practices (e.g., soil conservation practices) result in a shift in the relative distribution of flows and, subsequently, a new set of NPS pollutant loads must be analyzed. Interaction of surface and subsurface processes is a major consideration in the decision-tree analysis for selection of agricultural nonpoint source controls developed by Stewart et al. (2)

The impact of the hydrologic system is important in any attempt to measure the nature and extent of NPS loads through field sampling. Intensive, continuous sampling over short periods of time may measure little of the total extent of the problem. Runoff itself is stochastic as is the time between runoff events which, in turn, influences the quantity of pollutants available for transport. Of equal concern are the limitations inherent in grab sampling over longer periods in that peak loads may be missed entirely. A detailed presentation of monitoring methods and procedures is given in Appendix D of this manual.

4.2.3 Representative Nonurban Nonpoint Source Loading Data

Data from studies which attempt to measure nonpoint source loads vary over several orders of magnitude. This is not at all surprising when viewed in light of the possible variations in land use activities and the features of the hydrologic system acting on these activities. Indeed, the wide variation in data alone should serve as caution against relying heavily on extrapolation for assessment decisions.

Generally, there are three types of data bases available. They are (1) lysimeter or soil column studies, (2) small plot or individual field scale studies, and (3) drainage basin studies. Interpretation of data from these studies should be modified by the types of land use they represent, their location relative to the watershed system depicted in Figure 4-1, and the time period over which they were developed.

One other nonpoint source load not included in any of these is precipitation. In areas where surface waters constitute large areas, pollutant inputs via precipitation can account for a significant portion of the total load. Typical precipitation loading to land areas is given in Table 4-8.

Each of the three study types represents different components of the watershed system. Soil column or lysimeter studies represent only vertical movement of pollutants and effects of surface runoff, groundwater (shallow or deep) flow, or interflow (subsurface flow returns to surface runoff) are not included. Small plot or field scale studies represent direct surface runoff of pollutants and effects of groundwater flow, vertical movement, down-slope deposition, and different land uses, are not included. Portions of the

TABLE 4-8

NONPOINT SOURCE POLLUTANT LOADINGS FROM PRECIPITATION[a]

Type of Loading	Pollutant Range		
	Total Nitrogen	Total Phosphorus	Acids, pH
Areal loading lb/ac/yr	4.4-8.9	0.045-0.055	-
Concentration mg/l	0.1-12.8	0.005-0.10	4.3-5.6

[a] Source: (15, 24-29)

interflow effect may be observed. Drainage area studies include the effects of all flow components and different land uses. Drainage studies, depending upon the size of the basin they represent, may also reflect the effects of stream assimilation capacity. Also, larger drainage basins may be subject to groundwater export to or import from adjacent basins, making NPS pollutant mass balance calculations difficult.

Results of typical soil column studies are given in Table 4-9. Extrapolation of these data to NPS loads would result in overestimation of groundwater loads, especially for nitrogen. Such studies are very helpful, however, when investigating the impact of waste residuals, spoil materials, chemicals, etc., on the soil-plant-water complex. For example, the potential problems of heavy metal or pathogen leaching from areas (where runoff is controlled) on which municipal or industrial sludges are spread can be evaluated by analysis of similar data.

Small plot and field scale (small watershed) studies dominate the literature available on nonurban, nonpoint source loading. Such studies are very useful because they represent the relative impact of different land uses and land use activities (including management practices recommended for controls), and because they provide an estimate of direct surface runoff water quality. Generally, the larger the area included in the study, the more realistic the extrapolation of the data because more components of the hydrologic system are included.

Data from typical studies representing various land uses and pollutants are included in Tables 4-10 through 4-13. Comparison of the areal contribution data from these studies with similar data from the soil column studies illustrate the moderation provided in small watershed studies by the increased geographical scale and the inclusion of more watershed processes over those provided by soil column studies.

Data collected during drainage basin studies are usually considered to be most meaningful in evaluating the water quality impact of nonpoint source loads. Two conditions are necessary to make such studies suitable for water quality impact assessment. First, the measured water quality must be determined primarily by processes occurring in or on the land surface and

TABLE 4-9

AGRICULTURAL NONPOINT SOURCE LOADING FROM SOIL COLUMN STUDIES[a]

	Nitrogen Yield, lb/ac/yr			Phosphorus Yield, lb/ac/yr		
	Number[b]	Mean	Range	Number[b]	Mean	Range
Total	15	25	0.3-98	9	1.04	0.05-6.9
Inorganic	28	17	0.3-73	6	0.47	0.01-2.2

[a] Extracted from Chapter 2, Table 2-12

[b] Number of studies.

TABLE 4-10

AGRICULTURAL NONPOINT SOURCE LOADING DATA FROM SMALL PLOTS (0.02-0.80 ACRES)[a]

Crop	Management System	Pollutant Loss, lb/ac/yr		
		NO_3^--N	NH_4^+-N	Inorganic P
Corn	Return residue & rye as cover crop	1.25	0.29	0.115
	Return residue & rye as cover crop	0.35	0.12	0.04
	Residue burned, no cover crop	2.19	0.88	0.436
	Residue burned, no cover crop	0.35	0.18	0.14
	No-till	10.22[b]		1.44
	Conventional	9.64[b]		0.20
	Continuous - silage	7.51[b]		0.48
	Continuous corn silage cover crop	8.37[b]		0.53
Beans	Return residue	1.30	0.36	0.16
	Residue removed	26.0	0.44	0.33
Soybeans	Continuous field cultivator	7.81[b]		0.44
	No-till	4.84[b]		1.84
Wheat	Return residue plus rye grass & alfalfa cover crop	0.83	0.37	0.15
	Return residue plus rye grass & alfalfa cover crop	0.44	1.15	0.18
	Residue burned, no cover crop	1.01	0.32	0.28
	Residue burned, no cover crop	0.53	0.13	0.07
Meadow		1.40[b]		0.43

[a] Source: (27,30)

[b] Contains NO_3^- + NH_4^+-N.

TABLE 4-11

TYPICAL PESTICIDE LOADINGS MEASURED ON SMALL PLOTS (44-5700 FT^2)[a]

Pesticide	Amount applied (lb/ac)	Type of application[b]	Crop	Pesticide loss in runoff (lb/ac)	Range of pesticide loss in runoff increments
Aldrin	1.3	SR	Cultivated	0.068	
Atrazine	3.0	Inc. SR	Fallow	0.0741 sediment 0.278 water	5-138 μg/g 500-11,000 μg/l
	1.5	Inc. SR	Fallow	0.031 sediment 0.111 water	4-15 μg/g 50-600 ppb
	2.7	SR	Fallow	0.176	100-10,340 μg/l water
	2.0	S	Corn	0.1	100-200 μg/l 0.5-10 μg/g
	4.0	S	Corn	0.19	100-3800 μg/l 0.5-4 mg/g
	2.0	SR	Corn	0.05	50-2000 μg/l
Dicamba	0.18-1.09	SR	Fallow Sod	0.013	0-4800 μg/l
Dichlobenil	6.0	Inc. SR	Fallow	0.117 sediment 0.270 water	4-37 μg/g 100-900 μg/l
Dieldrin	1.3	SR	Cultivated	0.061	1.6-14 μg/g sediment
Diuron	0.75	S Ponded	Cotton	0.0004	1-4 μg/l
2,4-D-Amine	2.0	SR	Cultivated	0.047	
2,4-D-Butylether	2.0	SR	Cultivated	0.7	640 μg/l
2,4-D-Isooctyl	2.0	SR	Cultivated	0.8	1380 μg/l
Endosulfan	0.9	S	Cont. Potatoes	0.003	1.0-19 μg/l
	0.9	S	Rot. Potatoes	0.002	Trace-18 μg/l
	0.65	S	Oats	0.00007	Trace-3 μg/l
Endrin	1.3	S	Cont. Potatoes	0.012	1.0-49 μg/l
	1.3	S	Rot. Potatoes	0.008	Trace-48 μg/l
	0.27	S	Sugarcane	0.003	<0.01-2.07 μg/l
	0.36	S	Sugarcane	0.001	0.15-5.0 μg/l
Fenac	3.0	S	Sugarcane	0.086	1-310 μg/l
GS 14254	2.0	S	Alfalfa	0.0004	100-3800 μg/l 0.5-10 μg/g
	4.0	S	Alfalfa	0.0012	100-2000 μg/l 0.75-10 μg/l
Linuron	2.0	S Ponded	Cotton	0.0006	2-124 μg/l
Methoxychlor	22.0	SR	Grass	0.09	0.1-8.8 μg/l
Picloram	0.5	F	Grass	-	349-838 ppb
	0.25	S	Range	-	17 ppb
	0.9-1.8	SR	Fallow Sod	0.053	15-560 μg/l
Prometryne	2.5	S	Cotton	0.013	
Toxaphene	24.6	F	Cotton	0.089	~60 μg/l
Trifluralin	1.25	Inc.	Cotton & Soybeans	0.0005	0.2-1.9 μg/l
2,4,5-T	0.5	F	Grass	-	495-769 ppb
	10.0	SR	Grass	0.005	1-380 μg/l
	0.9-1.8	SR	Fallow Sod	0.03	7-3300 μg/l

[a] Source: (31)

[b] S=Surface; Inc.=Incorporated; F=Foliar; SR=Simulated Rainfall

TABLE 4-12

AGRICULTURAL NONPOINT SOURCE LOADING DATA FROM SMALL WATERSHEDS (0.7-150 ACRES)[a]

		Pollutant Loss, lb/ac/yr					
Crop	Management System	NO_3-N	NH_4-N	Kjeldahl N	P[b]	Suspended Solids	COD
Corn	Contour	1.29	0.85	2.96	0.26		
	Contour	0.47	0.31	22.46	0.52		
	Contour	0.84	1.30	41.35	1.15		
	Contour[c]	2.05	1.38	5.23	0.442		
	Contour[e]	1.30	0.37	31.02	0.923		
	Contour[c]	1.17	1.88	61.66	1.900		
	Terraced[c]	0.21	0.11	0.28	0.08		
	Terraced[c]	0.13	0.03	0.46	0.018		
	Terraced[c]	0.14	0.52	6.28	0.257		
	Corn & Oats rotated	0.33	-	0.81	0.27	255	43
Brome Grass	Rotation/-grazing	1.02	0.59	0.46	0.224		
	Rotation/-grazing	0.15	0.08	0.19	0.072		
	Rotation/-grazing	0.84	0.38	2.58	0.456		
	Hay/grazing	0.21	-	0.65	0.09	3.6	12
Pasture	Grazing	0.36	-	1.00	0.22	10.5	25

[a] Source: (32,33)

[b] Total loss values represent the inorganic P of the solution and the $NaHCO_3$-extractable P of the sediment.

[c] 2.5 times recommended rate applied.

TABLE 4-13

TYPICAL PESTICIDE LOADINGS MEASURED ON SMALL WATERSHEDS[a]

Pesticide	Amount applied (lb/ac)	Type of application[b]	Crop	Plot size, ac	Pesticide loss in runoff (lb/ac)	Range of pesticide loss in runoff increments
Atrazine	3.0	S	Corn	1.7-3.8	0.48	1.77-735 µg/g sediment
Dieldrin	5.0	Inc.	Primarily Corn	1.7	0.00035 water 0.11 sediment	1.9-20 µg/l water 1.6-14 µg/g sediment
	5.0	Inc.	Primarily Corn	2.7	0.035	0.4-4.1 µg/l
Picloram	2.5	F	Grass	3.0	0.00005	7-12 ppb
Propachlor	6.0	S	(Corn Surface)	1.7-3.8	0.138	117-491 µg/l water
Toxaphene	9.0	F	Cont. Cotton	38.5	0.0864	<10-28 µg/l
Trifluralin	0.98	Inc.	Cont. Cotton	38.5	0.00176	
2,4,5-T	2.5	F	Grass	3.0	0.0005	7-26 ppb

[a] Source: (31,34)

[b] S=Surface; Inc. = Incorporated; F = Foliar

not in the stream. In other words, the data should reflect pollutant loadings only, rather than a combination of loading <u>and</u> stream processes. (If all pollutants were conservative this would not be a problem.) Second, the measured output should be from a "hydrologically closed" watershed. That is, interbasin transfer of water (and pollutants) should be minimum or at least measurable.

Ideally, a number of basins having only one land use should be studied. However, basin studies tend to lump the effects of land use. A recent EPA study (13) analyzed a number of drainage basins and attempted to correlate general land use to measured water quality. Results from this and other studies are summarized in Tables 4-14 and 4-15.

4.3 General Characteristics of Nonpoint Source Load Estimation Methods

The previous section discussed the various kinds of loading data available and evaluated their usefulness in assessment studies. The importance of data interpretation within the hydrologic system framework was stressed. For assessment studies there are various approaches available for estimating nonurban, nonpoint source loads. The evaluation of these techniques must also be made within the hydrologic system framework and associated land use configurations. The following section includes a general discussion of the basic properties common to most loading methods (models), followed by a more detailed description of key models now available for application to assessment studies.

Mathematical modeling of complex phenomena is a rapidly growing science for which few widely recognized standards or definitions exist. At the risk of violating the sensibilities of a few modeling practitioners (perhaps even more than a few), the following definitions are offered:

Empirical methods---calculation procedures based on analysis of data or a certain known relationship among variables.

Deterministic methods---models based on a rigorous representation of known relationships (physical or mathematical).

Stochastic methods---models based on the concepts of probability theory and the idea that future events are determined by random processes.

TABLE 4-14

GEOLOGIC CLASSIFICATION AND MEAN VALUES FOR STREAM NUTRIENT CONCENTRATIONS AND EXPORTS FROM 223 SUBDRAINAGE AREAS IN THE EASTERN UNITED STATES[a]

Land use	Geologic classification and grouping code(s)	Number of subdrainage areas	Concentrations, mg/l				Export, kg/km^2/yr			
			T-P	O-P	T-N	I-N	T-P	O-P	T-N	I-N
Forest		53								
	Sedimentary; some or all limestone (10)	19	0.011	0.006	0.860	0.287	6.4	3.6	498.7	159.6
	Sedimentary; without limestone (20)	11	0.014	0.007	0.766	0.337	9.0	4.5	467.6	192.2
	Sedimentary; all (10, 20)	30	0.012	0.006	0.825	0.306	7.4	3.9	487.3	171.5
	Predominantly sedimentary (10, 14, 20)	31	0.012	0.006	0.818	0.302	7.3	3.9	482.3	169.1
	Igneous; volcanic origin (30)	0	-	-	-	-	-	-	-	-
	Metamorphic (40)	16	0.017	0.007	0.520	0.103	10.3	4.6	337.4	65.2
	Igneous; plutonic origin (50)	0	-	-	-	-	-	-	-	-
	Igneous and metamorphic (40, 45)	18	0.017	0.007	0.533	0.119	10.3	4.6	342.1	74.6
	Predominantly igneous and metamorphic (40, 41, 42, 45)	22	0.016	0.007	0.625	0.135	9.7	4.3	380.7	80.7
Mostly Forest		170								
	Sedimentary; some or all limestone (10)	55	0.037	0.015	1.056	0.488	16.3	6.3	472.1	233.2
	Sedimentary; without limestone (20)	48	0.035	0.014	0.817	0.288	18.0	6.9	441.8	161.2
	Sedimentary; all (10, 20)	103	0.036	0.014	0.945	0.395	17.1	6.6	458.0	194.3
	Predominantly sedimentary (10, 14, 20, 23, 24, 25)	118	0.036	0.014	0.930	0.374	17.1	6.5	456.5	186.7
	Igneous; volcanic origin (30)	0	-	-	-	-	-	-	-	-
	Igneous; volcanic origin (Present but not dominant) (23, 43)	4	0.038	0.018	0.975	0.328	13.1	6.2	332.2	115.5
	Metamorphic (40)	32	0.035	0.014	0.762	0.277	20.7	8.2	452.0	166.0
	Igneous; plutonic origin (50)	1	0.026	0.010	0.951	0.138	7.4	2.8	269.5	39.1
	Predominantly igneous; plutonic origin (50, 52, 54)	6	0.032	0.013	1.049	0.317	13.6	9.1	476.2	134.6
	Igneous and metamorphic (40, 43, 45, 50, 54)	40	0.036	0.014	0.798	0.269	19.2	8.2	427.7	149.8
	Predominantly igneous and metamorphic (40, 41, 42, 43, 45, 50, 52, 54)	52	0.035	0.014	0.827	0.284	18.2	8.1	433.1	152.3
Agriculture		91								
	Sedimentary; some or all limestone (10)	80	0.136	0.059	4.315	3.296	30.5	12.4	996.8	748.3
	Sedimentary; without limestone (20)	11	0.123	0.055	3.497	2.335	23.6	10.3	865.4	660.1
	Sedimentary; all (10, 20)	91	0.135	0.058	4.225	3.190	29.7	12.2	982.3	738.6

Abbreviations: T-P = Total Phosphorus; O-P = Orthophosphorus; T-N = Total Nitrogen; I-N = Inorganic Nitrogen

[a] Source: (13)

TABLE 4-15

RUNOFF AREAL LOADING RATE - POUNDS/SQUARE MILE/DAY[a]
(Average Range)

Land Use	Total Nitrogen	Total Phosphorus	BOD_5	TSS
Agriculture	15 (1.9-58)	1.0 (0.05-3.9)	40 (6.3-57)	2500 (449-6594)
Forest	4 (1.3-16)	0.25 (0.01-1.4)	8 (6.3-11)	400 (71-620)
Pasture	8 (3.9-13.3)	0.5 (0.4-1.0)	17 (9.4-27)	670 (19-1320)
Feedlots	1700 (1080-2290)	370 (200-610)	-	-

[a] Extracted from Chapter 2, Table 2-13

Simulation methods---models containing components of each of the above methods which attempt to simulate the behavior of processes known to influence the variable of interest.

Regression equations like the Universal Soil Loss Equation and the urban runoff equations used in Chapter 3 are examples of empirical approaches. The data bases upon which empirical models are built determine their ability to satisfy the needs of any given task. Use of such methods in solving problems outside the range of the original data base is risky and should be done only with full recognition of the possible errors.

Deterministic models are most elegant in their treatment of any problem. However, for NPS load assessment studies, the current lack of understanding of nonurban watershed dynamics and the apparent inability to measure all the necessary parameters make such models almost impossible to use. Some associated specific problems can be solved in this manner. Pipe and rigid-boundary, open-channel flow are amenable to deterministic modeling.

Stochastic models are becoming popular tools in water resource problem-solving (14). These methods require large amounts (in space and time) of data generally not available for water quality assessment studies. But when prediction of precipitation and streamflow is needed, and data for long periods of record are available, such techniques can be effectively applied.

Simulation models are especially attractive for use in nonpoint source assessment because they permit application of the state-of-the-art for each process of interest. For example, erosion modeling is still largely an empirical science, while overland flow can be treated deterministically. Simulation can combine both approaches to estimate sediment transport. Another key feature of simulation models is their ability to predict system responses from system changes - an obvious need when evaluating alternative future policies.

Application of any of the above models to estimation of NPS loads requires calibration and testing (some prefer the term verification). For some empirical models, like regression equations, both are trivial tasks because the exact form of the model is determined by the available data. The test for "accuracy" is inherent and is reflected in the statistical measures of correlation. Other empirical models like the urban loading method presented in Chapter 3 are calibrated by applying the model to the measured data and calculating the parameters and coefficients which appear as unknowns in the equations. Empirical models will yield better results when tested against data from areas having the same properties as those associated with the calibration data set. Indeed, the major weakness of empirical approaches is their inability to accomodate changes in the watershed. Calibration of deterministic models consists of estimating the physical constants applicable to the system under study. Testing is inherent because deterministic formulations are developed from well-understood, thoroughly-tested theory. Simulation models require calibration through trial and error or least-squares fitting procedures. The deterministic features of simulation models require inputs that are independent of time-varying, measured data (e.g., flow, water quality). Testing is normally accomplished by split-sample procedures. That is, part of the data set is used for calibration to adjust model parameters and the remaining data are simulated to determine how well the model predicts "future" loadings.

Nonurban, nonpoint source models can also be evaluated by comparing their properties to the watershed-hydrologic system described in section 4.2.2. This can be done for all models regardless of their classification among empirical, deterministic, stochastic, or simulation. Three fundamental properties can be listed: (1) spacial resolution, (2) temporal resolution, and (3) transport assumptions. Any NPS loading method or model can be evaluated by separation according to these properties. The resulting information is useful in determining the appropriate model for application to any given problem. The following sections describe these properties.

4.3.1 Spatial Resolution

Data are used during the development of most loading models and the resulting model spatial scale is directly related to the spatial scale of the data sources. Spatial scales for NPS assessment models are almost the same as those for the data described in section 4.2.3 with one exception. Models for conditions at a point, corresponding to the soil column studies, have limited application to NPS loading estimation and are not included here. Such models may be useful in studying leaching problems from residuals disposal areas or for investigating the impact of certain practices on the soil-plant-water complex but these are not normally a part of a general NPS assessment study.

NPS loading models have been developed for field scale or small plot areas, first-order watersheds, and complex drainage basins. Figure 4-2 illustrates these three levels of spatial resolution.

Field Scale: The field scale unit is the basic building block for the larger area models. The "field" varies in size from a few to a few hundred acres. Runoff loads calculated from such areas are limited to direct surface runoff and a small portion of the shallow, subsurface flow (interflow). Only single land uses are represented but these models are ideally suited for evaluating the relative effectiveness of alternative management practices. For example, all soil conservation farm planning is based on techniques developed for field size areas.

Models having only a field scale spatial resolution are subject to the same caveats as the concomitant data bases. Only a portion of the hydrologic system is represented and certain attenuating processes (sediment deposition, adsorption during subsurface flow, etc.) are not included. As a result, the sum of field loadings for a large basin usually exceed measured water quality at a point downstream in the same basin.

First-Order Watershed: A first-order watershed is hydrologically defined as any watershed which is drained by a stream having no tributaries above its confluence. For purposes of this manual, a modification to that definition is proposed - a first-order watershed is any watershed whose water quality is not influenced by in-stream processes (chemical, biological, etc.). The size of such watersheds varies for different regions but is generally limited to less than two square miles. Several land uses are possible at this scale, so the models should have the capability for predicting multi-land use loadings. All components of the hydrologic cycle can be observed in first-order watersheds so that models should (not all do) predict loadings in both surface and subsurface flows.

Complex Drainage Basin: Estimating NPS loading at some point "downstream" in a large watershed having varying land use and a complex hydraulic drainage network cannot be accomplished unless in-stream water quality changes are

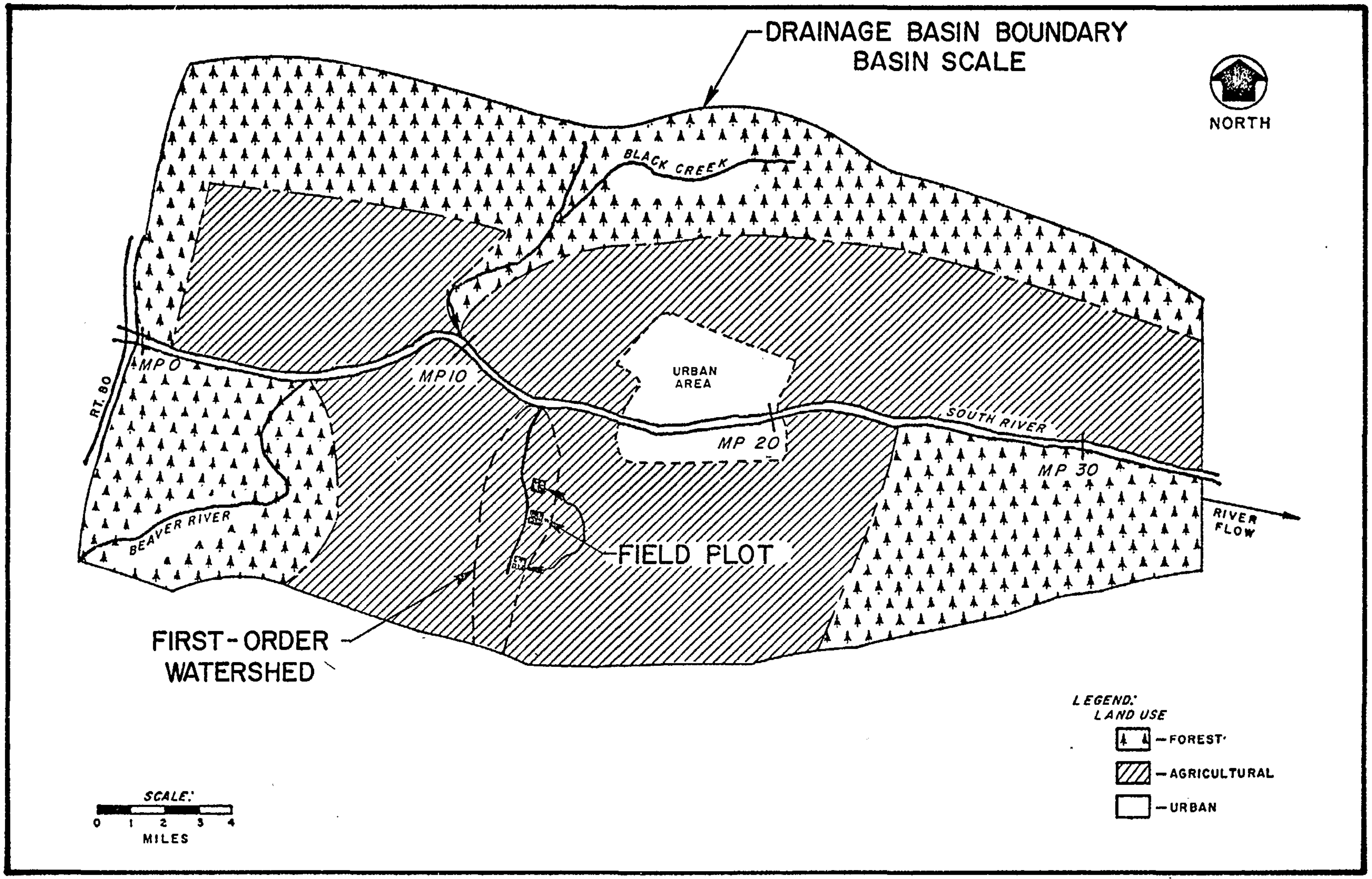


FIGURE 4-2
(SOUTH RIVER, U.S.A.)
HYPOTHETICAL EXAMPLE
SPATIAL SCALE COMPARISON

included as part of the model. For this reason, no generalized "loading only" model has been developed and tested. Indeed, testing might well prove impossible. Empirical models and regression equations can be developed with data collected at any downstream point but the result is an equation that lumps together land use and water quality impacts. The additional set of factors included in the model make accurate extrapolation almost impossible. It is possible, however, to construct models based on first-order watershed scale calculations which may provide good estimates of loads with the accompanying assumption that such loads are conservative. Although these models are difficult to verify, their outputs can be useful in evaluating the relative benefits of various control alternatives applied to large areas.

4.3.2 Temporal Resolution

The second fundamental property of NPS loading models is the time period over which they predict loads. Again, comparison of these time intervals with the behavior of the watershed system response described in section 4.2.2 provides insight into the most appropriate choice of analysis methodology. Three approaches representing different time intervals are available as follows: (1) single event, (2) annual average, and (3) continuous simulation over any specified time interval. Figure 4-3 illustrates the output of each approach. Each time interval will now be examined in more detail.

Event Models: Event models are predicated on the assumption that a "design" load can be estimated and its water quality impact determined. This "worst case" analysis has been popular in hydraulic structure design but its use in nonurban, NPS loading estimation is currently quite limited. The major difficulty with this approach is estimation of the antecedent conditions, especially those related to pollutant availability for transport by runoff. Recall from the earlier discussion of land use that land use activities and their timing combine with various environmental conditions to determine the concentration, form, and location of pollutants for subsequent transport via the hydrologic cycle. Since these processes occur between individual events, their results must be established as boundary conditions for each model run.

Annual Average Models: The major water quality impact of some pollutants is exerted over long time periods. For example, nutrient loads to a large impoundment may be adequately described by the total annual load. If sediment

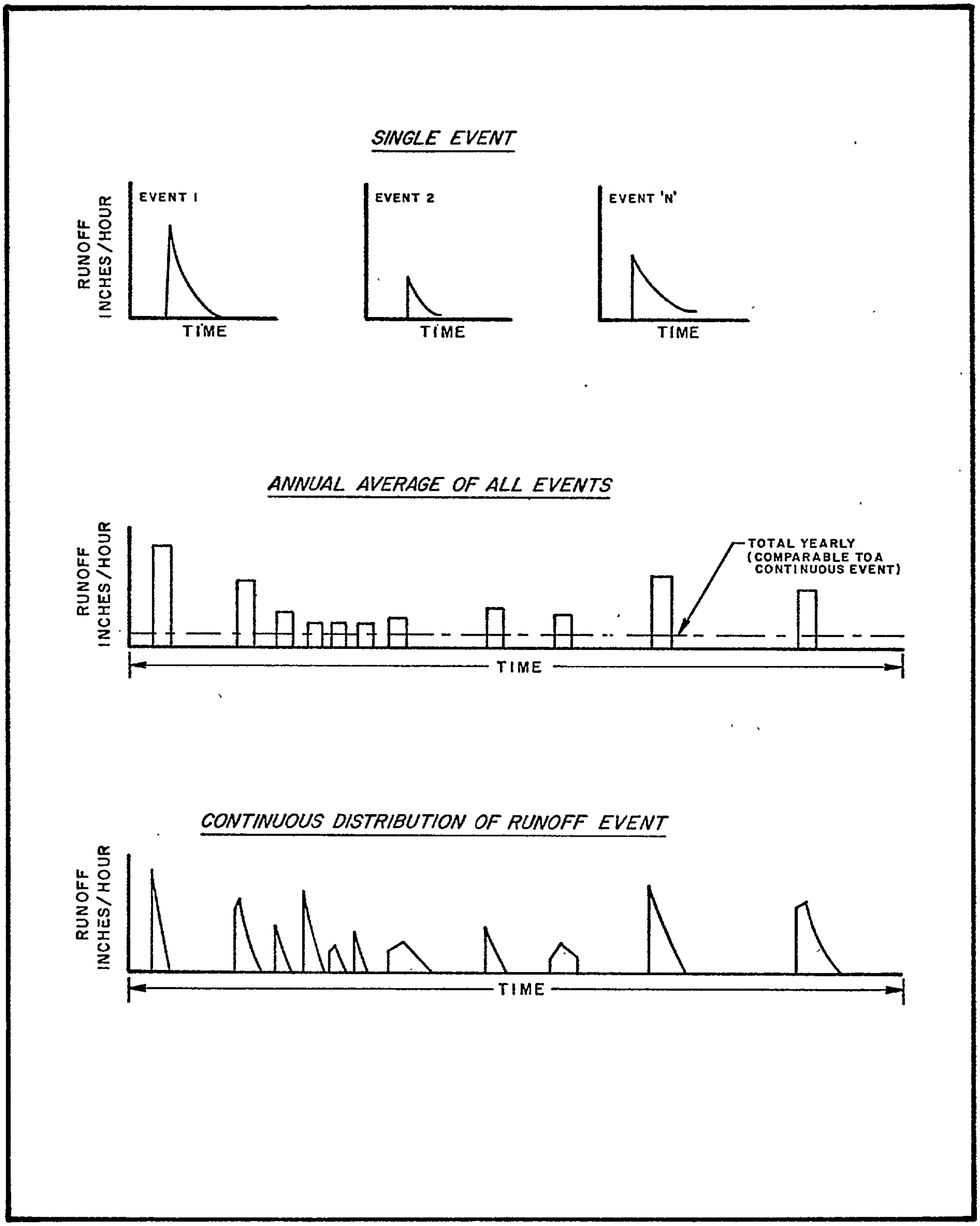


FIGURE 4-3
TEMPORAL RESOLUTION OF RUNOFF PROCESS

deposition is the problem, the total annual load may suffice for an assessment decision. For these situations (and for others not so well suited) models have been developed that predict annual average loadings. In some cases, total water yield is estimated as the transporting medium, while in others, sediment yield is used as the transport medium. Most notable among these models are those based on erosion prediction by the Universal Soil Loss Equation (15). Annual average loading can be distributed on a daily basis as was done in the examples of Chapters 2 and 3.

Continuous Simulation Models: The most complete description of the watershed response is provided by models that attempt to predict all loadings by continuously simulating the hydrologic cycle and its interaction with land use activities. Such techniques require more data inputs and calculation time. The need to predict loadings in this manner may depend on the pollutants of interest and their anticipated impact. For example, pollutants like pesticides that exhibit a short-term water quality impact from peak loading can hardly be predicted by annual average techniques or event models that require *a priori* specification of the timing of peaks in pollutant availability on the watershed. An added advantage of continuous simulation is the ability to predict event or annual average loadings. It is, of course, possible to construct a continuous model from single event models by addition of the between-event processes.

4.3.3 Transport Assumptions

The hydrologic cycle provides the pathways and energy to transport pollutants to surface or groundwater. Pollutants will be transported with the sediment carried by overland flow or dissolved in both overland and sub-surface flow. The physical-chemical processes that determine the relative distribution of pollutants between particulate and dissolved forms are poorly understood and even more difficult to describe mathematically to the point where the theory can be incorporated into NPS loading models. A recognition of the partitioning phenomenon must be made, however, in both interpreting measured data and predicting loads via models. Models have been designed that assume all pollutants are attached to (or behave as) sediment while others attempt to partition pollutants between the two transporting media.

4.3.3.1 Sediment-Based Transport

Sediment-based transport models assume pollutant loads are proportional to sediment loads. Loads are calculated by predicting sediment loss and applying the proportionality relationships for each pollutant. Because sediment is transported by direct surface runoff, sediment-based models are more useful in predicting pollutants associated with soil surface conditions. Dissolved constituents are not necessarily ignored, however. Sediment transport is also proportional to runoff volumes and if the relative distribution between water and sediment does not change, total pollutant losses can be estimated. That is, if the relationship between pollutants and sediment is determined by measurements taken for the total runoff (water and sediment) it may be possible to estimate total loads by only predicting sediment losses.

Two problems arise from the sediment-transport assumption. First, much larger quantities of water than sediment appear in the drainage from watersheds. If dissolved constituents are ignored, significant NPS loads will also be ignored. The fact that subsurface flow accounts for a higher percentage of the total runoff as the watershed size increases further highlights this problem. Nitrate loading estimates are not included in sediment-based models. The second problem arises from the interaction of pollutants and sediment particles. Sorption is a function of surface area which in turn is determined by the particle size. Relationships of surface area to textural classes have been developed by Frere et al. (11). Using a three-level distribution, the specific surface area can be calculated by:

$$SS = 200\ (\%Cl) + 40\ (\%Si) + 0.5\ (\%Sa) \tag{4-1}$$

where SS = specific surface area, m^2/g

$\%Cl$ = clay content of soil, fraction of total

$\%Si$ = silt content of soil, fraction of total

$\%Sa$ = sand content of soil, fraction of total

Equation(4-1)shows that the clay content of soil largely determines the surface area available for interaction with pollutants.

The impact of equation(4-1)on sediment-based loading models results from the mechanics of the erosion process. Analysis of eroded and <u>in situ</u> soil samples for a given area show that erosion is a selective process resulting in a greater percentage of finer material (clays, silts) in the eroded soil than in the original material. The net result is a different relationship between pollutants and sediment in runoff than in the soil profile. Most erosion models predict only gross soil movement. That is, no distinction is made among soil particles sizes. To accommodate this problem, an "enrichment ratio" is often applied to predicted loads to increase the concentration of pollutants in or on eroded soil. Numerical estimates of the enrichment ratio are included in section 4.4.5.

Sediment-based transport models can also estimate loadings for pollutants that behave like inorganic sediment during transport. Organic matter (plant residues, animal wastes, etc.) and crystalline or precipitated chemicals may not be sorbed to soil particles but may be part of the total suspended solids measured in runoff water. If such materials have specific gravities less than inorganic sediments, their presence will increase the measured enrichment ratio because of preferential movement by runoff water.

4.3.3.2 Partitioned-Based Transport

Land use activities combined with environmental conditions within a watershed determine the type, form, and distribution of pollutants. A whole series of complex processes combine to determine for any given pollutant the relative distribution between dissolved and particulate forms. In some cases the distribution is a simple one-way shift from particulate to dissolved as a result of decay or leaching. Usually, however, equilibrium is reached with shifts dependent on pollutant concentration and environmental conditions.

If partitioning processes are included in loading models, the dissolved and particulate loads can be calculated. For example, ammonium (NH_4^+) is transported in both runoff water and adsorbed on sediment. If partitioning constants for NH_4^+ are known for a given soil, loads in water and sediment can be estimated. Currently available models assume adsorption reaches instantaneous equilibrium and represent the process by a variation of the Freundlich equation. The Freundlich equation is:

$$C_a = KC^{1/n} \qquad (4-2)$$

where C_a = pollutant adsorbed per unit weight of soil
K, n = empirical constants
C = pollutant concentration in dissolved phase

Note that when $n = 1$, equation (4-2) reduces to the linear case and a simple partitioning coefficient, K, characterizes the process.

The addition of partitioning capability to NPS loading models obviously enhances their ability to yield reliable results. Unfortunately, the data requirements, model sophistication, and computer run-time requirements also increase. Considerable thought should be given to design of the study plan if such models are chosen for assessment studies so that the most efficient model use can be achieved. Optimum strategies are impossible to specify now because so little experience is available to draw upon.

4.3.4 Classification of NPS Models

Nonpoint source models should be evaluated in the same manner that measured runoff data are analyzed. Namely, how do model properties and capabilities compare with the behavior of the watershed system depicted by Figure 4-1. A complete analysis of each available model along with sample runs, etc., is beyond the scope of this manual, but it is possible to classify the key models or techniques according to the fundamental properties (spatial, temporal, and transport) of importance. Table 4-16 shows the classification of selected NPS loading models.

Appendix A, Model Applicability Summary, presents a brief summary of major stormwater and water quality models and includes some discussion related to two of the models shown in Table 4-16 (AGRUN and ARM). For those models not included in Appendix A, a similar analysis is provided in the following sections.

NPS: The Nonpoint Source Pollutant Loading Model (NPS) was developed by Hydrocomp, Inc., for EPA. The model was specifically designed for use in planning studies and is compatible with existing water quality impact models. The model is comprised of subprograms to represent the hydrologic processes in a watershed, including snow accumulation and melt, and the processes of pollutant accumulation, generation, and washoff from the land surface. The hydrologic components, derived from the Stanford Watershed Model, have been

TABLE 4-16

SELECTED CHARACTERISTICS OF NONURBAN NONPOINT SOURCE MODELS

	Model				
Characteristic	NPS	AGRUN	ACTMO	ARM	MRI
Spatial Resolution					
Field scale	X		X	X	X
First-order watershed	X	X	X	X	X
Basin		X			X
Temporal Resolution					
Runoff event	X	X	X	X	
Annual average	X		X	X	X
Continuous	X		X	X	
Transport Assumption					
Sediment	X	X			X
Partitioned			X	X	
Reference	20	35	11	10	15

previously tested and verified on numerous watersheds across the country. The sediment and pollutant transport components have been tested on several urban and rural watersheds for selected pollutants and are currently undergoing additional testing. The simulation of pollutants is based on sediment as an indicator. Erosion processes are simulated and the resulting loads are converted to pollutant loads by user-specified "potency factors" that indicate the pollutant strength of the sediment for each pollutant simulated.

The NPS model can simulate loads from a maximum of five different land uses in a single production run. In addition to runoff, water temperature, dissolved oxygen, and sediment, the NPS model can simulate up to five user-specified pollutants from each land use category.

Documentation of the model, complete with a user manual and program listing, is available from EPA in a report entitled "Modeling Nonpoint Pollution From the Land Surface," EPA-600/3-76-083, (July 1976).

ACTMO: The Agricultural Chemical and Transport Model (ACTMO) was developed by the Agricultural Research Service, U.S. Department of Agriculture. The model consists of three components simulating hydrology, erosion and sedimentation, and interactions of agricultural chemicals (fertilizers and pesticides) with the soil-water-plant system. The USDAHL-74 model (16) was used for the hydrologic component and the Universal Soil Loss equation was modified to generate erosion/sedimentation. ACTMO is one of two models (ARM is the other) that simulates the partitioning of pollutants between water and sediment. The hydrologic model has been tested on several watersheds, the sediment model has been tested in two locations and the chemical transport model is essentially untested. Current status of model development is unknown.

Documentation of the model is available from ARS-USDA in a report entitled, "ACTMO - An Agricultural Chemical Transport Model," ARS-H-3, (June 1975).

MRI: The Midwest Research Institute (MRI) developed for EPA a series of loading functions for assessment of water pollution from nonpoint sources. These loading functions assume the form of algebraic equations that can be solved analytically without the aid of computers. Functions for essentially all nonpoint sources and pollutants are included. For most cases, modifications of the USLE are used. Daily loads are calculated from annual average estimates. In addition, a methodology is proposed for estimating the maximum and minimum thirty-day loads. This set of functions is consistent with the loading models for urban areas in Chapter 3 and will be further expanded via examples in section 4.5.

Documentation of each loading function complete with supporting data and references is included in the EPA report entitled "Loading Functions for Assessment of Water Pollution From Nonpoint Sources," EPA-600/2-76-151, (May 1976).

4.4 Nonpoint Source Loading Methods Based on the Universal Soil Loss Equation (USLE)

Most nonpoint source models estimate pollutant loads by relating pollutants to sediment. The problem is thus reduced to calculating erosion and sedimentation. The Universal Soil Loss Equation (USLE) is an entrenched analytical tool used for the purpose of soil conservation planning. Because of its wide-spread use and successful testing over the years, many NPS loading models have been built around it. Both desk-top analyses like the MRI loading functions and computer simulation models like STORM, AGRUN, and ACTMO, make use of the equation in one way or another. Future development of NPS loading models will likely continue inclusion of USLE variations. For these reasons, the basic equation, its limitations, extensions, and associated data bases are included in the following sections. The descriptions are somewhat abbreviated to avoid needless repetition of excellent references on the subject (1, 2, 17).

4.4.1 The Equation

The equation is:

$$A = RKLSCP \tag{4-3}$$

where A = average annual soil loss in tons/acre

R = rainfall and runoff erosivity index

K = soil erodability factor

LS = dimensionless topographic factor representing the combined effects of slope length and steepness

C = the cover and management factor

P = factor for supporting practices

Note that equation (4-3) includes factors for precipitation (and to a lesser extent, runoff), soil type, topography, vegetative cover, and structural controls. Although the form of equation (4-3) is often argued, most of the erosion processes are included. The influence of runoff on erosion is only partially implicit in R because of the way in which the data were correlated. That is, R is calculated directly from rainfall but field data against which R was correlated included the lumped effects of rainfall and runoff. A major weakness still prevails if the size of the area expands beyond a field of a few acres. The influence of runoff in channels on erosion and deposition is not included. When the equation is used for calculating annual average loads at a given location, R, K, and LS are fixed, areal properties and yearly variations in sediment loads result solely from changes in management or structural controls.

Perhaps the most attractive feature of the USLE, in addition to its ease of use, is the data base available to aid the user in estimating the equation factors. The U.S. Department of Agriculture's Soil Conservation Serivce uses the equation on a nation-wide basis and considerable effort has been devoted to determination of factors for a wide array of geographical locations, soil types, cropping systems, topographical configurations, and tillage operations. Detailed guidance on selection of the most appropriate numerical values for each factor is included in several of the references given in the list of references for this chapter (for example; (1, 2, 17, 18)). *Under no circumstances should equation 4-3 be used in assessment studies before carefully reading these references.*

The USLE Data Base: The data base for the USLE has been reduced to a series of maps, nomographs, and tables. These data are reproduced in subsequent sections for easy reference. A more detailed description of each factor is given below to aid in parameter selection.

R - The rainfall factor is included in equation (4-3) to represent the influence of precipitation on erosion. R is numerically defined as the number of EI units (erosivity index) for the specified time period. EI is calculated as the product of two rainstorm parameters: kinetic energy of the storm in hundreds of foot-tons per acre times its maximum 30-minute intensity in inches per hour. Data from weather stations having 22 years or longer of recording-raingage records were analyzed to determine the long-term, annual average R values for various locations (18). Results are shown in Figure 4-4. Note that for use in locations within the shaded portions of Figure 4-4 adjustments to accomodate the influence of snowmelt runoff are required. Procedures for correction, along with specific recommendations for certain areas, are available from the Soil Conservation Service Regional Technical Center, Portland, Oregon. The data were further analyzed to determine the yearly distribution of R for each region. Curves for each geographical region have been generated (1, 2). The R value can be estimated by interpolation between isolines of Figure 4-4 or by analysis of local rainfall data. For local data, the kinetic energy can be estimated by the following equation (18):

$$E = 916 + 331 \log X \qquad (4\text{-}4)$$

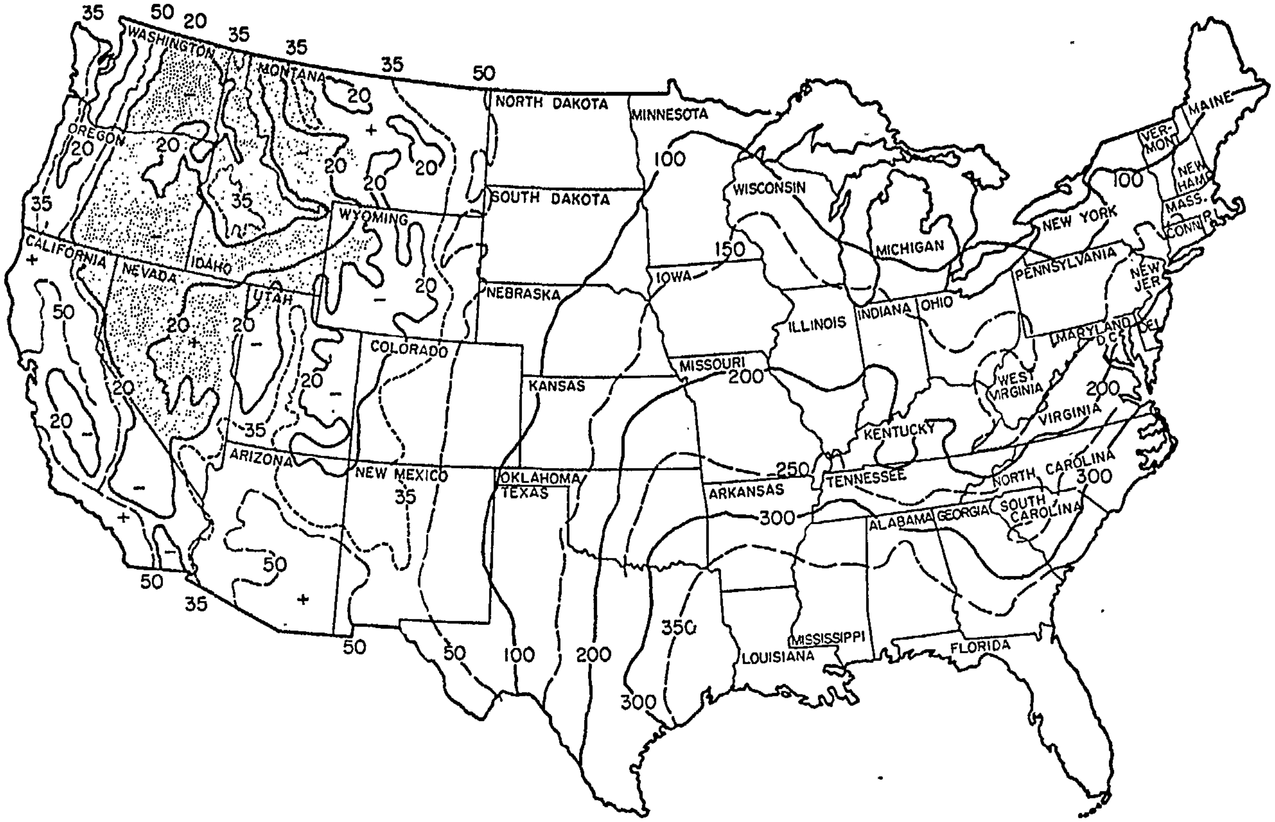
where E = kinetic energy, foot-tons/acre

X = rainfall intensity, inches/hour

The product EI is then determined by multiplication of E by the maximum 30-minute rainfall intensity observed for each storm from which X was abstracted.

K - The soil erodability factor reflects soil properties and is a measure of the susceptibility to erosion. Numerical estimates for certain soils were determined by measurements of soil loss per unit of R for a standard set of conditions established on small plots. A generalized procedure for factor estimation was then developed as a function of standard, measurable soil properties. Results are included in Table 4-17 and the nomograph of Figure 4-5. State and local offices of the Soil Conservation Service also have K values tabulated for specific soils and should be consulted for advice.

LS - The steepness and length of slope for a given area impact on erosion rates. The LS factor represents the combined effect of these two variables and numerical estimates have been determined by analysis of experimental data (1). Results are shown by the solid lines in Figure 4-6. Two important features of these data should be noted. First, the data were taken from studies involving slopes with a specific range of steepness and length (refer



REFERENCE: (2)

FIGURE 4-4
AVERAGE ANNUAL VALUES OF THE RAINFALL-EROSIVITY FACTOR, R

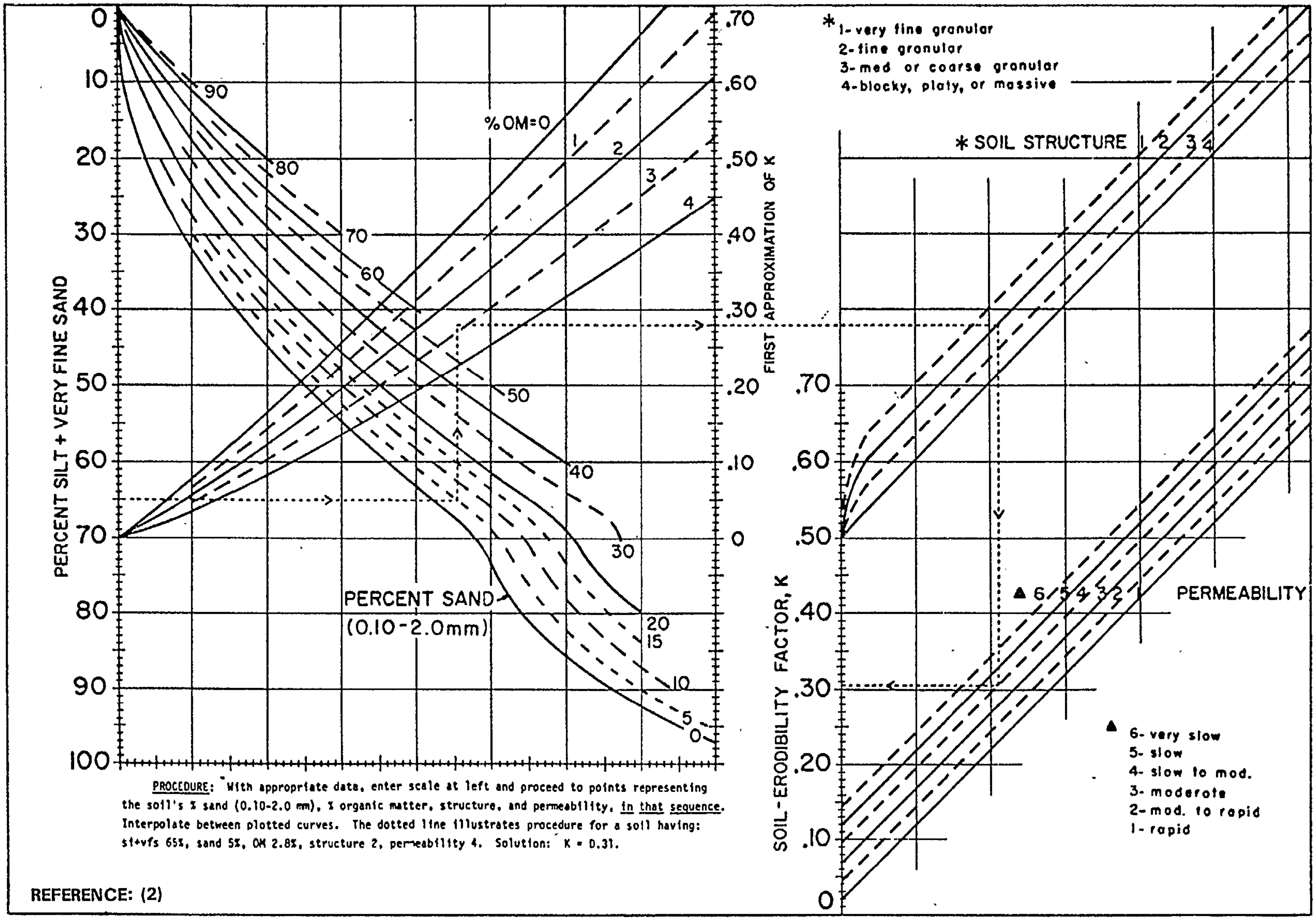
TABLE 4-17

INDICATIONS OF THE GENERAL MAGNITUDE OF THE SOIL-ERODIBILITY FACTOR, K[a]

	Soil Erodibility Factor, K[b]		
	Organic Matter Content		
Texture Class	0.05%	2%	4%
Sand	0.05	0.03	0.02
Fine sand	0.16	0.14	0.10
Very fine sand	0.42	0.36	0.28
Loamy sand	0.12	0.10	0.08
Loamy fine sand	0.24	0.20	0.16
Loamy very fine sand	0.44	0.38	0.30
Sandy loam	0.27	0.24	0.19
Fine sandy loam	0.35	0.30	0.24
Very fine sandy loam	0.47	0.41	0.33
Loam	0.38	0.34	0.29
Silt loam	0.48	0.42	0.33
Silt	0.60	0.52	0.42
Sandy clay loam	0.27	0.25	0.21
Clay loam	0.28	0.25	0.21
Silty clay loam	0.37	0.32	0.26
Sandy clay	0.14	0.13	0.12
Silty clay	0.25	0.23	0.19
Clay		0.13-0.29	

[a] Source: (2)

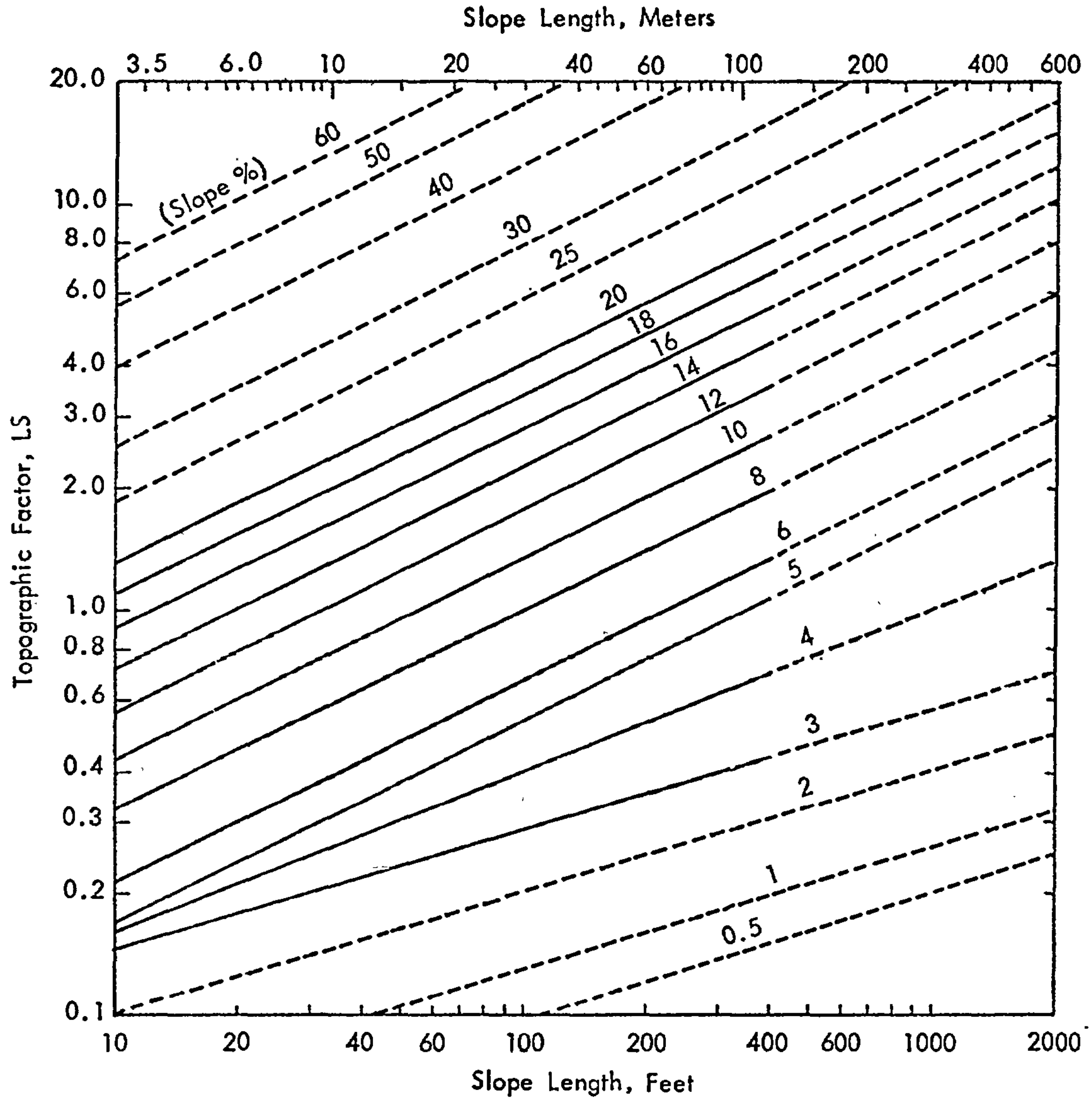
[b] The values shown are estimated averages of broad ranges of specific-soil values. When a texture is near the borderline of two texture classes, use the average of the two K values. For specific soils, use of Figure 4-5 or Soil Conservation Service K-value tables will provide much greater accuracy.



__PROCEDURE__: With appropriate data, enter scale at left and proceed to points representing the soil's % sand (0.10-2.0 mm), % organic matter, structure, and permeability, in that sequence. Interpolate between plotted curves. The dotted line illustrates procedure for a soil having: si+vfs 65%, sand 5%, OM 2.8%, structure 2, permeability 4. Solution: K = 0.31.

REFERENCE: (2)

FIGURE 4-5
SOIL-ERODIBILITY FACTOR (K) NOMOGRAPH FOR U.S. MAINLAND SOILS



REFERENCE: (15)

NOTE:
THE DASHED LINES REPRESENT ESTIMATES FOR SLOPE DIMENSIONS BEYOND THE RANGE OF LENGTHS AND STEEPNESSES FOR WHICH DATA ARE AVAILABLE.

FIGURE 4-6

SAMPLE PLOT OF TOPOGRAPHIC FACTOR (LS) VERSUS SLOPE AND SLOPE LENGTH to note on Figure 4-6). Second, the factors apply to uniform sopes only. Although procedures to correct for the effects of nonuniform slopes have been developed (15), the impact of slope concavity or convexity is not reflected here. The dashed lines of Figure 4-6 represent the extrapolation of the relationship beyond the data base. Validity of this extension is currently unknown.

C - Crop cover and management factors act to mitigate erosion rates. While an annual average C value is often used in the USLE, estimated values, reflecting crop growth stages can also be used. Values range from 0.001 for undisturbed forests to 1.0 for tilled continuous fallow (open, continuously plowed areas). Tables 4-18 through 4-20 summarize appropriate C values for agricultural and silvicultural systems. In cases where the USLE is applied to other land use activities, the C value is approximated by a comparison of the cover conditions to similar cover conditions for agricultural situations. For example, construction activities result in bare, exposed, and disturbed soil surfaces and a C value of 1.0 should be used.

P - Certain other structural or management options related to the landscape serve to mitigate erosion. Such practices are collectively known as supporting practices and include contouring, terracing, strip cropping, etc. The impact of these practices on erosion are estimated through P, with values ranging from 0.25 to 1.0. Table 4-21 summarizes the various P values appropriate for each supporting practice.

4.4.2 A Few Words of Caution

Statistical analyses of the USLE's predictive capability have been performed and a recent paper by the equation's developer summarized these results along with important words of caution for users (17). These precautions are especially noteworthy for those who expand the USLE to aid in estimation of NPS pollutant loads.

The accuracy of the equation was determined by comparing its average annual prediction with measured data from 189 field plots scattered across the country. The overall measured mean soil loss was 11.3 tons per acre. The average prediction error was 1.4 tons with 84% of the predictions within 2

TABLE 4-18

GENERALIZED VALUES OF THE COVER AND MANAGEMENT FACTOR, C, IN THE 37 STATES EAST OF THE ROCKY MOUNTAINS[a,b]

Line number	Crop, Rotation, and Management[d]	Productivity Level[c] High	Productivity Level[c] Moderate
		C Value	C Value
Base value:	continuous fallow, tilled up and down slope	1.00	1.00
Corn			
1	C, RdR, fall TP, conv (1)	0.54	0.62
2	C, RdR, spring TP, conv (1)	0.50	0.59
3	C, RdL, fall TP, conv (1)	0.42	0.52
4	C, RdR, wc seeding, spring TP, conv (1)	0.40	0.49
5	C, RdL, standing, spring TP, conv (1)	0.38	0.48
6	C, fall shred stalks, spring TP, conv (1)	0.35	0.44
7	C(silage)-W(RdL, fall TP) (2)	0.31	0.35
8	C, RdL, fall chisel, spring disk, 40-30% rc (1)	0.24	0.30
9	C(silage, W wc seeding, no-till pl in c-k W (1)	0.20	0.24
10	C(RdL)-W(RdL, spring TP) (2)	0.20	0.28
11	C, fall shred stalks, chisel pl, 40-30% rc (1)	0.19	0.26
12	C-C-C-W-M, RdL, TP for C, disk for W (5)	0.17	0.23
13	C, RdL, strip till row zones, 55-40% rc (1)	0.16	0.24
14	C-C-C-W-M-M, RdL, TP for C, disk for W (6)	0.14	0.20
15	C-C-W-M, RdL, TP for C, disk for W (4)	0.12	0.17
16	C, fall shred, no-till pl, 70-50% rc (1)	0.11	0.18
17	C-C-W-M-M, RdL, TP for C, disk for W (5)	0.087	0.14
18	C-C-C-W-M, RdL, no-till pl 2d & 3rd C (5)	0.076	0.13
19	C-C-W-M, RdL, no-till pl 2d C (4)	0.068	0.11
20	C, no-till pl in c-k wheat, 90-70% rc (1)	0.062	0.14
21	C-C-W-M-M, no-till pl 2d & 3rd C (6)	0.061	0.11
22	C-W-M, RdL, TP for C, disk for W (3)	0.055	0.095
23	C-C-W-M-M, RdL, no-till pl 2d C (5)	0.051	0.094
24	C-W-M-M, RdL, TP for C, disk for W (4)	0.039	0.074
25	C-W-M-M-M, RdL, TP for C, disk for W (5)	0.032	0.061
26	C, no-till pl in c-k sod, 95-80% rc (1)	0.017	0.053
Cotton[e]			
27	Cot, conv (Western Plains) (1)	0.42	0.49
28	Cot, conv (South) (1)	0.34	0.40
Meadow			
29	Grass & Legume mix	0.004	0.01
30	Algalga, lespedeza or Sericia	0.020	
31	Sweet clover	0.025	

TABLE 4-18
(Continued)

Line number	Crop, Rotation, and Management[c]	Productivity Level[c] High C Value	 Moderate
Base value:	continuous fallow, tilled up and down slope	1.00	1.00
Sorghum, Grain (Western Plains)[e]			
32	RdL, spring TP, conv (1)	0.43	0.53
33	No-till pl in shredded 70-50% rc	0.11	0.18
Soybeans[e]			
34	B, RdL, spring TP, conv (1)	0.48	0.54
35	C-B, TP annually, conv (2)	0.43	0.51
36	B, no-till pl	0.22	0.28
37	C-B, no-till pl, fall shred C stalks (2)	0.18	0.22
Wheat			
38	W-F, fall TP after W (2)	0.38	
39	W-F, stubble mulch, 500 lbs rc (2)	0.32	
40	W-F, stubble mulch, 1000 lbs rc (2)	0.21	
41	Spring W, RdL, Sept TP, conv (N & S Dak) (1)	0.23	
42	Winter W, RdL, Aug TP, conv (Kans) (1)	0.19	
43	Spring W, stubble mulch, 750 lbs rc (1)	0.15	
44	Spring W, stubble mulch, 1250 lbs rc (1)	0.12	
45	Winter W, stubble mulch, 750 lbs rc (1)	0.11	
46	Winter W, stubble mulch, 1250 lbs rc (1)	0.10	
47	W-M, conv (2)	0.054	
48	W-M-M, conv (3)	0.026	
49	W-M-M-M, conv (4)	0.021	

[a] Source: (2)

[b] This table is for illustrative purposes only and is not a complete list of cropping systems or potential practices. Values of C differ with rainfall pattern and planting dates. These generalized values show approximately the relative erosion-reducing effectiveness of various crop systems, but locationally derived C values should be used for conservation planning at the field level. Tables of local values are available from the Soil Conservation Service.

[c] High level is exemplified by long-term yield averages greater than 75 bu. corn or 3 tons grass-and-legume hay; or cotton management that regularly provides good stands and growth.

[d] Numbers in parentheses indicate number of years in the rotation cycle. Number (1) designates a continuous one-crop system.

TABLE 4-18
(Continued)

[e] Grain sorghum soybeans, or cotton may be substituted for corn in lines 12, 14, 15, 17-19, 21-25, to estimate C values for sod-based rotations.

Abbreviations defined:

B = soybeans
C = corn
c-k = chemically killed
conv = conventional
cot = cotton
F = fallow
M = grass & legume hay
pl = plant
W = wheat
wc = winter cover

lbs rc = pounds of crop residue per acre remaining on surface after new crop seeding
% rc = percentage of soil surface covered by residue mulch after new crop seeding
70-50% rc= 70% cover for C values in first column; 50% for second column
RdR = residues (corn stover, straw, etc) removed or burned
RdL = all residues left on field (on surface or incorporated)
TP = turn plowed (upper 5 or more inches of soil inverted, covering residues)

TABLE 4-19

C FACTORS FOR PASTURE, RANGELAND, AND IDLE LAND[a,b]

Vegetative Canopy			Cover that Contacts the Surface					
Type and Height of Raised Canopy[c]	Canopy Cover,[d] %	Type[e]	Percent Ground Cover					
			0	20	40	60	80	95-100
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	0.45	0.24	0.15	0.090	0.043	0.011
Canopy of tall weeds or short brush (0.5 m fall height)	25	G	0.36	0.17	0.09	0.038	0.012	0.003
		W	0.36	0.20	0.13	0.082	0.041	0.011
	50	G	0.26	0.13	0.07	0.035	0.012	0.003
		W	0.26	0.16	0.11	0.075	0.039	0.011
	75	G	0.17	0.10	0.06	0.031	0.011	0.003
		W	0.17	0.12	0.09	0.067	0.038	0.011
Appreciable brush or bushes (2 m fall height)	25	G	0.40	0.18	0.09	0.040	0.013	0.003
		W	0.40	0.22	0.14	0.085	0.042	0.011
	50	G	0.34	0.16	0.085	0.038	0.012	0.003
		W	0.34	0.19	0.13	0.081	0.041	0.011
	75	G	0.28	0.14	0.08	0.036	0.012	0.003
		W	0.28	0.17	0.12	0.077	0.040	0.011
Trees but no appreciable low brush (4 m fall height)	25	G	0.42	0.19	0.10	0.041	0.013	0.003
		W	0.42	0.23	0.14	0.087	0.042	0.011
	50	G	0.39	0.18	0.09	0.040	0.013	0.003
		W	0.39	0.21	0.14	0.085	0.042	0.011
	75	G	0.36	0.17	0.09	0.039	0.012	0.003
		W	0.36	0.20	0.13	0.083	0.041	0.011

[a] Source: (15)

[b] All values shown assume: 1) random distribution of mulch or vegetation, and 2) mulch of appreciable depth where it exists.

[c] Average fall height of waterdrops from canopy to soil surface, m = meters.

[d] Portion of total-area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[e] G = Cover at surface is grass, grass-like plants, decaying compacted duff, or litter at least 5 cm (2 in.) deep.
W = Cover at surface is mostly broadleaf herbaceous plants (as weeds) with little lateral-root network near the surface and/or undecayed residue.

TABLE 4-20

C FACTORS FOR WOODLAND[a]

Stand Condition	Tree Canopy[b] Percent of Area	Forest Litter[c] Percent of Area	Undergrowth[d]	C Factor
Well stocked	100-75	100-90	Managed[e]	0.001
			Unmanaged[e]	0.003-0.011
Medium stocked	70-40	85-75	Managed	0.002-0.004
			Unmanaged	0.01-0.04
Poorly stocked	35-20	70-40	Managed	0.003-0.009
			Unmanaged[f]	0.02-0.09

[a] Source: (15)

[b] When tree canopy is less than 20%, the area will be considered as grassland or cropland for estimating soil loss.

[c] Forest litter is assumed to be at least 2 inches deep over the percent ground surface area covered.

[d] Undergrowth is defined as shrubs, weeds, grasses, vines, etc., on the surface area not protected by forest litter. Usually found under canopy openings.

[e] Managed = grazing and fires are controlled.
Unmanaged = stands that are overgrazed or subjected to repeated burning.

[f] For unmanaged woodland with litter cover of less than 75%, C values should be derived by taking 0.7 of the appropriate values in Table 4-19 The factor of 0.7 adjusts for the much higher soil organic matter on permanent woodland.

TABLE 4-21

P VALUES FOR EROSION CONTROL PRACTICES ON CROPLANDS[a]

	Erosion Control Practice				
Range of Slope	Up and Down Hill	Cross-Slope Farming Without Strips	Contour Farming	Cross-Slope Farming With Strips	Contour Strip-Cropping
2.0-7	1.0	0.75	0.50	0.37	0.25
7.1-12	1.0	0.80	0.60	0.45	0.30
12.1-18	1.0	0.90	0.80	0.60	0.40
18.1-24	1.0	0.95	0.90	0.67	0.45

[a] Source: (1)

tons of the measured losses. Further analysis also showed that larger errors were associated with measured data collected over shorter time periods than the 22-year cycle chosen for the R data base.

Considerable error can result if the equation factors are estimated incorrectly for large areas where watershed sediment yield is the objective. The author states..."Applying the equation to a complex watershed by using overall averages of slope length and gradient with estimated watershed-average value for factors K and C would be incorrect. To use the equation correctly, the combination of selected factor values must reflect the manner in which the parameters are associated in each subarea....Perhaps the greatest potential source of prediction error is superficiality in selecting factor values....If the selected values do not truly represent the conditions to be evaluated, neither will the computed soil loss." (17).

4.4.3 Sediment Delivered to the Stream

Gross erosion as predicted by the USLE suffers the same limitation as pollutant loss data collected at the outlet of plots or small fields - the load to the stream is significantly less than these values because other components of the hydrologic system act to attenuate their magnitude. For sediment,this attenuation is a function of many variables including soil characteristics, watershed area, slopes, slope length, relief/length ratio, and drainage density. Erosion from gullies or the stream channel itself is a contributor to downstream sediment load but is not included in the USLE predictions.

Correction for the efficiency of a watershed system to yield eroded sediments to a point downstream is made by application of a sediment delivery ratio (SDR). The SDR is defined as the ratio of sediment delivered at a location in the stream system to the gross erosion from the drainage area above that point.

Ideally, one could structure a model using sediment transport theory and route both water and sediment through the system. Failing that, most investigators have chosen to develop empirical relationships (based on data) for sediment delivery including one or more of the variables listed above. The results of a recent development for use with NPS loading functions are given in Figure 4-7. Drainage density in Figure 4-7 is defined as the ratio of total channel-

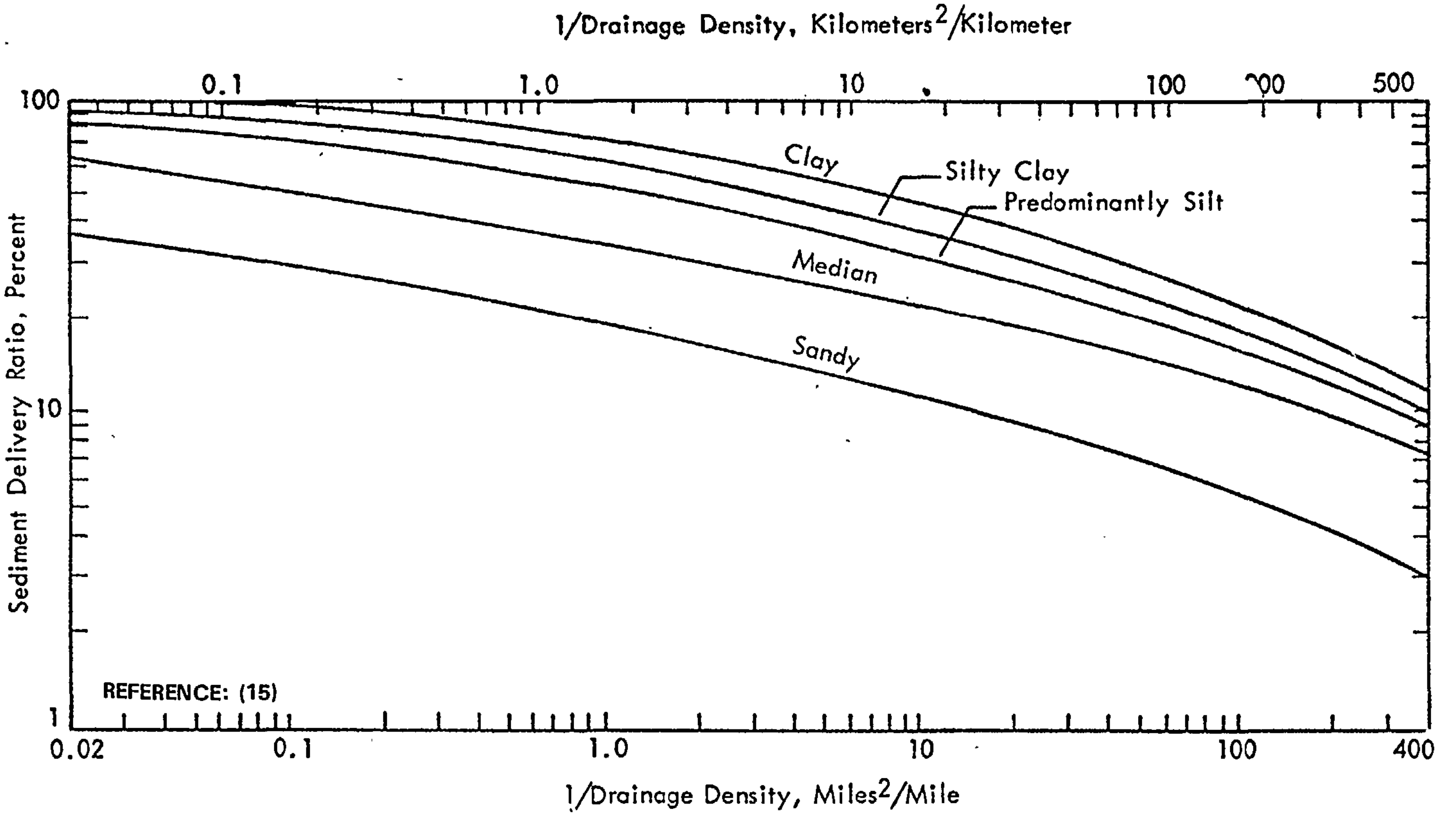


FIGURE 4-7
SEDIMENT DELIVERY RATIO FOR RELATIVELY HOMOGENEOUS BASINS

segment lengths to the basin area. Note also the different relationship for each soil particle size class. This distinction is made to accommodate the greater ease with which finer materials are transported.

Application of the SDR to the USLE enables the analyst to estimate loads to a specific point in the stream. A sediment yield equation is thus given by:

$$Y(S)_E = A(RKLSCP)S_d \tag{4-5}$$

where $Y(S)_E$ = sediment loading to stream, tons/yr

A = area, ac

$RKLSCP$ = factors of USLE

S_d = sediment delivery ratio

If data are available for the area of interest, S_d should be validated, if possible, by analysis of the data. Usually, reservoir sedimentation rates are the most commonly available data sources.

4.4.4 A Few More Words of Caution

The USLE does not estimate erosion from gullies, stream banks, or head cuts. Delivery ratios based on locally measured data may include the lumped effects of these sources as well as the sources estimated by the USLE. The most appropriate value is given by:

$$S_d = \frac{SY}{SH + GU + CH} \tag{4-6}$$

where SY = sediment yield at point of interest

SH = USLE related erosion

GU = gully erosion

CH = channel erosion

Note that if S_d is determined from measured data that accounts for only SH in the denominator of equation 4-6, the resulting ratio will be too high when applied to new values of SH for prediction of SY. To illustrate this possible source of error, data from a recent paper on sediment yield from two watersheds in the Mississippi Delta were analyzed (19). The study reported estimates of each source of erosion (sheet, gully, and channel) and the total watershed sediment yield. Calculated 15-year mean sediment delivery ratios were 0.28 and 0.57, respectively. Calculation of similar values, but based on estimates for sheet erosion only (per USLE), yielded ratios of 0.42 and 0.80, respectively. Use of these uncorrected delivery ratios in NPS assessment studies would result in the estimated values being 150% and 140% of the actual loads.

4.4.5 MRI Loading Functions

The final step in developing a series of pollutant loading functions based on sediment delivered to the stream is correlation of pollutants to sediments. Two options are available. A direct ratio of pollutants to sediment can be determined by field sampling followed by calibration of equation (4-5). A similar procedure is recommended for models like the NPS (20) and STORM (21). The other option is a correlation between pollutants and <u>in situ</u> soil along with a factor to correct for the enrichment process discussed in section 4.3.3. Midwest Research Institute (15) used this approach and developed a series of loading functions having the form:

$$Y(P)_E = aY(S)_E \, C_s(P) r_P \qquad (4\text{-}7)$$

where $Y(P)_E$ = pollutant, P, load to streams
a = dimensional constant
$Y(S)_E$ = sediment loading to stream (using equation (4-5))
$C_s(P)$ = concentration of pollutant, P, in soil profile
r_P = enrichment ratio for pollutant, P

The enrichment ratio, r_P, corrects for the observed phenomena that pollutants in eroded sediments are somewhat more concentrated than the same pollutants in the watershed soil profile. Numerical values are simply the ratio of the two concentrations. The need for such a ratio is not surprising in view of the surface area differences among soil fractions. That is, the much greater surface area associated with the finer, more erodable particles (see equation (4-1)).

Equation(4-7)is further modified for certain pollutants to account for the "unavailability" of portions of the load. For example, if equation(4-7)were used to estimate only total phosphorus loadings, determining the water quality impact would be difficult because it is only the biologically available phosphorus that exerts the impact. This correction is made by simply multiplying equation (4-7) by an availability factor (percent of load that is immediately available for use by plants or animals).

A summary of the MRI loading functions is given in Tables 4-22 and 4-23. Details of their development and additional information is included in the EPA report entitled "Loading Functions for Assessment of Water Pollution From Nonpoint Sources," EPA-600/2-76-151,(May 1976).

The most accurate results from equation (4-7) can be expected when the long-term average annual loads are estimated. The temporal distribution of the annual loads is important in water quality impacts, however, especially for those pollutants exerting short-term impacts. To accommodate the necessity to estimate peak loads over a one-year period, two methods are proposed. First, if one assumes that the only variation in the USLE factors for a given area is that attributed to rainfall, then, it is possible to distribute the loads according to the distribution of R. That is, all factors in the USLE are considered fixed except for R. The variation in R is determined by cumulative distribution curves developed by the Soil Conservation Service (1) and reproduced by MRI (15). Figure 4-8 generated (15) from the cumulative distribution curves, is expressed in units of percent of gross surface erosion expressed on a daily basis. The percent of the annual erosion for each time period is easily determined by multiplying each data point (for each month) by the number of days in the month. The shape of the curve will obviously remain the same.

The assumption that all factors of the USLE will remain the same throughout the year is unlikely for land uses subject to operations like agriculture. For these situations, the curve is further modified by the C factor. Periods for which the C factors are different are determined and the product RC is used to distribute the loads. Results for continuous corn in central Indiana is given in Figure 4-9. Note the synergistic effects of both the high R values and high C values for the mid-June to mid-July period resulting in maximum loadings for the year.

The MRI Loading Functions Data Base: The MRI document repeatedly stresses the value of obtaining locally derived data inputs from knowledgeable sources. This advice is indeed appropriate, and typical sources are listed in Appendix C as well as the MRI report (15). In cases where local data are

TABLE 4-22
LOADING FUNCTIONS SUMMARY[a]

Loading Type or Source	Loading Function	Description
Sediment (Agriculture)	$Y(S)_E = \sum_{i=1}^{n} (A \cdot R \cdot K \cdot L \cdot S \cdot C \cdot P \cdot S_d)_i$	Applicable to agricultural land; may be modified with multipliers to represent silviculture, construction, surface mining
Nitrogen	$Y(NT)_E = a \cdot Y(S)_E \cdot C_s(NT) \cdot r_{NT}$ $Y(NA)_E = Y(NT)_E \cdot f_N + Y(N)_{Pr}$ $Y(N)_{Pr} = A \cdot \frac{Q(OR)}{Q(Pr)} \cdot N_{Pr} \cdot b$ $Y(NA) = Y(NT)_E \cdot f_N + Y(N)_{Pr}$	The first loading function represents total nitrogen yield from sediment. The second loading function represents total available nitrogen in sediment. The third function represents the precipitation borne nitrogen loading that is transported via overland flow. The last function is total available nitrogen.
Phosphorus	$Y(PT) = a \cdot Y(S)_E \cdot C_s(PT) \cdot r_{PT}$ $Y(PA) = Y(PT) \cdot f_p$	The first loading function represents total phosphorus yield. The second loading function represents total availabel phosphorus. There is no phosphorus in precipitation.
Organic Matter	$Y(OM)_E = a \cdot Y(S)_E \cdot C_s(OM) \cdot r_{om}$	Can be used to estimate BOD loadings.
Pesticides (Herbicides, Insecticides, Fungicides)	$Y(HIF) = a' \cdot C_s(HIF) \cdot Y(S)_E \cdot r_{HIF}$	Based on average pesticide residual (nonpeak loads). Used for insoluble pesticide, if average soil concentration known.
	$Y(HIF) = \sum_{i=1}^{n} Q_i C_i a$	Used for water soluble and insoluble pesticide loadings
Salinity	$Y(TDS)_{IRF} = a \cdot A \cdot C(TDS)_{GW} \cdot (IRR + Pr-CU)$	Source-to-stream methodology
	$Y(TDS)_{IRF} = a \cdot (Q(str)_B \cdot C(TDS)_B - Q(str)_A \cdot C(TDS)_A) - Y(TDS)_{BG} - Y(TDS)_{PT}$	Stream-to-source methodology
	$Y(TDS)_{IRF} = (TDS)_{IRF} \cdot A$	Requires areal salt loading rates based upon accumulated results of stream monitoring program.
Sediment 1) Silviculture 2) Construction 3) Surface Mining	$Y(S)_E = \sum_{i=1}^{n} (A \cdot R \cdot K \cdot L \cdot S \cdot C \cdot P \cdot M \cdot S_d)$	M is a multiplier indicating the effect of different types of disturbances. Nutrient loads are assumed to be only inorganic.
Acid Mine Drainage	$Y(AMD) = N[Ka \cdot (I_{AU} + I_{IU} + I_{AS} + I_{IS}) - Kb \cdot Q(R) \cdot C(AUK)_{BG}]$ $Y(AMD) = a \cdot A \cdot Q(R) \cdot (C(SO_4) - C(SO_4)_{BG} - C(SO_4)_{PT})$ $Y(AMD) = a \cdot Q(str) \cdot (C(SO_4) - C(SO_4)_{BG} - C(SO_4)_{PT})$	The appropriate equation to be used for calculating acid mine drainage depends upon the region of the county. The first two loading functions reflect a source-to-stream approach; the next two reflect a stream-to-source estimating procedure.
Heavy Metals/ Radioactivity	$Y(HM)_s = a \cdot C_s(HM) \cdot Y(S)_E$ $Y(HM) = a \sum_{i=1}^{n} Q_n \cdot C(HM)_n$ $Y(HM) = a \cdot A \cdot Q(R) \ (C(HM) - C(HM)_{BG})$ $Y(HM) = a \cdot Q(str) \ (C(HM) - C(HM)_{BG})$	Estimating procedure for heavy metals and radioactivity loads are identical except that heavy metals go in as "ppb" and radioactivity as "picocuries/liter"
Feedlots	$Y(i)_{FL} = a \cdot Q(FL) \cdot C(i)_{FL} \cdot FL_d \cdot A$	Overland runoff approach; not related to sediment
Terrestrial Disposal	$Y(i)_{LF} = a \cdot C(i)_{LF} \cdot Q(LF) \cdot LF_d \cdot A$	Subsurface/groundwater return flow, in the form of leachate

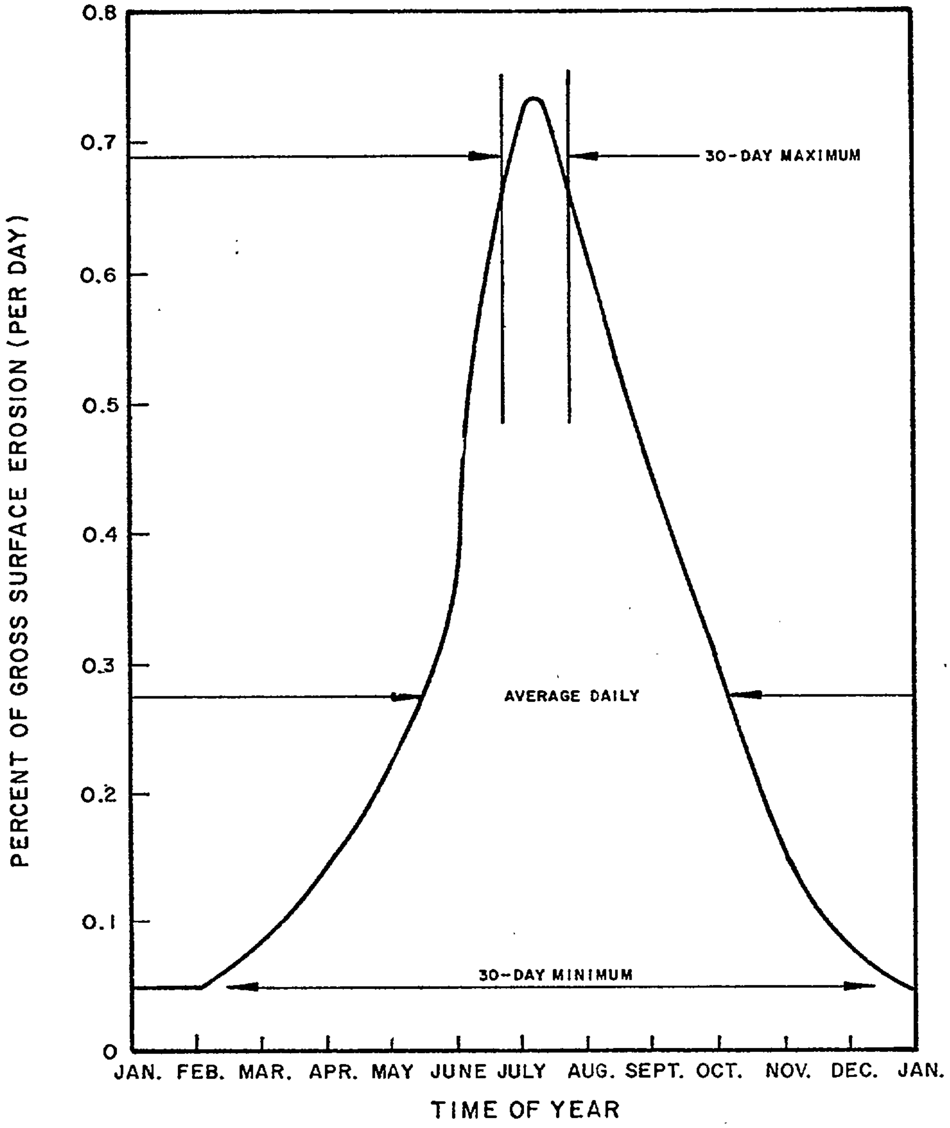
(a) These loading functions are derived and explained by MRI in Reference (15)

TABLE 4-23

LIST OF SYMBOLS USED IN TABLE 4-22

A	Source area, ha
AMD	Acid mine drainage
AS; AU	Active surface or underground mine
BG	Background source
C	Cover management factor
$C_s(i)$	Concentration of pollutant i in sediment
$C(i)_{source}$	Concentration, C of pollutant i in source
$C(Alk)_{BG}$	Concentration of background alkalinity, mg/liter
D	Overland distance between erosion site and receptor water, ft
DD	Drainage density, km^{-1}
E	Annual average erosion rate, MT/ha/year
f_N	Ratio of NA:NT in eroded sediment
f_P	Ratio of PA:PT in eroded sediment
FL	Small feedlot source
FL_d	Feedlot delivery ratio
HIF	Herbicide, Insecticide, Fungicide; and pesticide
HM	Heavy metals or radioactivity
I	Load index for acid mine drainage
IRF	Irrigation return flow
IRR	Irrigated water added annually to crop root zone, cm/year
IS; IU	Inactive surface or underground mine
K	Soil erodibility factor
L	Slope length factor
LF; LF_d	Landfill, landfill delivery ratio
LS	Topographic factor
NA	Available (or mineralized) nitrogen
N_{Pr}	Nitrogen yield rate per unit area from precipitation, kg/ha/year
NT	Sum of nitrogen of all chemical forms
OM	Organic matter
OR	Overland runoff
P	Percolation rate, cm/year
P	Conservation practice factor
Pr	Annual average precipitation, cm/year, storm precipitation, cm
PA	Available phosphorus
PD	Population density, number/ha
PT	Total phosphorus; also point source
Q_i; Q	Runoff due to a storm event, cm
Q(FL)	Feedlot runoff, cm/year
Q(LF)	Landfill leachate flow rate, cm/year

Q(OR)	Overland runoff, cm/year
Q(P)	Total precipitation flow rate, cm/year
Q(Perc)	Percolation flow rate, cm/year
Q(R)	Direct runoff, cm/year
Q(Str)	Stream flow rate, liters/sec
Q(t)	Runoff over a period of time, t
R	Rainfall erosivity factor
R_r	Rainfall erosivity factor due to rainfall
R_s	Rainfall erosivity factor due to snowmelt
r_{NT}	Enrichment ratio for nitrogen (ratio of concentration of nitrogen in sediment to that in soil)
r_{OM}	Enrichment ratio for organic matter (ratio of concentration of organic matter in sediment to that in soil)
r_{PT}	Enrichment ratio for phosphorus (ratio of concentration of phosphorus in sediment to that in soil)
S	Slope gradient factor; also sediment
S_d	Sediment delivery ratio (ratio of the amount of sediment delivered to a stream to the amount of on-site erosion)
TDS	Total dissolved solids
U	Composite topographic factor for irregular slopes
Y(AMD)	Acid mine drainage loading, kg/year
Y(HIF)	Total pesticide loading, kg/year
Y(HM)	Heavy metal loading, kg/year
$Y(i)_{FL}$	Loading of pollutant i from small feedlots, kg/year
$Y(i)_{LF}$	Loading of pollutant i from landfills, kg/year
$Y(i)_U$	Loading of pollutant i from urban areas, kg/year
$Y(N)_{Pr}$	Nitrogen loading from precipitation runoff, kg/year
Y(NA)	Available nitrogen loading, kg/year
$Y(NT)_E$	Total nitrogen loading from erosion, kg/year
$Y(OM)_E$	Organic matter loading, kg/year
Y(PA)	Available phosphorus loading, kg/year
Y(PT)	Total phosphorus loading, kg/year
Y(RAD)	Loading of radioactive substances, microcuries/year
$Y(S)_E$	Sediment loading from surface erosion, MT/year
$Y(S)_U$	Loading of street solids from urban areas, kg/year
$Y(TDS)_{BG}$	Salinity (TDS) load from background, kg/year
$Y(TDS)_{IRS}$	Salinity (TDS) load in irrigation return flow, kg/year
$Y(TDS)_{PT}$	Salinity (TDS) load from point sources, kg/year
$Y(i)_{BG}$	Yield of pollutant i from background, kg/year



REFERENCE: (15)

FIGURE 4-8
PROJECTED VARIATION OF SOIL EROSION

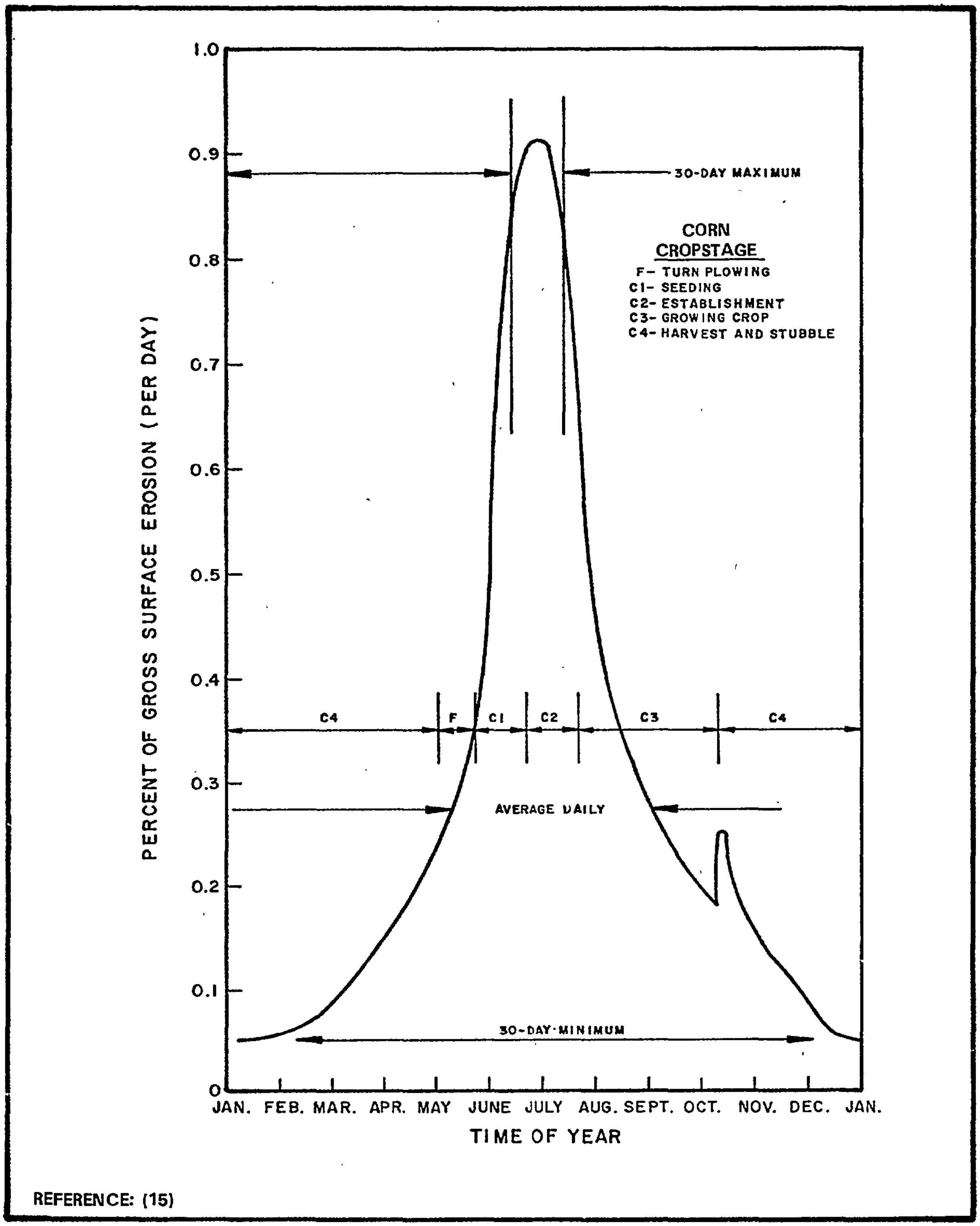


FIGURE 4-9
PROJECTED VARIATION OF SOIL EROSION

unavailable or where only a first-cut analysis is desired, procedures are proposed for parameter estimation. Factors associated with sediment loading were discussed in sections 4.4.1, 4.4.2, and 4.4.3. The following data apply to the pollutant associated parameters of selected loading functions.

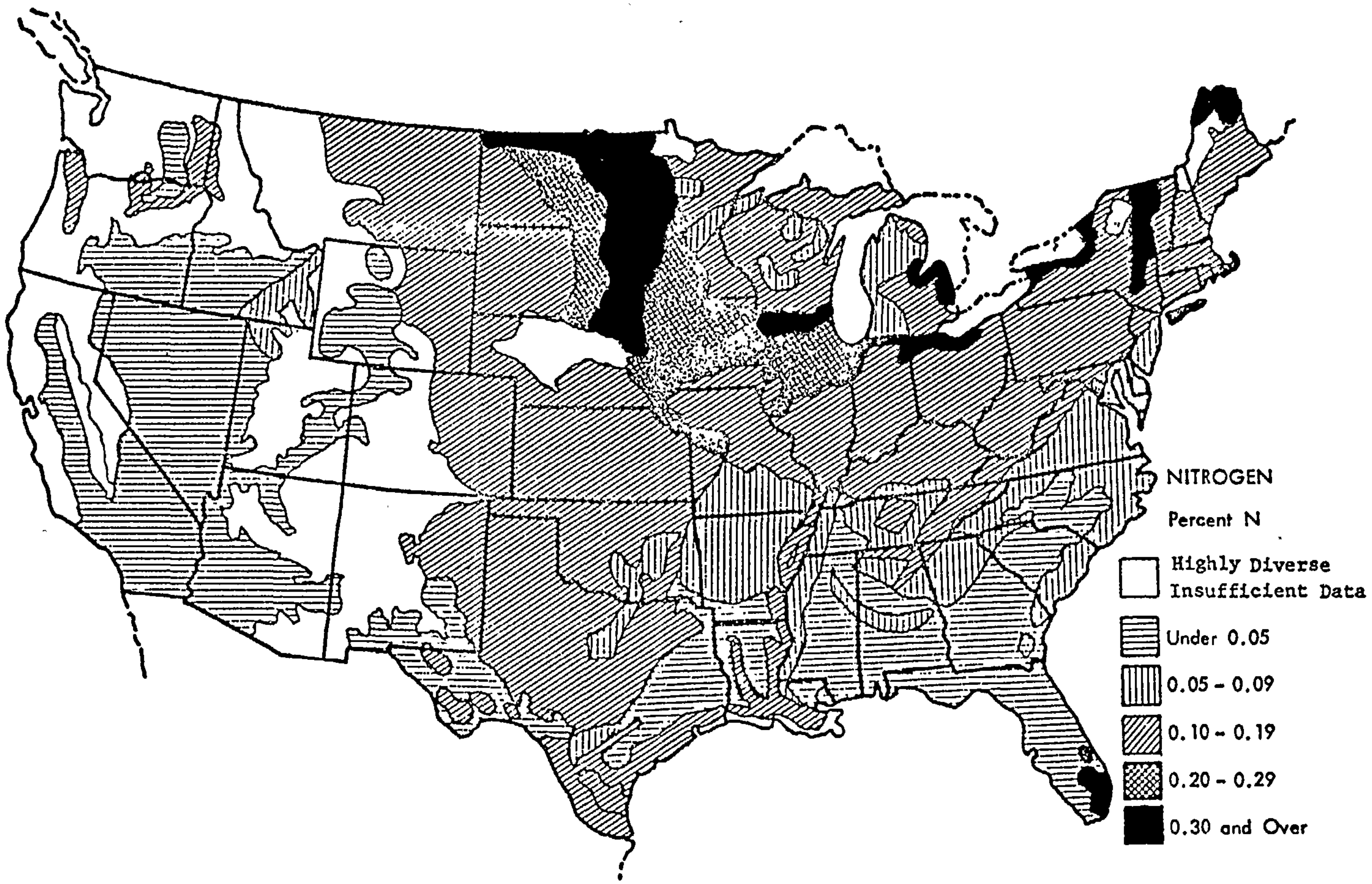
Soil-Pollutant Concentrations - Cs(P) of equation (4-7) - Soil sampling data is the most reliable and current data available for estimation of Cs(P). Failing that, however, one can use the maps given in Figures 4-10 and 4-11 for soil nitrogen and phosphorus content. Organic matter estimates are made by multiplying the nitrogen values by 20. An alternative method for estimating nitrogen concentrations is also included in the MRI document (15) but will not be repeated here.

Enrichment Ratios - (r_p in equation(4-7))- Estimation of enrichment ratios without the benefit of locally derived data requires extrapolation at its best (or worst). Only a few experimental studies have reported measured values and no theoretical approaches to prediction have been attempted. Table 4-24 gives typical values for nitrogen and phosphorus. MRI used values of 2.0 and 1.5 for nitrogen and phosphorus, respectively, in solution of example problems. No experimentally determined values for organic matter are reported, but MRI suggested a range of from 1.0 to 5.0 and used a value of 2.5 for the completed example.

Availability Factors - The fraction of total pollutants that are available for immediate use by aquatic plants, etc., is also difficult to estimate with certainty. For nitrogen, MRI suggested an upper limit of 15%, reported a value of 8% and used 6% in their completed example. For phosphorus, values of 5% and 10% were reported and 10% was chosen for use in their completed example. No availability factors were given for organic matter, but the fraction of organic matter exerting BOD is important from a water quality impact perspective. Locally determined BOD values for surface soil organic matter should provide useful information.

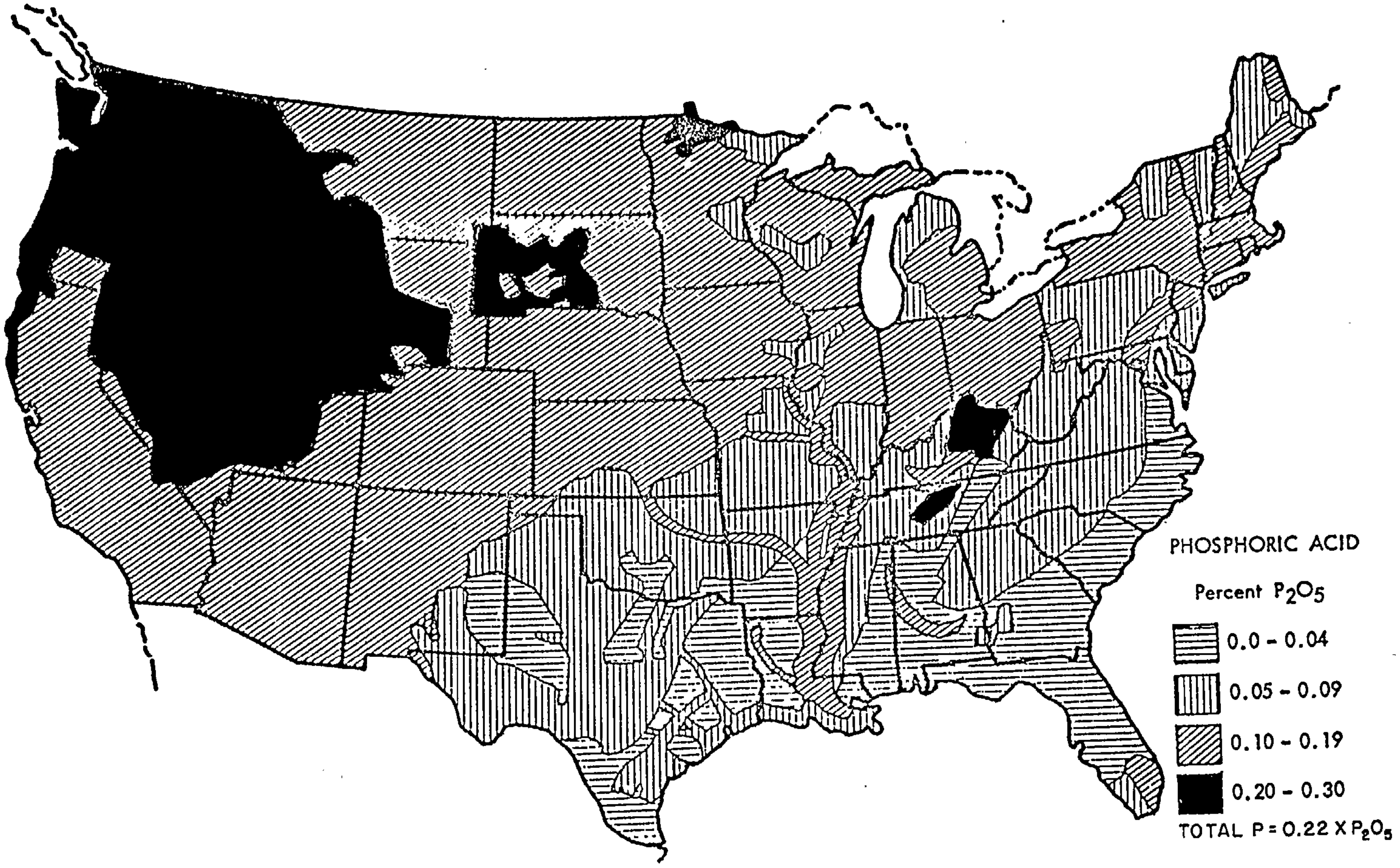
4.5 Methodology for Preliminary Assessment of Nonurban Nonpoint Source Loadings

The techniques and data bases discussed in the previous sections can be used to assess the nature, magnitude, and extent of nonpoint source pollutant



REFERENCE: (15)

FIGURE 4-10
PERCENT NITROGEN (N) IN SURFACE FOOT OF SOIL



REFERENCE: (15)

FIGURE 4-11

PERCENT PHOSPHORUS (P) IN SURFACE FOOT OF SOIL

TABLE 4-24

SUMMARY OF EXPERIMENTALLY DETERMINED ENRICHMENT RATIOS[a]

	Ratio	
Land use	Nitrogen	Phosphorus
Fallow	3.88	1.59
Rye winter cover crop	4.08	1.56
Manure spreading	4.28 3.35	1.47 1.47
Agricultural[b]	2.0 2.7	- -

[a] Source: (15)

[b] No specific activity reported.

loading. Assessment can be preliminary or comprehensive, depending on the choice of the methdology and the resources available for analysis. Whatever the approach taken, interpretation of the data base (both measured and abstracted from previous studies), and loading predictions should only be made with due consideration for the hydrologic-watershed system described in section 4.2.2.

The approach chosen for presentation in the following sections can result in at best a preliminary assessment and was designed to be consistent with the analysis for urban loadings presented in Chapter 3. The general approach, however, is useful as a guide for assessment regardless of the particular set of tools (models) chosen for use. The methodology is limited to a problem assessment only - evaluation of alternative control strategies, selection of controls, and economic analyses are not included.

4.5.1 Statement of the Problem and the Solution Methodology

The Problem: As part of a water quality impact assessment study in a planning area, a specific (typical) watershed is chosen to evaluate the nature, extent, and magnitude of NPS loads. Subsequently, an analysis of the water quality impacts for the water quality limited stream segment into which the watershed drains will be made. The pollutant types, magnitude, and timing are desired.

The Methodology: The assessment procedure has been divided into six steps, as shown in Figure 4-12. Each step consists of an activity requiring input information of the type described. Completion of the activity results in a series of outputs that are, in turn, available for inputs to the next step or for decision-making. For steps II, III, IV, and VI, references to previous sections are given to indicate the location of necessary or useful information. To illustrate the methodology each step in Figure 4-12 will be completed for a hypothetical example consistent with the problem stated above. Data for step I will be abstracted from a report by the Ohio River Basin Sanitation Commission (Reference: 22). Steps II through VI will be completed using information contained in previous sections of this chapter and certain other assumptions (hopefully reasonable) that are necessary to complete the example.

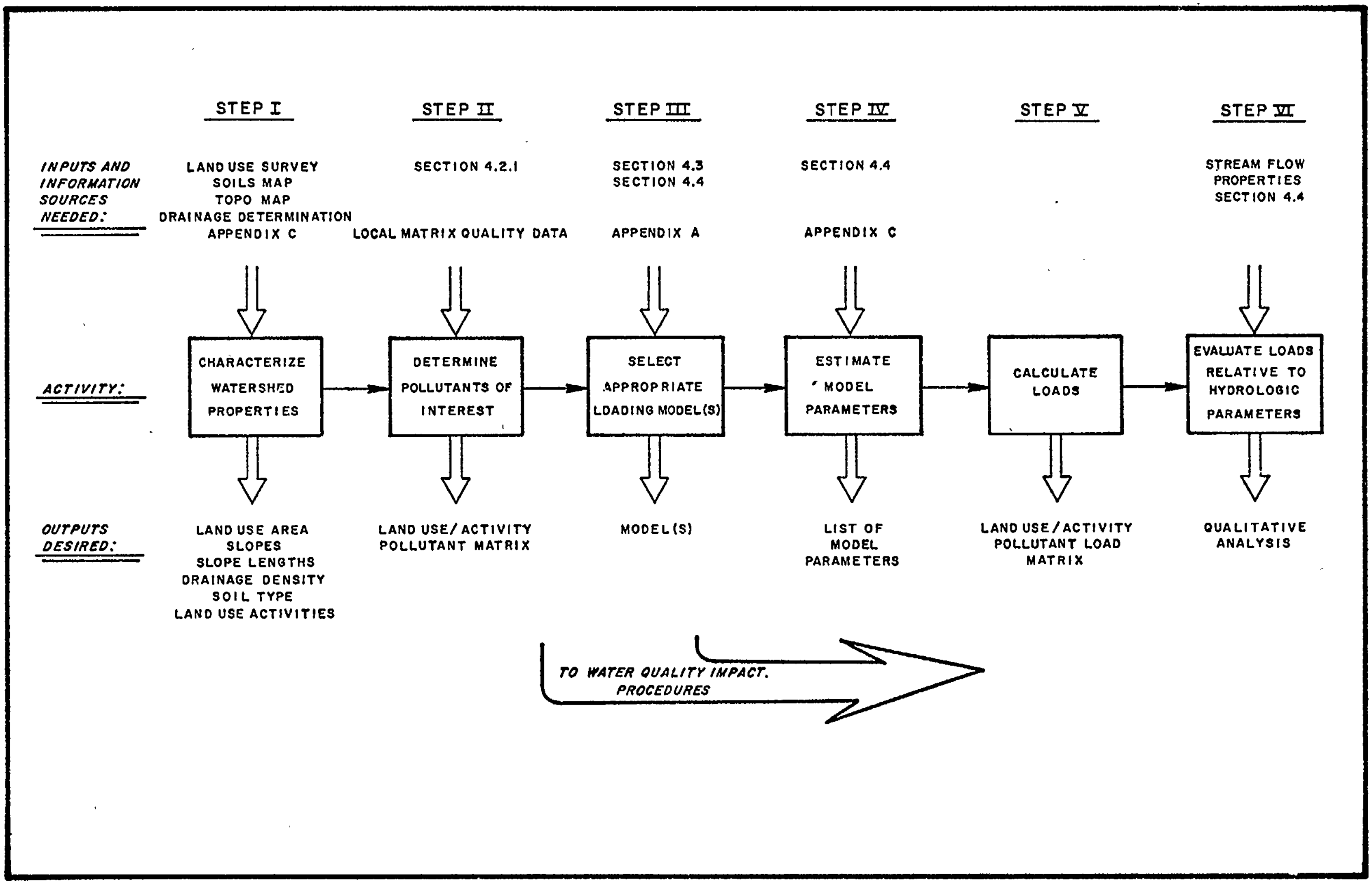


FIGURE 4-12

SCHEMATIC REPRESENTATION OF NPS LOAD ASSESSMENT METHODOLOGY

4.5.2 Example Case Study

The methodology shown in Figure 4-12 will now be illustrated such that the "problem" defined above can be solved. Each step will be considered separately.

Step I: The study area chosen is a hypothetical watershed located in central Indiana, cleverly referred to as Indiana Creek Watershed. A schematic of the watershed, complete with land uses noted, is given in Figure 4-13. Vital statistics were obtained from the county and state Soil Conservation Service, a recent land use survey, and standard USGS topographical maps. Analysis of these data resulted in Table 4-25. Note that the slopes and soil types are amazingly uniform.

Step II: The objective of step II is a listing of the potential pollutants arising from the land uses and land use activities on the watershed. From Tables 4-1 through 4-7, a new matrix was developed for Indiana Creek as shown in Table 4-26. Additional information from water quality surveys suggests nutrients and organics (BOD_5) are the most serious problems.

Step III: Normally, specific loading models should only be selected after careful consideration of the known water quality problems, analysis of the available resources, and the importance (economic or political) of the decision to be made with the resulting information. Such an effort should never be taken lightly. Fortunately, the task in this case is straightforward. Loading functions developed by MRI which are based on equation (4-5) will be used. Utilization of this approach permits desk-top calculations. Equations for each pollutant of interest in Table 4-26 are listed below.

Sediment, tons/year

$$Y(S)_E = \sum_{i=1}^{3} A_i (RKLSCP)_i \, S_d \tag{4-8}$$

Total Nitrogen, pounds/year

$$Y(NT) = \sum_{i=1}^{3} a \cdot Y(S)_{E_i} \, C_s(NT)_i \, r_N \tag{4-9}$$

Available nitrogen, pounds/year

$$Y(NA) = f_N \, Y(NT) \tag{4-10}$$

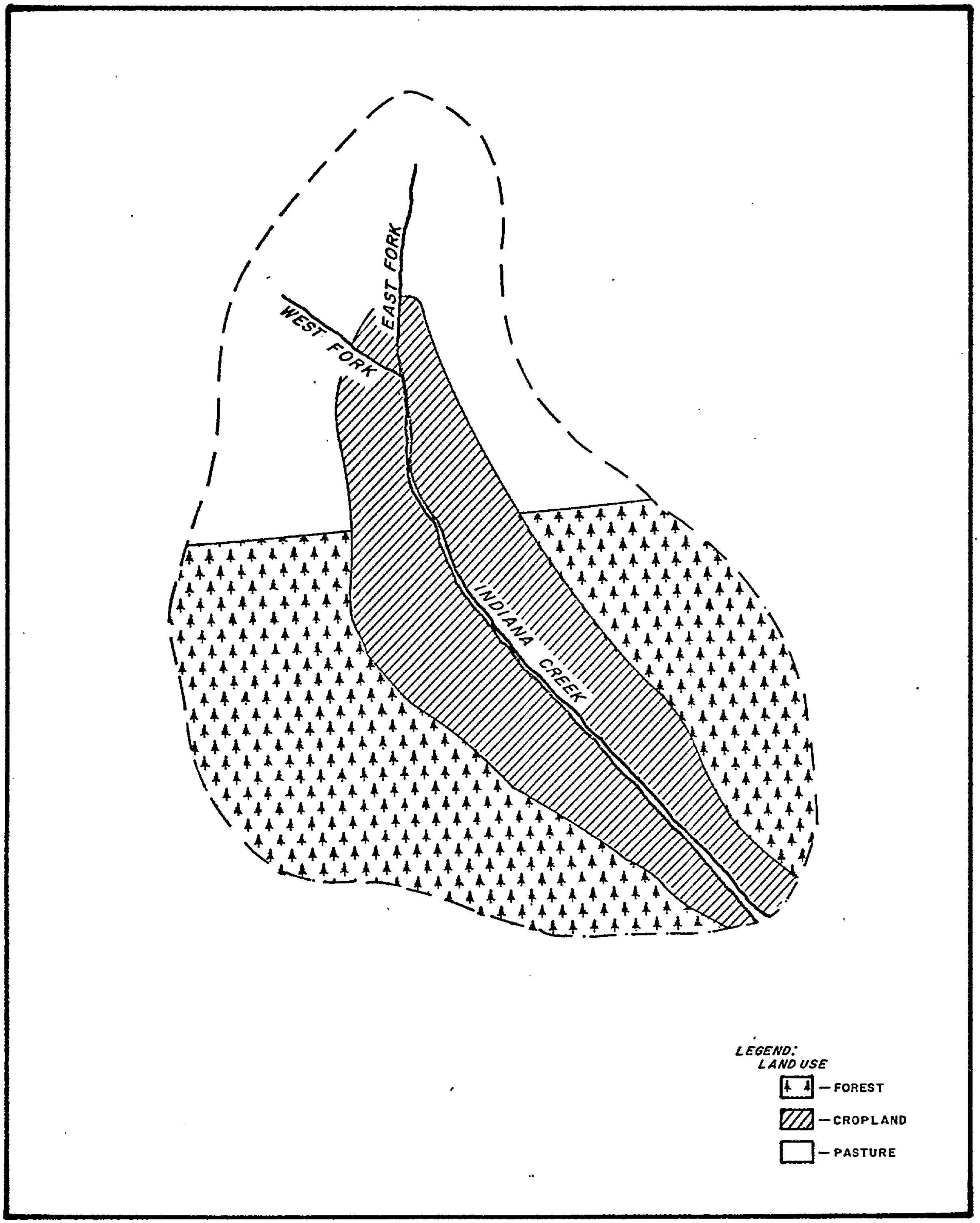


FIGURE 4-13
HYPOTHETICAL INDIANA CREEK WATERSHED

TABLE 4-25

SUMMARY OF INDIANA CREEK WATERSHED PROPERTIES

Watershed Properties	Land Use Cropland	 Forest	 Pasture	Totals
Area, ac	280	450	270	1000
Predominant slopes, %	6	15	8	-
Predominant slope length, ft	300	120	200	-
Soil type	Fayette silt loam, 4% organic matter	Silt loam	Fayette silt loam, 4% organic matter	-
Total stream segment length, mi	-	-	-	1.0
Land use activities	Continuous corn, contour plowing, conventional tillage, yield 80-90 bu/ac, residue left	Medium stocked, 50% tree canopy cover, 80% litter cover, managed undergrowth	80% ground cover	

TABLE 4-26

SUMMARY OF POTENTIAL POLLUTANTS IN INDIANA CREEK DRAINAGE

Land Use Activity	Potential Pollutants Sediment	 Nitrogen	 Phosphorus	 Organics (BOD_5)	 Pesticides
Cropland	X	X	X	X	X
Pasture	X	X	X	X	
Forests	X	X	X	X	

Total phosphorus, pounds/year

$$Y(PT) = \sum_{i=1}^{3} a \cdot Y(S)_{E_i} \, C_s(PT)_i \, r_P \tag{4-11}$$

Available phosphorus, pounds/year

$$Y(PA) = f_P \, Y(PT) \tag{4-12}$$

Organic matter, pounds/year

$$Y(OM) = \sum_{i=1}^{3} a \cdot Y\ (S)_{E_i} \, C_s(OM)_i \, r_{OM} \tag{4-13}$$

BOD_5, pounds/year

$$Y(BOD_5) = f_B \, Y(OM) \tag{4-14}$$

where i = land use: cropland = 1, pasture = 2, forest = 3

A = land area for land use i, acres

RKLSCP = factors of USLE, see equation (4-3)

S_d = sediment delivery ratio

f_N, f_P f_B = availability constants for nitrogen, phosphorus, and organic matter, expressed as fraction of total that is available

r_N, r_P r_{OM} = enrichment ratios for nitrogen, phosphorus, and organic matter

$C_s(N)$, $C_s(P)$, $C_s(OM)$ = soil profile concentrations of nitrogen, phosphorus, and organic matter, gm/100 gms

a = 20

Of the pollutants listed in Table 4-26 for Indiana Creek, only pesticides were omitted in the above list of loading functions. This in no way implies pesticides are of no real concern and should be omitted. Rather, it reflects current inability to adequately assess pesticides except at a rather detailed level quite beyond the scope of this manual and, in particular, this example. Also, none of the water quality impact analyses given in this manual include pesticides. A more qualitative analysis of the pesticide problem will be given in step VI.

Step IV: Equations (4-8) through (4-14) contain a number of parameters that must be estimated. Some represent properties of the entire watershed while others are land use or pollutant specific. Each set of parameters will be selected based on the material given in previous sections.

Sediment - The sediment delivery ratio is determined from Figure 4-7. Note from Table 4-25 that the watershed has an area of 1000 acres (1.56 square miles) and a stream length of 1.0 miles, yielding a drainage density of 0.64. The inverse drainage density is 1.56. The soil type is silt loam which is predominately silt. The resulting S_d value from Figure 4-7 is 0.50. The R value in equation (4-8) is common to the entire watershed and is estimated to be 175 as interpolated from Figure 4-4. The K value is also assumed constant for the watershed since each land use has the same soil type and from Table 4-17 is estimated to be 0.33.

The remaining sediment-related parameters are land-use specific and are estimated as follows:

Topographic factors from Figure 4-6:

Cropland - LS = 1.2
Pasture - LS = 1.4
Forest - LS = 2.8

Cover and Management factors, from Tables 4-18, 4-19, 4-20:

Cropland - C = 0.38
Pasture - C = 0.013
Forest - C = 0.003

Erosion Control factors, from Table 4-21:

Cropland - P = 0.50
Pasture - P = 1.0
Forest - P = 1.0

Pollutants - The parameters associated with pollutants other than sediment are included in equations (4-9) through (4-14). Assuming no local soil surveys, appropriate values are abstracted from Figures 4-10 and 4-11, and Tables 4-24 and 4-25.

From Figures 4-10 and 4-11, the concentrations of nitrogen and phosphorus (calculated from $P=0.22xP_2O_5$) in the soil are:

$C_s(NT)$ = 0.145% (mid-point of range)
$C_s(PT)$ = 0.032% (mid-point of range)
$C_s(OM)$ = 4.0% (from Table 4-25)

Note that $C_s(OM)$ is based on the given soil characteristics and exceeds the suggested value based on $C_s(NT)$. A value of 20 times $C_s(NT)$, or 2.9, would result from the guidance in section 4.4.5

From section 4.4.5
f_N = 0.06
f_P = 0.10

No guidance was given on the relationship of BOD to organic matter so a value of f_B = 0.06 was arbitrarily chosen.

From section 4.4.5
r_N = 2.0
r_P = 1.5
r_{OM} = 2.5

The summary of the selected model parameters for the Indiana Creek Watershed is given in Table 4-27.

Step V: Calculation of loads require application of the parameters of Table 4-27 to equations (4-8 through 4-14). Calculations by land use are illustrated as follows:

Cropland -

$$Y(S)_E = 280 \cdot 175 \cdot 0.33 \cdot 1.20 \cdot 0.38 \cdot 0.5 \cdot 0.5$$
$$= 1843 \text{ tons/year}$$
$$= 10{,}100 \text{ pounds/day, sediment}$$

$$Y(NT) = 20 \cdot 1843 \cdot 0.145 \cdot 2.0$$
$$= 10{,}689 \text{ pounds/year}$$
$$= 29.3 \text{ pounds/day, total nitrogen}$$

$$Y(NA) = 0.06 \cdot 29.3$$
$$= 1.76 \text{ pounds/day, available nitrogen}$$

TABLE 4-27

SUMMARY OF LOADING FUNCTION PARAMETERS FOR INDIANA CREEK WATERSHED

	Land Use		
Parameter	Cropland	Pasture	Forest
S_d	0.5	0.5	0.5
R	175	175	175
K	0.33	0.33	0.33
LS	1.20	1.40	2.80
C	0.38	0.013	0.003
P	0.50	1.0	1.0
C_s(NT)	0.145	0.145	0.145
C_s(P)	0.032	0.032	0.032
C_s(OM)	4.0	4.0	4.0
r_N	2.0	2.0	2.0
r_P	1.5	1.5	1.5
r_{OM}	2.5	2.5	2.5
f_N	0.06	0.06	0.06
f_P	0.10	0.10	0.10
f_B	0.06	0.06	0.06

$Y(PT) = 20 \cdot 1843 \cdot 0.032 \cdot 1.5$

= 1769 pounds/year

= 4.85 pounds/day, total phosphorus

$Y(PA) = 0.10 \cdot 4.85$

= 0.49 pounds/day, available phosphorus

$Y(OM) = 20 \cdot 1843 \cdot 4.0 \cdot 2.5$

= 368,600 pounds/year

= 1010 pounds/day, organic matter

$Y(BOD_5) = 0.06 \cdot 1010$

= 60.6 pounds/day, BOD_5

Pastures - Similar calculations for pastures yield:

$Y(S)_E$ = 777 pounds/day
$Y(NT)$ = 2.3 pounds/day
$Y(NA)$ = 0.14 pounds/day
$Y(PT)$ = 0.4 pounds/day
$Y(PA)$ = 0.04 pounds/day
$Y(OM)$ = 78 pounds/day
$Y(BOD_5)$ = 4.7 pounds/day

Forests - Finally for forests:

$Y(S)_E$ = 598 pounds/day
$Y(NT)$ = 1.7 pounds/day
$Y(NA)$ = 0.1 pounds/day
$Y(PT)$ = 0.29 pounds/day
$Y(PA)$ = 0.03 pounds/day
$Y(OM)$ = 60 pounds/day
$Y(BOD_5)$ = 3.6 pounds/day

A summary of the results of the loading calculations are given in Table 4-28 using the land use-pollutant matrix format.

Step VI: The most meaningful way to predict NPS loads includes prediction of runoff as the transporting medium. The USLE-based loading functions only implicitly include hydrology through the R value. Expansion of the assessment procedure in this chapter to include a quantitative representation of

TABLE 4-28

SUMMARY OF POLLUTANT LOADINGS FOR INDIANA CREEK WATERSHED

	Pollutants, lb/day						
	Sediment	Nitrogen		Phosphorus		Organic Matter	
Land Use Activity		Total	Available	Total	Available	Total	BOD_5
Cropland	10,100	29.3	1.76	4.9	0.49	1010	60.6
Pasture	777	2.3	0.14	0.4	0.04	78	4.7
Forest	598	1.7	0.10	0.3	0.03	60	3.6
Total Loading	11,475	33.3	2.00	5.6	0.56	1148	68.9

the watershed hydrology would require a quantum jump in both effort and sophistication. Such an effort is recommended as a mandatory step in any comprehensive analysis, but is not included here.

A qualitative analysis of the watershed hydrology can be accomplished within the ground rules assumed by this manual and is helpful in evaluating the timing and potential impacts of selected pollutants. Recall from section 4.4.5 that the MRI loading functions can be used to predict the time-distributed load by one of two methods. First, assume all equation factors remain constant except for R and the loads are linearly distributed in time as a function of R. Second, allow the R value to be weighted according to the time-varying cover conditions represented by C. The R-distributed and the RC-distributed curves are given in Figures 4-8 and 4-9, respectively.

The superposition of time-varying land use activities, watershed system inputs, and streamflow characteristics on Figure 4-9 provides a qualitative assessment of the interaction of the major system components including the hydrology. Such an analysis for purposes of general "enlightenment" of planning agency personnel has been proposed (23) and will be included here.

A stream-gaging station downstream of Indiana Creek Watershed was selected to represent the hydrologic characteristics of the area. Long-term flow records were abstracted from published flow data and plotted as the annual average monthly distribution in percent of total annual value as shown by curve Q in Figure 4-14. Curve RC is the same as that of Figure 4-9 but expressed as percent of total annual. That is, each monthly value has been multiplied by the number of days in that month. The Q curve requires some further explanation. If the curve was generated from data taken in Indiana Creek, the response would more directly trace the rainfall events imbedded in the RC curves. Downstream, however, the curves can interact as shown because of the areal variability of rainfall and the extraction of water via evapotranspiration. Plotted beneath the two curves are the time "windows" during which certain land use activities and man-induced inputs to the watershed occur. Only the cropland system is represented because the RC curve was calculated

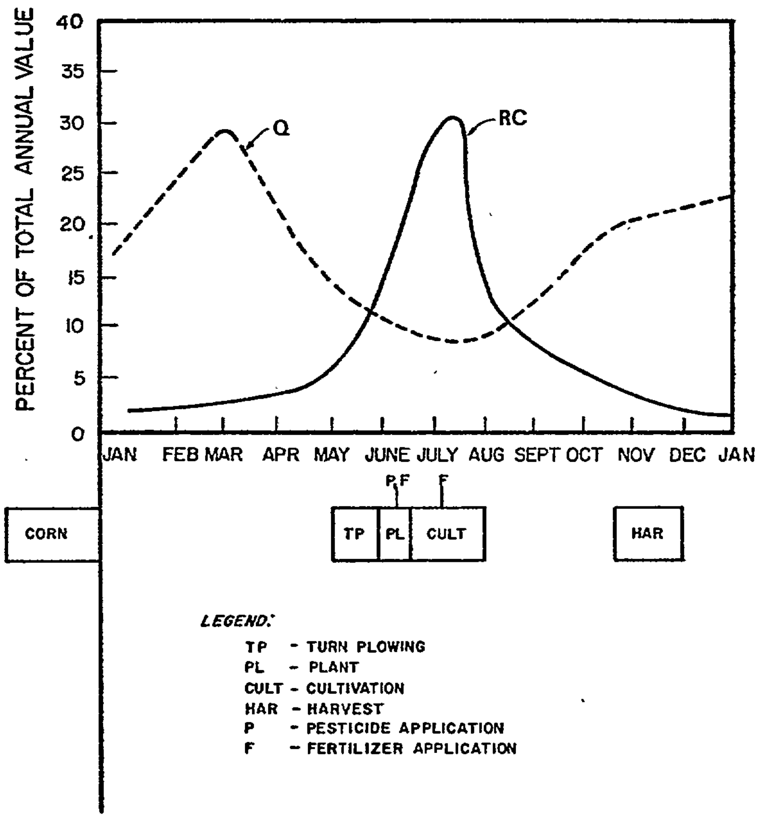


FIGURE 4-14
INTERRELATION OF NON-POINT SOURCE
LAND USE AND HYDROLOGIC TRENDS

using the cropland data. If forestry activities were included a longer time period would be more appropriate.

Note from Figure 4-14 that during the period from May 15 to August 15 several watershed operations occur that can interact to produce nonpoint source loads. The soil surface is essentially bare during a time of highly erosive rainstorms (denoted by RC curve) and relativey low stream flow. Also, the addition of potential pollutants (fertilizers and pesticides) occurs during the same time period. In addition to an evaluation of potential loads, Figure 4-14 suggests time during which field measurements of stream water quality will be most meaningful.

4.6 Specific Nonurban, Nonpoint Source Category Information

As discussed in earlier sections, nonpoint sources of pollutants include nonurban land use activities such as construction, agriculture (including irrigation return flows), silviculture, hydrologic modifications, mining, and residual management practices. In this section of the manual, these specific source categories are discussed individually. The primary intent is to familiarize the planner with the nature of the NPS load-generating potential of each category, the types of pollutants which can be anticipated, and with control measures or management practices which have been identified to be appropriate and which may be considered in the planning process.

The discussion covers each of the categories broadly, and identifies specific documents or publications which may be referred to by the planner to develop the in-depth information and guidance which will be required to address those NPS problems which are important in his planning area.

The primary reference publications for each NPS category include a series of EPA documents, either published or to be released in the near future. These include:

(a) EPA Office of Research & Development Reports - These are guideline manuals which provide technical information relating to specific land use categories or types of activity.

(b) EPA 304(e) Documents - These provide general information and background on identification of NPS pollution and on potential control measures.

(c) EPA Water Planning Division - This set of documents is presently being issued. Separate manuals for each NPS category address the Best Management Practices (BMP) which have been identified as pertinent to that category. They are designed to assist the planner interface effectively with technical specialists in identifying and assessing the significance of NPS problems from various categories. BMP documents will cover agriculture, silviculture, construction, hydrologic modifications, mining and residuals management.

These sources of information, along with numerous other reported studies, yield a valuable data base for continuing analysis and implementation of control programs. The discussion provided in the balance of this chapter, together with both the cited references and the secondary references which can be developed from their review, can provide the planner with a starting point and the framework for a planning effort appropriate to his study area, for addressing NPS pollutants that are significant in his area.

4.6.1 Agriculture (Excluding Irrigated Agriculture)

Nonpoint source pollutant problems resulting from agricultural activities are difficult to reduce to a simple set of specific sources, loads, controls, and implementation. Problems arise because of the pervasiveness of activities within watersheds; the almost infinite array of activities/practices possible; the changes in practices which can usually occur annually; and the diversity of chemicals, application rates, and methods available to individual producers. The conjunctive use of agricultural lands for crop production and disposal (or utilization) of animal wastes is yet another spectrum of activities having potential for loadings which require a specific set of controls. These factors exist, unfortunately, in addition to the normal variations associated with the hydrologic system described in Section 4.2.2

4.6.1.1 Activities and Practices

The control of nonpoint source pollutant loadings is different from control of point sources in one key way - control is obtained only through source management and cannot be achieved by only knowledge of effluents (runoff). This fact requires the analyst to become familiar with each land use activity before any realistic problem assessment or subsequent control program can be developed. Comprehensive analysis requires a definition of activities more detailed than those in Table 4-3. No complete description of each activity for each crop and for every part of the country exists. Several reports are available, however, that describe cultural systems for specific crops (36-39). These references, in addition to local information available from state universities, provide excellent overviews of the mechanics for specific agricultural crop production. For example, references on wheat production describe the nature and timing of field operations, fertilizer and pesticide application recommendations, compatible rotations, etc., in use where wheat is grown.

4.6.1.2 Pollutants

The knowledge of the use and scheduling of these activities permits an evaluation of pollutant-generating practices, the delineation of pollutants, and most importantly, an insight into feasible management practices for control. The advantages of having a general knowledge of agricultural production are considerable when implementation of controls is desired.

Certain activities are common to all crops and can be considered collectively. Most notable among these (related to nonpoint source pollutants) is the application of fertilizers and pesticides. Several references are available that give detailed descriptions of pesticide properties, intended use, toxicity data, persistence, and relative mobility in soils (40).

Similarly, fertilizer technology and evnironmental behavior has been reviewed in recent publications (41, 42). Some of these references are production-oriented, but the basic data give useful insights to potential problems and their control.

Data for the distribution of various crops throughout the country and the associated planting and harvesting dates have been compiled and reported in a USDA Handbook (43). Listings by crop and geographical areas are included. The resulting time "windows" for planting and harvesting also suggest times when chemical applications are made and pollutant-loading analyses, similar to that illustrated by Figure 4-14, can be completed. More comprehensive analyses (e.g., via ARM, ACTMO) also require information of this type.

In addition to crop production management, agricultural waste management is an activity which is requiring more attention as semi-confined and concentrated livestock and poultry feeding increases. In some situations these waste sources are technically (and legally) considered to be point sources of pollution and, therefore, subject to the appropriate point source discharge permit regulations. Point source controls notwithstanding agricultural lands remain the ultimate recipient of solid and liquid wastes and nonpoint source load potentials must be evaluated. Two key EPA reports, now in preparation and soon to be published, describe livestock operations, wastes generated, and available controls (44, 45).

4.6.1.3 Control Practices

Chapter 4 has thus far been devoted to a discussion of NPS loading assessment. However, the specification of needed controls is the ultimate objective of a loading assessment. One of the most complete and useful source of information currently available for the selection of agricultural pollution controls is the two-volume report jointly prepared by the USDA and EPA referred to several times previously (2, 4). This manual points out controls available for erosion, runoff, nutrients, and pesticides. A methodology for selection of control practices, complete with flow charts and examples, is given in Volume I. A detailed interpretive review of existing literature is given in Volume II. These volumes are complete, have been designed specifically for use in development of nonpoint source control plans, and represent the state-of-the-art for agricultural sources. They are recommended for use in the development of water quality management planning. Similarly, the manuals for agricultural waste management, which are being prepared (44, 45) have been specifically designed to facilitate their use in environmental planning and decision-making.

Additional references (11, 13, 14, 15, 16, 20, 27, 32, 35) potentially useful in problem assessment, selection of controls, and implementation of plans have been previously discussed and will not be addressed here.

4.6.2 Silviculture

The major differences between agricultural and silvicultural activities are their frequency and areal extent within a given watershed. An agricultural rotation can extend from one to five years, while silviculture cycles are keyed to tree growth and extend from 20 to 80 years. Land use activities impacting nonpoint source loading, therefore, occur over relatively short time periods followed by a much longer recovery period. For example, clear-cutting to remove vegetation is a drastic change in the watershed, but recovery (if proper management steps are taken) is relatively rapid.

4.6.2.1 Activities and Practices

The broad categories suggested in Section 4.2.1.2 apply to silvicultural systems nation-wide. The U.S. Forest Service has further defined these activities in a joint Forest Service-EPA report (46). The major activities are categorized and defined as follows:

1. Vegetative manipulation by mechanical means: Any activity (excluding timber harvest and road construction) that uses mechanized equipment to alter or remove vegetation is included in this category. Mechanical site preparation for reforestation after timber harvest or for species type conversion is the most significant operation. Because such practices are somewhat limited by ground slopes, the most widespread use of these techniques is in the lower elevations common to the southern and southeastern United States. Dissmeyer (47) has described several treatments including chopping; chopping and burning; chopping, burning, and bedding; KG-blading; windrowing and burning; KG-blading, windrowing, burning, and bedding; and bulldozing, windrowing and burning. The area subjected to these treatments varies widely and depends on the size of cut or extent of type conversion desired.

2. Road and trail construction and maintenance: Access systems are a necessary part of any timber harvest system and include a variety of permanent roads, spur roads, skid trails, and landings. Some are continually maintained, while others are returned to natural vegetation

after harvest is complete. The pollutants most characteristic of silviculture activities are those associated with road construction and maintenance. The EPA-FS report cited previously (46) is an excellent overview.

3. Fire: Fire management systems include the use of fires for undergrowth control, slash disposal, and wildifire control. Wildfire itself can expose vast areas to erosive and leaching processes.
4. Grazing: Although not a silvicultural activity, grazing may constitute a major land use within timber areas. The pollutants are similar to those in agricultural activities.
5. Timber harvest: Only those activities directly associated with cutting and removal of trees are considered here. Access roads and reforestation, integral parts of harvesting operations, have been discussed separately. Various harvesting systems, extent of areal disturbance, and pollutants have been described (48, 49).
6. Recreation: Multiple use of forest lands is a common practice on public lands. Camping, recreational travel, fishing, and hunting can result in problems ranging from waste disposal to site maintenance.
7. Chemical use: Intensive timber management often includes applications of fertilizers and pesticides. Forest fertilization of established tree stands is a growing practice in northwest and southwest. Although less is known about pesticide usage, both insecticides and herbicides are used. A recent review of silviculture chemicals usage in the northwest is available (50).

4.6.2.2 Pollutants

The major impact exerted on the forested watershed results from the exposure of soil surface to erosive and leaching processes. The major pollutant (by volume) is sediment. Some operations, particularly timber harvest, increase water yield, accelerate oxidation of organic matter, and result in increased levels of plant nutrient discharge. The most comprehensive review of these problems and a description of the available data for each is included in the EPA-FS report (46).

4.6.2.3 Control Practices

Control of pollutants generated by silviculture is achieved through source management. A comprehensive selection methodology for controls is being developed. Several reports (46, 48, 51) describe control measures and further elaborate on specific activities listed above.

4.6.3 Construction

Construction activity is estimated to influence about 1 million acres (somewhat less than 0.2% of the total land area of the U.S.) at any one time. This includes construction for housing, highways, dams, and the like. The major pollutant resulting from construction is sediment. Although gross yields are relatively small compared with other NPS categories because of the smaller area affected, site-specific loads can be high and localized adverse impacts often result.

Several elements make control or reduction of pollutants from construction activity more practical than from other NPS activities. This results from (a) the relatively localized nature of the activity, (b) the on-site concentration of men, facilities and equipment during the construction process and (c) the ability to plan or modify specific activities in order to minimize pollution potential.

Because the occurrence of runoff is a function of the local climatic events which can be highly variable, pollutants from construction sites can change drastically and unpredictably. Also, the nature and quantity of pollutants depend upon the project phase, soil characteristics, local topography, and geologic conditions, the magnitude of people and equipment involved, extent of protective vegetative covering on the site, and other factors.

Control measures for this category will be directed at (a) minimizing the generation of pollutants, and (b) minimizing the runoff at those which are generated.

4.6.3.1 Activities and Practices

In analyzing the potential for the generation of pollutants from this NPS category, it is useful to make a distinction between construction activities, and construction practices. Activities are characterized in such a way that a distinction is made among (a) transportation and communication networks, (b) housing, (c) energy networks (power plants and transmission lines), (d) water resource development (dams, canals), and (e) recreational developments (parks, ski slopes). Generally, all will give rise to nonpoint pollution. However, basic differences in area affected, proximity to water sources, and predominant construction practices employed make it likely that specific problems and controls will be more or less consistent within an activity and different from other activities.

Construction practices may be defined as specific categories of job operations - for example, clearing and grubbing, grading, construction of facility, and site restoration. For any type of activity, each of the practices have a particular potential for the generation of pollutants. The potential for both the type of pollutant and the magnitude of the load differ with the particular practice, so that this characterization provides a useful frame of reference within which to assess pollutant loads and control measures.

Construction practices refer to the individual jobs performed in a construction project over a period of time. Each practice has a different potential to produce pollutants.

Clearing, grubbing, and pest control are construction practices typical in the first stage of a construction activity. During these operations unwanted vegetation including trees, bushes and sod is removed from the original site. In addition, herbicides may be applied to remove unwanted vegetation from the area.

Rough grading is the next step in most construction projects. During this phase the soil is moved from one place to another to obtain desired surface elevations. Grading may involve cutting and filling of slopes for highway projects, excavating or filling for buildings, or excavating for pipeline or canal installations. Heavy machinery used in this practice can be responsible for creating substantial amounts of petroleum waste products. In addition, the effects of moving and disturbing soils and subsoils can alter their physical characteristics to make them easily eroded when impacted by rainfall.

Rough grading is followed by final grading and facility construction. The constructed facility is different for transmission structures, highways, buildings and dams, but it is at this time that all construction sites encounter much physical activity. Equipment and machinery are in use, products and materials are on the site and workers are the most active. Pollutants generated during facility construction include chemicals, trash, sanitary wastes, cement and trace metals.

The final construction practice is that of site restoration. This includes cleanup of the site, final grading in some areas, tilling, establishment of permanent vegetation, removal of temporary structures and any other activities that restore the site to a balanced landscape. It is principally at this time that fertilizers are applied and nutrient runoff can be generated.

4.6.3.2 Pollutants

Sediment is the principal pollutant by weight that can result from construction. During a precipitation event, soil particles are eroded, transported, and deposited at other locations. Sediment may be deposited on land at the bottom of steep slopes or transported to a water body. Once in the water body, the soil particles may be further transported to areas of deposition where the stream velocity is less than that required to keep the sediment particles in suspension.

Pesticides are another type of nonpoint source pollutant. They include insecticides, herbicides, and rodenticides. These chemical compounds may adhere to the soil particles or dissolve in the runoff water and be transported off the construction site during runoff events.

Other nonpoint pollutants include petroleum products such as gasoline, diesel fuel, oil, grease and solvents. Some oils are used for dust control which requires that they be applied directly to the soil. These products become pollutants through their general use and misuse on the construction site.

Other chemicals may be used on the construction project for site preparation, facility construction, or for site restoration. Chemicals used to develop desirable soil properties are lime, fly ash, asphalt, phosphoric acid, salt and calcium chloride. Additional chemicals used during construction include pastes, oils, paints, solvents, dying compounds, cleaning compounds, and concrete curing compounds.
In addition, the construction site usually contains an excess of damaged building products, packaging and other miscellaneous garbage. Garbage can take the form of food containers, cans, coffee containers, empty cigarette packs, and general refuse. If they are not collected, they may eventually be transported to the nearest stream or water body.

Sanitary wastes are generated on construction sites by the work force. This type of pollution is generally controlled by using portable facilities. However, sanitary wastes may still reach the water bodies and be a source of pathogenic organisms.

Metals are used extensively for most types of construction projects. When pipes, beams, wire mesh etc., are left exposed to weather conditions, they eventually oxidize. During precipiation events, surface runoff may transport these oxides off the construction site to the stream.

Cement can become another source of water pollution if loose materials are not stored in a dry location out of the weather. The most common source of pollution from cement results from the washing of cement transporting vehicles or batching facilities. This washing is generally done in a batching plant area or where cement is to be placed often without regard for closeness to a stream.

Nitrogen and phosphorus compounds are used at construction sites for fertilizers. Some of these compounds may adhere to soils and can be washed to adjacent streams during runoff events. In addition, nutrients can dissolve in water and pollute both surface waters and groundwater.

4.6.3.3 Control Practices

Best Management Practices on a construction site are measures to control erosion, sediment and stormwater runoff. Each method is designed to minimize the pollutants being transported off the site as a result of a runoff event.

Erosion control measures prevent the transport of soil particles. These practices reduce both energy and velocity of the runoff thereby reducing its erosive powers. Erosion controls minimize soil exposure, control runoff, shield the soil and bind the soil.

A method to reduce erosion is surface roughing of slopes. Roughing includes tracking and scarification of slopes to slow the movement of runoff. Diversion structures such as soil or stone dikes, ditches, terraces and benches may also be used to control runoff by diverting runoff from sensitive areas. When diversion structures are used they may be used in conjunction with disposal structures which transport the diverted runoff without causing further pollution. Disposal structures include flexible downdrains, sectional downdrains, flumes, and level spreaders.

Vegetation can also be used to reduce erosion by protecting the ground surface from rainfall impact, binding the soil particles in place, and filtering out sediment which may be transported. Temporary stabilization is attained by planting fast growing annual and perennial plants, and provides short-term protection during construction delays or until permanent vegetation has become established. Permanent stabilization involves the planting of permanent vegetative materials such as grasses, legumes, vines, shrubs, native herbaceous plants, and trees.

Soil stabilization on a temporary and or permanent basis can also be attained through the use of nonvegetative controls. Nonvegetative temporary stabilization practices include the use of mulches, nettings and chemical binders which hold the soil in place or protect it from rainfall energy. More permanent controls include gravel slopes, retaining structures, bank protection, and instream grade stabilization structures.

Control of pollution from construction sites also involves sediment control. Sediment controls prevent sediment transport off the constuction site. These controls include both filtering and ponding of runoff.

Filtering of surface runoff can be effected by vegetation. Common vegetative filters include natural buffer strips, installed buffer strips, center strips, and soil inlet filters. These filters remove sediment and debris from the runoff. The filters also slow the runoff thereby reducing its erosive capability.

Structural devices often are used in sediment control to intercept and detain runoff allowing the sediment to settle out, and to remove large debris. Structural devices are designed for the lifetime of the construction project or may be permanent and continue to operate after construction is complete. These devices include gravel inlet filters, sediment traps, and wet and dry sedimentation basins.

Although sediment is the major pollutant by weight generated as a result of construction activity, the other pollutants discussed earlier in the section must also be controlled. Partial control would be accomplished through the use of sediment control practices identified in this section. However, total removal of all other pollutants could be attained only by appropriate wastewater treatment, and would be expensive. The best methods to control these other pollutants are good housekeeping practices. These practices are not expensive to implement, however, they do require care and awareness by the workers, supervisors, engineers and planners. These practices are discussed in detail in the literature (16).

Good housekeeping practices involve the proper application of materials such as nutrients and pesticides when necessary. They also include proper disposal of solid and sanitary wastes. Finally, the construction activity should use the most effective type and amount of materials and properly dispose of the unused materials along with the other solid wastes.

Although the need for controlling runoff from a construction site has gained a degree of recognition, that of controlling excess runoff after development has not been fully recognized. Replacing natural open spaces by paved areas and buildings lowers the overall water infiltration capacity of a site which, in turn, leads to increased stormwater runoff.

When stormwater enters directly into a stream, the increased velocity may cause accelerated erosion of the stream channel. Where this runoff enters existing sewer systems, the system capacity may not be adequate to treat the increased flow and this could result in the discharge of poorly treated or untreated wastewater to the stream.

Stormwater management methods provide means to controlling runoff, reducing stream channel erosion and sediment discharge, and the release of pollutants contained in the stormwater runoff. Among those methods is the temporary storage and regulated release of runoff from small storms.

The following is a list of typical stormwater management practices which have been used in urban areas or on construction sites in non-urban areas:

1. Roof Top Ponding
2. Parking Lot Ponding
3. Diversion Structures
4. Ponds
5. Retention Basins
6. Porous Pavement
7. Holding Tanks
8. Infiltration Systems
9. Stream Channel Storage and Control
10. In-Line Sewer Storage

The best water pollution abatement plan is one which minimizes or prevents erosion, sediment and runoff damages. No one single management practice can fulfill this task. An adequate water pollution abatement plan will contain many of the control practices discussed in this section. In addition, the best plans are based on a site inspection of the area and a data evaluation. If evaluated early enough, a construction activity or project may be adjusted or relocated depending on the sensitivity of the area. Any plan evolved should include feedback to allow for adjustments depending on the field activities.

4.6.3.4 Annotated Bibliography

1. "Methods for Identifying and Evaluating the Nature and Extent of Nonpoint Source Pollution," Environmental Protection Agency Publication 430/9-73-014 (October 1973)(52).

This report includes a discussion of construction activities as one of the major nonpoint sources of pollution significantly influenced by the commercial activities of man.

Estimates are presented of the land area influenced and erosion rates for construction relative to agriculture, silviculture and mining. Significant pollutants likely to be contributed by each source are discussed.

Approximately 30 pages, devoted to the discussion of construction as a source of nonpoint pollution, provide a concise summary of types of activities and types of practices which can influence the potential for generating NPS loads, the type of pollutants and methods of pollutant transport. The report distinguishes between construction activity and construction practice. It characterizes five general classes of activity, each of which could be expected to influence NPS load generation in a different manner and possibly dictate different broad approaches to problems. For any of the types of activity (e.g. housing, dams and canals, etc.), a series of construction practices are identified and their relation to NPS load generation are discussed. Potential pollutants which could be generated from construction activities are identified and their source and potential significance are discussed.

The section of this report dealing with construction can provide the 208 planner with a good overview and perspective on the potential for pollution from construction activities.

2. "Processes, Procedures, and Methods to Control Pollution Resulting from All Construction Activity," Environmental Protection Agency Publication 430/9-73-007 (October 1973) (53).

This report presents information on processes, procedures and methods for controlling sediment, stormwater, and pollutants other than sediment which result from construction activities. The processes examined include site planning, preliminary site evaluation and design, use of planning tools, and structural and vegetative design considerations. The procedures examined include relative processes at Federal, State and local levels necessary to control land disturbing activities. Methods discussed include on-site erosion sediment and stormwater management control structures, as well as soil stabilization practices.

This report emphasizes planning as an essential element in the control of nonpoint pollution originating from construction sites. A pollution control plan should be an integral part of the project and should start as early as the site selection. In addition, control of pollution as well as the other engineering factors should control the timing of certain construction activities. The project design should also include plans to protect the environment, such as the installation of temporary stream crossing structures prior to construction, proper disposal of petroleum wastes, etc.

The major section of the text discusses the on-site methods which are available for the control of erosion, sediment, and stormwater. Structural, nonstructural and vegetative control techniques are addressed. Sketches and photographs of a variety of pollution control devices are provided although detailed design considerations are not covered. It provides a useful introduction to the devices which are available and discusses how these devices are useful in the control of nonpoint pollution.

For the planner, this report will be useful for obtaining background information on the processes and methods available to control nonpoint pollution originating from construction activities. For an engineer, it emphasizes the importance of the total site evaluation and planning ideology, and can be used as a source of references to the more technical manuals which are available.

3. EPA, Nonpoint Source Pollution Control Guidance Construction Activity, (in Progress) (54).

The objectives of this report, presently in draft form, are to identify management practices which will reduce or control generation of pollutants from construction activities, and to provide guidance to 208 planner in evaluation or selecting measures which may be appropriate in this area.

The document contains four chapters. Chapter 1 discusses approaches to identifying and assessing the existence of problems. The discussion is general in nature and describes possible approaches. The approaches discussed include:

(a) making a general survey of existing and recently completed construction projects to determine the extent of pollution.

(b) examining water quality data (coincident with the construction activity) to determine if the activity was harmful to the waterbody.

This chapter also discusses the various physical observations which should be made to help indicate whether significant erosion has taken place. In addition, methods for making gross estimates of the quantity of eroded soils are presented in this chapter.

Chapter 2 discusses the procedures which may be helpful to make an analysis of the construction activity and its effect on the surrounding water bodies. The section is brief and very general and primarily identifies types of data which may be useful in the evaluation of nonpoint runoff and control and the sources from which it may be procured. Discussion of control approaches provides a broad overview, and essentially identifies the principles to be considered.

Chapter 3 discusses several of the techniques available for the control of erosion, sediment and stormwater. Erosion control practices include both vegetative and structural devices. Vegetative control practices are described in general terms while the engineering details are contained in cited literature. As a guide for the design of erosion, sediment, and stormwater control structures, some preliminary information and sketches are given. In addition, the references necessary for a more detailed examination and design of the control devices are presented in the text. Using the Universal Soil Loss equation to estimate sediment runoff, the final section in this chapter estimates the amount of sediment generated on a construction site after control devices have been added. Also, estimates of the changes in the input parameters of the soil loss equation with changes in control devices are made.

The thrust of Chapter 4 is a presentation of a methodology for the assessment of potential pollution problems and their magnitude. Sediment is the major pollutant addressed. Using the Universal Soil Loss Equation, sediment losses from areas without control devices are estimated. Charts and tables are presented to enable the users to calculate the USLE input parameters. Finally, references are given throughout the chapter directing the reader to more detailed information on the methodology.

This manual will assist both a nontechnical planner and an engineer in assessing and evaluating pollutants which result from construction activities. However, for a complete design or evaluation, it may be necessary to augment this information with data that is presented in the cited references.

4. "Comparative Costs for Erosion and Sediment Control, Construction Activities," Environmental Protection Agency Report 430/9-73-016 (July 1973) (55).

This report presents installation costs for certain erosion and sediment control devices which may be used to minimize nonpoint runoff from construction activities. This information is documented for more than 25 pollution control practices in current use in both the eastern and western United States. Most of the data presented were obtained from the Walnut Creek Watershed in central California and the Occoquan Watershed in Virginia.

The report describes erosion and sediment control structures. It includes photographs and sketches of the control structure being discussed. Detailed installation cost data is provided for the particular control devices.

Cost data is provided for the removal of sediment which originates at construction sites and which has been translocated to a deposition point by rainfall runoff. Removal methods examined include excavation and dredging. Limited data are presented for the cost of removing sediment from runoff by water treatment.

The text also discusses the USLE as a method of estimating sediment loss. All graphs and charts that are necessary to make quantitive estimates of soil losses using the USLE are presented. The USLE is also used to analyze the effectiveness of erosion and sediment structural and nonstructural control measures.

The text includes a cost evaluation example problem which developed from information presented in previous sections. The example problem gives additional insight to the chapter since it shows the reader exactly what input data is needed, how it is handled and processed and how the data can be used to make performance and cost effectiveness estimates.

The material presented in this report aids either an engineer or a planner who knows what the NPS problem is and which control solutions are available. This report will then help the engineer calculate the cost and effectiveness of the given control structures or techniques. An actual cost figure would require an adjustment of the estimate through one of the acceptable indices.

5. "Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas," U.S. Dept. of Agriculture, Soil Conservation Service (56).

The report presents specifications for the design of erosion and sediment control structures. The standards and specifications were prepared by the Soil Conservation Service while providing technical assistance through the Soil Conservation Districts in Maryland. A general background to erosion phenomena and sediment control is presented in the report. However, this presentation is short and should not replace the background information on erosion, sediment and the respective control techniques and procedures which are presented in other references discussed earlier.

The first section of the report is concerned with temporary structural practices. The text defines the structural practices, gives the purpose, describes the condition where the practice applies, and lists the design criteria and construction specifications. In addition, each structure is illustrated with a standard engineering drawing which includes a copy of the construction specifications. The drawing with the specifications is useful in that it contains all the information necessary for a field engineer or supervisor. The text also gives any of the mathematical equations which are necessary for the structural design. If graphs or nomographs are needed to solve the design equations, then they are also presented.

The report then describes the standards and specifications for permanent structural practices. Because these are permanent structures, their design and their specifications receive more attention than did the temporary structures. For most of the structures detailed hydraulic considerations are included in the design. However, all equations, nomographs and illustrations are included.

Vegetative control practices are discussed next and on the same level of detail as both temporary and permanent structures. When applicable, vegetative seeding procedures and the installation dates for certain vegetative covers are given, as well as fertilizer application dates and application rates.

Finally, standards and specifications for other nonstructural practices as discussed. These activities include bank stabilization, topsoiling, tree protection and other miscellaneous topics not covered in the earlier sections.

It should be recognized that this is a highly technical manual, and is less useful than some of the others cited in the planning stages of a water pollution control project. In these early stages, the other more general background literature, cost literature and effectiveness of control literature should be consulted. The information in this manual will be most useful when the proper control structure has been selected and it is time to design the site specific structures and implement their construction.

4.6.4 Hydrologic Modification

Hydrologic modifications result from activities that alter the natural flow patterns of surface and groundwater. Water quality impacts may be realized from hydrologic modifications both during facility construction and after the project is operational. The pollutant problems resulting from the construction phase of the project are discussed in the section on construction activities, Section 4.6.3, while the problems resulting from the completed hydrologic modification are discussed below.

The various types of hydrologic moficiations which may already exist, or may be proposed for a 208 planning area are identified and discussed in this section. The different types of pollutants which may affect the study area are presented, and various control strategies are analyzed. An annotated bibliography is also provided to indicate more detailed sources of information for the assessment of hydrologic modifications.

4.6.4.1 Activities and Practices

Hydrologic modifications can be grouped into channel modifications, impoundments and reservoirs, dredging and other resource recovery operations and water use systems. Channel modifications are generally for flood control or for improved drainage, and they provide an increased channel capacity. This allows for the passage of a greater volume of flow. Flood control modifications include channel clearing and snagging, channel excavations and channel realignment. Other modifications are sometimes made to tributaries of the main channel for flood and stormwater control. These include the construction of retarding basins and debris basins. Modifications of the drainage pattern may be accomplished with drainage ditches and surface or groundwater pumping.

A general result of channel modifications is an increase in stream velocity. The velocity increase is caused by a decrease in the stream roughness or by the straightening of the channel. However, the increased velocity may sometimes exceed the channel stability velocity and will cause bottom scour and bank erosion. In addition, when work is performed in the stream channel, the stream shoreline is usually disturbed by the movement of equipment. During these construction activities, shoreline vegetation may be removed or killed. The destruction of vegetation can result in an increase in the incident solar radiation. This, in turn, can result in an increase in stream temperature.

Hydrologic modifications which result in the depletion or lowering of groundwaters can impair quality in both surface and subsurface waters. Fish and wildlife habitats may be reduced from the increased infiltration of stream waters, seasonal water temperature patterns may be modified, and seawater intrusion may occur in coastal areas.

Impoundments are constructed for power generation, flood control and water supply. Those impoundments used only for power generation are known as run-of-the-river impoundments. Run-of-the-river impoundments have a time of flow passage of approximately a few days or less and, therefore, do not confine a large volume of water. Another type of impoundment is a reservoir storage impoundment which can have a time of flow passage of many months. These are the large reservoirs which are used for water supply and flood control. These storage reservoirs often result in significant water quality problems.

The depth of the large reservoirs combined with the incident solar radiation usually creates thermal stratification of the water impounded on the upstream side of the dam. The more common pollutional problems downstream of the dam are caused by the release of water which is warmer or colder than the downstream water temperature and the release of water which is low in dissolved oxygen and supersaturated with nitrogen.

Instream resource recovery operations such as dredging are another type of channel modification. These operations can cause nonpoint source pollution when the extraction of the materials from the streambed is taking place. If the benthos contains settled sewage or other waste products, these pollutants may become resuspended during the dredging operations. Additional problems can occur from the changes in bottom substrates and biological habitats and alterations in water velocity and current patterns.

4.6.4.2 Pollutants

Sediment is normally generated during the construction of hydrologic modifications. After the completion of the channel modifications it is possible that an increased stream sediment content will also persist because of the higher stream velocities that are present in the stream. Sediment deposition may also be a problem at the upstream end of impoundments: as the water flows into the impounded area, stream velocities decrease and the suspended sediment may settle to the stream bed.

Instream water temperatures are affected by incident solar radiation which reaches the stream if channel modifications remove or reduce the natural vegetative covers. In addition, thermally stratified water layers are created in reservoirs because of the incident solar radiation and the depth of the water. Depending on the outlet point of the reservoir, the thermal increases or decreases can occur in the downstream water temperatures.

Other pollutants such as nutrients, herbicides, insecticides, trace metals, other chemicals and soluble organic compounds, may exist in the benthic muds. When the muds are disturbed, such as in dredging or other channel alterations, these pollutants can be resuspended. In the case of impoundments, the natural leaching processes can transport these pollutants from the benthos to the water column.

4.6.4.3 Control Practices

Control practices to be followed during the construction phase of hydrologic modifications are the same as those discussed in Section 4.6.3.3 for general construction activity. Best management practices for channel modifications attempt to reduce the erosive force of the stream flow. Site preparation and reduction of thermal stratification are practices which reduce the nonpoint source pollution for impoundments.

Best management practices for control of nonpoint source pollution originating from channel modifications reduce the stream velocity and, therefore, reduce its erosive forces. The velocity reduction may be obtained by increasing the channel roughness or decreasing the channel slope. Rocks and stones (of local origin whenever possible) can be used to increase channel roughness and/or stabilize banks. Riffle and pool areas can also be built into the channel modifications to dissipate energy and, additionally, provide areas of stream reaeration. If the channel cross-sectional area is increased for flood control, a low-flow channel should be provided within the new channel. This low-flow channel can be constructed with the same physical characteristics as the original channel. When major channel modifications are performed, the bends in the river should be designed to prevent erosion of the banks. The usual method for this is to have steep outside banks and shallow inside banks. In addition, stones can be used on the outside banks to protect them from erosion.

Temperature increases can be controlled by the proper planning of vegetative cover. Whenever possible, natural vegetation should be left undisturbed during construction. In addition, permanent vegetation should be planted upon project completion to provide soil stabilization and shade cover conditions similar to those which existed before construction.

Control of spoils removed from the channel is an additional measure necessary to control nonpoint source pollution. These spoils are usually placed along the side of the stream. They should be prevented from returning to the river through erosion and runoff. This can be accomplished through proper sloping and immediate revegetation. In addition, spoils should be placed so they do not harm the natural vegetation.

Best management practices for control of nonpoint source pollution originating from impoundments include proper site preparation, control of release waters, minimization of stratification, and temporary algal and weed controls. Soil areas within the impoundment should be prepared for flooding. Lumber, stumps, and man-made debris should be removed from the impoundment area. Grass, shrubs, organic mulches, and rich topsoils should also be removed. Finally, the bed should be covered by two inches or more of sand to prevent the leaching of materials out of the soil.

Outlets located at several levels can be used selectively to release waters in which the temperature and dissolved oxygen are consistent with downstream water quality. Conversely, thermal stratification in deep reservoirs could be avoided by mixing the impounded water via diffusers or other mechnaical means. Also, release waters could be reaerated mechanically or by outfall design in order to prevent the discharge of water having a dissolved oxygen deficiency.

In large impoundments, eutrophication may be a problem. The only control practice which provides long-term results for eutrophication control is to limit the nutrients entering the reservoir. Proper site preparation will aid in nutrient reduction. However, the amount of nutrients reaching the stream from upstream point and nonpoint sources can provide enough nutrients to eutrophy an impounded area. These sources should be controlled by point source nutrient removal processes and nonpoint source nutrient control practices.

Temporary solutions to eutrophication problems are often used when long-term solutions are not possible. Temporary solutions include harvesting of algae, cutting and removal of aquatic weeds, drowning of aquatic weeds and the use of herbicides.

4.6.4.4 Annotated Bibliography

1. "The Control of Pollution From Hydrographic Modifications," Environmental Protection Agency Publication -- EPA 430/9-73-017 (57).

This report presents a broad, general description of the environmental effects of hydrographic modifications. (Note that the term "hydrographic modification" is used instead of "hydrologic modification", although their meanings are the same.) The preparation of the report was mandated by Section 304(e) (1) and (2) part (F) of the Federal Water Pollution Control Act Amendments of 1972, Public Law 92-500. Qualitative guidance is provided for identifying and evaluating pollution problems and possible control measures. Although quantitative, predictive methods are referenced, none are described in the report. In this respect, "The Control of

Pollution From Hydrographic Modifications" is well suited for an introductory overview of the wide range of possible problems and solutions.

The report discusses four general types of hydrographic modifications. These include:

1. Channel modification projects.
2. Impoundments and reservoirs.
3. Effects of urbanization.
4. Dredging operations.

For each type of project the current level of government involvement is discussed, including the activities of Federal, State and local agencies. Current practices are identified, as are the sources and types of pollutants which may affect the modified waterway. Methods of pollutant transport in stream channels and impoundments are examined, and water quality modeling and data collection are briefly described. The mechanics of groundwater pollution, such as that which occurs from seawater intrusion, is also discussed.

Methods, processes and procedures to control the pollution impact of projects are presented. Examples of these include design modifications to minimize adverse channelization impacts, structural and nonstructural alternatives to channelization, aeration and destratification of reservoirs, land use control, and productive uses of dredge spoil.

2. "Impact of Hydrologic Modifications on Water Quality," Environmental Protection Agency Publication -- EPA 600/2-75-007(1975) (58).

This report describes the scope and magnitude of water pollution problems caused by hydrologic modification activities that disturb natural flow patterns of surface water and groundwater. Although no new field measurements were made, a large body of available data is analyzed and presented.

The study has two principal purposes. The first is a description of the magnitude of water quality problems caused by the following hydrologic modifications:

1. Construction
2. Dams and impoundments
3. Channelization
4. Dredging
5. Land reclamation activities.

Quantitative estimates are made of the amount of sediment that enters the Nation's surface waters as a result of highway and urban construction. This section has the format of a nationwide assessment, and is primarily intended for use in EPA's program planning process. This analysis is supplemented by an Appendix with numerous case studies from each of the five categories of hydrologic modifications from around the United States. These summaries constitute a useful and concise compilation of extensive information and data.

The second purpose of the report is to develop "source loading functions" for predicting the quantities of water pollutants released by out-of-stream construction activities. These loading functions can be used by technical investigators to estimate the amount of sediment entering a watercourse from construction sites of known size and locations. The loading functions, which are adaptations of the USLE, are based on measurements of sediment yields and other parameters at ten construction sites. The accuracy and limitations of the functions are analyzed.

The report also includes a brief discussion of methods for controlling pollution from hydrologic modifications, including out-of-stream and instream approaches.

3. "EPA Nonpoint Pollution Control Guidance, Hydrologic Modifications," (in progress), Environmental Protection Agency, Water Planning Division (59).

This document presents the information needed by a water quality management or planning group in developing the management practices to minimize water pollution due to hydrologic modifications. The report is directed towards activities which modify the hydrology of an area, such as channelization, dams, dredging, and instream construction; as well as activities which unintentionally impact hydrologic characteristics, such as the land development which accompanies rapid urbanization. The information is qualitative, and specifications for technical design are not included.

The report describes approaches for assessing water quality problems from existing hydrologic modifications. The different types of modifications and the various pollutants which may effect them are discussed. These pollutants include:

1. Sediment
2. Nutrients
3. Thermal effects
4. Pesticides and other chemicals.
5. Biological microorganisms.

Possible sources for available information are identified and survey techniques for expanding the information are discussed.

Methods for the analysis and selection of control alternatives are also presented. Data needs are described, including climatic, geologic and topographic information. Selected practices for the control of water quality problems associated with hydrologic modifications are discussed in more detail.

The report includes a section on identifying potential problems from proposed projects. Analyses were made of primary problems such as flooding and landslides which could result from poor planning and design and secondary problems such as sediment control during construction.

The Appendix of the report presents some of the pertinent rules and regulations of the United States Army Corps of Engineers and the Environmental Protection Agency related to hydrologic modifications. They are reproduced from the Federal Register and include:

1. U.S. Army Corps of Engineers, "Permits for Activities in Navigable Waters or Ocean Waters"
2. Environmental Protection Agency, "Navigable Water, Discharge of Dredged or Fill Material"

The Appendix is useful for determining the legal considerations that may be involved in planning decisions.

4. "Report on Channel Modifications - The President's Council on Environmental Quality, United States Government Printing Office," (March, 1973) (60).

This report, prepared for the Council on Environmental Quality discusses forty-two selected channelization projects performed by the United States Army Corps of Engineers, Soil Conservation Service, Tennessee Valley Authority, and the Bureau of Reclamation. An assessment is made of the environmental, economic, and engineering aspects of the channel modifications. The methodology used in preparing this report is a good guide for those evaluating additional projects.

The projects chosen for discussion represent a variety of climatic and topographical conditions, soil type, aquatic and habitat systems, rural and urban locale, and a range of project purposes and sizes. Each site evaluation conducted by the study group consisted of a general project briefing session, a field survey of the area, and a concluding session with public participation.

Professional observations serve as a basis for evaluating the physical effects of wetland drainage and land use changes, cutoff of oxbows and meanders, watertable changes and stream recharge, erosion, sedimentation, and channel maintenance. The results of extensive biological investigations conducted by the Philadelphia Academy of Natural Sciences are presented. These are integrated into a useful format for evaluating the effects of channel modifications on fish and wildlife resources, habitat, species diversity, and productivity. Although there is little quantitative information presented based upon measurements of water quality or hydraulic parameters, the assessment procedures derived from professional observation and biological studies may be useful to the planner of additional projects.

5. "Planning and Design of Open Channels," United States Department of Agriculture, Soil Conservation Service, Technical Release No. 25, December, 1964, Revised March, 1973(61).

This technical release is presented as a guide for use by field personnel in the evaluation, planning, and design of open channels. The material is directed toward the more complex type of channel work done by the Soil Conservation Service. This includes floodways and drainage-type channels in which channel degradation and bank erosion are of primary concern.

The report discusses general planning considerations such as the adequacy of outlets, legal requirements, and the rights-of-way. Guidance is provided for the preliminary survey, drawings, strip maps, and profiles. Examples of drawings are included.

Site investigations are described to evaluate the resistance of the soils in the bed and banks of the channel to erosion forces, to evaluate the sediment transport relationships, to determine slope stability against sloughing and sliding, to estimate earth loads that may act on structural members, and to determine the rate of water movement through the soils. Information for identifying and analyzing stratigraphic units (layers of soil) is presented, and procedures for sampling and testing the channel bed and slope are suggested.

The report presents a methodology for determining channel capacity and relating this to design discharges. Designs for special transition structures in waterways are examined. Criteria for slope stability are analyzed, and methods for improving channel slope stability are suggested.

The revised version of the report includes a section on environmental considerations in channel design, installation, and maintenance. Although no definitive criteria or standards are established for SCS projects, useful guidance is provided for the planner. Issues of aesthetic quality, fish, and recreation resources are addressed. Methods are presented for protecting these beneficial uses through design and during construction. The section on environmental considerations includes appendices with approaches and charts for rating the recreational and environmental quality of a project. The report provides useful technical information for the actual design and implementation of channelization projects.

6. "Water Quality Management Planning Methodology for Hydrographic Modification Activities," Texas Water Quality Board, (December, 1976) (62).

This report, prepared for the Texas Water Quality Board, develops a methodology for the evaluation of hydrographic modifications, the pollutant loads generated by associated activities, impacts on receiving water bodies, and pertinent control strategies. Although portions of the report pertain specifically to conditions in Texas, the approach and techniques presented are useful to planners seeking a straightforward analysis framework for evaluating hydrographic modifications.

The effects of hydrographic modifications are outlined by examining typical contaminants and relating them to instream impacts. These impacts include:

1. Aesthetic value
2. Dissolved oxygen depletion
3. Sediments and deposits
4. Excessive aquatic growth
5. Public health threats
6. Improved recreation value
7. Ecological damage
8. Reduced commercial value.

Water quality problems common in impoundments and reservoirs are identified and quantitative estimating techniques are presented. Issues of temperature, density, stratification, evaporation, and eutrophication are addressed. Solutions are given for pollutant concentrations in run-of-the-river impoundments, large vertically mixed and vertically stratified reservoirs, and two dimensional near-field effects. Simple, empirical methods for relating the eutrophic status of a lake to the nutrient loads are discussed. Example problems are included to demonstrate the techniques.

The pollution impact of channel modifications and dredging operations are also presented. Some of the issues addressed are the relationship between bank vegetation and water temperature, the effect on reactions, reaeration and the assimilative capacity of the stream, and the physical, chemical, and biological characteristics of dredged material. Relevant data needs and sources of information are discussed.

The report includes an analysis of pollution control methods for each type of hydrographic modification activity. Examples of those presented are site preparation, control of loads, selected withdrawals from reservoirs, aeration and destratification, control of nuisance organisms, reclamation of disturbed areas, and disposal site controls.

7. Reports from the United States Army Corps of Engineers.

A number of studies related to hydrologic modifcations have been conducted by the U.S. Army Engineer Waterways Experiment Station in Vicksburg, Mississippi. A few of the available reports are summarized below.

a. "Mathematical Simulation of the Turbidity Structure Within an Impoundment," Research Report H-73-2 (63).

In this report, the thermal and turbidity structures of an impoundment are simulated with a mathematical model. The model was verified with observed data from the Hills Creek Reservoir in Oregon and used to predict the turbidity structure of a proposed impoundment. The effectiveness of selective withdrawal, in particular, a low-level outlet for controlling turbidity is examined. Examples of input and output from the computer program used for the analysis are presented.

b. "Selective Withdrawal from Man-Made Lakes," Technical Report H-73-4 (64).

This report presents the results of laboratory investigations to determine the withdrawal-zone characteristics created in a density-stratified impoundment by releasing flow through a submerged orifice, over a free and submerged weir, or through a combination of the above. Density stratification from differentials in both temperature and salinity are examined. Generalized relationships for describing the vertical limits of the withdrawal zone and the vertical velocity distribution within the zone are developed. This flow rate distribution can then be applied as a weighting function to the reservoir profile of water quality parameters to determine their concentrations in the reservoir release.

c. "Ecological Evaluation of Proposed Discharge of Dredged or Fill Material into Navigable Waters," Miscellaneous Paper D-76-17 (65).

This document is intended to provide interim guidance for the evaluation of discharges of dredged material as mandated in Section 404(b) of Public Law 92-500. The selection and interpretation of appropriate tests for dredging sites and dredged material are discussed. General approaches for technical evaluation are partitioned into physical effects, chemical-biological interactive effects, and procedures for site comparison. The report presents detailed stepwise procedures for conducting an elutriation test (to simulate the release of dissolved solids from dredged material), estimating a mixing zone, performing bioassays, conducting total sediment analyses, and evaluating biological community structure.

d. "Techniques for Reducing Turbidity Associated with Present Dredging Procedures and Operations," Contract Report D-76-4(66).

The reduction of turbidity from present dredging procedures is examined by analyzing the following operations: Cutter performance, ladder, suction, hull, pipeline, connections, barges, tenders, personnel, inspection, contracts, plans, and specifications. The suggested techniques consist principally of good dredging procedures already known, but not always followed by dredging contractors and their personnel.

e. "B. Everett Jordon Lake Water Quality Study," Technical Report H-76-3(67).

This report is an example of a study on a large impoundment. Physical and mathematical models are used to investigate the hydrodynamics of stratified flow within the lake, to predict the temperature and dissolved oxygen regimes of the lake immediately upstream of the dam, and to estimate the temperatures and dissolved oxygen content of the release. The report provides an interesting example of applications of different modeling techniques used by the Army Corps of Engineers.

4.6.5 Mining

Mining refers to the extraction, transport, processing and storage of minerals and disposal of mineral wastes. Mining activities impact more than ten million acres of land in the United States. The types and quantities of pollutants generated from these mining operations are dependent on the substance being mined, rock type, climate, topography, geologic structure, method of mining, and hydrologic characteristics of the site. The detrimental effects of mining activities can be prevented, reduced, or eliminated by preventing the formation of pollutants at the mine site, containing the pollutants within the mining area once they are generated, and controlling pollutant contributions emanating from mining operation areas.

4.6.5.1 Activities and Practices

The extraction of mineral deposits from the earth is accomplished by surface mining, underground mining, well extraction, and a number of lesser methods including in-situ leaching.

Surface mining takes several forms such as strip, open pit, dredging, and hydraulic mining. Strip mining involves the removal of overburden to expose an underlying deposit for extraction. Open pit mining is a similar procedure for areas with little overburden. Open pit mines contribute minimal spoil, but result in deep open holes. Dredging is underwater mineral recovery. The material may be removed from artificial impoundments or natural bodies of water. Hydraulic mining incorporates the use of a high velocity water jet directed at unconsolidated deposits. Underground mining is directed at the extraction of minerals deep in the earth. Shafts are sunk to gain access to the deposit. Surface mining generally creates a more visible defacement and disturbance of the earth's surface than does underground mining. The placement and distribution of the mineral deposit determines the size, depth and method of the mining operation.

A mining operation progresses along a number stages of development including exploration; access and support facility construction; mineral extraction and processing; mine closure; mineral storage; and waste disposal.

4.6.5.2 Pollutants

Both surface and groundwaters can be adversely affected by active and inactive or abandoned mining operations. Pollution arises because hydrologic characteristics of surface or subsurface runoff may be altered when the earth is disturbed to gain access to mineral deposits. The degree of alteration and pollution generated depends upon the size, depth, and method of the disturbance and also on the chemical and physical properties of the disturbed material.

The major forms of water pollution resulting from mining operations include physical, chemical, and hydrologic changes. Most chemical pollution results from the oxidation of sulfide minerals. In the presence of water and an increased amount of oxygen due to exposure, accelerated oxidation of the ore may produce acid and salts, particularly iron salts and sulfuric acid. When these solutions contact mineral and soil formations, the acid may in turn selectively extract heavy metals.

Refuse waste materials that result from mining activities can be significant sources of pollution when surface or groundwater percolation is intercepted and produces a contaminated leachate. In addition, mineral wastes which contain pyrite will oxidize and form soluble iron salts and sulfuric acid. These may be flushed into nearby surface waters or percolate into groundwaters during a precipitation event.

Radioactivity arising from mining activities is primarily a concern in the western United States with regard to seepage of uranium and radium at sites where uranium ores are mined and processed. The effects of very long term exposure to low levels of radioactivity are of concern and are being studied.

The most common form of physical pollution is sediment. Surface mining creates large areas of disturbed land which are often highly erodible. The processing of raw minerals to concentrate ore creates vast piles of fine grained waste materials which are a potential source of sediment pollution. During contour strip mining operations, the practice of placing overburden on the downslope side of the outcrop can result in excessive siltation in water courses.

In summary, the most serious pollutants associated with mining and mineral operations are mine drainage contaminants and sediment. Mine drainage contaminants may include such constituents as acids, alkalis, iron, aluminum, manganese, zinc, cobalt, lead, mercury, cyanide, fluoride, copper, arsenic, cadmium, nickel, phosphate, sulfate, chloride, and radioactive mineral contaminants.

4.6.5.3 Control Practices

Mine water pollution control is generally achieved by preventing pollution production or containing pollutants on the mine site and controlling pollutant delivery to the receiving water. For both surface and underground mining, effective pollution control pre-planning can prevent, reduce or eliminate pollution from active mining areas and pollution that may occur after completion of mining. Such site characteristics as geology and ground water patterns, climate, soil and slope stability, chemical/ physical nature of the overburden, past mining history and characteristics of receiving waters should thoroughly be investigated to determine the proper choice of mining methods and pollution control techniques.

4.6.5.3.1 Surface Mining - On-site Abatement

a. Controlled Mining Procedures: Certain mining procedures provide better control of water pollution than other techniques. The adaptation of one or more of the techniques is governed by the specific type of mining operation. Some of the controlled mining procedures are discussed below.

Overburden can be separated by type. Some overburden materials are conducive to plant life, while others are sterile or have the potential for polluting. The purpose of overburden segregation is to keep these classes of material separated during mining so they can be effectively utilized during later regrading.

Longwall strip mining is being researched as an alternative to conventional strip mining as a procedure which may reduce the pollution potential of the mining operations. A vertical trench is cut into a hill perpendicular to the mineral outcrop and the deposit is extracted with automatic mining equipment. There is little surface disturbance required when using this technique and most of the sediment related problems of strip mining are eliminated.

Other mining techniques in use are modified block cut, head-of-hollow fill, box-cut mining, area mining, auger mining, and selective mineral extraction. Any of these mining operations may result in more or less nonpoint pollution, depending on the characteristics of the particular site.

b. Water Infiltration Control: Infiltration results from subsurface water movements, or downward percolation of surface waters from rainfall. Pollution from water infiltration can be avoided by decreasing surface permeability, using impermeable barriers between the water source and the pollution forming material, diversion of water around the mine site, and underdrain utilization.

c. Handling Pollution-Forming Materials: Pollution-forming mining wastes discarded on the land surface may be exposed to oxidation, weathering, erosion and leaching. Mine backfilling and sealing, relocation of wastes to a more suitable hydrologic location, and flooding of underground mines to eliminate oxidation are several methods of controlling pollution from these materials.

Another technique is the utilization of mine wastes as saleable products. Reprocessing mine wastes for secondary extraction can eliminate or reduce waste piles.

d. Regrading: Regrading is the mass movement of earth to achieve a more desirable topography. The main purpose of regrading is to provide a suitable base for revegetation, burial of wastes, and reduction of erosion. The techniques will vary according to the topography of the final land surface.

e. Erosion Control: Erosion control is accomplished by several basic methods. One of these is the protection of erodible surfaces from movement by water. This is accomplished by water diversion and by covering procedures. Chemical stabilization, revegetation, mulching, slope control, and concentrated flow handling are methods of decreasing erosion potential at mine sites. Revegetation, specifically, is one of the most effective erosion control techniques. A dense ground cover stabilizes the surface with its root system, shields the surface from rainfall, and reduces velocity of surface runoff.

4.6.5.3.2 Underground Mining - On-site Abatement

a. Controlled Mining Procedures: For underground mining, water pollutants are generated after mining is completed by oxidation of the mined materials. Air circulating through an abandoned mine can therefore continue to oxidize susceptible materials. Flooding of a mine to exclude air is the only practical method of eliminating this source of oxygen under present technology. However, if a mine is flooded, the flooded water must be contained within the mine or it can itself become a source of pollution.

Most of the water entering underground mines passes vertically through the mine roof from overlying strata. Collapse of a mine roof is sometimes responsible for increased vertical flow. The chance of the collapse of overlying strata can be reduced by employing one or a combination of the following: pillars, roof support, limiting the width of openings, and by backfilling the voids.

b. Water Infiltration Control: These techniques are designed to reduce the amount of water entering underground mines, and subsequently to reduce the amount of drainage leaving the mine. Choice of techniques and extent of their use will depend on hydrologic conditions in the area and cost effectiveness of each technique. Infiltration generally occurs as a result of rainfall recharge to the ground water reservoir. Rock fracture zones and faults have strong influence on ground water flow patterns. Infiltration can usually be reduced by avoiding these zones during mining. Boreholes and other fractures can be sealed up to reduce the movement of water.

Filling surface depressions, smoothing overlying areas, or other surface regrading techniques can be used to decrease water infiltration.

c. Mine Sealing: This practice is usually employed to promote inundation of underground mine workings in order to reduce oxidation of pyritic materials. Mine sealing involves construction of a physical barrier in a mine opening to prevent passage of air and water. A mine seal can be constructed in many ways, using many different types of material. A mine seal must have internal strength capable of withstanding water pressure, and it must be tied into the floor, roof, and sides of a mine opening. Some seal types and methods which have been successfully demonstrated include double bulkheads, gunite, grout curtains, clay and air seals.

4.6.5.4 Annotated Bibliography (Mining Activity)

1. U.S. Environmental Protection Agency, "Methods for Identifying and Evaluating the Nature and Extent of Nonpoint Sources of Pollutants," EPA 430/9-73-014 Washington, D.C.,(October 1973) (68).

This report provides documentation of presently available knowledge in four areas of nonpoint sources of discharge, namely, agriculture, silviculture, mining and construction. In the seventy-page chapter on mining, the nature and extent of pollution from mining activities is discussed, data interpretation aids are given and prediction methods pertaining to pollution sources from mining are presented.

The report discusses the nature of the sources, the type and relative importance of pollutants from each source, and the pollution loads related to natural and operational factors. General discussions on acid mine drainage, sediment, leachate, and radioactivity are included. Discussions are given on groundwater pollution and subsidence from abandoned underground mines. Details of some coal mine drainage pollution problems are presented. Pollution sources from other mining activities discussed in less detail include the mining of hard rock minerals; stone, sand and gravel; noncoal sedimentary materials; and oil and gas.

Statistical information on inactive and abandoned mines, and sources of current statistics on active mining operations are also provided.

The empirical aids for mine drainage data interpretation that are presented in the report consist of nomographs which permit a check of the anion-cation balance of a sample. The use of the nomographs and interpretation of the results obtained from them are discussed, as is the conversion of raw water quality data into useful form. Trends (correlations) that were observed for stream quality data in mine drainage areas are also presented. A discussion of sampling techniques and procedures, and analytical methods that are usually performed for mining-related wastewater, is presented. Brief descriptions of models that have been developed for predicting pollution quantities are also presented together with indications of their applicability and success. These include models to determine flow and chemical characteristics of mine drainage, limestone requirements for pyritic spoils and overburden, infiltration of water into spoil banks, leachate quantities from spoil banks, pollution potential from spent oil shale, mine drainage volumes in localized areas, and sediment loadings.

This report presents a general overview of information to identify and evaluate the extent of nonpoint sources of pollutants stemming from mining activities. The reader may use this information as a guide to more elaborate and comprehensive discussions of the previously mentioned topics relating to mining activities. A list of seventy references are presented at the end of the mining section of this report and are footnoted in appropriate sections of the text.

2. U.S. Environmental Protection Agency, "Processes, Procedures and Methods to Control Pollution from Mining Activities," EPA-430/9-73-011, Washington, D.C.,(October 1973) (69).

This report provides information that identifies and evaluates available technology for control of water pollution from mining activities. Information is presented on techniques of at-source water pollution control applicable to the mining industry, whose practicability and feasibility have been demonstrated, or strongly indicated by the results of research. The control methods included in this manual are identified and described by way of brief text, generalized illustrations, and unit cost indications where possible.

The manual is divided into three major components, namely, surface mining, underground mining, and treatment of mine drainage. Discussions of various controlled mining procedures, methods to control water infiltration, and techniques used to handle polluted mine waters are presented for surface mining and underground mining operations. The section on surface mining also contains methods of handling pollution forming materials, discussions of various types of regrading, erosion control procedures, and revegetation techniques. Numerous methods of mine sealing underground mines are also presented. The treatment section of the report discusses neutralization of mine drainage utilizing various limestone and lime processes. Subsections under treatment of mine drainage include discussions on sludge disposal, evaporation processes, reverse osmosis, electrodialysis, ion exchange processes, freezing (crystallization), and iron oxidation.

The manual serves to acquaint the reader with the many available techniques to control pollution from mining activities. It does not provide the degree of detail that would be needed for it to be used alone as a pollution control reference. However, the manual may be used to guide the reader to the appropriate reference or references for specific, detailed, comprehensive information on how to apply a particular technique.

3. EPA Technology Transfer Seminar Publication, "Erosion and Sediment Control, Surface Mining in the Eastern U.S.," Volume I: Planning, Volume II: Design, EPA-625/3-76-006,(October 1976) (70).

This manual consists of two volumes. Volume I covers the basic concepts of erosion and sediment control and implementation of the control plan. Volume II discusses design and construction considerations, erosion control materials, and provides a sample of an erosion and sediment control plan.

The mechanics of soil erosion and sedimentation and the physical factors which determine the nature and extent of these processes are discussed in Volume I. Included are presentations of the methods of erosion and sediment control and maintenance of control practices. The various aspects of an erosion and sediment control plan and implementation of this plan are also discussed.

Volume II of this manual discusses the design procedures and construction specifications for selected control structures. Included are diversion structures, sediment traps, downdrain structures, level spreaders, grassed waterways, ripraps, check dams and sediment basins. Erosion control products and materials, such as chemical binders, mulches, and other stabilization materials are also discussed. A sample of an erosion and sediment control plan, selected state mining laws, and reclamation information are also presented.

This manual contains comprehensive discussions of various aspects of erosion and sediment control. Volume I provides numerous figures and photographs which aid the reader in obtaining a thorough explanation of the need for control, basic control principles, available technology for erosion and sediment control, and procedures for preparing and implementing a control plan. Similarly, Volume II is a detailed design manual, containing numerous tables and figures of design parameters and construction specifications for various control structures. Example problems and a sediment basin design example are also included. The manual provides lists of references after various sections, as well as a glossary. It should be noted that the control information presented is directed primarily toward preventing excessive soil loss and resulting damage associated with coal surface mining operations in the eastern portion of the United States, specifically the Appalachian, eastern interior, and western interior coal regions. However, much of the material and certainly all of the basic erosion and sediment control philosophy are applicable to all categories of surface mining in all regions of the country.

4. "EPA Guidance for Identification and Assessment of Mining Nonpoint Pollution Problems" (in Progress)(71).

The problem assessment procedures that are presented in this document are intended to help state and areawide 208 agencies to better understand and carry out their planning responsibilities specifically related to mining nonpoint sources under Public Law 92-500. This guidance presents a simplified approach for planners to use in making an initial identification of mining nonpoint source problems and suggests alternative judgements and decision points that are involved in the definition of mining planning requirements. The document suggests a task outline for identification and assessment of mining nonpoint sources and provides procedures for chosing the location and description of mining pollution, interpretation of water quality data, existing pollution load description, quantitative load characterization, and mining pollution load analysis.

5. Bituminous Coal Research, Inc., Mine Drainage Abstracts (72).

These abstracts are an important information source regarding what has been published in the area of mine drainage research and in treatment and control technology. Annual supplements published each year contain information received that year pertaining to all aspects of mine drainage. The bibliography and supplements are available at a nominal cost from the Library, Bituminous Coal Research, Inc., 350 Hochberg Road, Monroeville, Pennsylvania 15146.

4.6.6 Residuals Management Practices

Residuals management practices deal with the disposal of residual waste materials. Residual waste materials are generally considered to be but are not limited to wastewater treatment plant sludges, septage effluent and pumpage, water treatment sludges, municipal refuse, combustion and air pollution control residuals, industrial waste sludges, feedlot manure, mining wastes, and dredge spoils. The residuals management practices vary widely depending on the residuals and the land area.

4.6.6.1 Activities and Practices

Residual management can be separated into utilization practices and disposal practices. The utilization practices are designed to reduce the amount of residual by conversion to a resource. Disposal practices are strictly designed to dispose of the residual waste products.

Residual utilizations differ for each of the types of residuals. Wastewater and water treatment plant sludges can be used for fertilizers and soil conditioners after they are stabilized and dewatered. In addition, the lime and alum can be removed from these sludges and reused. Municipal refuse may be either recycled, or incinerated for use as an energy source. A common use of combustion and air pollution control residuals is for coarse aggregate material used in roadbed construction. There are numerous recovery methods and uses for industrial waste sludges. Each reuse depends directly on the characteristics of the sludge and therefore on the industrial product. Finally, mining wastes can be used for road construction materials or for incinerator combustion materials.

Disposal practices for residual wastes are diverse and in some cases, complex. Each practice generally includes the operations of site selection, evaluation, disposal, control and monitoring. The disposal operations are categorized as land reclamation, land spreading, sanitary landfills, ocean disposal, and trench disposal. Any single disposal method can not normally be used for all of the previously cited residuals.

Land reclamation includes the reuse of strip mines and marginal lands, and the creation of wildlife habitats. In each of these practices, residuals are deposited on land through spray irrigation, pipeline slurries, and hopper scows. These practices are generally used for the residuals listed below.

1. Wastewater sludge
2. Septage
3. Water treatment sludge
4. Feedlot wastes
5. Dredge spoils

A second residual disposal practice is land spreading. Land spreading is normally accomplished by spray irrigation, ridge and furrow application, plow-furrow-cover application, plow-injection application, subsoil injection, truck application and manure spreading. These operations are used for the liquid slurry residuals listed below.

1. Wastewater sludge
2. Septage
3. Feedlot wastes
4. Dredge spoils

Another residual disposal practice is sanitary landfilling. In sanitary landfill operations, residuals are applied to the land, compacted, covered with soil and compacted again. This process is repeated until the landfill is completed. The landfill is closely monitored to insure that both the groundwater and surface water are not contaminated by leachate. Generally, municipal refuse comprises the largest fraction of residual in a sanitary landfill. A landfill can accept sewage, septage and water treatment sludges if proper pretreatment and dewatering is practiced. Sanitary landfills are often used for disposal of combustion air pollution control residuals and incineration residuals. Industrial wastes and sludges are usually deposited in separate areas of the sanitary landfill, while mixing, feedlot and dredging residuals are not normally disposed of in sanitary landfills.

Ocean disposal is another residual management practice although operations of this type are being significantly restricted. This practice has been utilized extensively in the past for disposal of all types of residuals. However, recent legeslation limits the mixed concentration of the residual and water to 0.01 of the concentration which is detrimental to the appropriate sensitive marine organisms. This will tend to eliminate ocean disposal of all residuals except dredge spoils.

Trench disposal is another land application method. Residual wastes are placed in trenches approximately two feet deep. The trench is then immediately covered with soil to prevent odor escape and surface water contamination. The only residuals which have normally been disposed of in this manner are digested and raw dewatered sludges. Trench disposal is not a commonly used method, since the period required for stabilization may be as long as five years.

4.6.6.2 Pollutants

Pollutants generated from residual management practices vary according to the practice employed and the residual itself. The pollutants have been listed in Table 4-5 according to the residual waste product.

Nitrogen and phosphorus are potential pollutants which originate in wastewater sludge, septage, and feedlot manure. Organic materials are generally derived from septage, water treatment, municipal refuse, feedlot and dredge spoil residuals. Potential sources of heavy metals are wastewater sludge, septage, water treatment sludge and municipal refuse. Micro-organisms originate from all biological residuals.

To assess the pollutants from these residual management practices, the planner must examine the residual. Next, he must examine the method of disposal. Finally, he must examine the control practices used in the site to monitor the possible pollution of surface and groundwater. It is not likely that the quality or the quantity of the pollutants can be assessed without site specific water quality data. However, these data normally exist for some of the residuals management practices in the planning area.

4.6.6.3 Control Practices

Residuals managment is practiced to dispose of residual waste materials while at the same time eliminating and minimizing one of the nonpoint sources of pollution.

The first step in a management plan is to minimize the volume of waste material for disposal. A second requirement is the treatment of the residual to the state-of-the-art treatment level available. Such treatment methods include stabilization, dewatering, neutralizing, etc.

Examination of the disposal site is required. The site should be chosen so that the residual waste does not become a source of pollution upon disposal. For example, an industrial sludge containing heavy metals should not be used to fertilize a corn field. Likewise, salt water dredge spoils should not be applied to land as fertilizers due to their high salt content.

It is during the site planning that the meteorologic, geologic, and hydrologic characteristics of the disposal site should be examined. In developing a disposal plan, disposal site conditions should be compatible with the properties of the residual in order to give maximum protection to both the ground water and surface water.

For a sanitary landfill, pollution control practices should include the installation of proper drainage systems to divert surface water away from the fill area. An impermeable liner can also be installed to protect the groundwater. Additional control measures depend largely upon the specific waste under consideration.

Other general management practices for pollution control of residuals will depend on the residual and the conditions of the disposal site. These methodologies are not described here. However, the many pollution controls options for the residual management practices are available in the literature.

4.6.6.4 Annotated Bibliography

1. "Effects of Land Disposal of Solid Wastes on Water Quality," Department of Health, Education, and Welfare, SW-Z, Bureau of Solid Waste Management, Cincinnati,(1968) (73).

This pamphlet is an informative review of current literature which describes the influence of solid waste disposal practices on water quality. Definitions, site descriptions, water quality criteria, potential hazards, case histories, recommendations, and tentative guides are included. The information is designed to give some insight into the problems that may occur and the methods for solving them.

A section describing open dumping and sanitary landfill as major land disposal methods is presented. Included are discussions of infiltration and percolation, solid wastes decomposition processes, gas production and movement, leaching, groundwater travel and direct runoff, and the role each plays in contributing to surface and groundwater pollution due to land disposal.

Evidence that physical characteristics, biological quality, and chemical composition of surrounding waters are affected by quality and quantity of solid waste conditions is well known. The physical characteristics of major concern are turbidity, odor, taste, and color. Biological water quality refers to bacteria present in the water, usually by leaching. Chemical composition is concerned with mineral and organic substances present in solid wastes which are capable of causing gross pollution of underground water supplies. Of major concern are chlorides, organic matter, hydrogen sulfide, carbon dioxide, methane, ammonia, and nitrates.

Case studies of water quality investigations related to solid waste disposal operations are presented. These case histories are samples of past investigations and some present research efforts presented to clarify the potential pollution problem associated with refuse disposal sites.

Requirements for proper land disposal are presented. Recommended land disposal methods for surface water wet areas such as swamps and marshes, tidal areas, ponds, quarries, and similar depression type areas are discussed. Actual requirements enforced in several municipalities are outlined.

In addition to requirements, suggested guides to enable management to judge the acceptability of a waste disposal site are listed. These include the areas of study required prior to design and construction of sanitary landfills. Also included is a listing of guides to good practices which is essentially a list of "DO's" and "DON'T's" with regard to land disposal practices.

2. "Residual Management by Land Disposal," Procedings of the Hazardous Waste Research Symposium, EPA Report No. 600/9-76-015, Cincinnati, (July 1976) (74).

The report contains information on extramural research projects funded by the Solid and Hazardous Waste Research Division of the U.S. Environmental

Protection Agency, Municipal Research Laboratory in Cincinnati, Ohio. The papers presented in the proceedings are separated into five sections, as they were categorized in the symposium.

The first section, "Introduction and Orientation" presents an overview of Federal research and legislative development programs for hazardous waste disposal.

The second section, "Identification of Pollution Potential" deals with techniques for gathering and interpreting information about problems with disposal of hazardous wastes. The authors indicate that there is a need for a program of disposal research. They also describe some of the methods used to observe the effects of land disposal of hazardous wastes on environmental quality.

Available techniques for dealing with potential disposal problems are presented in the third section, entitled "Modification of Disposal Sites and Waste Streams." Chemical stabilization of the waste or soil and/or use of impermeable liners in order to prevent groundwater contamination are discussed.

The fourth section, "Special Disposal Problems," is a discussion of problems encountered in disposal of specific types of wastes. The wastes are highly concentrated and/or highly toxic substances such as hexachlorobenzene, vinyl chloride, and pesticides.

Movement of contaminants in soil is discussed in the fifth section, "Predicting Trace Element Migration." The section includes discussions of predictive and modeling procedures, techniques and problems of detecting contaminant movement, and determination of the soil properties and contaminant characteristics which control movement.

3. Solid Waste Management: D. Joseph Hagerty, Joseph L. Pavoni, John E. Heer, Jr.; Van Nostrand Reinhold Environmental Engineering Series, (1973) (75).

The book offers a look at the environmental problems and solutions associated with solid waste collection and disposal. Included are data which show how future systems for collection, disposal, and recovery will have to be designed and selected. In addition, discussions are included regarding the pros and cons of presently used systems, innovations in collection equipment, and specialized practices and facilities for transport and collection of solid waste.

Equipment is described which is capable of being modified to specific ways of handling almost any type of solid waste. The many types of compaction and size reduction equipment are discussed. Presentations of separation techniques and material and energy recovery systems are included. Ways and means to reduce the volume to be collected while recovering larger amounts of highly valuable materials are shown. Recovery of valuable materials is an incentive to the pursuit of environmental goals.

In its discussion of sanitary landfills, the book includes information on the design and planning stages, equipment requirments, site selection considerations, relative costs of landfilling, prevention of resulting pollutional problems, and use of the completed sanitary landfill.

A discussion of incineration is also included. The basic process principles are discussed, and information is given regarding the planning, design, construction, and operation of refuse incinerators and their accessory apparatus.

The authors outline the economic advantages, environmental aspects, and other factors involved with the new developments and equipment in the field of solid waste management. Discussions of high-temperature incineration, power generation through incineration, pyrolysis of wastes, new landfill stabilization techniques, and landfill disposal of hazardous wastes are included. Finally, the text outlines the legal, environmental, and public relations aspects of solid wastes management.

4. "Management of Hazardous and Toxic Wastes," Paul N. Cheremisinoff, William F. Holcomb, Pollution Engineering, Vol. 8 No. 4, April 1976, pp. 24-32 (76).

Hazardous wastes continue to be a significant problem, even as air and water pollution control and solid waste disposal methods progress. The authors point out that the generation rate of non-radioactive hazardous wastes is estimated in excess of 10 million tons annually and increasing, or about 10 percent of all waste material generated by industry.

Included are several definitions of "hazardous wastes" as given by The Hazardous Materials Transportation Act of 1974, Federal Water Pollution Control Act Amendments of 1972, and the National Solid Wastes Management Association. The authors also indicate that disposal of wastes on land is essentially unregulated except in the case of radioactive wastes. The EPA is in favor of a program which would establish a nationwide Federal and State regulatory program.

An extensive discussion of present disposal practices is included. In making use of existing technology, hazardous wastes can be generally dealt with by reduction in quantity generated by process modification or raw materials changes; by concentration of wastes at the source by evaporation, precipitation, etc.; by stimulation of waste exchange -- recovered acid, caustic, or solvent wastes may be sold or recycled; by recovery and recycle of metals, energy content and other useful resources contained in the wastes; by destruction of some hazardous wastes by special incineration methods; by detoxification and neutralization for land disposal; and by construction of specially designed landfills, insulated from groundwater, and properly monitored and secured.

Finally, a section on "International Disposal Techniques for Other-than-high-level Solid Radioactive Wastes" is presented. In the section, disposal methods such as shallow land burial, disposal into mines and deep geological formations, deep sea disposal, deep well disposal, and packaging are discussed.

5. "Stop Leachate Problems," Michael Dilaj, John F. Lenard, Water and Wastes Engineering, Volume 12 No. 10, October 1975, pp 27-40(77).

Controlling leachates is one of the most important aspects of sanitary landfilling. The degradation of surface and groundwaters as a consequence of landfill operations remains a problem, especially in humid areas where precipitation is considerable.

The authors visualize leaching as a special case of extraction of substances adsorbed onto solid particles. The transfer of pollutants from the refuse to the groundwater is accomplished by vertically or horizontally moving water which directly passes through, and has intimate contact with the refuse.

Leaching takes place only if a section of the landfill is at saturation, or field capacity. Any additional moisture beyond field capacity generates leachate. Landfills containing large amounts of paper can retain significant quantities of water without any leachate formation.

The authors discuss the type of information required prior to design of the landfill. Subsurface characteristics of the site, including geologic and hydrologic patterns are required. Well studies, borings, and soil surveys provide the data for design. From the data, water table contour maps, cross-sections of the landfill site, groundwater flow patterns, soil transmissibility, groundwater velocities, and infiltration capabilities are determined.

Further discussion of each of the aforementioned developments is presented. Finally, a case history design of a sanitary landfill in Ledyard, Connecticut is discussed. The methods described in the article were used in the design.

6. The Report to Congress: Waste Disposal Practices and Their Effects on Groundwater, Executive Summary, U.S. Environmental Protection Agency, Office of Water Supply, Office of Solid Waste Management Programs, April 22, 1976 (78).

The report is a summary of an investigation into (a) disposal of waste (including residential waste) which may endanger underground water which supplies, or can reasonably be expected to supply, any public water systems, and (b) means of control of such waste disposal.

The study covers waste disposal activities which result in the actual collection and disposal of liquid, semi-solid, and solid wastes. In addition, resulting contaminants from the disposal practices are defined and their various routes to the groundwater system are outlined. Some of the wastes studied are industrial wastes contained in surface impoundments, municipal and industrial refuse and sludge, septic tank and cesspool wastes, municipal sewage, stormwater runoff, waste brine from petroleum industry, solid and liquid mixing wastes, and animal feedlot wastes.

The report is divided into fifteen sections. The first three sections are concerned with the importance of groundwater as a resource, its nature and extent, and the ways in which it becomes contaminated. The following nine sections of the report are concerned with common waste disposal practices. The disposal practices are discussed in conjunction with the types of wastes discussed in the preceding paragraph. The next section discusses contamination of groundwater due to sources other than waste disposal practices. The final two sections of the report are concerned with existing federal legislation with regard to groundwater contamination and discussions of state and local alternatives for groundwater quality protection.

7,8. Solid Waste Disposal, Volume 1: Incineration and Landfill and Volume 2: Reuse/Recycle and Pyrolysis, Baum, B. and Parker, C., Ann Arbor Science Publishers, Inc., Ann Arbor, Michigan, Vol. 1 1973, Vol. 2 1974(79).

Volume 1 of this book provides valuable information through a detailed examination of two methods for the disposition and disposal of solid wastes. The practice of incineration is treated in terms of its history and its design criteria for both municipal and industrial wastes, capital and operations costs, instrumentation and control of air pollutants. Landfill practices and the design, construction, administration, and economics of sanitary landfill operation are discussed.

Volume 2 explores the vital aspects of reuse, recycling and reclamation of plastic and non-plastic solid waste, with particular emphasis on ways to preserve our natural resources, reduce pollution, conserve energy and reduce costs. Finally, the authors analyze government activity and legislation in this area and project into the future of solid waste management and utilization, including a discussion of the latest disposal methods.

Recovery of solid waste now represents less than 5% of total solids disposed of by homes, industry, commerce and government. The importance of this 2-volume study is that it not only surveys current problems and methods of solid waste disposal, but also presents solutions that can play a significant role in protecting and preserving our environment.

9. "Disposal of Sewage Sludge into a Sanitary Landfill," Report No. SW-71d, U.S. Environmental Protection Agency(1974) (80).

This report describes the results of a three-year investigation of the environmental and economic effects of disposing liquid sewage sludge and septic tank pumpings into a sanitary landfill. The objectives of the study were to determine: (1) the capacity of solid waste to assimilate the moisture in liquid sewage sludge and septic tank pumpings; (2) the significant parameters affecting that capacity; (3) the optimum means for nuisance-free admixture of liquid sludge with solid waste in a landfill; (4) the effects of combined liquid sludge-solid waste disposal on the environment, landfill equipment, operating efficiencies, and personnel performance; (5) the effects of liquid sludge on landfill compaction and solid waste decomposition; and (6) the most economically feasible methods for dewatering, transporting, and disposing liquid sludge.

The three-year study included laboratory evaluations of water absorption by solid waste, pilot-scale simulation of landfill conditions, full-scale field test cells for controlled landfill simulation, full-scale demonstration of liquid sewage sludge disposal into a sanitary landfill, and characterization of the sewage sludges and solid wastes generated by the City of Oceanside. A special nationwide survey of the disposal of sewage sludge and septic tank pumpings into sanitary landfills was made by contacting responsible State public health authorities and municipal landfill managers.

10. "Residual Waste Best Management Practices: A Water Planner's Guide to Land Disposal," Environmental Protection Agency Publication, EPA-440/9-76-022 (in progress) (81).

This handbook describes residual wastes from nine most frequently encountered sources and relates management of these wastes to exhaustive enumeration of Best Management Practices. The solid waste problem assessment procedures that are presented in this report will provide the potential users-planners, engineers, administrators, lawyers, elected officials, and others with a reference for carrying out their residual waste management responsibilities under areawide or state water quality management planning programs and other regional and local activities.

Suggestions are presented for the identification and assessment of residual wastes nonpoint sources and methods are given for the location and description of pollution associated with solid waste disposal. Also included are methods of interpretation of water quality data and description and characterization of loads associated with disposal of solid wastes.

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CHAPTER 5

ANALYSIS OF STREAM IMPACTS FOR URBAN AND NONURBAN SOURCES

5.1 Introduction

The preliminary assessment methodology presented in Chapter 2 provides analysis techniques which can be used to identify existing and potential water quality problem areas. The methods are set in a rather broad planning context which sacrifices details within the study area in favor of regional load assessment and problem area identification. In effect, Chapter 2 provides the 208 planner with planning tools which permit him to reduce the dimensionality of his problem, that is limit the number of river miles and water quality variables which should be subjected to more definitive analysis.

Chapter 5 presents methodologies for more detailed assessments of existing and potential problem areas identified in the preliminary problem assessment phase of the 208 program. In this regard, Chapter 5 includes another level of detail for assessing water quality in streams. Simplified modeling methods are described for determining receiving water quality responses to continuous and intermittent loads described in Chapters 3 and 4. The time and space scales of the modeling framework are reduced from those in Chapter 2. Spatial resolution in the analysis described in Chapter 5 ranges from local to sub-basin scale. The time scale permits an evaluation of water quality response characteristics over the course of a few weeks. This is in contrast to the steady state average annual water quality responses generated in Chapter 2. The principal water quality variables considered in these time and space scales are BOD, dissolved oxygen, suspended solids, total nutrients and coliform organisms. The principal emphasis in the chapter is on urban loads.

Special techniques are described for simplifying the structure of the water quality analysis in streams. These include guidelines for aggregating loads or for treating local distributed loads as point sources and simple methods for incorporating dispersion estimates into the stream analysis. Finally the chapter presents an example stream analysis which draws on load generation techniques and modeling methods presented in Chapters 2, 3 and 4. The example proceeds from the development of the variable loads through model application. Special sub-sections of the chapter cover model calibration and methods for developing time dependent results from steady state water quality models.

Lakes, estuaries and coastal areas are also considered in Chapter 5. The level of detail employed in these analyses is less sophisticated than that for streams, and is similar in scope to the preliminary assessment techniques employed in Chapter 2. The reason is that lakes, estuaries and coastal areas are characterized by complex transport mechanisms, the analysis of which is not easily structured in the simplified form which characterizes advective transport in streams. Furthermore, the nature of water quality problems in these systems frequently requires multi-dimensional analyses or solutions involving complex reactions and interactions between variables, and are beyond the scope of the present manual.

Chapter 5 does, however, present methods for identifying and analysing existing and potential water quality problem areas in complex water bodies. In certain cases, these methods will be satisfactory by themselves. In others, the analysis will point to problems which can only be satisfactorily resolved with numerical modeling techniques. Criteria for making these distinctions are presented in this chapter.

Finally, Chapter 5 presents guidelines for using numerical computer models to analyse specific problems which are not amenable to analysis techniques described in this manual.

Chapter 5 presents a simplified procedure for analysing a set of complex and very sophisticated water quality problems, those associated with stormwater. The analysis includes methodologies for assessing storm water impacts in streams which can be performed with an electronic calculator or with relatively simple computer programs which are readily available. The purpose of this analysis is to reduce the complexity of the water quality modeling effort to the point where reliable assessments of storm water impacts and their controls can be made with a minimum expenditure of resources.

The methods in this manual are not viewed as a replacement for more sophisticated storm water modeling frameworks. Instead, they are designed to be part of the 208 planner's library of analysis tools. Until this time, there has been no reliable storm water evaluation methodology which could be used to determine storm water control requirements in the time frame of a few weeks. Most storm water programs require at least that time to develop the data input for the model. The present method is designed to satisfy the need for a reliable yet inexpensive evaluation of storm related problems.

The method employs steady state water quality models of stream systems. In this regard, the fundamental tools are classical and familiar to most 208 planners. The method, however, yields results which contain a great deal of time variable information. Consider a stream which is subjected to pulse loadings from storm events. The water quality response can be viewed as a series of pulse storm water responses which spread as they move downstream. The steady state model must be interpreted in a very specialized way in this case. First, the response represents the worst water quality condition at each point downstream, as the pulse travels downstream. Secondly, the model results are attenuated in the downstream direction, reflecting the natural attenuation of the peak response due to longitudinal dispersion. Other refinements and modifications to the steady state model are also included in this chapter. Each of these are included to maximize the time variable information content developed from the model.

5.2 Stream Water Quality Analysis Methods (Non-Steady State)

Steady state stream analysis has been generally considered to be of limited usefulness in evaluating the water quality response due to non-steady state loads. The problem centers on the inability of the steady state model to represent transient water quality responses due to short term load variations. However, because of the ease with which steady state water quality models can be developed and applied, they are frequently used to make estimates of extreme conditions which might result from a time variable load. For example, it is a common practice to use short term loading extremes to estimate extreme water quality occurrences. An analysis of this type is presented in Section 2.7.1.2 (pg. 2-101) where a steady state model is employed to evaluate dissolved oxygen response in the hypothetical South River under an extreme storm event (extreme high flow, extreme wet weather load). The analysis is reasonable, since the duration of the loading event is sufficiently long to approach steady state in the River.

Recent developments, however, have extended the capabilities of simplified analyses in assessing transient water quality responses. Analysis techniques presented in this chapter consider the time variable response to continuous and intermittent loads using modified steady state models. The methods are consistent with others in this manual in that the analysis techniques can be performed without the assistance of computerized numerical models. Before considering these methods, a brief discussion of model verification and calibration is presented at this point because it is clear that an adequate representation of water quality responses can be developed only if the adequacy of the model can be demonstrated by a comparison against field data. These models can be computerized numerical models or, as is used in this chapter, the equations in Table 2-17 (pg. 2-83).

5.2.1 Criteria for Model Verification

Water quality projections have an associated level of reliability which is related to the models ability to reproduce observed receiving water responses in the particular study area being considered. That is, if a model has been developed, its reliability in making forecasts is assessed by its ability to compute water quality similar to that measured in field data collection programs. The methods by which reliability is developed is called model verification analysis.

There are a number of levels of model verification which can be pursued ranging from gross comparisons of computed water quality with a single set of grab samples to a comprehensive verification, which utilizes data sets collected at many combinations of river flow, seasonal factors (temperature, rainfall, etc.) and loading.

The degree to which any model should be verified is normally decided on the basis of:

1. The degree of confidence which must be maintained in the model projections.
2. The potential range of costs which could result from management decisions that the model outputs will impact.
3. The range of flow, temperature, and loading conditions which the model will be used to evaluate.
4. Resources available for model development and testing.

Generally the last item, cost, is an overriding concern since model verification is costly and time consuming. As the model application proceeds in a continuing planning process, factors 1 through 3 may dictate additional model verification work to develop more detailed input to the planning process.

Calibration is a term applied to verification analyses which are limited. Normally full model verifications require model comparisons with three to four complete sets of observed data, while calibrations normally rely on a reduced data base with fewer, or less complete data sets.

Since stream models applying methods contained in this chapter are to be used in generating step 1 planning guidance, some model calibration is justified and encouraged. The level of verification required must be decided on a site specific basis using the four criteria cited above as guidelines. In most cases because of a lack of extensive historical data, model calibration against 1 or 2 sets of available data will be a practical upper limit.

5.2.1.1 Model Calibration Analysis

Model calibration consists of four tasks: Data preparation and analysis, preparation of model input, model application and comparison of model results with measured water quality data.

Chapter 2 describes possible sources of historic water quality data and presents some examples of data presentation methods which are appropriate in a model calibration exercise. For example see Figures 2-2 (pg. 2-12), 2-3 (pg. 2-13), and 2-13 (pg. 2-47). Normally, companion data are available to establish the appropriate river flows, Table 2-3(a) (pg. 2-23), and seasonal temperature, Figure 2-12(b) (pg. 2-35). Chapter 2 provides some guidance in the preparation of loading data. Efforts should be made wherever possible to gather loading data from measurements collected during the receiving water quality surveys. Where data collection is required to develop a data base for model verification, the Monitoring and Surveillance Appendix to this manual provides guidance in developing adequate programs.

The data is then assembled in a manner consistent with the input requirements to the model. Finally, the model results are compared to measured data and an evaluation is made as to whether the model adequately reproduces the observations. If not, the model input data (temperature, flow, load measurements, depths, etc.) should be refined or modifications made in the model coefficients.

When the calibration is judged to be acceptable, the following criteria should be satisfied:

1. Model loads and river flows should be those which were measured.
2. Model coefficients should be within acceptable limits described in Chapter 2.
3. Model coefficients should be internally consistent and should not vary indiscriminately from segment to segment.
4. The computed water quality should at least reproduce major trends in the observations. It is not necessary for the computed water quality profile to go through every data point.

An example of an acceptable calibration using the South River data is demonstrated in Figure 5-1. The calibration analysis indicates that the South River model is only in marginal agreement with the observed suspended solids concentrations. Additional effort should be directed toward refining the model loads which were poorly defined in the existing data base. Calibration of other water quality variables appears to be adequate. A second calibration analysis would enhance the reliability of the model considerably.

Section 5.2.1.1 has reviewed the basic elements of model calibration in general terms, allowing a certain amount of latitude because of the Phase 1 planning function of models developed in this chapter and the site specific nature of water quality problems and verification requirements. A more definitive description of model calibration and verification procedures can be found in references (1, 2) and in the model applicability Appendix.

5.2.2 Time Variable Load Characteristics

5.2.2.1 Continuous Loads

Time variable loads are broadly categorized as continuous and intermittent. Continuous loads are those which are always or nearly always discharging within the time scale of the analysis. The magnitude of the loading, however, can be variable in time. For most applications the non-steady state characteristics of continuous discharges can be ignored because the load variability is small relative to the mean loading, at least within the time scale of the problem. For example, most municipal and industrial discharges have relatively constant dry weather mass loads. This is shown schematically in Figure 5-2(a).

Other municipal loads may exhibit large load variations reflecting the inflow characteristics of combined sewer systems. In such cases, the load might be treated as constant only during dry weather periods, and the loading dynamics might have to be incorporated into the receiving water analysis during wet weather simulations. The degree to which this distinction has to be made is dependent upon the magnitude of the load

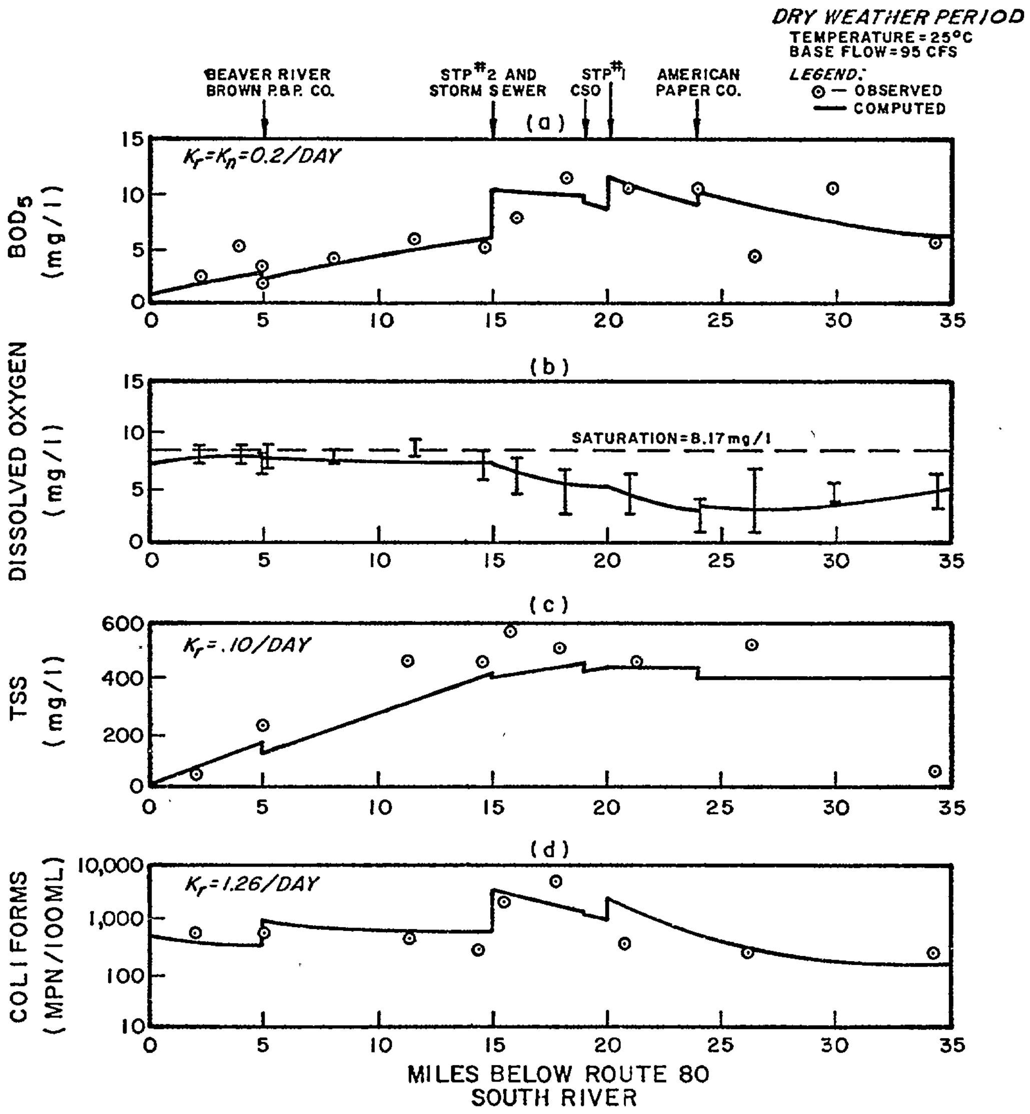


FIGURE 5-1
HYPOTHETICAL SOUTH RIVER-MODEL CALIBRATION ANALYSIS

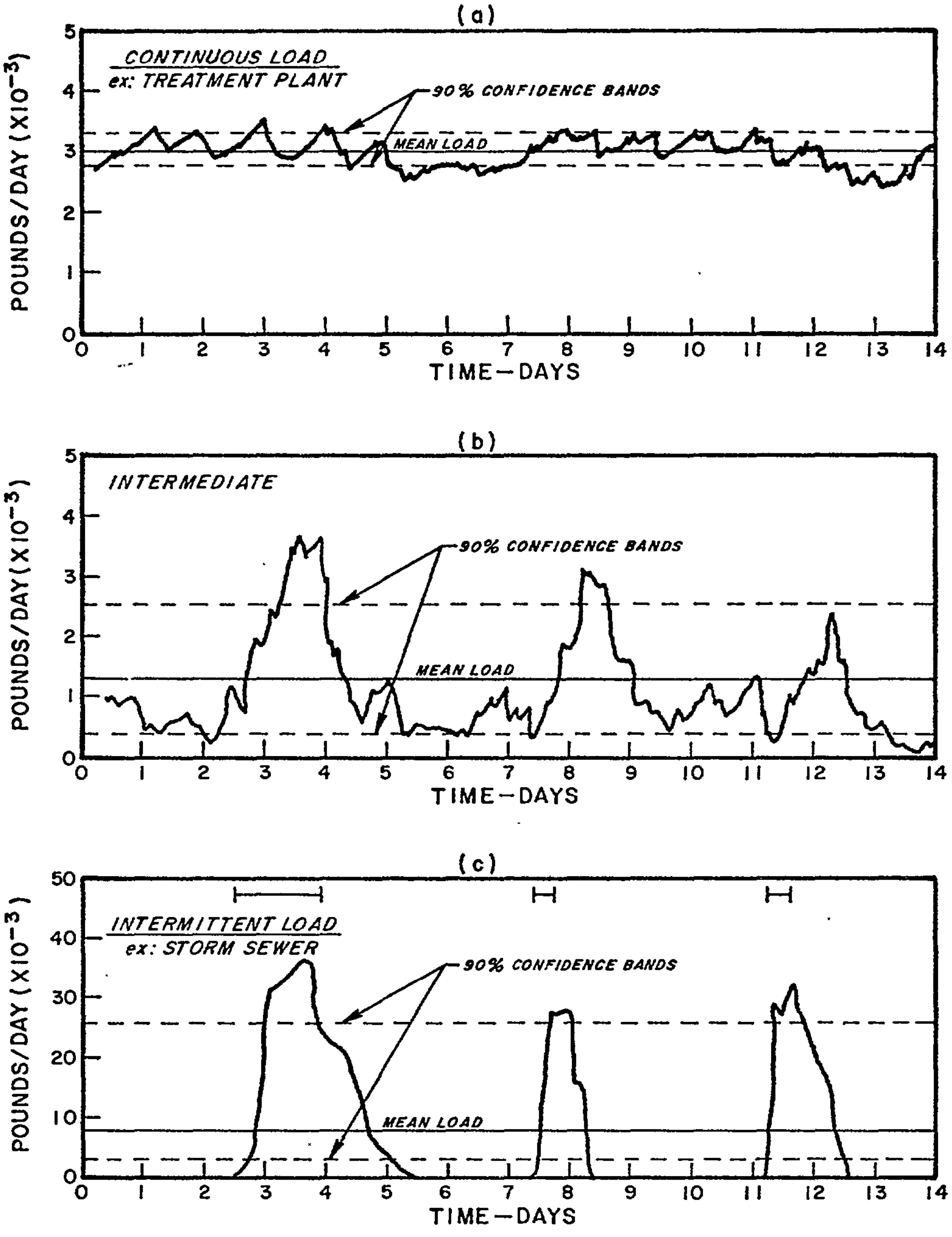


FIGURE 5-2
EXAMPLES OF LOAD TYPES

variability and must be evaluated on a case by case basis. In general the hydraulic capacity of the plant and the combined sewer system puts an upper limit on the load variability.

5.2.2.2 Intermittent Loads

The second category of time variable loads, the intermittent load, is characterized by an on-off type of behavior. Typical of this type of load is the loading from a storm water outlet. During dry periods, the discharge is zero, and during and immediately after the storm, a major pulse load occurs. Figure 5-2(c) typifies this behavior. Normally intermittent loads cannot be effectively treated as steady state. The loading is simply not sustained for a period of time which approaches the time-to-steady state in the receiving water. A number of water quality analysis techniques exist for evaluating receiving water responses to this type of pulse load. Most of these involve detailed time variable integrations of receiving water equations with detailed loading histories as inputs.

5.2.2.3 General Classification of Loads

Between the continuous and the intermittent loads is a spectrum of the time variable loadings which possess characteristics of both types. Typical of these loads are loadings from wastewater treatment facilities servicing combined sewer systems and industrial loads having high volumes of batch production. It is important for the engineer to analyse the load mix to a receiving water and segregate the various loads into steady state and time variable categories. Each loading type is treated differently in the water quality simulation analysis.

5.2.3 Stream Response Characteristics

The water quality response in streams varies depending upon the nature of the loading function. Stream response to continuous point and distributed sources is discussed in Chapter 2. Steady state responses to conservative, reactive and coupled system variables is given in that chapter. Time variable loads from continuous discharges and other intermittent loadings result in similar water quality responses. However, in this case, the response has an associated variability which is largely dependent upon the variability of the loading function.

In order to understand the relationship between load variability and water quality variations one can view the stream as a purely advective system, that is a plug flow system. If such a system is loaded with a series of pulse loads of a conservative tracer as indicated in Figure 5-3(a), measurements at a downstream point would yield a series of pulse responses as indicated in Figure 5-3(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 5-3(c). Dispersion in stream systems can be neglected where the time of travel is short. However, in other situations it is a factor in attenuating the response to discrete pulse loading events.

Analysis of the dispersion problem can be accomplished using the curve displayed in Figure 5-4. The Figure displays percent attenuation of the peak concentration as a function of travel time in the stream and the duration of the rainfall event. Steady state model responses can be adjusted accordingly to account for dispersion effects. A complete discussion of the theoretical basis for Figure 5-4 is contained in reference 3. Computer programs discussed in Appendix A can also be used to evaluate the problem further. In any event, methods presented in this manual will yield conservative water quality responses downstream of the point at which dispersion becomes a relevant factor. Dispersion in stream systems is included in the present analysis.

Chapter 3 described the statistical characteristics of intermittent load histories such as that displayed in Figure 5-3. The underlying density function was shown to be gamma distributed, having a mean, W_r, and a coefficient of variation, ν_w. The coefficient of variation is defined

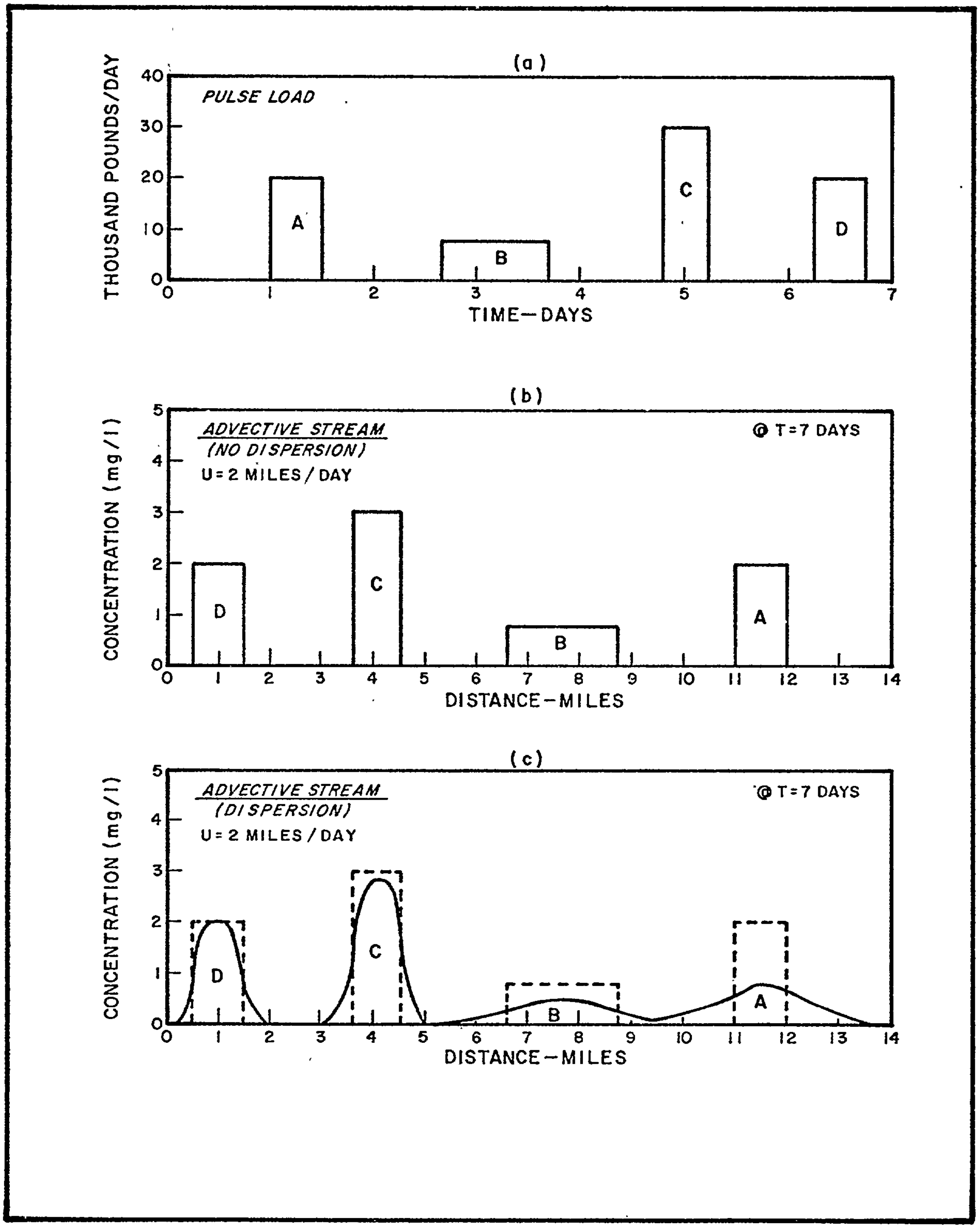
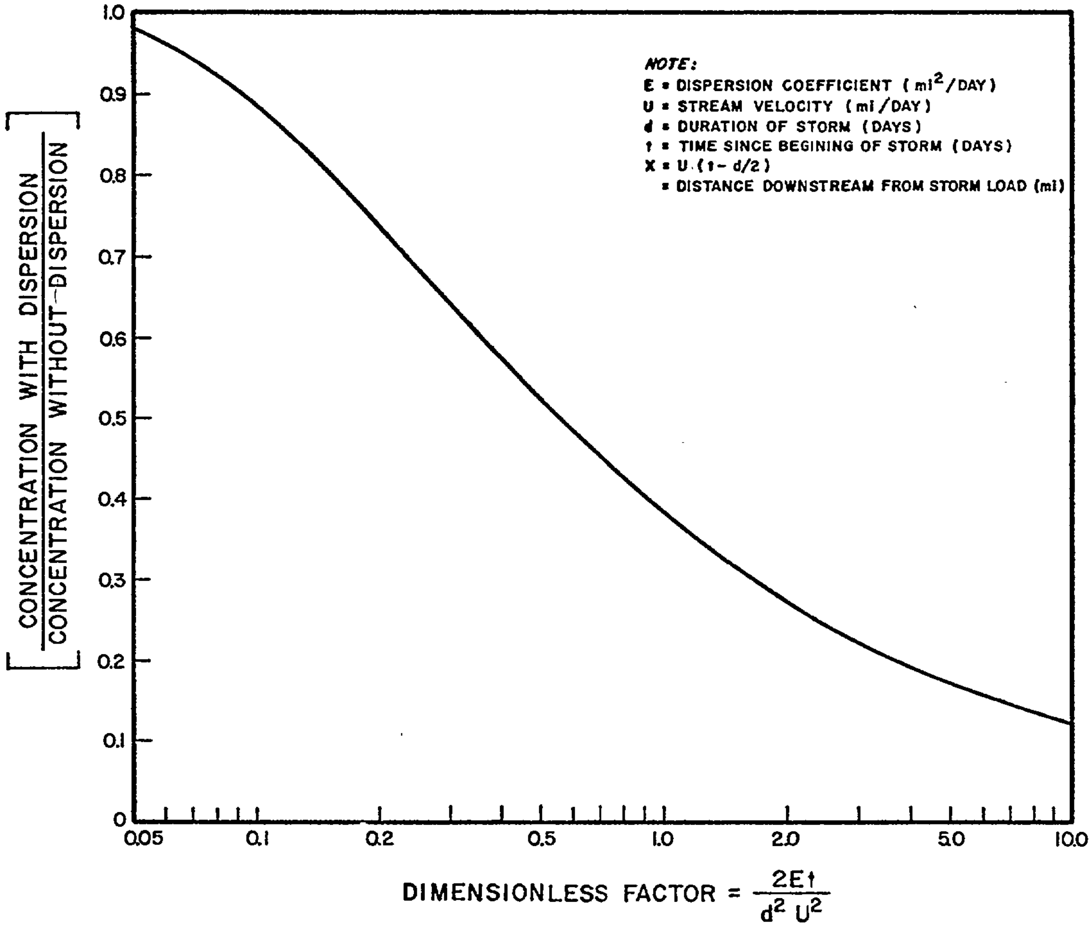


FIGURE 5-3
STREAM RESPONSE CHARACTERISTICS TO PULSE LOADS



NOTE:

E = DISPERSION COEFFICIENT (mi^2/DAY)
U = STREAM VELOCITY (mi /DAY)
d = DURATION OF STORM (DAYS)
t = TIME SINCE BEGINING OF STORM (DAYS)
X = U·(t − d/2)
= DISTANCE DOWNSTREAM FROM STORM LOAD (mi)

FIGURE 5-4
EFFECT OF DISPERSION ON POLLUTANT CONCENTRATION AT MIDPOINT OF STORM PULSE

as the standard deviation divided by the mean. The subscript w denotes that this is a statistic of the load W(t).

The analysis methods for evaluating in-stream water quality responses due to intermittent loads utilizes the statistics of receiving water quality due to that load. The behavior of the system can be thought of as analogous to the pulse load example presented in Figure 5-3, the difference being that instead of inputting a time history of discrete load events, the statistics of the history of those events is input to the stream model. The model then generates a statistical water quality response consisting of a mean and a standard deviation at all points downstream of the load. The frequency distribution of the water quality response is identical to that of the input load for the case of a single load or a closely grouped set of similar loads. For cases where there are numerous loads, all having different undefined density functions, the Central Limit Theorem suggests that the downstream water quality frequency distribution approaches a normal distribution. However, in cases where the spatial extent of the study area is small, and dominated by storm related loadings the underlying water quality frequency distribution is best described by that for the loads (i.e., a gamma distribution).(4)

The concept is shown diagramatically in Figure 5-5. Here two loads are shown: a continous steady state load and an intermittent load. The statistics of the loads can be generated using the methods presented in Chapter 3. The continuous load is characterized completely by its mean, $\overline{W}$, (Chapter 2), the intermittent load is characterized completely by its mean, W_r, its coefficient of variation, ν_w, and its distribution function.

The stream model shown in the Figure is the subject of discussion in the next section. It consists of the stream characteristics described in Chapter 2: channel geometry, reaction rates, and transport characteristics. Its function is to translate the load statistics from various sources into a water quality response having a computed mean and standard deviation. An alternate method for achieving similar results using empirical relationships is demonstrated in Section 5.3.4.

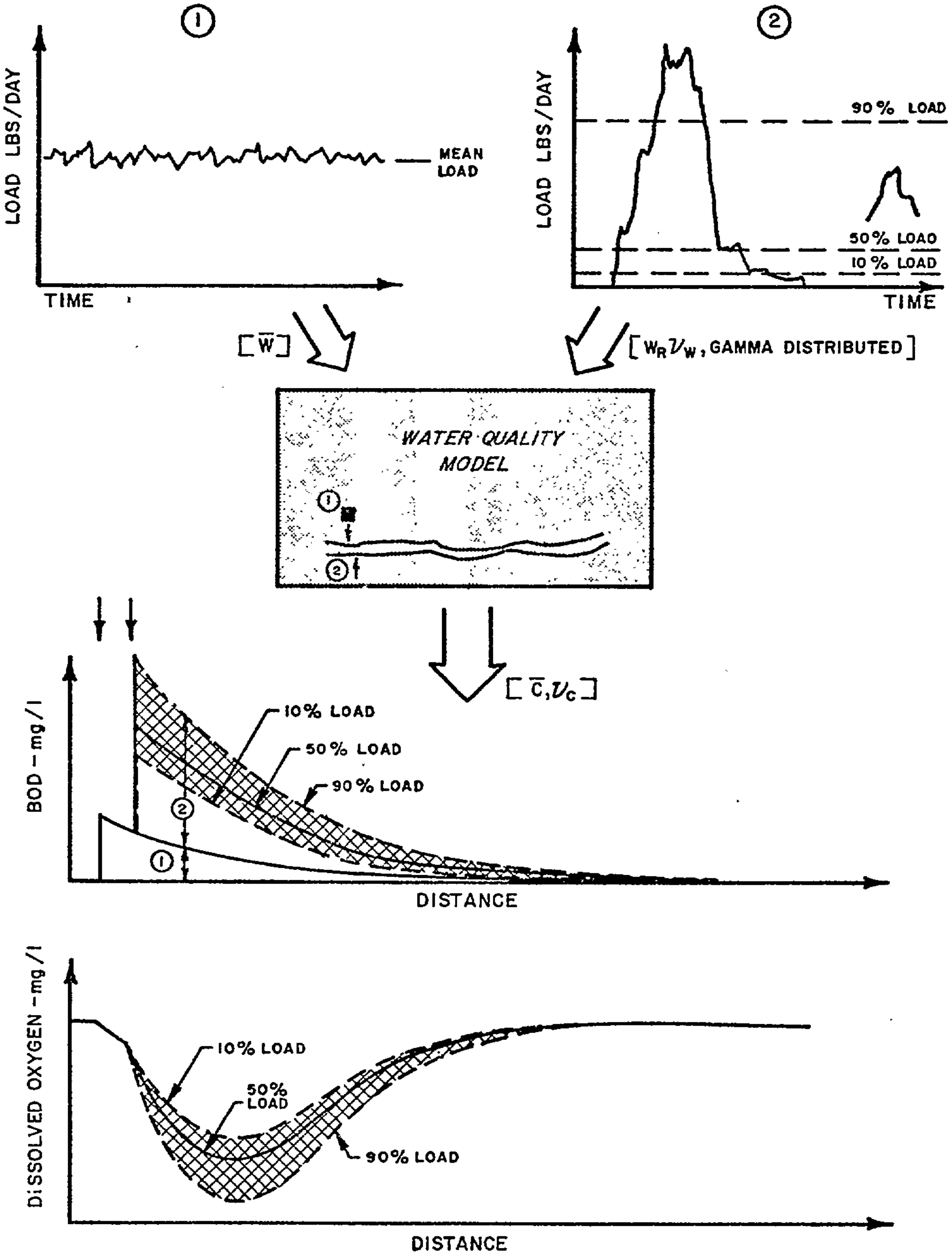


FIGURE 5-5

WATER QUALITY RESPONSE SIMULATOR
BOD-DISSOLVED OXYGEN EXAMPLE

5.2.3.1 Time Variable Water Quality Simulator

In Chapter 2 a summary of equations for computing pollutant concentrations in receiving waters are presented (Table 2-17, pg. 2-83). The reader is referred to Section 2.6.3.1 through 2.6.3.4 (pg. 2-82 to 2-97) for a detailed discussion of the coefficients in these equations.

In the special case of constant flow advective systems, the variability characteristics of the response function as a function of load variability are well known. 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(5). That is:

$$\nu_c = \frac{\sigma c}{\bar{c}} = \nu_w = \frac{\sigma w}{\bar{w}} \qquad (5\text{-}1)$$

where: ν_c, ν_w are the coefficients of variation of concentration and load

σ_c, σ_w are the standard deviation of concentration and load

$\bar{c}$, $\bar{w}$ are the mean concentration and mean load.

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 using the above relationships. This is a valid and recommended approach to analysing variable load impacts on streams where the constant flow assumption holds.

For example if one knows that a single point source has a mean load of 10^3lb/day, a standard deviation of 10^2lb/day, and a normal probability density function, it is a trivial problem to determine the mean load response and superimpose on that response a normal probability distribution having a coefficient of variation of 0.10. The 68% variability around the mean response in this case would be: $\nu_c \cdot \bar{c}$, or $0.1 \cdot \bar{c}$. This formulation only applies to constant flow systems.

However, in situations where intermittent loads such as storm related loads exist the impact of the load on the advective flow is often a major factor. Thus:

$$\nu_c \neq \nu_w$$

and other approaches must be developed. One such approach is presented in the example problem of Chapter 5. An alternative approach is presented in Reference (6).

As in Chapter 2, the unit response in water quality due to a single load to a linear system can be computed independently of all other loads. This is also true in the time variable simulator. However, a different set of ground rules must be established for combining the water quality responses due to a number of intermittent loads.

One base premise can be established: the water quality responses due to mean loads calculated from equations presented in Table 2-17 (pg. 2-83) are additive. That is the total mean water quality response is the sum of responses due to mean loads at all discharge points. However, the frequency distribution of the response function is normally not known even if the load frequency distributions are defined. Thus questions regarding expectations of water quality responses having specified levels are difficult to answer given the mean loads and their probability density functions.

The approach taken in this manual is one where the load statistics are developed in sufficient detail to permit statements regarding the expectation of various loads. The storm related loads are then interpreted in terms of the frequency with which they are expected to occur during wet weather (due to storms) and during longer term periods which include storms (i.e., season, year). The storm loads of given frequency and their associated flow are then used to compute water quality responses which will occur with approximately the same frequency as those loads. For example, if one determines that the 60% storm event (that which will be exceeded by only 40% of the storms) is a loading event which will only be exceeded 2% of the time, the water quality response due to that load is expected to occur with roughly the same frequency, 2% of the time. Within this analysis framework the storm related loads are expected to be correlated to a relatively high degree. That is, large storms cause correspondingly large loads at all wet weather discharges in an urban area.

The pollutant concentrations developed using this method will generally be within the accuracy of the level of stream analysis presented in this chapter. Methods for combining unit responses, discussed in Section 2.6.3.1, are appropriate within the context of Chapter 5.

5.2.3.2 Aggregating Loads for Representation as Point Sources

In some cases where a number of similar loads are located close to each other relative to the spatial scale of the water quality problem being modeled, it is possible, and in most cases desirable, to aggregate loads into a single point source. This treatment of loads simplifies the analysis with little effect on the accuracy of the calculated receiving water quality.

In general loads can be aggregated with no more than a 5% error in accuracy if they lie within a distance described by:

$$x \leq 0.05 \frac{U}{K} \tag{5-2}$$

where: U = river velocity (miles/day)
K = first order reaction rate (per day)

For example, consider the carbonaceous BOD load distribution shown in Figure 5-6(a). If the river flow is estimated to be 10 miles/day and the first order BOD reaction rate is taken as 0.2 per day, the distance over which loads can be aggregated is 2.5 miles:

$$x \leq 0.05 \frac{(10)}{0.2} \leq 2.5 \text{ miles}$$

The resulting aggregated point source loads are shown in Figure 5-6(b). The basis for equation(5-2)is a simple computation which evaluates the equation for a first order reactive substance (Table 2-17 pg. 2-83) at the point where the discharge occurs and at a downstream point where the concentration has been reduced by 5%. Equation(5-2)results when the equations at the two points are subtracted.

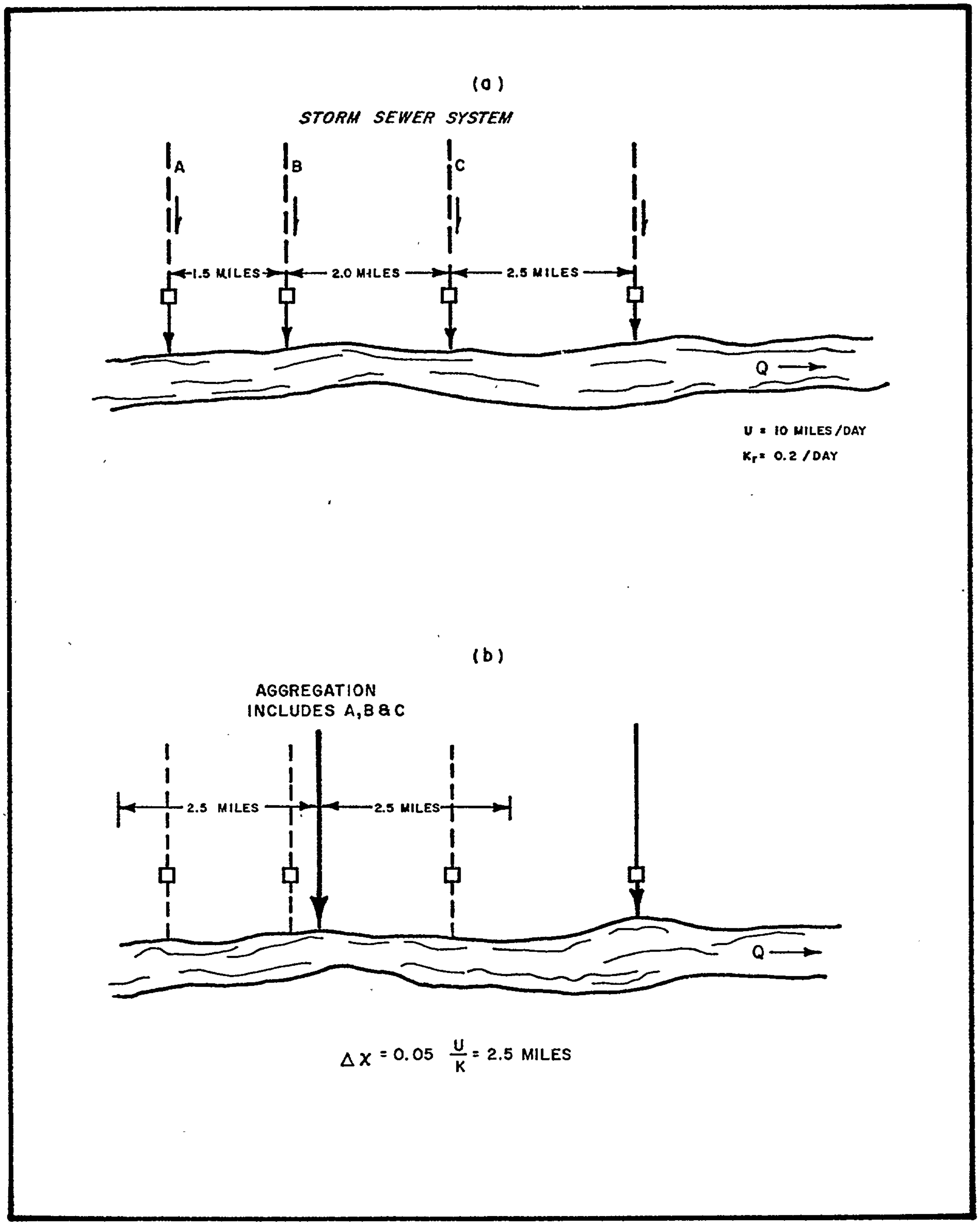


FIGURE 5-6
METHOD FOR AGGREGATING LOADS IN STREAMS

5.2.4 Application of Stream Impact Analysis Methods

Analysis methods presented thus far are illustrated in this section. The principal emphasis will be to apply the load generation techniques from Chapter 3 and the stream impact analysis methods discussed thus far in Chapter 5 in a typical 208 planning area setting. The hypothetical Jefferson City study area which was analysed in a preliminary manner in Chapter 2 is used for this purpose.

Analysis methods demonstrated here are more detailed than those from Chapter 2. Some of the simplifying assumptions regarding urban land use, sewerage systems and river characteristics are replaced with detailed representations more consistent with study area characteristics which the 208 planner is likely to encounter. In addition, a set of realistic, but hypothetical, problem constraints are imposed to demonstrate a broader scope of planning problems and opportunities. While the example analysis is designed to be instructional in nature, it is site specific to the Jefferson City study area. Therefore the 208 planner should use the methodology behind the example as a pattern for structuring a water quality impact analysis in his specific study area. He should not simply attempt to reproduce the computations presented in this section in another study area.

As indicated above, the problem setting is basically the same as the Jefferson City-South River problem setting from Chapter 2. The following modifications are instituted, however, to make the study area more realistic:

1. The urban area is described in terms of its major land use classifications.
2. A second treatment plant is included at an upstream location.
3. The major features of the sewer systems are described.
4. A water treatment plant is located above the city.
5. A realistic set of population projections are developed.

These details are presented in Section 5.2.4.1.

5.2.4.1 Problem Setting

The hypothetical Jefferson City is located on the South River (Figure 2-5 pg. 2-18). The urban area consists of two sewer districts designated Sewer District No. 1 and Sewer District No. 2. These are shown in Figure 5-7. District No. 1 located on the north side of the River has a 20 year old combined sewer system which services an area of 8,000 acres. A primary wastewater treatment plant having an average daily design capacity of 9 MGD services the area. The plant is presently operated at its full design capacity. Present plans are to convert this plant to a secondary treatment facility within the planning period. Both the combined sewer system and the plant have a peak hydraulic capacity of 36 MGD. Combined sewer overflow regulators bypass excess flows to the South River at three locations indicated in the figure.

Projected land use types within sewer District No. 1 at the end of a 20 year planning period are shown in Figure 5-8. The area is an established urban area having a central commercial district surrounded by light industry and residential housing. Industrial wastewaters are presently collected at a central location and pumped untreated to the primary treatment plant through a force main.

Sewer District No. 2 was constructed 10 years ago to service the rapidly growing area to the west and south of the central City. Development style housing and a new commercial district contribute to low to moderate population density within this area at present. Planning projections for the area indicate a trend toward more dense residential housing in the future. Figure 5-8 shows the projected land use patterns. A separate sanitary sewer system services the district and conveys treated wastewaters to a secondary treatment plant at the west end of the City. A force main conveys sanitary wastewaters from the south side of the South River to the plant which has an average daily design capacity of 12 MGD. The plant presently treats 11 MGD. A storm sewer system services District No. 2. Its design capacity is in excess of 900 MGD. Overflows operated by weir type regulators are activated by excess flows.

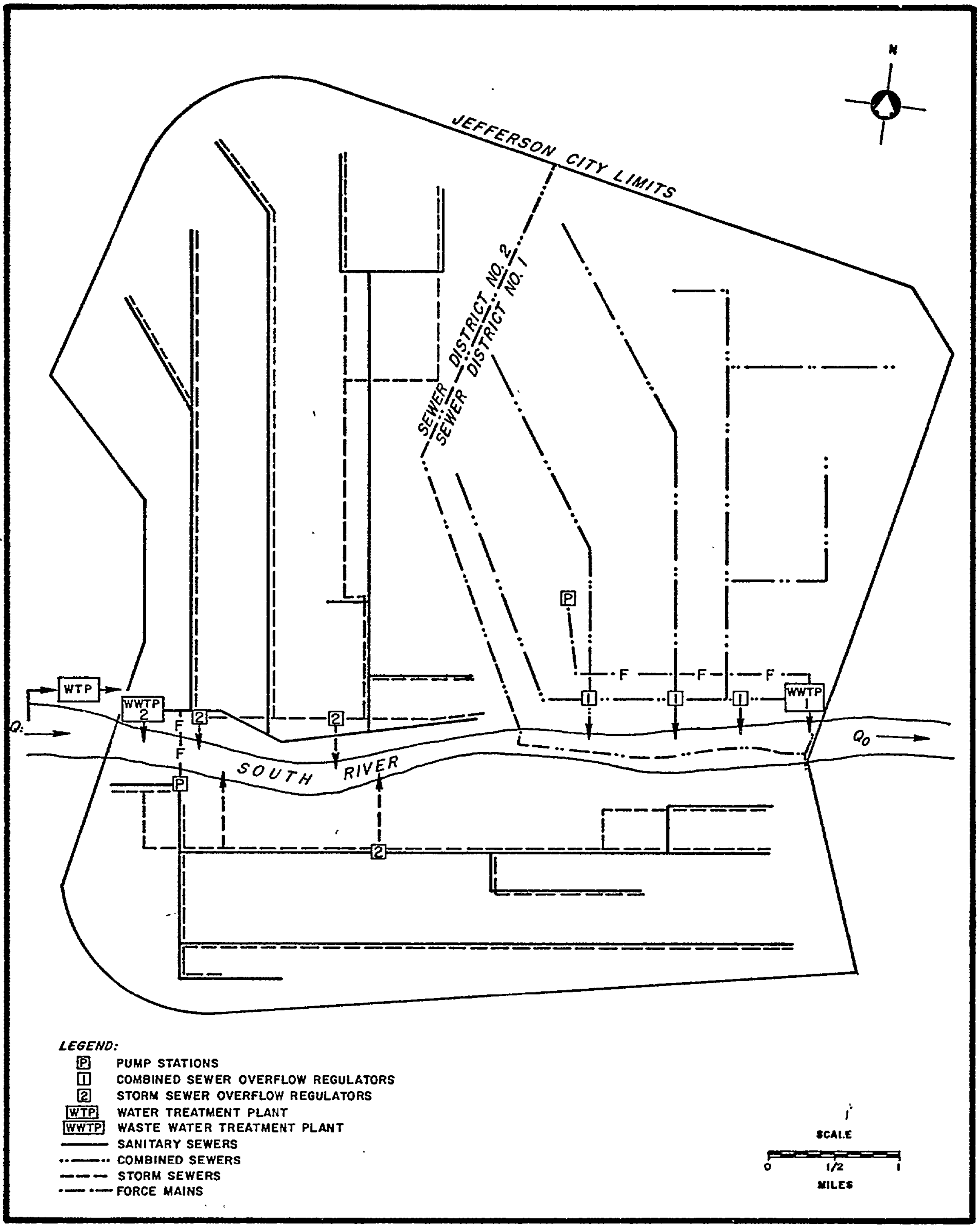


FIGURE 5-7

JEFFERSON CITY STUDY AREA
COLLECTION SYSTEM

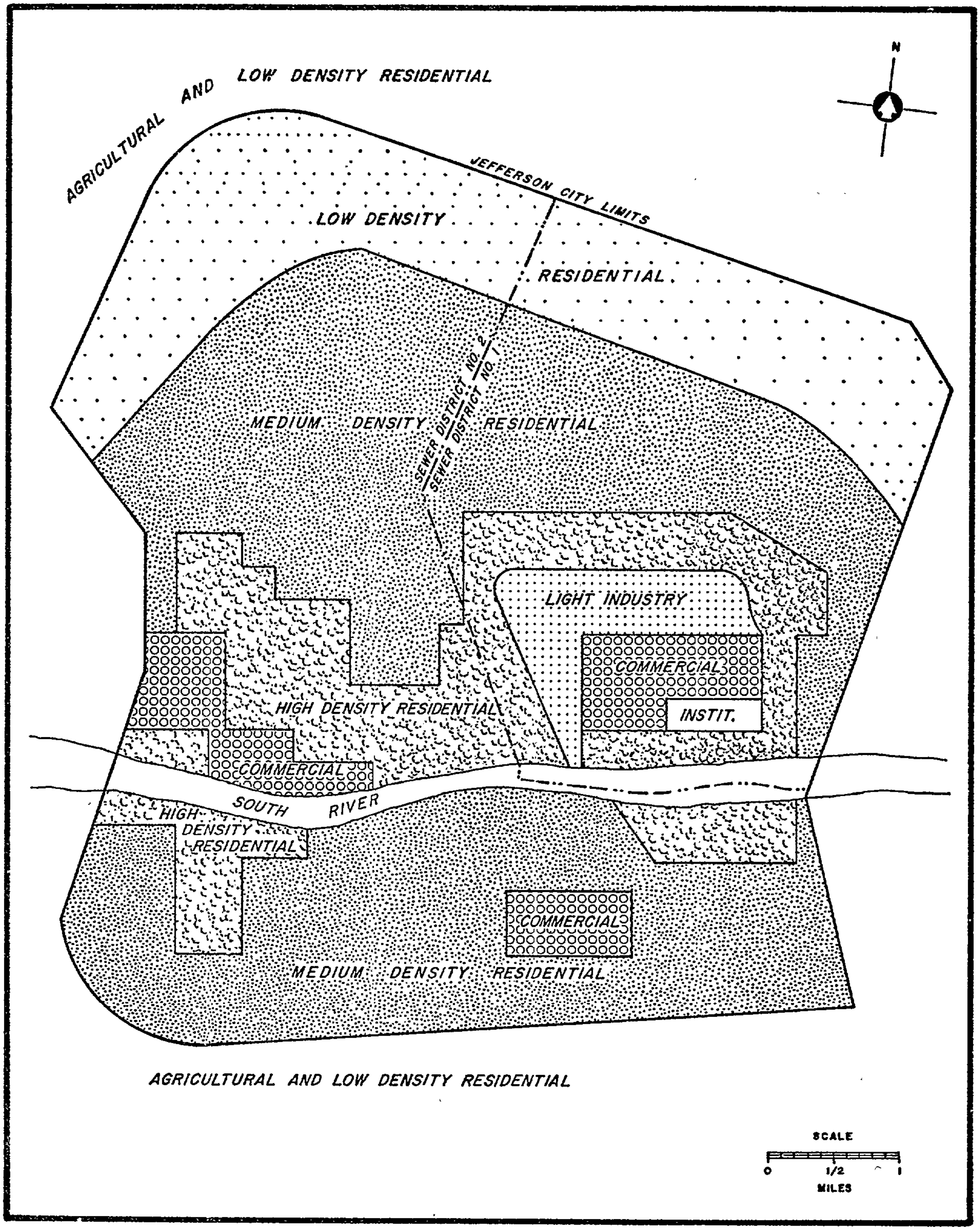


FIGURE 5-8

JEFFERSON CITY STUDY AREA
LAND USE CLASSIFICATION

A water treatment plant having a design life of 30 years was just constructed in a rural area west of town to service Jefferson City. The entire water supply for the City is withdrawn from the South River at this point.

Continued growth is expected throughout the urban-suburban area. Recent population studies conducted as part of the water supply plan are displayed in Figure 5-9. The population figures have been reworked along sewer district boundaries. In general, District No. 1 is expected to have only moderate growth during the next 25 years, while District No. 2 is expected to increase in population by between 35 and 110 percent.

5.2.4.2 Load Estimation

Load estimates for the Jefferson City study area are generated in a preliminary fashion in Chapter 2. The reader is referred to that chapter and the load generation techniques in Chapter 3 for specific details which are not repeated here. Non-urban loads from upstream and surrounding forest and rural areas are derived from Chapter 2.

The rainfall characteristics in the hypothetical South River Basin are taken as those for U.S. Weather Bureau Station 052220 displayed in Figure 3-15 (pg. 3-60). Two periods are considered in this chapter: the average summer storm condition (June through September), and the period of peak storm intensity, July. The rainfall characteristics during these two periods are displayed in Table 5-1.

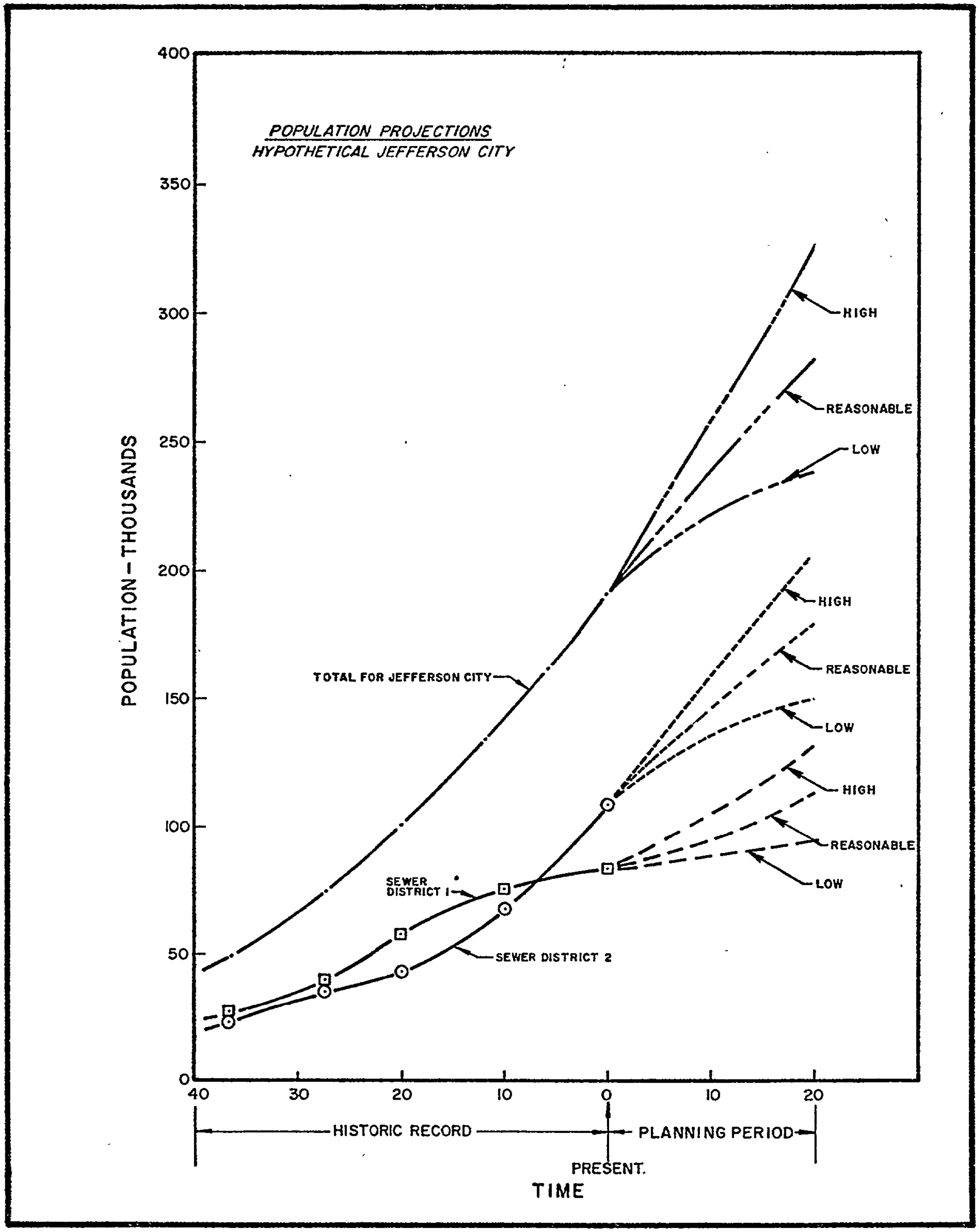


FIGURE 5-9

POPULATION PROJECTION
JEFFERSON CITY

TABLE 5-1

RAINFALL CHARACTERISTICS
HYPOTHETICAL SOUTH RIVER STUDY AREA

(a) SUMMER STORM PERIOD - JUNE THROUGH SEPTEMBER

Characteristic	Mean	Variation
Storm Intensity	I = 0.055 in/hr	$\nu_i = 1.55$
Duration	D = 3.0 Hrs.	$\nu_d = 1.15$
Volume	V = 0.18 in	$\nu_v = 1.90$
Time Between Storms	Δ = 80 Hrs.	$\nu_\delta = 1.15$

(b) JULY STORM PERIOD

Characteristics	Mean	Variation
Storm Intensity	I = 0.062 in/hr.	$\nu_i = 1.50$
Duration	D = 2.5 Hrs.	$\nu_d = 1.00$
Volume	V = 0.17 in	$\nu_v = 1.20$
Time Between Storms	Δ = 70 Hrs.	$\nu_\delta = 1.20$

Projected land use classifications within the two sewer districts for the 20 year planning period are displayed in Table 5-2. The information in this table was developed by the 208 study program and indicates anticipated land use classifications 20 years in the future. The total land area in District No. 2 is twice that for District No. 1. Both districts are expected to be dominated by medium density housing.

TABLE 5-2

LAND USE CLASSIFICATION IN THE HYPOTHETICAL JEFFERSON CITY
(Based on Population and Land Use Projections)

	S. D. No. 1		S. D. No. 2		Total	
Classification	Area (Acres)	%	Area (Acres)	%	Area (Acres)	%
Low Dens. Hous.	2,120	26.6	2,640	17.6	4,760	20.7
Med Dens. Hous.	2,800	35.1	8,800	58.6	11,600	50.5
High Dens. Hous.	1,600	20.0	2,640	17.6	4,240	18.4
Commercial	540	6.8	940	6.2	1,480	6.4
Institutional	120	1.5	-	-	120	0.5
Light Industry	800	10.0	-	-	800	3.5
Totals	7,980	100.0	15,020	100.0	23,000	100.0

Using the methods described on page 3-30 the percent imperviousness of the urban area within the two sewer districts is computed to be:

Sewer District No. 1 - 40.9% impervious
Sewer District No. 2 - 42.5% impervious

The runoff coefficients for the two Districts developed from Figure 3-8 are:

Sewer District No. 1, $C_v = .45$
Sewer District No. 2, $C_v = .47$

These are in agreement with the value of 0.42 developed in the preliminary assessment (pg. 2-59 and Figure 2-15).

Runoff from the study area for the storm periods described in Table 5-1 are computed using methods from Section 3.4.3.2. A sample analysis of this type is presented on pages 3-59 and 3-60. The anticipated probability distribution of runoff flows 20 years in the future are presented in Table 5-3.

TABLE 5-3

RUNOFF FLOW, $Q_R = C_V IA$
(IN CUBIC FEET PER SECOND)
20 YEARS IN FUTURE

% of Storms Less Than	Multiple of the mean Factor	S. D. No. 1		S. D. No. 2	
		Summer Storms	July Storms	Summer Storms	July Storms
20	(.10)	20	23	39	44
50(MEAN)	(0.4)	79(198)	93(233)	155(388)	175(437)
75	(1.4)	277	326	543	611
90	(2.7)	535	602	1048	1180
95	(3.9)	772	870	1513	1704

where: $\nu_q = \nu_i$ = 1.55 for the average summer storm condition;
ν_i = 1.50 for the peak intensity storm period, July;
The factor is the multiple of the mean from Figure 3-11b for $\nu = 1.55$.
( ) = The mean runoff flow in cfs calculated using Equation 3-15.

The product of the factor and the mean flow yields the runoff flow having the indicated probability. For example the 75% runoff from Sewer District No. 1 in July is 1.4 (233) = 326 cfs.

The loads associated with these flows can now be computed using methods presented in 3.4.3.3 and demonstrated on pages 3-60 through 3-62. In general pollutant concentration is assumed to be independent of flow, an adequate assumption in the absence of site specific data. That is, the mean concentration during storm events is only weakly correlated to the mean flow during that event. However, best engineering practice is to obtain measurements of pollutant concentrations at various storm water flow conditions and verify concentration-flow independence on a site specific basis. In cases where significant long term correlations do exist, an appropriate adjustment to the analysis procedure would have to be made.

The basic data for computing the storm loads for the hypothetical Jefferson City urban area are contained in Table 3-3. Without a correlation for the specific site between the concentration and total runoff flow, the variability of the flow and concentration cannot both be used as in equation 3-20 to determine the variability of the load. The correlation between concentration and runoff flow is not known on the South River. The coefficient of variation for the loads from the two storm conditions are, therefore, taken as the coefficients of variation of the runoff flow. That is, it is assumed that concentration is essentially constant from event to event and:

Average summer storm condition; $\nu_w = 1.55$

July storm condition; $\nu_w = 1.50$

The assumption of constant concentration appears to be an acceptable simplification based on data collected in numerous U.S. cities. However, an alternative procedure which includes observed variability in concentration is presented on page 3-62 and can be applied where the flow-concentration relationships are available from an observed data base. The loads from intermittent sources in the South River study area are displayed in Table 5-4.

Point source loads from the two municipal wastewater treatment plants in Jefferson City are expected to increase during the planning period. The loadings are developed from the "reasonable" set of population projections assuming the same treatment as presently exists at expanded treatment facilities in Sewer District No. 2, and upgrading of the Sewer District No. 1 plant to at least secondary treatment. The loadings from both plants are estimated in Table 5-5. For example, in Sewer District No. 1, the total nitrogen load using the reasonable population projections (Figure 5-9) is computed as: $w = 20 \text{ mg/l} \times 10^{-6} \times 8.34 \times 1.12 \times 10^{5}$ people $\times$ 150 gpc/day = 2818 lbs/day. Tributary and industrial loads are those presented in Chapter 2.

TABLE 5-4

SUMMARY OF STORM RELATED LOADS TO THE HYPOTHETICAL SOUTH RIVER

(1) BOD_5 Loads[a] - (Thousand Pounds/Day)

% of Storms Less Than	(Factor)[b]	S. D. No. 1		S. D. No. 2	
		Summer Storms	July Storms	Summer Storms	July Storms
20	(0.1)	9.96	12.0	5.6	6.4
50(Mean)	(0.4)	39.8(99.6)	48.0(120.0)	22.6(56.6)	25.5(63.7)
75	(1.4)	139.	168.	79.2	89.2
90	(2.7)	269.	324.0	153.	172.
95	(3.9)	388.	468.0	221.	248.

[a] $\bar{c}$ = 27 mg/l; separate storm sewer - District #2. $\bar{c}$ = 108 mg/l; sewer - District #1.

[b] Factors are from figure 3-11b.

(2) Total Coliform Loads[c] (10^{14} MPN/Day)

% of Storms Less Than	(Factor)[d]	S. D. No. 1		S. D. No. 2	
		Summer Storms	July Storms	Summer Storms	July Storms
20	(0.1)	.29	.03	.05	.05
50(Mean)	(0.4)	1.16(2.9)	1.36(3.4)	.18(.47)	.22(.53)
75	(1.4)	4.06	4.76	.66	.74
90	(2.7)	7.8	9.18	1.27	1.43
95	(3.9)	11.3	13.3	1.83	2.07

[c] $\bar{c}$ = 3 x 10^5 MPN/100 ml; separate storm sewer - District #2
$\bar{c}$ = 6 x 10^6 MPN/100; combined sewer - District #1

[d] Factors are from Figure 3-11b.

(3) Total Nitrogen Loads[e] - (Thousand Pounds of N/day)

% of Storms Less Than	(Factor)[f]	S. D. No. 1		S. D. No. 2	
		Summer Storms	July Storms	Summer Storms	July Storms
20	(0.1)	.83	1.0	.48	.54
50(Mean)	(0.4)	3.3(8.3)	4.0(10.0)	1.9(4.8)	2.2(5.4)
75	(1.4)	11.6	14.	6.7	7.6
90	(2.7)	22.4	27.	13.0	14.6
95	(3.9)	32.4	39.0	18.7	21.1

[e] $\bar{c}$ = 2.3 mg N/l; separate storm sewer - District #2. $\bar{c}$ = 9.0 mg N/l; combined sewer - District #1

[f] Factors are from Figure 3-11b.

(4) Total Phosphorus Loads[g] (Thousand Pounds of P/day)

% of Storms Less Than	(Factor)[h]	S. D. No. 1		S. D. No. 2	
		Summer Storms	July Storms	Summer Storms	July Storms
20	(0.1)	.51	.62	.20	.21
50(Mean)	(0.4)	2.0(5.1)	2.5(6.2)	.76(1.9)	.84(2.1)
75	(1.4)	7.1	8.7	2.7	2.9
90	(2.7)	13.8	16.7	5.1	5.7
95	(3.9)	19.9	24.2	7.4	8.2

[g] $\bar{c}$ = 0.9 mg P/l; separate storm sewer - District #2. $\bar{c}$ = 5.5 mg P/l; combined sewer - District #1

[h] Factors are from Figure 3-11b.

(5) Total Suspended Solids Loads[i] (Thousand Pounds/Day)

% of Storms Less Than	(Factor)[b]	S. D. No. 1		S. D. No. 2	
		Summer Storms	July Storms	Summer Storms	July Storms
20	(0.1)	34.	41.	127	144
50(Mean)	(0.4)	138(344)	165(414)	509(1274)	574(1435)
75	(1.4)	481.	580.	713	804
90	(2.7)	929	1118	3439	3874
95	(3.9)	1342	1615	4970	5597

[i] $\bar{c}$ = 608 mg/l; separate storm sewer - District #2. $\bar{c}$ = 372 mg/l;
$\bar{c}$ = 372 mg/l; combined sewer - District #1

TABLE 5-5

MUNICIPAL TREATMENT PLANT LOADS - 20 YEARS IN FUTURE HYPOTHETICAL JEFFERSON CITY

Discharge	Flow	Total Nitrogen mg/1	Total Nitrogen lbs/day	Total Phosphorus mg/1	Total Phosphorus lbs/day	Total S.S. mg/1	Total S.S. lbs/day
S.D. #1 plant	16.9	20	2818	10	1409	19.8	2791
S.D. #2 plant	30.4	20	5070	10	2535	19.8	5020

Discharge	Flow	BOD_5 mg/1	BOD_5 lb/day	Total Coliforms No./100 ml	Total Coliforms No./day
S.D. #1 plant	16.9	30	4228	1000	$6.4x10^{11}$
S.D. #2 plant	30.4	30	7606	1000	$1.1x10^{12}$

5.2.4.3 Stream Impact Analysis Conditions

The first step in the stream impact analysis is to reverify the conclusions resulting from the preliminary assessment procedures presented in Chapter 2. Those conclusions are summarized in Table 2-26. It is possible that the reestimation of loadings could significantly change one or more of these conclusions.

This task is accomplished by simply repeating the analysis for present and projected loading conditions and verifying that the water quality responses resulting from the more refined loads and the statistical water quality analysis methods of Chapter 5 lead to the same conclusions regarding the probability of potential problems. This of course, assumes that the basic water quality model has been adequately calibrated to the specific site at least for steady state conditions as discussed in Section 5.2.1.

Bottom demands are not included in the South River example. It should be pointed out that many problem cases will have significant dissolved oxygen water quality impacts because of bottom demands. In these cases the analysis framework can be extended to include bottom effects by including bottom demand terms in the equations displayed in Tables 2-17 as discussed in Section 5.5 and illustrated in Table 5-21.

The procedure for applying the time variable water quality method using the statistical loads is illustrated in this section. Because of space limitations a complete set of analyses for the South River will not be developed. Rather, a selected group of problem cases will be analyzed to demonstrate techniques in applying the statistical water quality method and interpreting its outputs. Toward this end, the method application to reverify the conclusions generated in Chapter 2 is omitted and the analysis focuses on the specific problem cases outlined in Table 5-6.

TABLE 5-6

CRITICAL WATER QUALITY CONDITIONS
HYPOTHETICAL SOUTH RIVER

	Water Quality Indicator			
River Flow Condition	Dissolved Oxygen	Sanitary W.Q.	Eutrophication	Suspended Solids
Drought flow	X			
Average summer flow			X	
Peak summer storm flows	X	X		X

Table 5-6 indicates flow conditions for which specific water quality problems will be analyzed. Dissolved oxygen concentrations in the Chapter 2 analysis were shown to be critical during drought flow conditions and during peak storm runoff conditions. The point sources dominated the drought flow response while combined sewer overflows were a principal contributor to the low dissolved oxygen during storm periods. Sanitary water quality problems were maximum during peak storm conditions. The combined sewer overflows contributed a major portion of the loading in this case. The preliminary eutrophication analysis indicated probable water quality problems during long term average loading conditions indicative of the average summer flow condition. The possible suspended solids problem case is reanalysed in this chapter during wet weather periods when urban non-point sources are significant.

The reader should note that the South River study area has been modified somewhat in Chapter 5 to make the example more illustrative. Therefore, water quality responses in the River are expected to be different than those presented in Chapter 2. In addition, the analyses presented here represent conditions 20 years in the future. Therefore, land use types, loadings and river flows developed for future conditions (Sections 5.2.4.1 and 5.2.4.2) are applied in the analysis. Subsequent analysis in Chapter 6 will deal with load allocation techniques, control practices and methods for developing minimum cost solutions to the principal water quality problems.

Existing combined sewer overflow loads from Jefferson City are located at Milepoints 18.5, 19.0 and 19.5, as shown in Figure 5-7. Rather than treating these loads as a uniformly distributed load from Milepoint 15 to Milepoint 20, as was done in Chapter 2, it is reasonable to combine these three discrete point source loads into one point source load located at Milepoint 19, using the criteria presented in Section 5.2.3.3, equation (5-2). Assuming it is desired to maintain an accuracy of approximately 5% when aggregating these loads, it is only necessary to check that the loads are within a distance x given by: $x \leq 0.05\ U/K$. Information contained in Tables 2-3 (pg. 2-34) and 2-16 (pg. 2-81) can be used to determine the river velocity within segment 3 at the lowest flow condition for which the combined sewer overflows contribute a loading to the analysis. The summer average flow satisfies this criteria in that the time averaged combined sewer overflow load is used in this analysis. The river velocity for this period is computed to be 8.48 miles per day. Therefore, loads within $x \leq 0.05\ (8.48)/0.20 \leq 2.1$ miles can be aggregated in computing BOD-dissolved oxygen responses in the South River. The BOD decay rate of 0.18 per day developed in the calibration analysis has been temperature adjusted to 25°C using equation (2-7) (pg. 2-87) in this computation. Hence, it is justifiable to locate the three CSO loads at Milepoint 19 without causing a significant change in the results of this analysis. An additional segment beginning at Milepoint 19 is added to the stream in order to input the load at this location. The segment characteristics are the same as those for segment 3.

The urban runoff load from portions of Jefferson City located within Sewer District No. 2, which was treated as a uniformly distributed load in Chapter 2, will also be treated as a point source load in this example, located at Milepoint 15 as shown in Figure 5-12. The hypothetical South River example now includes 6 constant parameter segments, bounded at Milepoints 0, 5, 15, 19, 20, 24 and 33. A summary of the geometry for the revised model segmentation is developed from Table 2-3(b) (pg. 2-34).

Another consideration is the impact of increasing populations during the planning period and the consequent impact on stream flows within the study area. This effect is minimal in the case of Sewer District No. 2 where the wastewater discharge is immediately below the water intake point. The only difference in flow in the case is consumptive loss which can generally be estimated as 10% of the raw water intake flow. This flow difference is estimated to be 3 cfs.

$$Q_{loss} = .1 \text{ (180,000 people) x } 110 \frac{\text{gallons}}{\text{person/day}} \text{ x } 1.54 \text{ x } 10^{-6} \frac{\text{cfs}}{\text{gpd}}$$

$$Q_{loss} = 3.05 \text{ cfs}$$

The population increase in sewer District No. 1 is estimated to be 19,000 people. Thus the withdrawal flow is estimated to be:

$$Q_{SD1} = 19{,}000 \text{ people x } 110 \frac{\text{gallons}}{\text{person/day}} \text{ x } 1.54 \text{ x } 10^{-6} \frac{\text{cfs}}{\text{gpd}}$$

$$Q_{SD1} = 3.2 \text{ cfs; say 3 cfs}$$

Ninety percent of this flow is reintroduced at Milepoint 20. The resulting drought flow conditions in the river are summarized in Table 5-7.

TABLE 5-7

FUTURE DROUGHT FLOW CONDITIONS IN THE HYPOTHETICAL SOUTH RIVER

Segment	Milepoints	Chapter 2 Flows(1)	River Flow (cfs) Change	Average Drought
1	0- 5	54	-	54
2	5-15	91	-	91
3	15-19	(101)	-6	95
4	19-20	103	-	97
5	20-24	109	+3	106
6	24-33	116	-	113

(1)Table 2-16, (pg 2-81)

5.2.4.4 Computing Water Quality Response Frequency

The frequency with which extreme water quality events will occur due to intermittent loads is generally considered to be a joint probability function incorporating factors such as rainfall intensity and duration, storm water flow concentrations, the interval between storms, and a number of receiving water characteristics such as base flow, temperature, and background concentration. Simplifying assumptions which are based in part on observations and in part on intuitive reasoning are developed in this section to arrive at a best estimate of the relationship between storm related load frequency and the frequency of extreme water quality responses.

One can show that the percent of time that rainfall occurs is simply D/Δ since $\sum_i d_i/\delta_i$ is the total duration of all rainfall events divided by the total time over which the record was gathered. Consider the rainfall characteristics for the South River shown in Table 5-1(a). The percent of time that rainfall occurs is estimated as:

$$P_r(I > 0) = D/\Delta = (3.0/80) \cdot 100 = 3.75\%$$

and the period without rain is:

$$P_r(I = 0) = 100 - P_r(I > 0) = 100 - 3.75 = 96.25\%$$

The probability that a storm related load, W_r, is less than or equal to a given value is estimated using the following equation.

$$P_r(W_r \leq W) = P_r(I = 0) + P_r(I > 0) \cdot P_r(W_r \leq W) \qquad (5\text{-}3)$$

This simply states that the probability that a storm related load is less than or equal to a value, W, is estimated as the probability of not experiencing rainfall, plus the joint probability of experiencing rainfall having a load less than or equal to W. For example, to compute the probability associated with a load which is only exceeded 1% of the time in the South River example:

$$P_r(W_r \leq W) = (100\% - 1\%) = 0.99$$

$$P_r(I = 0) = 0.9625$$

$$P_r(I > 0) = 0.0375$$

$$P_r(W_r \leq W) = \frac{0.99 - 0.9625}{0.0375} = .73$$

That is, the 99% load (that which is only exceeded 1% of the time), W, to the river is the load associated with the 73% storm event. Similarily the 1.5%, 2% and 3% loads can be shown to be associated with the 60%, 46% and 20% storm loads.

A major difficulty in defining statistical loads occurs when two variables (flow and concentration) are varying. The dilemma is one in which the probability of the loading event being exceeded is fixed and the analyst must determine the associated flow.

$$P_r(W_r \leq W) = P_r(c_r Q_r \leq W) \qquad (5\text{-}4)$$

If for example, the probability of a given load being exceeded is 30%, ($P_r(W_r \leq W) = .70$), a wide range of flow-concentration combinations exist which satisfy equation(5-4). The approach to the problem taken in this manual is to assume the concentration associated with the storm event constant, and associate all of the load variability with flow. Therefore, $P_r(W_r \leq W) = P_r(Q_r \leq Q)$. These methods are intended to be for guidance purposes only and can be modified to reflect local conditions and site specific observations regarding the relationship between flow and concentration between storm events.

5.2.4.5 Application to South River Water Quality

Equation(5-1) stated that in cases where the river flow is constant the coefficient of variation of the response function for a particular load is a constant for all distances downstream in a river, and is equal to the coefficient of variation of the point source wastewater input. This is not, however, the case in the South River where storm related runoff events have a significant impact on the stream flow.

The 20%, 50% and 75% storm loads are used to illustrate receiving water responses in this section. These loads are computed to occur 3%, 2% and 1% of the time from equation(5-3). For the urban runoff loading, which has a coefficient of variation of between 1.50 and 1.55 (Section 5.2.4.2), the 75% load is a factor of 1.4 times the mean loading (Table 5-4). The response due to the 75% load may therefore be computed by multiplying the mean load by the constant factor, 1.4, and by performing the analysis as illustrated in Table 2-20 (pg. 2-90) and 2-21 (pg. 2-95) using this load.

Table 5-8 presents an example calculations for the BOD response in the South River during the peak summer storm period using the former technique. The computation begins at segment 3 and illustrates the manner in which the 20%, 50%, and 75% profiles due to the variable load and the constant wastewater treatment plant load may be computed. The average flow, cross-sectional area and stream velocity are first determined, as well as the appropriate temperature adjusted reaction rate. The initial concentration due to the upstream sources (L_o) is then calculated from a mass balance at the beginning of the segment.

The variable storm sewer runoff load and the constant wastewater treatment plant load is entered at the appropriate probability levels desired, here, the 20%, 50% and 75% levels for the storm sewer and the average daily load for the treatment plant. Substitutution of these values into the solutions presented in Table 2-17 (pg. 2-83), results in a spatial

TABLE 5-8

REACTIVE CONSTITUENT (BOD_5) SAMPLE IMPACT CALCULATION
AVERAGE SUMMER FLOW - HYPOTHETICAL SOUTH RIVER EXAMPLE

Segment 3 - Milepoints 15.0 - 19.0

Q = 1124.0 cfs = 513 + 611 (Tables 2-16, 5-3)

A = 1596.4 sq. ft.

U = (Q/A) • (16.36) = 11.51 mi/day

K_r = 0.25/day @ 25°C

L_o = Upstream BOD_5 = 3.03 mg/l

$L_o = (L_o) \cdot (Q_o/Q_1) = (3.03) \cdot (451/1124)$

= 1.21 mg/l (Upstream Q = 451 cfs (Table 2-16))

W_{75} = 75% load = 1.4 x mean load = 89200 lb/day (Table 5-4)

W = 89200 + 7606 = 96806 lbs/day (Tables 5-4, 5-5)
(Storm) (Point)

w = 416 lb/mi • day (AG + FOR) (Table 2-15)

Rewriting Equations of Table 2-17 with conversion factors for units,

$$L(x) = L_o e^{-K_r x/U} + (W/5.4Q) e^{-K_r x/U} + (3.04 \cdot w/(A \cdot K_r)) \cdot (1 - e^{-K_r x/U})$$

for MP 15, X = 0; $L(15) = 1.21\ e^{-0.25(o)/11.51} + (96806/(5.4) \cdot (1124.))\ e^{-.25(o)/11.51}$

$$+ (416) \cdot (3.04/1596.4 \cdot 0.25)) \cdot (1 - e^{-0.25(0)/11.51})$$

$$L(15) = 1.21 + 15.95 + 0.00 = 17.16 \text{ mg/l}$$

for MP 19^-, x = 4; $L(19) = 1.21\ e^{-0.25(4)/11.51} + (96806/(5.4) \quad (1124))\ e^{-.25(4)/11.51}$

$$+ (416) \cdot (3.04/(1596.4 \cdot 0.25)) \cdot (1 - e^{-0.25(4)/11.51})$$

$$L(19^-) = 1.11 + 14.62 + .26 = 15.99 \text{ mg/l}$$

Since the storm sewer load is a transient load, 15.99 mg/l overestimates the expected response at MP 19. Dispersion will diminish the response due to the storm sewer load in the manner shown in Figure 5-4. This effect may be included by considering the response due to the storm sewer load alone, both with and without dispersion.

In this case,

E = 1 mi^2/day

d = 4 hrs = 0.167 days

x = 4 mi.

U = 11.51 mi/day

t = X/u + d/2 = .431 days

and $\frac{2Et}{d^2U^2} = \frac{2(1.0)(.431)}{(.167)^2(11.51)^2} = .233$

From Figure 5-4, $\frac{\text{Concentration with dispersion}}{\text{Concentration without dispersion}} = 0.70$

The response at MP 19 due to the storm sewer, without dispersion, is given by:

$$\frac{W}{5.4Q}\ e^{-K_r x/U} = \frac{89200}{5.4(1124)}\ e^{-.25(4)/11.51} = 13.5 \text{ mg/l}$$

The concentration with dispersion = 0.7 (13.5) = 9.45 mg/l.

The difference, 13.5 - 9.45 = 4.05 mg/l is the amount by which dispersion will reduce the response due to the storm sewer. Hence, including the effects of dispersion on the transient storm load, the BOD concentration at MP 19 can be estimated as 15.99 - 4.05 = 11.94 mg/l.

A similar procedure is carried out for segments 4,5 and 6, with care being taken to account for the effects of dispersion on the response due to both the SS and CSO loads.

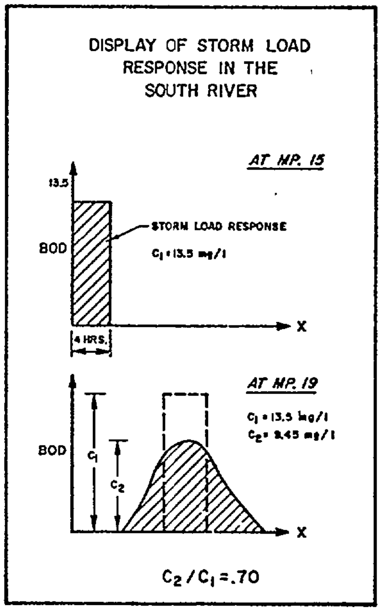

DISPLAY OF STORM LOAD RESPONSE IN THE SOUTH RIVER

TABLE 5-8
(Continued)

COUPLED SYSTEM (BOD-DO) SAMPLE IMPACT CALCULATION
AVERAGE SUMMER FLOW ANALYSIS
HYPOTHETICAL SOUTH RIVER EXAMPLE

Segment 3 - Milepoints 15.0 - 19.0

Q = 1124.0 cfs = 513 + 611 (Tables 2-16, 5-3)

A = 1596.4 sq. ft.

H = 5.39 ft.

$U = (Q/A) \cdot 16.36 = 11.51$ mi/day

$K_r = 0.25$ @ 25°C

$K_d = K_r = 0.25$ @ 25°C

$K_a = 12.96\ U^{1/2}$ (ft/sec)$/H^{3/2}$(ft)

$= 12.96{:}\ 70^{1/2}/4.43^{3/2} = .869$/day @ 20°C

$K_a = .869\ (1.024^{T-20}) = .869(1.024^5) = .98$/day @ 25°C

Def: $D_o = (D_o) \cdot Q_o/Q_1 = (.682) \cdot (451/1124) = .27$ mg/1 (Q_o = 451, Table 2-16)

CBOD: $L_o = (L_o) \cdot Q_o/Q_1 = (3.03) \cdot (451/1124) = 1.21$ mg/1

$W = 7606 + 89200 = 96806$ lb/day(Point Source + Storm Source)(Tables 5-4, 5-5)
(Point)(Storm)

$w = 416$ lb/mi · day (AG + FOR) (Table 2-15)

NBOD: $L_o = 0$

$W = 5070 + 3800 = 8870$ lb/day(Point + Storm Source) (Tables 5-4, 5-5)

$w = 0$ (Assuming AG + FOR non-reactive)

UOD: $L_o = 1.21 \cdot 1.5 + (0) \cdot 4.57 = 1.81$ mg/1

$W = 96806 \cdot 1.5 + 8870 \cdot 4.57 = 185{,}740$ lbs/day

$w = (416) \cdot 1.5 + (0) \cdot 4.57 = 624$ lbs/mi · day

Rewriting equations of Table 2-17 with conversion factors for units:

$$(D(x) = Do\ e^{-K_a x/U} + L_o \cdot \frac{K_d}{K_a - K_r} \cdot (e^{-K_r x/U} - e^{-K_a x/U}) + (W/Q \cdot 5.39) \cdot \frac{K_d}{K_a - K_r} \cdot (e^{-K_r x/U} - e^{-K_a x/U})$$

$$\frac{w}{A \cdot K_r} \cdot \frac{K_d}{K_a - K_r} \cdot 3.04 \cdot (\frac{K_r}{K_a} e^{-K_a x/U} - e^{-K_r x/U} + \frac{K_a - K_r}{K_a})$$

At MP 15, $x = 0$, $D_o = .27$ mg/1 and D.O. = 7.9 mg/1

At MP 19^-, $x = 4^-$,

$$D(4^-) = .27\ e^{-.98(4)/11.51} + 1.81\ \frac{.25}{.98 - .25}\ (e^{-.25(4)/11.51} - e^{-.98(4)/11{,}51})$$

$$+ (185740./(1124. \cdot 5.4)) \cdot \frac{.25}{.98 - .25}\ (e^{-.25(4)/11.51} - e^{-.98(4)/11.51})$$

$$+ (\frac{624}{1596.4 \cdot .25}) \cdot (\frac{.25}{.98 - .25}) \cdot 3.04 \cdot (\frac{.25}{.98} e^{-.98(4)/11.51} - e^{-.25(4)/11.51}$$

$$+ \frac{.98 - .25}{.98})$$

$D(4^-) = .192 + .127 + 2.1528 + .016 = 2.49$ mg/1

D.O. = 8.17 - 2.49 = 5.68

This D.O. level is somewhat lower than might actually be expected, since the analysis thus far does not include the effects of dispersion on the transient storm sewer load. The deficit response to the storm sewer load alone, at MP 19, without the effect of dispersion is determined as follows:

$W = 89200 \cdot 1.5 + 3800 \cdot 4.57 = 150870$ lbs UOD/day

Substituting this value of W into the third term of Equation 2-17, the deficit at MP 19 due to the SS is determined to be 1.75 mg/1

From the example BOD calculations, previous page, the expected response with the effects of dispersion included was shown to be 70% of the non-dispersive case, or .7(1.75) = 1.22 mg/1. Hence, the difference of 1.75-1.22 = .53 mg/1 is the amount by which the steady state plug flow analysis overestimates the expected oxygen deficit. Using this information, the computed deficit at MP 19 becomes:

$D(4^-) = 2.49 - .53 = 1.96$ mg/1

D.O. = 8.17 - 1.96 = 6.21 mg/1

Similar calculations are made for segments 4, 5 and 6. Note that the SS and CSO loads must be handled individually to account for the effects of dispersion.

distribution of five day BOD within the segment being analyzed. Similar computations follow for the remaining segments, with care being taken to carry the appropriate concentration and flow values for a given probability level through to the end of the study area. The dissolved oxygen response has been computed in a similar manner. Care must be taken to attenuate the effects of the transient storm sewer and combined sewer overflow loads, so as to account for the effects which dispersion has on them. The procedure for doing this is described in Section 5.2.3 and illustrated in the example calculations as well. These computations are illustrated for one river segment in Table 5-8. The computed dissolved oxygen concentrations for the South River study area are presented in Figure 5-10.

5.2.4.5.1 Dissolved Oxygen Concentration - Drought Flow Periods

Drought flow conditions in the South River study area are found to be 50 cfs for the 7 consecutive day-10 year low flow at the upstream end of the study area. During these periods only dry weather discharges from municipal and industrial sources and tributaries contribute oxygen demanding substances to the receiving waters.

The projected water quality response in the South River is indicated in Figure 5-11. Figure 5-11(a) shows the dissolved oxygen concentration relative to the water quality objectives for the study area at the end of a 20 year planning period. A highly probable water quality problem is indicated in the figure. Dissolved oxygen concentrations are expected to approach 0.0 mg/l in the region between Milepoints 20 and 25 and the entire region between Milepoint 17 and 33 is expected to be below the water quality objective level.

Figure 5-11(b) indicates the impacts due to specific discharges. Municipal wastewater treatment plant No. 2 is the largest single contributer to the problem contributing 65% of the total response at the critical location, Milepoint 23. The industrial discharge and tributary inflow have a minor impact on the projected dissolved oxygen conditions.

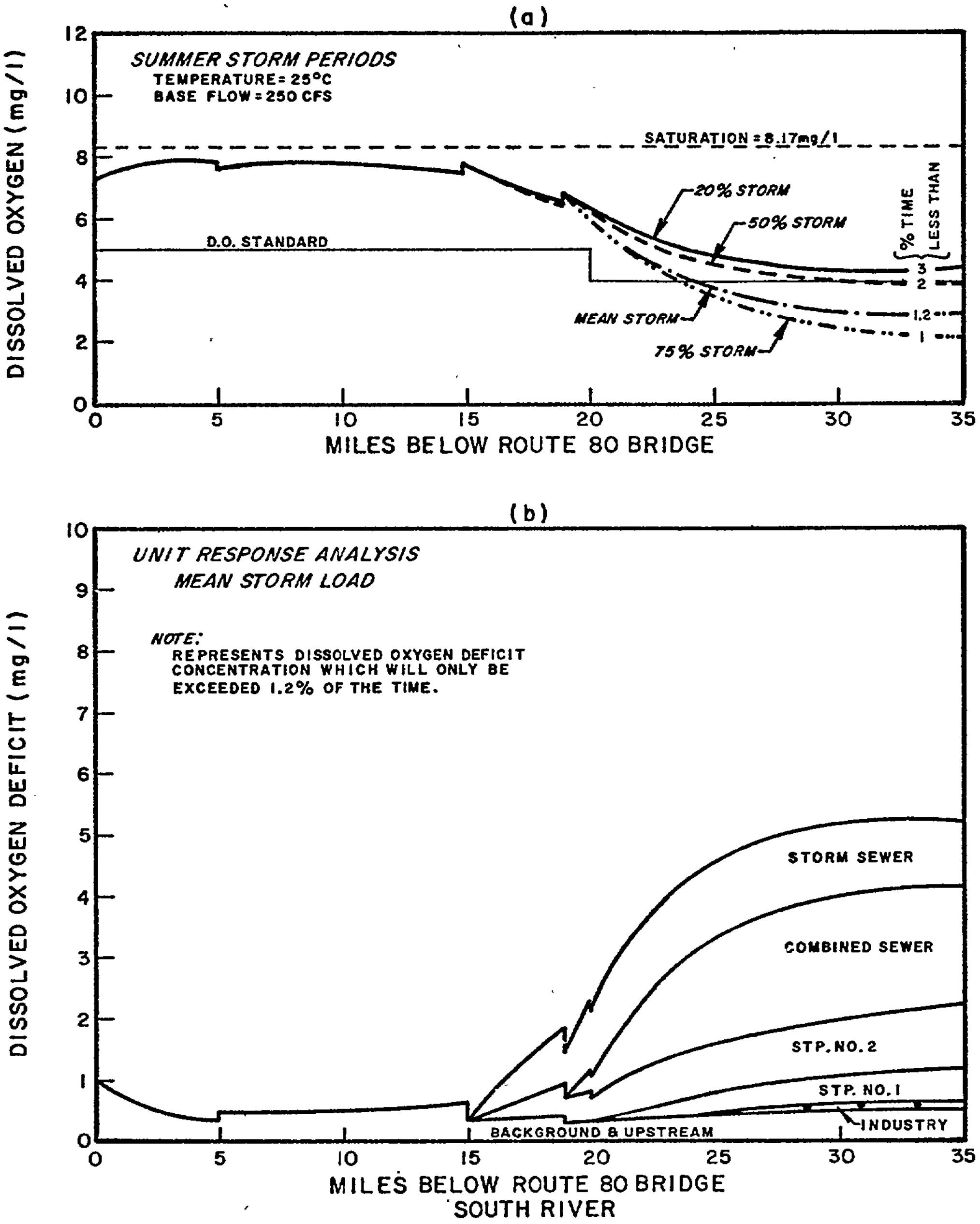


FIGURE 5-10
PROBABILISTIC ANALYSIS OF WET WEATHER DISSOLVED OXYGEN
HYPOTHETICAL SOUTH RIVER

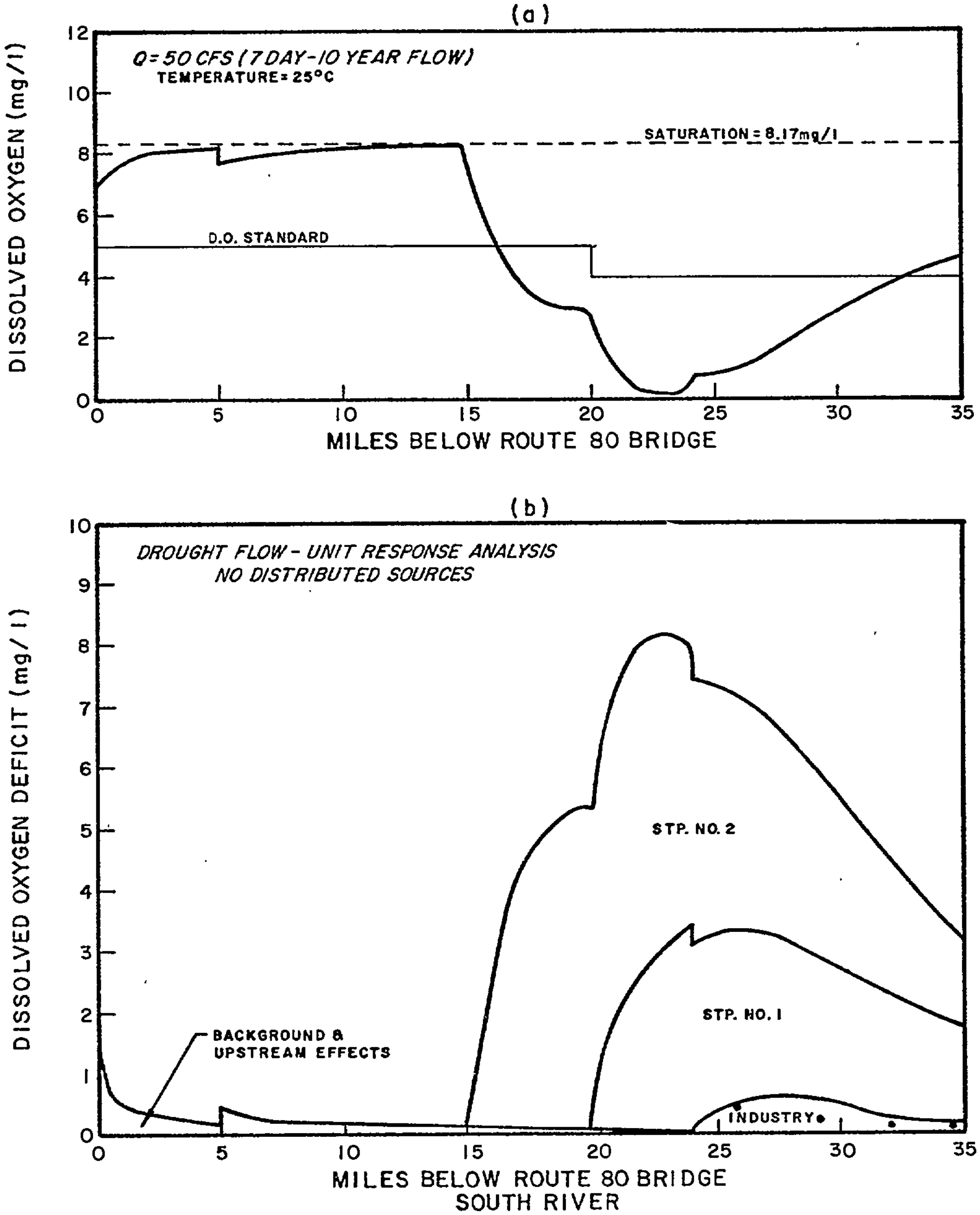


FIGURE 5-11

DRY WEATHER DISSOLVED OXYGEN ANALYSIS
HYPOTHETICAL SOUTH RIVER

5.2.4.5.2 Nutrient Enrichment-Average Summer Conditions

Nutrient enrichment in the South River study area was noted to be a potential water quality problem in Chapter 2. Because nutrient impacts tend to be on a longer time scale, the analysis was made using average summer flows, the long term average summer storm loads (W_o), the point sources, and upstream and background effects. As indicated in Equation 3-21, $W_o = W_R D/\Delta$, and $D/\Delta = 0.0375$ during the summer in the South River area, that is, it rains only 3.75% of the time. The results developed from the equations in Table 2-17 are presented in Figures 5-12(a) for total nitrogen and 5-12(b) for total phosphorus, where the components of the nutrient response due to various loads within the study area are shown. The predicted peak concentrations of 3.95 mg N/1 and 1.40 mg P/1 are considerably higher than those required to indicate a probable problem.

Figures 5-12(a) and 5-12(b) show that point source loads are expected to be the largest single source of nutrients to the system at the end of the 20 year planning period. Therefore, nutrient removal at one or both of these plants appears to be a potential solution to the problem, if further analyses of nutrient-phytoplankton dynamics (using methods described in Appendix A) demonstrate a need for nutrient reductions. Such an analysis would indicate the nutrient which should be removed as well as the nutrient levels required to maintain an objective phytoplankton level.

5.2.4.5.3 Total Suspended Solids-Summer Storm Periods

Expected total suspended solids concentrations in the South River using the solids decay rate of 0.1/day (pg. 5-8) during the 20%, 50%, mean (68%) and 75% summer storms are shown in Figure 5-13(a). These results indicate a probable problem during summer storm periods. The smaller (i.e., 20%) storms have a smaller impact at Milepoint 15, the location of maximum concentration. The river concentration increases further downstream during these storms, however, due to the continual inflow and background load, with the smaller

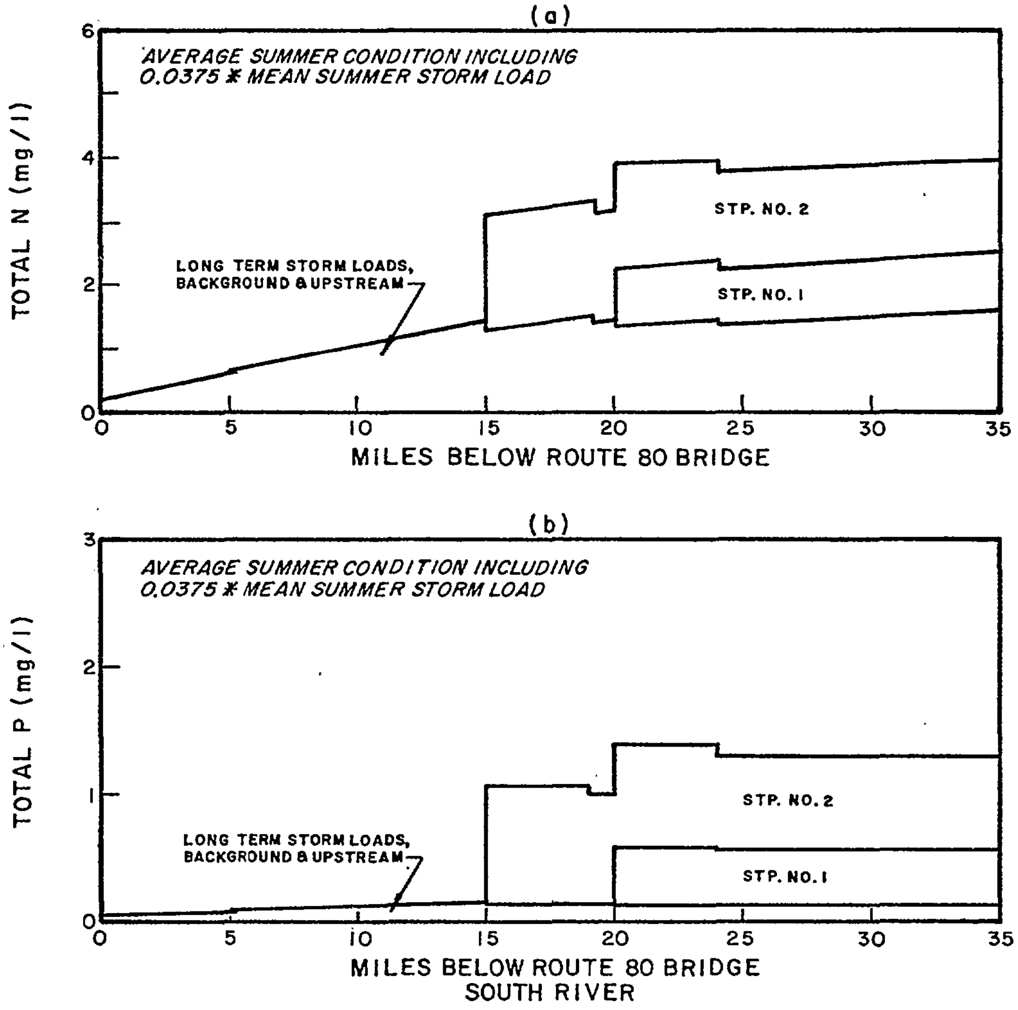


FIGURE 5-12
LONG TERM TOTAL NUTRIENT IMPACTS
HYPOTHETICAL SOUTH RIVER

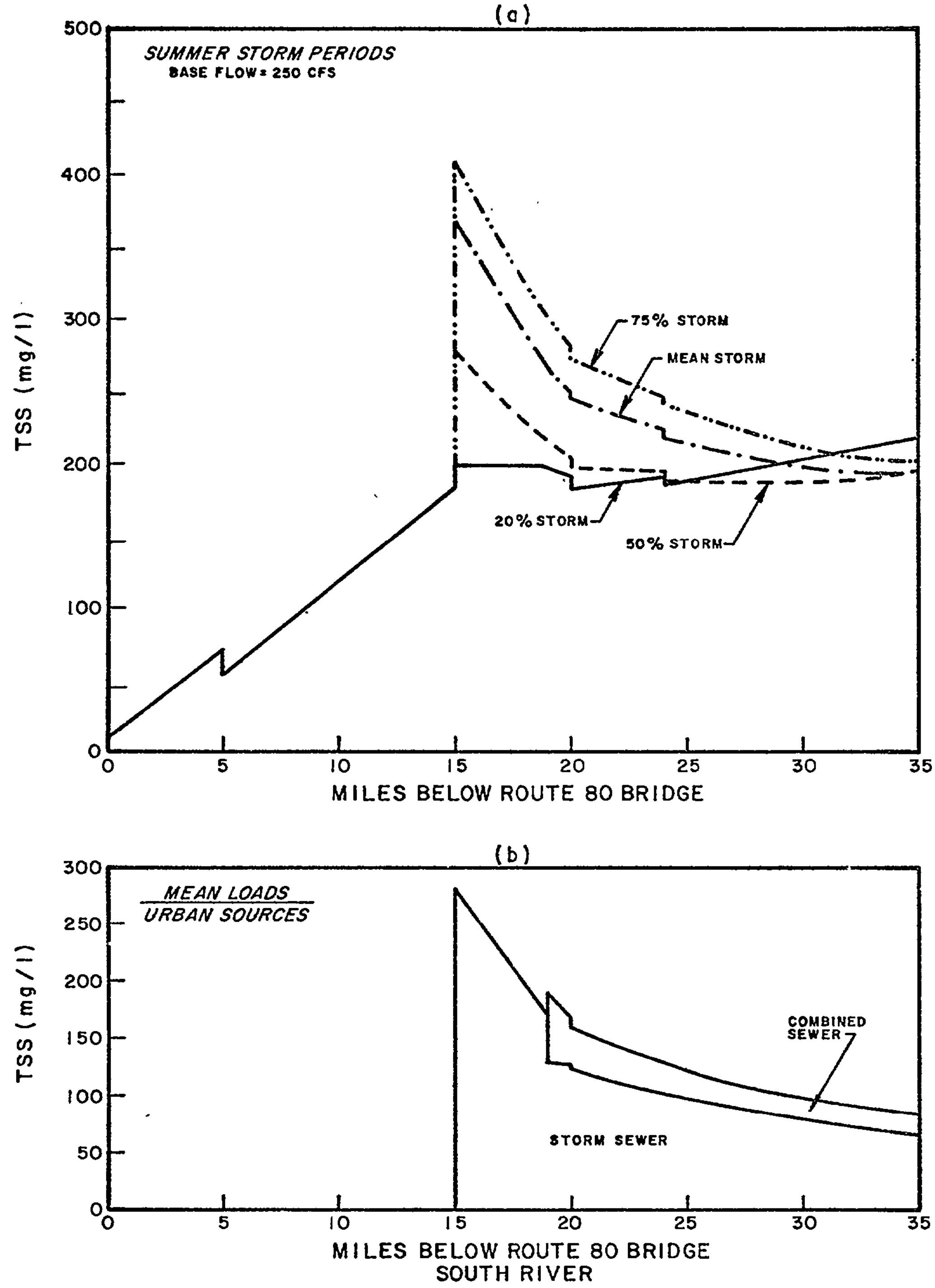


FIGURE 5-13
WET WEATHER TOTAL SUSPENDED SOLIDS IMPACTS
HYPOTHETICAL SOUTH RIVER

diluting flow in the river. It should be noted that the storm impacts are also reduced to compensate for dispersion effects as indicated in Figure 5-4. The component contributions of the urban sources of total suspended solids during the mean storm are shown in Figure 5-13(b). The municipal treatment plant loads have a negligible impact on the total suspended solids, while the problem is in large part due to storm sewer sources. Another major contributer to the problem is upstream and background sources from agricultural lands which contribute 30 percent of the total observed response at Milepoint 15 during the mean storm. The results indicate a possible need to evaluate control of both non-urban and urban non-point sources of total suspended solids to control the problem.

5.2.4.5.4 Total Coliform Concentration-Summer Storm Periods

Figure 5-14 demonstrates for the coliform die-away rate of 1.26 (pg. 5-8) the 20 year projections for total coliform concentrations in the South River during the 20%, 50%, mean (68%) and 75% storms. Very high concentrations are predicted at Milepoint 19, the location of the combined sewer area load, indicating a highly probable problem. The peak concentration during the mean storm is about 1,200,000 MPN/100 ml.

The mean storm unit response for total coliform organisms is shown in Figure 5-15. The combined sewer systems in sewer District No. 1 contribute heavily to the problem followed by the separate sewer areas. Note that the logarithmic scale used in Figures 5-14 and 5-15 make comparisons of different storm responses difficult, with small vertical differences towards the top of the figures representing large differences in total coliform concentrations, while larger vertical differences towards the bottom of the figures represent small actual differences in total coliform concentrations.

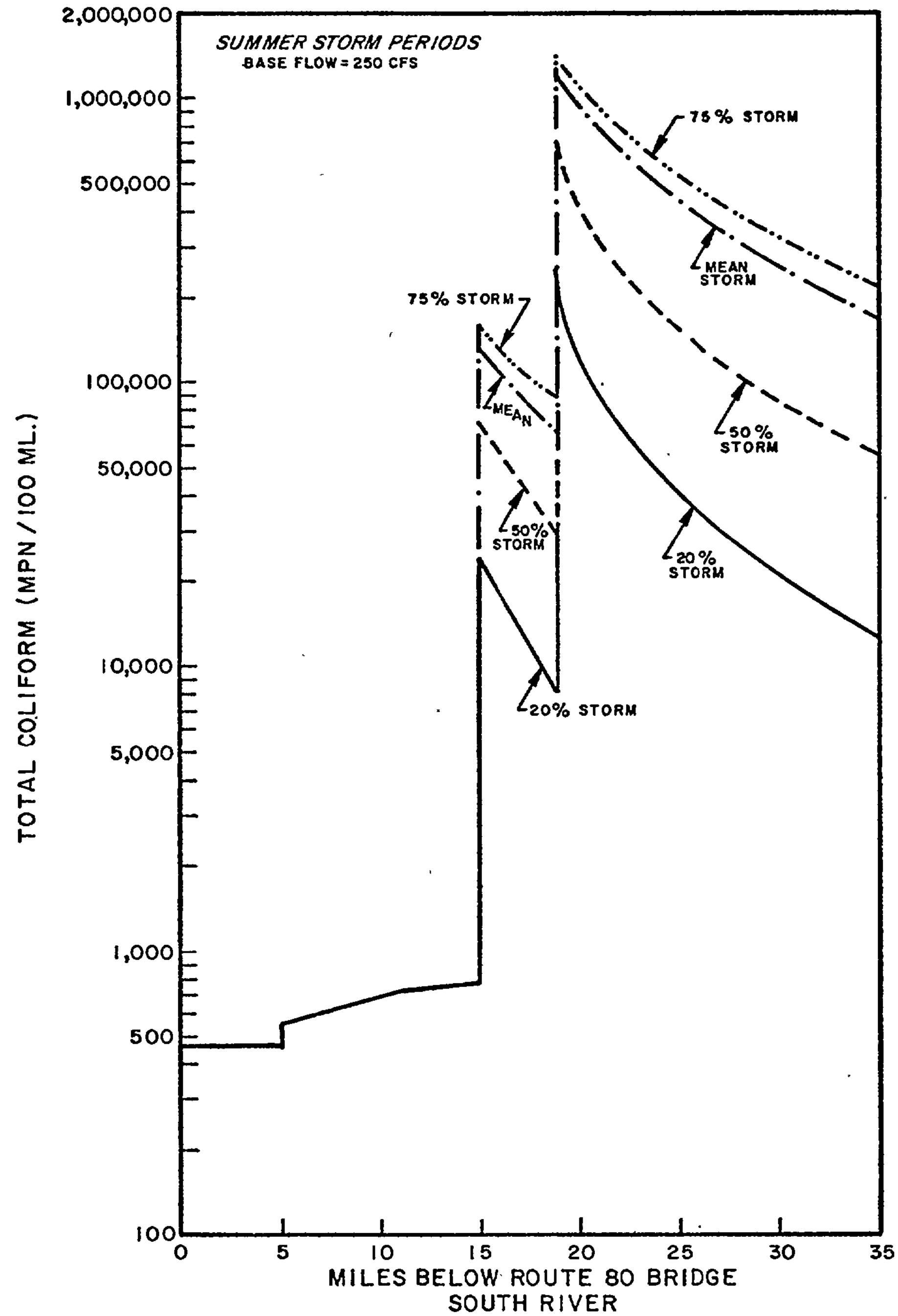


FIGURE 5-14
WET WEATHER TOTAL COLIFORM CONCENTRATIONS
HYPOTHETICAL SOUTH RIVER

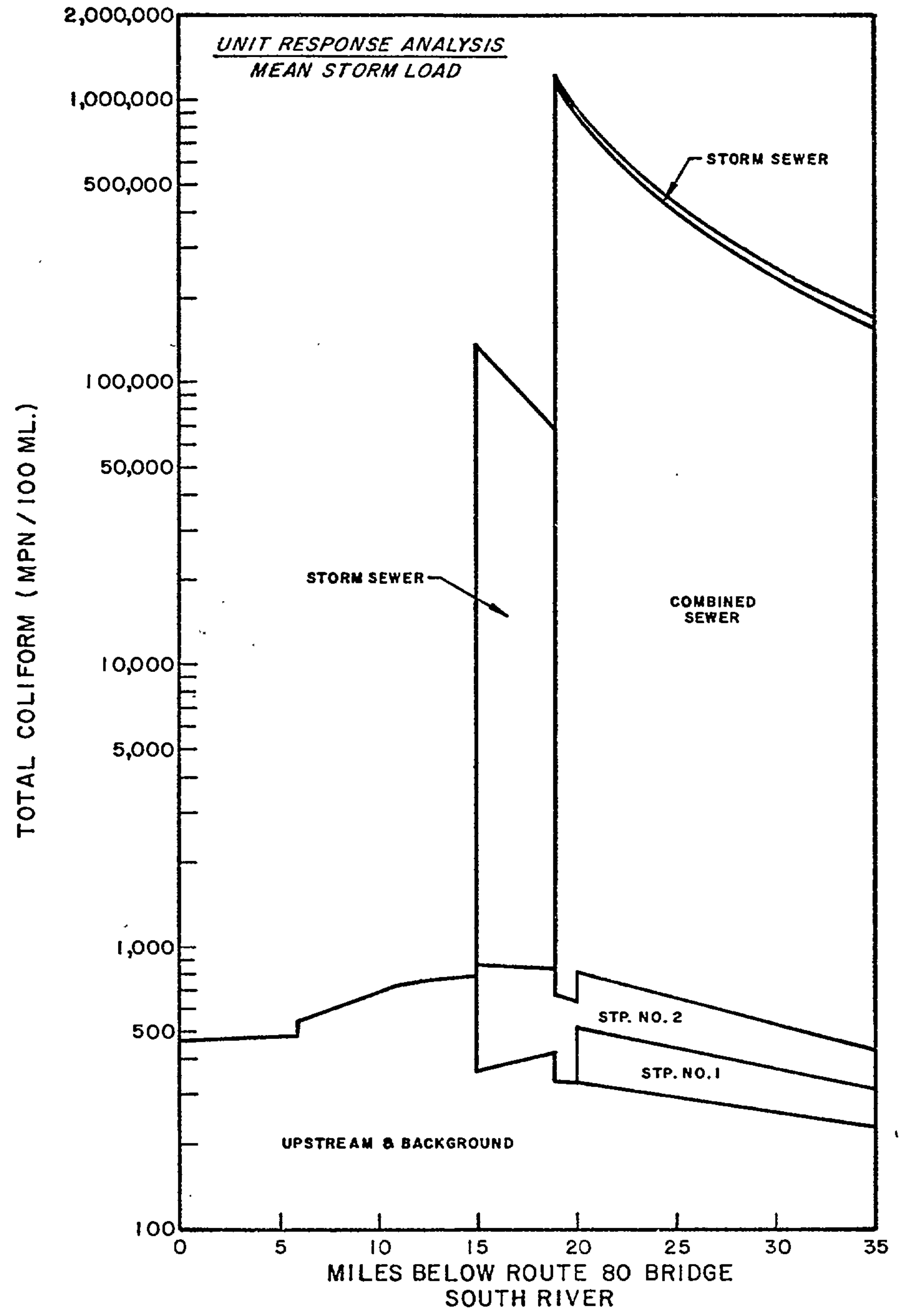


FIGURE 5-15
UNIT RESPONSE TO MEAN TOTAL COLIFORM STORM LOADS
HYPOTHETICAL SOUTH RIVER

5.3 Estuaries

Estuaries are those water bodies in which a significant hydrodynamic transport mechanism is mixing caused by astronomical tides and other similar mixing mechanisms. Estuaries normally consist of two sections which are characterized by the relative magnitude of advective flow to tidal mixing or dispersion. In purely estuarine systems the downstream portion is normally dominated by tidal mixing and freshwater advective flow is less important in transporting physical and chemical constituents. The upper reaches of the estuary are usually influenced by tidal action to a lesser degree because of the damping effect of bottom drag within the estuary. This portion of the estuary is referred to as a tidal river and is characterized by a significant advective transport component.

The analysis of water quality in one dimensional estuaries is somewhat more complicated than in streams principally because of tidal mixing. Materials that are discharged at one point in the system affect water quality in both the upstream and downstream direction because of tidal reversals. The classical method of incorporating this mixing transport in estuaries, and one which finds wide use in engineering practice today, is through the use of dispersion coefficients, normally designated by E. In practice the dispersion coefficient is an estimator of the net rate at which mass is transported from regions of high concentrations to regions of low concentrations. The effect of dispersion is to spread materials discharged to the estuary in the longitudinal direction both upstream and downstream. Due to this phenomenon, a segmented model of an estuary requires simultaneous solution for all segments. The simplified stream analysis, in which upstream concentrations are independent of downstream effects, is not applicable.

Estuaries also differ from streams in the time scale of their response to continuous and intermittent loadings. Generally the response time to loadings is longer in estuaries than in streams. For example, after a continuous source begins discharging into an estuary, concentrations will build up for several days to several weeks before a steady state value is achieved. This is in contrast to advective systems where the concentration buildup at the discharge point is immediate, and equal to the mass balance concentration between the stream and the discharge. Intermittent loads are also acted upon differently in estuaries. Pollutant concentrations due to pulse loads are quickly attenuated because of longitudinal mixing.

This section of the manual considers methods for evaluating water quality responses in estuaries and tidal rivers. The analysis is structured for preliminary assessments similar in design to the preliminary water quality assessment methods for streams presented in Chapter 2. These methods are useful to the 208 planner in the initial development steps of the 208 water quality management plan, and will provide him with a broad overview of water quality problems, the relative magnitude of various waste sources, and the probable impact of the various sources on water quality. From that point the analysis can proceed toward the selection and implementation of appropriate technical procedures to analyse waste sources which are important in the planning area. These technical procedures may be numerical computer models or more refined extensions of the analysis procedures demonstrated in this manual.

In this Chapter, then, all estuarine analyses will be performed on a constant parameter simulator of a real system for which analytic solutions are available. If significant variations in flow, cross-sectional area, dispersion coefficients and kinetic rates, etc. occur within the study area, and if the results of this simplified analysis indicates potential water quality problems, a more sophisticated analysis framework - outstide the scope of this manual - is required (Appendix A).

The basic equations for calculating waste concentrations in estuaries and tidal rivers are presented along with guidance for the determination of model coefficients. Example problems are presented to demonstrate the analysis methods for point and non-point sources. Finally Section 5.3.4.1 presents empirical methods for assessing water quality variability. The examples presented in Section 5.3.4.1 are applicable to many local problem cases and should be taken as instructive tools illustrative of techniques which the 208 planner may follow.

5.3.1 Description of Model

For purposes of this chapter, a tidal river is defined as that portion of a water body that is subject to reversals of current direction but does not include estuaries where the effects of freshwater runoff may be small. Thus, tidal rivers that may oscillate in velocity and direction due to causes other than astronomical tides are included in the analysis. An example of the latter case is the flow oscillations in the tributaries of the Great Lakes caused by wind produced seiches. Estuaries are those water bodies that are dominated by tidal dispersion.

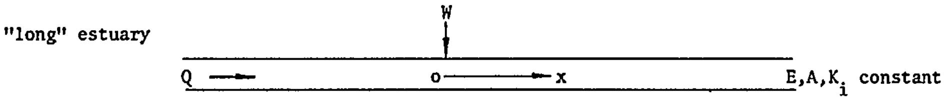
The appropriate general steady state solutions for the tidal rivers and estuaries are presented in Table 5-9 through 5-11 for conservative, reactive and sequentially reactive system variables (7). The reaction coefficient, K, is descriptive of the particular substance under consideration. The velocity, U, is that due to the freshwater discharge. The tidal velocity is not included in the equations, implying that water quality conditions are at mean tide.

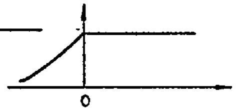
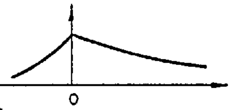
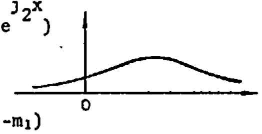
The coefficient, E, is the longitudinal dispersion coefficient. It is most significant in the saline portion of the estuary where a number of factors contribute to the intrusion of the salt into the estuary. The concentration of other substances, which are of concern in water quality analyses of estuaries, is affected in a manner similar to that of the salt. In the tidal, but non-saline sections of the river, the dispersion -- although not as pronounced as in the saline section -- is still a significant factor in the analysis of water quality.

As pointed out previously, the equations for calculating receiving water concentrations (Table 5-9) are applicable to constant parameter estuaries. For gradually varying areas, flows, etc, averaged values are to be input to the equations for preliminary problem assessments.

TABLE 5-9

STEADY STATE EQUATIONS FOR WASTE CONCENTRATIONS IN TIDAL RIVERS AND ESTURIES DUE TO POINT SOURCE



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)	$D = \frac{K_{12}}{K_2-K_1} L_o (e^{g_1 x} - \frac{m_1}{m_2} e^{g_2 x})$	$D_o = \frac{K_{12}}{K_2-K_1} L_o (1 - \frac{m_1}{m_2})$	$D = \frac{K_{12}}{K_2-K_1} L_o (e^{j_1 x} - \frac{m_1}{m_2} e^{j_2 x})$ $x_c = n(\frac{m_1}{m_2} \cdot \frac{1-m_2}{1-m_1} \div \frac{U}{2E}$ (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)

$m_i = \sqrt{1 + 4K_i E/U^2}$

$j_i = (U/2E)\ (1-m_i)$

$g_i = (U/2E)(1 + m_i)$

TABLE 5-10

STEADY STATE EQUATIONS FOR WASTE CONCENTRATIONS IN TIDAL RIVERS AND ESTUARIES DUE TO DISTRIBUTED SOURCE

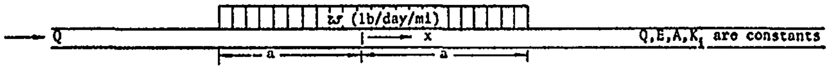
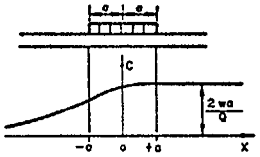
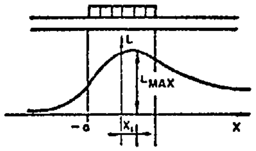
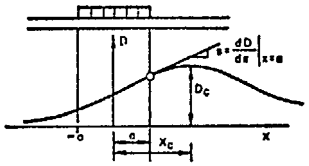
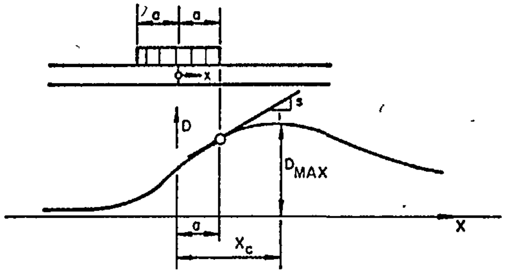
Type of waste				Distribution of Estuarine Waste Concentrations
Conservative	$C = \frac{w}{Q}\;\frac{E}{U}\;\; e^{u/E(x+a)} - e^{u/E(x-a)}$	$C = \frac{wa}{Q}\;\; 1 + \frac{x}{a} + \frac{E}{aU}\,(1 - e^{u/E(x-a)})$	$c = \frac{2wa}{Q}$	
Reactive (System 1)	$L = \frac{w}{Qm_1g_1}\;\; e^{g_1(x+a)} - e^{g_1(x-a)}$	$L = \frac{w}{Qm_1}\;\; \frac{e^{j_1(x+a)} - 1}{j_1} - \frac{e^{g_1(x-a)} - 1}{g_1}$	$L = \frac{w}{Qm_1j_1}\;\; e^{j_1(x+a)} - e^{j_1(x-a)}$	 $X_1 = a/m_1,\; 0 \leq X_1 \leq a$
Sequentially Reactive (System 2)	$D = \frac{w}{Q}\;\frac{K_{12}}{K_2-K_1}\;\; \frac{e^{g_1(x+a)} - e^{g_1(x-a)}}{m_1g_1} - \frac{e^{g_2(x+a)} - e^{g_1(x-a)}}{m_2g_2}$	$D = \frac{w}{Q}\;\frac{K_{12}}{K_2-K_1}\;\; \frac{e^{j_1(x+a)} - 1}{m_1j_1} - \frac{e^{g_1(x-a)} - 1}{m_1g_1} - \frac{e^{j_2(x+a)} - 1}{m_2j_2} - \frac{e^{g_2(x-a)} - 1}{m_2g_2}$	$D = \frac{w}{Q}\;\frac{K_{12}}{K_2-K_1}\;\; \frac{e^{j_1(x+a)} - e^{j_1(x-a)}}{m_1j_1} - \frac{e^{j_2(x+a)} - e^{j_2(x-a)}}{m_2j_2}$	 FOR X, SEE TABLE 5-9.

TABLE 5-11

LOCATION OF MAXIMUM CONCENTRATION FOR DISTRIBUTED SEQUENTIALLY REACTING WASTE

NOTE:
FIGURE SHOWN FOR CASE I.



$$s = \left[d\ D/dx \right] \quad x = a$$

$$s = \frac{e^{2aj_1} - 1}{m_1} - \frac{e^{2aj_2} - 1}{m_2}$$

Value of s	Location of D_{Max}	Implicit Equation For x_c and Equation For D_{MAX} when s = 0
I Positive	$x_c > a$	$\frac{e^{j_1(x_c+a)} - e^{j_1(x_c-a)}}{m_1} = \frac{e^{j_2(x_c+a)} - e^{j_2(x_c-a)}}{m_2}$
II Negative	$x_c < a$	$\frac{e^{j_1(x_c+a)} - e^{g_1(x_c-a)}}{m_1} = \frac{e^{j_2(x_c+a)} - e^{g_2(x_c-a)}}{m_2}$
III Zero	$x_c = a$	$D_{MAX} = \frac{W}{Q} \cdot \frac{K_{12}}{K_2 - K_1} \cdot \left[\frac{e^{2aj_1} - 1}{m_1 j_1} - \frac{e^{2aj_2} - 1}{m_2 j_2} \right]$

5.3.2 Methods of Analysis

5.3.2.1 Conservative Substances

The analysis for conservative substances is identical to that for streams. The maximum concentration at the point source discharge location is simply the mass rate of waste discharge divided by the freshwater flow (Table 5-11):

$$c_o = \frac{W}{Q} \tag{5-5}$$

Note that upstream migration of the waste occurs.

Equation (5-5) may be applied to substances such as total dissolved solids, and other materials which decay at such slow rates that they may be regarded as conservative.

5.3.2.2 Non-conservative Substances

Many substances decay in accordance with a single reaction or at least for practical engineering purposes may be assumed to decay in this fashion. As discussed previously, the reaction is assumed to be first order with a decay coefficient, K.

It should be noted that the following assumptions have been made:

a) steady state

b) constant coefficients exist, i.e., flow, cross-sectional area, reaction kinetics and dispersion characteristics are all constant along the length of the estuary under study.

Since most estuaries vary in cross section along the axis of flow this area must be estimated as the average over which the profile extends at mean tide. For highly reactive substances ($K \geq 2$/day) this distance may be in the order of 10 or 20 miles, while for moderately reacting material ($K \leq 0.5$/day) it may be as much as 50 miles. The difficulty in assigning a realistic average area over such distances is evident from a casual inspection of a geographic map of the coastal areas of the United States. A common physical feature of the topography not taken into account by the above model is the number of tributaries which feed many estuaries and the delta network which characterizes many estuarine mouths. Obviously, a more complete mathematical description of the estuarine structure is required for such situations. In spite of these difficulties, at least, some engineering approximation may be made and the error introduced is invariably on the conservative side.

Table 5-12 presents ranges of values for reaction coefficient in tidal rivers and estuaries for the pertinent substances.

TABLE 5-12

FIRST ORDER RANGE OF VALUES FOR REACTION COEFFICIENTS TIDAL RIVERS AND ESTUARIES (8)

Substance	K-per day	K-per day [1]
Coliform	2 - 4	1 - 3
BOD_5	0.2 - 0.5	0.2 - 2.0
Nutrients	0.1 - 0.25 or conservative (K = 0)	0.1 - 1.0

[1] From Table 2-19, for rivers.

5.3.2.3 Dissolved Oxygen Analysis

The analysis for dissolved oxygen is conducted in a similar manner. The waste discharge causes a drop in the dissolved oxygen concentration with a subsequent rise further downstream. The tidal river profile is therefore similar to that of the stream. Due to the tidal action, however, the deficit in dissolved oxygen is translated upstream and the associated dispersion flattens the profile. The tidal river profile is therefore projected further upstream and downstream in contrast to the stream profile. The equation of the dissolved oxygen deficit evaluation in estuaries is shown in Table 5-11. As may be seen from these equations the dissolved oxygen deficit profile is determined by the ratio, ϕ, and also the parameter n. The following sections relate to a discussion of these factors.

5.3.2.4 Reaction Coefficients

As in the case of the freshwater stream, the surface transfer coefficient, K_L, is a fundamental expression of reaeration phenomenon particularly appropriate to estuary analysis. It is related to the volumetric reaeration coefficient by the depth.

$$K_a = \frac{K_L}{H} \tag{5-6}$$

where: K_L is the surface transfer coefficient (ft/day)(Figure 2-20)
H is the average depth at mean tide (ft.)
K_a is the reaeration coefficient (1/day).

The reaeration coefficient is a function of the velocity and depth of flow. In the tidal river and estuarine case, the pertinent velocity is the average tidal current. The ranges of transfer and reaeration coefficients which may be encountered in estuaries are presented in Table 5-13.

TABLE 5-13

RANGE OF TRANSFER AND REAERATION COEFFICIENTS ESTIMATED FOR TIDAL RIVERS AND ESTUARIES (8)
(K_L in ft/day, K_a in 1/day)

	Average Tidal Velocity (fps)					
	1		1-2		2	
Mean Tidal Depth (ft)	K_L	K_a	K_L	K_a	K_L	K_a
10	4	0.5	5.5	0.6	7	0.8
10 - 20	3	0.2	4.5	0.3	6	0.4
20 - 30	2.5	0.1	3.5	0.14	5	0.2
30	2	0.06	2.5	0.08	4	0.12

The probable range of K_L is between 3 - 6 feet/day with limits from 2 to a possible 10 feet per day for a shallow estuary with high tidal velocity.

Anticipating the effect of treatment on the oxidation in the natural estuarine environment, the range of the deoxygenation or deaeration coefficient, K_d, may be from 0.2 - 0.5 with a probable average in the order of 0.3, (See Table 5-12). This range assumes that the estuary is no shallower than about 5 feet.

The assimilation ratio, ϕ, may readily be tabulated from the above data and is summarized in Table 5-14 for different conditions.

TABLE 5-14

TABULATION OF ASSIMILATION RATIO - ϕ
TIDAL RIVERS AND ESTUARIES (8)

Reaeration Coefficient K_a (1/day)	ϕ			
K_d=	0.2	0.3	0.4	0.5
0.08	0.4	0.27	0.20	0.16
0.15	0.75	0.50	0.38	0.30
0.30	1.5	1.0	0.75	0.60
0.60	3.0	2.0	1.5	1.2

Tables 5-13 and 5-14 indicate that the deeper main channel estuaries have ϕ values from 0.2 to 0.8, while the shallower tidal tributaries are in the range 0.8 to 3.0. The lower limit of each of these ranges indicates the more restricted tidal bodies of lower velocity, higher temperatures, and effluents from less advanced degrees of treatment, while the upper limit describes the free flowing, higher velocity estuary, and more advanced treatment in more moderate temperature regions of the country.

5.3.2.5 Estuarine Number

In addition to the assimilation ratio, ϕ, the estuarine number, $n = KE/U^2$, is the additional specification which characterizes water quality in tidal rivers and estuaries. The practical range of the dispersion coefficient, E, is from 1 to 20 (mi^2/day). The upper limit describes the highly-saline, high-tidal-velocity stretches in the vicinity of the estuarine mouth, while the lower limit applies to the upstream, non-saline, low tidal sections of the estuary. If slack water longitudinal profiles of salinity or chlorides are available, an estimate of the dispersion coefficient may be obtained. This is accomplished by plotting the salinity vs. distance on semi-log paper, fitting a straight line to the data and obtaining E, as described in Reference (7). The dispersion coefficient, E, with the advective velocity, U, provides sufficient hydrodynamic definition for each estuary. The advective velocity associated with the freshwater flow is determined by dividing the freshwater flow, Q, by the average cross-sectional area, A. The dispersion coefficient may therefore vary over a wide range due to the number of geophysical and hydrological factors which affect it, not only within the estuary itself, but also by the characteristics of the drainage basin. The advective velocity U, may range from 0.1 - 10 miles per day. The estuarine number, n, developed from this range of advective velocities and a practical range of dispersion coefficients is tabulated in Table 5-15 for a reaction rate of 0.3/day. For wastes with higher reaction rates, the estuary number increases accordingly. Thus, for coliform bacteria with a decay rate of 3/day, the estuary numbers, n, would be ten times the values tabulated below.

TABLE 5-15

RANGE OF ESTUARINE NUMBER, n, FOR TIDAL RIVERS (8)

$K_1 = 0.3/\text{day}$

Tidal Dispersion (sq. mi/day)	Advective velocity - mi/day				
	0.5	1.0	2.0	4.0	
2	2.4	0.6	0.15	0.04	= n
5	6.0	1.5	0.38	0.10	
10	12.0	3.0	0.75	0.19	
20	24.0	6.0	1.5	0.75	

A summary of the above tabulations with approximate physical descriptions of the types of tidal rivers and estuaries is presented in Table 5-16.

TABLE 5-16

CLASSIFICATION OF TIDAL RIVERS AND ESTUARIES (8)
(K=0.3/DAY)

Description	Assimilation Ratio (ϕ) Average Value	Range	KE (mi^2/day^2) Aver. Value	Range	Estuary Number "n" Aver. Value	Range
Large, deep, main channel in vicinity of mouth	0.3	0.1-0.5	10	5 -20	15	5 - 30
Moderate navigation channel, upstream from mouth saline, large tidal tributaries	0.5	0.2-1.0	3	2 - 5	5	2 - 10
Minimum navigation upstream, smaller saline or nonsaline tidal tributaries	1.0	0.5-2.0	1.5	1 - 2	2	0.5-5
Tidal tributaries, shallow and nonsaline	2.0	1.0-3.0	.5	.21	1	0.2-2

5.3.3 Examples of Estuarine Analyses

Examples of impact evaluations of single point source and multiple waste sources on estuaries are presented below. All analyses are carried out using the steady state equations of Table 5-10 for calculating waste concentrations in estuaries with constant geometry, hydrology and kinetics and having continuous pollutant inputs. For study areas having significantly varying geometry, flows or kinetic rates (e.g., reaeration coefficients), appropriate analyses may be conducted to determine the sensitivity of the result to the varying parameters. Thus, if the cross-sectional area varies widely, analyses can be performed for low, medium and high estimates of the constant area for the entire length of the estuary. If sufficiently diverse results occur under the three assumptions, and if the interpretation of the differences in water quality response show the existence of a water quality problem in one case and the lack of a potential problem in the estuary in another case, a more detailed analysis using computerized models is required. On the other hand, if the impacts vary little and the interpretation of the water quality results is consistent, relative confidence in the analysis results and management decisions may be made based on this simplified approach.

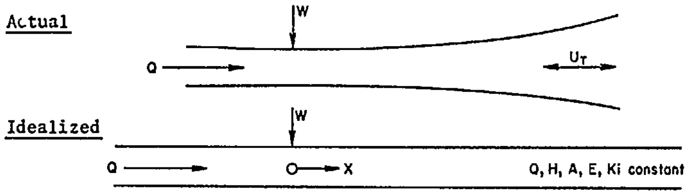
Use of the steady state framework will yield accurate results when reasonably continuous waste sources exist. For intermittent inputs (e.g. storm overflows), the calculated concentrations will generally be higher than those that would result from a more rigorous time variable analysis. For short-term inputs (several hours), the steady state analysis may significantly overestimate the estuarine impact, whereas for longer areawide storms of several days duration reasonably accurate impacts are usually predicted. Since the steady state analysis generates conservative results, its use as a preliminary screening device for relative impacts of waste sources is useful. If continuous sources are cited as the dominant cause of deteriorated water quality, the analysis may be considered as a valid input to management decisions. In those cases where problems are attributable to intermittent sources, care must be exercised in using the results. Where predicted estuarine concentrations are several orders of magnitude above desired levels (e.g. coliform bacteria), the implication of the intermittent source as a problem may be made with confidence. In marginal cases, time variable analysis is required - a topic beyond the scope of this manual.

5.3.3.1 Example of an Estuary Analysis with a Single Point Source

Table 5-17 contains low flow and summer average flow analyses for a continuous point source discharge. A secondary municipal STP is assumed with an effluent flow of 10 cfs and effluent concentrations of total nitrogen (assumed conservative), total coliform and ultimate oxygen demand consistent with data in Table 2-10 (pg. 2-53).

TABLE 5-17

EXAMPLE OF ESTUARY WITH SINGLE POINT SOURCE



Freshwater Flow (Q)

Low Flow = 50 cfs

Summer Av. = 300 cfs

Mean Water Depth (H) = 10 ft.

Av. Cross Sect. Area (A) = 2000 sq. ft.

Av. Tidal Velocity (U_T) = 0.6 knots

Dispersion Coeff. (E) = 2 mi^2/day

BOD-DO: $K_1 = K_r = 0.25$/day

: $K_{12} = K_d = K_r$

: $K_2 = K_a$

Coli: $K_1 = K_c = 2$/day

Point Source Flow: 10 cfs

PS Effl. Conc: 20 mg-TN/1,
120 mg-UOD/1,
1000 MPN/100 ml Tot. Coli.

LOW FLOW ANALYSIS

Av. Freshwater Velocity = Q/A = 50 cfs ÷ 2000 ft^2 = 0.025 fps

$U = 0.025 \text{ fps} \times 16.4 \times 16.4 \dfrac{\text{mi/day}}{\text{fps}} = 0.41 \text{ mi/day}$

a) Total Nitrogen Concentrations (Conservative)

$W_{TN} = 10 \text{ cfs} \times 20 \text{ mg/1} \times 5.4 \dfrac{\text{lb/day}}{\text{cfs-mg/1}} = 1080 \text{ lb/day}$

$C_{Max} = C$ @ $x = 0$, $C_o = W/Q$ (Table 5-9)

C_o = 1080 lb/day ÷ (50 cfs x 5.4) ≐ 4.0 mg-TN/1

Upstream of Point Source

$C = C_o e^{ux/E} = 4.0\, e^{(0.41 \text{ mi/day} \div 2 \text{ mi}^2\text{/day}) \times (\text{mi})}$ (Table 5-9)

$C = 4.0\, e^{0.205x}$, $x \leq 0$

b) Total Coliform Concentrations (Reactive)

Estuary Number $n = K_cE/U^2 = 2/\text{day} \times 2 \text{ mi}^2/\text{day} \div (0.41 \text{ mi/day})^2$

$n = 23.8$

$m = \sqrt{1 + 4m} = \sqrt{1 + 4 \times 23.8} = 9.81$

$j = (U/2E)(1-m) = (0.41/(2x2))(1-9.81) = -0.903$/mile

$g = (U/2E)(1+m) = (0.41/(2x2))(1+9.81) = + 1.108$/mile

Total Coli. Discharge = W_{coli} = 1 cfs x 1,000 MPN/100 ml

Max. T. Coli Conc. is @ x = 0: (Table 5-9)

$L_{MAX} = L_o = W/Qm_1 = \dfrac{10 \text{ cfs} \times 1{,}000 \text{ MPN/100 ml}}{50 \text{ cfs} \times 9.81} = 20 \text{ MPN/100 ml}$

Upstream of PS, $L = L_o e^{gx} = 20\, e^{1.108x}$, $x \leq 0$ (Table 5-9)

Downstream of PS, $L = L_o e^{jx} = 20\, e^{-0.903x}$, $x \geq 0$

c) Dissolved Oxygen Deficit Concentrations

U_T = Av. Tidal Vel = 0.6 (knots = $\dfrac{\text{naut. mi}}{\text{hr}}$) x 1.15 $\dfrac{\text{stat.mi.}}{\text{naut.mi.}} \times \dfrac{88 \text{ fps}}{60 \text{ mi/hr}}$ = 1.0 fps

MW depth = 10 ft, K_L = 4 ft/day;

$K_a = K_L/H$ = 4 ft/day ÷ 10 ft = 0.4/day

$n_1 = K_rE/U^2 = 0.25 \times 2/(0.41)^2 = 2.974$; $m_1 = \sqrt{1 + 4\, n_1} = 3.591$

$n_2 = K_aE/U^2 = 0.4 \times 2/(0.41)^2 = 4.759$; $m_2 = \sqrt{1 + 4n_2} = 4.476$

$j_1 = (U/2E)(1-m_1)=(0.41/(2x2))(1-3.591) = -0.266$/mile

$j_2 = (U/2E)(1-m_2)=(0.41/(2x2))(1-4.476) = -0.356$/mile

$g_1 = (U/2E)(1+m_1) = (0.41/(2x2))(1+3.591) = +0.471$/mile

$g_2 = (U/2E)(1+m_2) = (0.41/(2x2))(1+4.476) = +0.561$/mile

TABLE 5-17
(Continued)

EXAMPLE OF ESTUARY WITH SINGLE POINT SOURCE

Ultim. Oxygen Demand Discharged = 10 x 120 x 5.4 = 6480 lb/day

Max. DO Deficit Occurs @ x_c (Table 5-9)

$$x_c = \ln\left(\frac{m_1}{m_2} \cdot \frac{1-m_2}{1-m_1}\right) \div \frac{U}{2E}\,(m_2 - m_1)$$

$$= \ln\left(\frac{3.591}{4.476}\ \frac{1-4.476}{1-3.591}\right) \div \left[\frac{0.41}{2x2}\,(4.476 - 3.591)\right]$$

x_c = 0.07354/0.09071 = 0.811 miles

$$D_c = \frac{K_{12}}{K_2-K_1} \cdot L_o \cdot \left(e^{j_1x_c} - \frac{mi}{m2}\ e^{j_2x_c}\right),\ L_o = \frac{W_{UOD}}{Qm_1}\ \text{(Table 5-9)}$$

$$= \frac{0.25}{0.4-0.25} \cdot \frac{6480}{50x5.4x3.591} \cdot \left(e^{-0.266x0.811} - \frac{3.591}{4.476}\,e^{-0.356x0.811}\right)$$

D_c = 11.14 (0.2049) = 2.28 mg-DEF/1

$D = 11.14\ (e^{0.471}\ -0.802\ e^{0.561x}),\ x \leq 0$ (Table 5-9)

$D = 11.14\ (e^{-0.266x}\ -0.802\ e^{-0.356x}),\ X \geq 0$

SUMMER AVERAGE FLOW ANALYSIS

Assume no signif. geometric changes, and same dispersion coeff., kinetic coeff, loads.

U = 300 cfs/2000 ft^2 = 0.15 fps = 2.46 mi/day

Using U = 2.46 mi/day, calculations similar to low flow analysis are performed.

a) Total Nitrogen

C_o = 1080/(300x5.4) = 0.67 mg/l
$C = 0.67\ e^{2.46\ x/2} = 0.67\ e^{1.33x},\ x \leq 0$

b) Total Coliform

m = 0.661, m = 1.909, j = -0.559/mi, g = 1.789/mi

L_o = (10 x 1000)/(300 x 1.909) = 17.5 MPN/100 ml

$L = 17.5\ e^{1.789x},\ x \leq 0$
$L = 17.5\ e^{-0.559x},\ x \geq 0$

c) Dissolved Oxygen Deficit

$n_1 = 0.0826,\ m_1 = 1.153,\ j_1 = 0.0941/mi,\ g_1 = 1.324/mi$
$n_2 = 0.1322,\ m_2 = 1.236,\ j_2 = -0.1451/mi,\ g_2 = 1.375/mi$

$$x_c = \left[\ln\left(\frac{1.153}{1.236} \cdot \frac{1-1.236}{1-1.153}\right)\right] \div \left[\frac{2.46}{2x2}\,(1.236 - 1.153)\right] = 7.13\ \text{miles}$$

$$D_c = \frac{0.25}{0.4-0.25}\ \ \frac{6480}{300x5.4x1.153}\ \ \left(e^{-0.0941x7.13} - \frac{1.153}{1.236}\,e^{-0.1451x7.13}\right)$$

D_c = 5.783 (0.1797) = 1.04 ng-DEF/1

$D = 5.783\ (e^{1.324x} - 0.933\ e^{1.375x}),\ x \leq 0$

$D = 5.783\ (e^{-0.0941x} - 0.933\ e^{-0.1451x}),\ x \geq 0$

Comparative plots of resulting concentrations for both flow conditions appear on Figure 5-16.

Freshwater flows of 50 cfs and 300 cfs are used for the low flow and summer average condition and these are assumed constant throughout the estuary. The STP effluent flow of 10 cfs is not included in these values and appropriate sensitivity runs could be made to assess its effect. An average tidal velocity of 0.6 knots is used, a value generally obtainable from annually published NOAA Tidal Current Tables(9). The dispersion coefficient (E) is assumed to be 2 mi^2/day, a value generally representative of the more upstream portion of an estuary. Kinetic rates for the wastes are extracted from Table 5-12.

In the low flow analysis, the maximum total nitrogen concentration is calculated as 4.0 mg/l at the point of discharge. With a freshwater velocity of 0.41 mi/day and an estuary number of 23.8, a maximum total coliform concentration of 20 MPN/100 ml results at the discharge location, indicating minimal impact from this source. Using the average tidal velocity of 0.6 knots (= 1.0 fps) and the average mean water depth of 10 feet, a reaeration coefficient of 0.4/day is generated. The maximum dissolved oxygen deficit of 2.28 mg/l occurs approximately 0.8 miles downstream of the point of discharge. Similar computations are performed for the summer average flow condition. The comparative plots between the two flow conditions, displayed in Figure 5-16, give insight into the behavior of estuaries. The maximum total nitrogen concentrations of 4.0 and 0.67 mg/l are in inverse proportion to the flow, as in stream analyses. On the other hand, the peak total coliform concentrations of 20 and 17.5 MPN/100 ml indicates that the freshwater flow has little effect. The maximum dissolved oxygen deficit of 2.28 mg/l for the 50 cfs flow is reduced to 1.04 mg/l for the flow of 300 cfs showing some reduction to increased flow but not in proportion to the flows. In general, for the more reactive substances, the estuary numbers will be higher (n = 23.8, coliform; n = 2.974, UOD; n = 0, total nitrogen) and the impact of flow will be less.

5.3.3.2 Example of Estuary with Multiple Waste Source

Table 5-18 contains analyses for an estuary with a point source representative of a secondary municipal STP (Table 2-10 pg. 2-53) and a

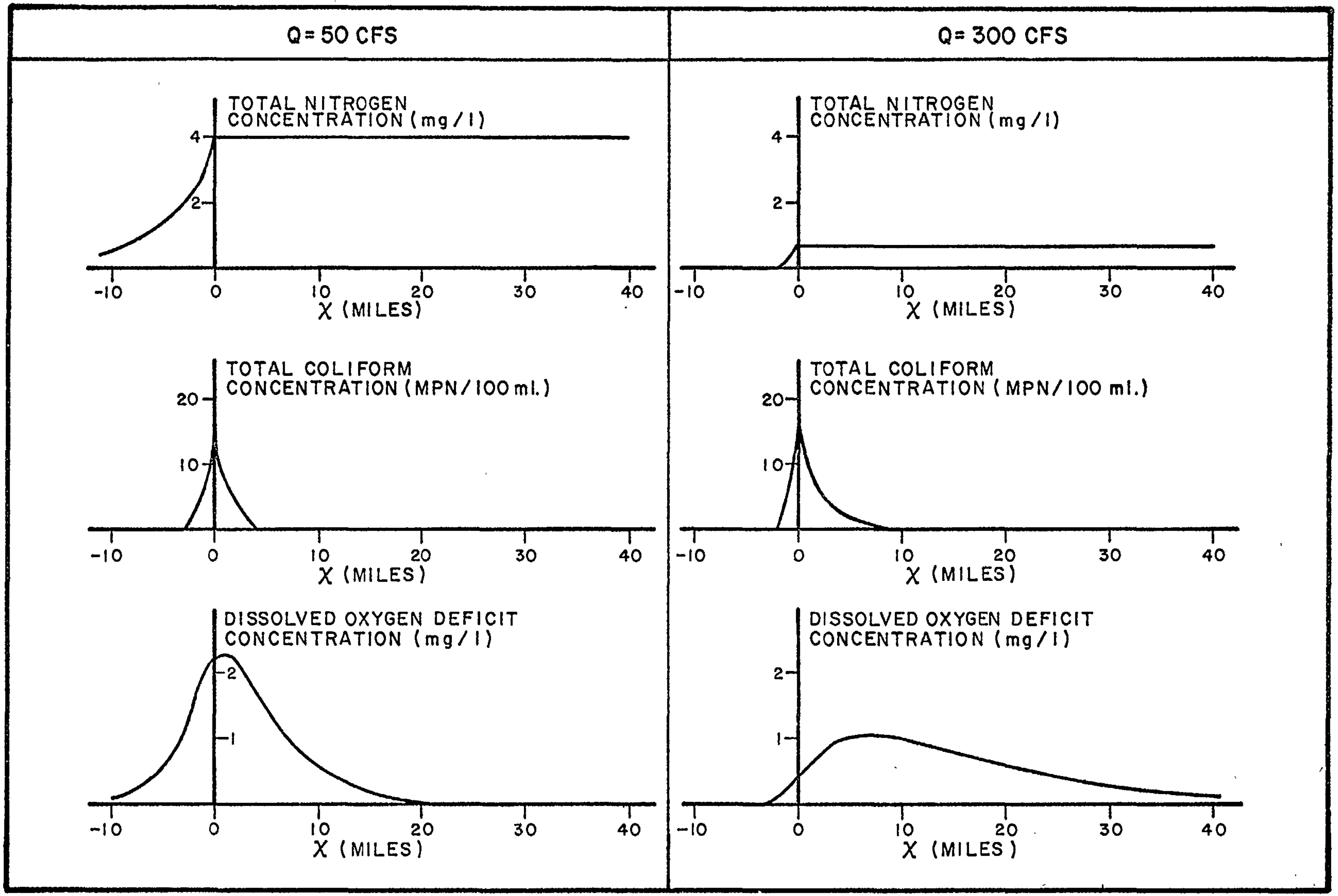
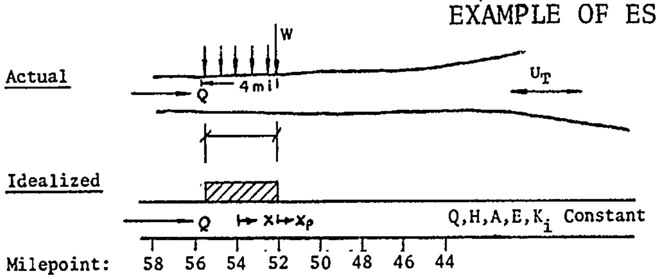


FIGURE 5-16
CONCENTRATION PROFILES FOR ESTUARY WITH SINGLE POINT SOURCE

TABLE 5-18

EXAMPLE OF ESTUARY WITH MULTIPLE WASTE SOURCES



Freshwater Flow (Q)

Low Flow = 50 cfs

Summ. Av. = 300 cfs

Summ. Storm = 900 cfs

Mean Water Depth (H) = 10 ft

Av. X-Sect. Area (A) = 5000 sq. ft.

Av. Tidal Veloc, (U_T) = 0.2 knots

Dispersion Coeff. (E) = 2 mi^2/day

BOD-DO: $K_1=K_r=0.25$/day

: $K_{12}=K_d=K_r$

: $K_2=K_a$

T.Coli: $K_1=K_c=2$/day

Point Source Flow: 20 cfs

PS Effl. Conc:

Total Coli. = 10,000 MPN/100 ml

UOD = 160 mg/l

Non-point Source (Consider Uniformly Distributed Over 4 miles.)

Flow: 20 cfs summer av, 100 cfs summer storm

T.Coli: 3×10^5 MPN/100 ml, UOD: 80 mg/l

	Point Source		Non-point Source	
Condition	T.Col	UOD(lb/day)	T.Coli	UOD(lb/mi/day)
Low Flow	20x10000	20x160x5.4=17280	0	0
Summ Av.	20x10000	20x160x5.4=17280	-	20x80x5.4/4=2160
Summ Storm	20x10000	20x160x5.4=17280	$100 \times 3 \times 10^5 \div 4$mi	

Total Coli Analysis-Summer Storm Condition

$Q = 900$ cfs, $U = (900/5000)\ 16.4 = 2.95$ mi/day, $E = 2\ mi^2$/day, $K_c = 2$/day

$m = 2 \times 2/(2.95)^2 = 0.460,\ m = \sqrt{1 + 4 \times 0.460} = 1.685$

$g = (2.95/(2 \times 2)) \cdot (1 + 1.685) = 1.980$/mi

$j = (2.95/(2 \times 2)) \cdot (1 - 1.685) = -0.505$/mi

From Table 5-11, T. Coli Conc. are calculated as follows:

Point Source ($X_p = 0$ at MP 52)

$$L = (20 \times 10{,}000/(900 \times 1.685))e^{1.980x_p},\ X_p \leq 0$$

$$L = 132\ e^{-0.505xP},\ X_p \geq 0$$

Non-Point Source (x = 0 at MP 54, a = 2 mi)

$$L = \frac{100 \times 3 \times 10^5/4}{900x1.685x1.980} (e^{1.980(x+2)} - e^{1.980(x-2)}\ ,\ x \leq -2$$

$$L = \frac{100 \times 3 \times 10^5/4}{900 \times 1.685} (e^{-0.505(x+2)} - e^{-0.505(x-2)})\ ,\ -2 \leq x \leq 2$$

$$L = \frac{100 \times 3 \times 10^5/4}{900x1.685(-0.505)} (e^{-0.505(x+2)} - e^{-0.505(x-2)})\ ,\ X \geq 2$$

L_{MAX} occurs at $X_1 = 2/1.685 = 1.187$ mi (Table 5-10)

	PS		NPS		Total		PS		NPS		Total
MP	XP	Conc.	x	Conc.	Conc.	MP	xp	Conc.	x	Conc.	Conc.
60	- 8	0	- 6	1	1	50	2	48	4	3094	3142
58	- 6	0	- 4	48	48	49	3	29	5	1867	1896
57	- 5	0	- 3	345	345	48	4	18	6	1127	1145
56	- 4	0	- 2	2498	2498	47	5	11	7	680	691
55	- 3	0	- 1	6375	6375	46	6	6	8	410	416
54	- 2	3	0	8677	8680	45	7	4	9	248	252
53	- 1	18	1	9794	9812	44	8	2	10	149	151
52.81	-0.81	26	1.19	9834*	9860*	42	10	1	12	54	55
52	0	132*	2	8495	8627	40	12	0	14	20	20
51	1	80	3	5126	5206	38	14	0	16	7	7

*Max. Conc.

TABLE 5-18
(Continued)
EXAMPLE OF ESTUARY WITH MULTIPLE WASTE SOURCES

DISSOLVED OXYGEN DEFICIT ANALYSIS SUMMER AVERAGE CONDITION

Q = 300 cfs, U = 0.984 mi/day, E = 2 mi^2/day, $K_1 = K_r = 0.25$/day

$K_2 = K_a$ = 4 ft/day ÷ 10 ft = 0.4/day

$m_1 = 0.516$, $m_1 = 1.750$, $g_1 = 0.677$/mi, $j_1 = 0.1845$/mi

$n_2 = 0.826$, $m_2 = 2.075$, $g_2 = 0.756$/mi, $j_2 = -0.264$/mi

From Table 5-11, DO DEF are calculated as follows:

Point Source

$$D = \frac{0.25}{0.4-0.25} \cdot \frac{17280}{300x5.4x1.750} \cdot (e^{0.677x_p} - \frac{1.750}{2.075} e^{0.756 X_p}), \; X_p \le 0$$

$$D = \frac{0.25}{0.4-0.25} \cdot \frac{17280}{300x5.4x1.750} \cdot (e^{-0.1845x_p} - \frac{1.750}{2.075} e^{-0.264x_p}), \; x_p \ge 0$$

$$X_p\Big|_c = \ln(\frac{1.750}{2.075} \quad \frac{1-2.075}{1-1.750}) \div (\frac{0.984}{2x2} (2.075 - 1.750)) = 2.37 \text{ mi}$$

Non-Point Source

$$D = \frac{0.25}{0.4-0.25} \cdot \frac{2160}{300x5.4} (\frac{e^{0.677(x+2)} - e^{0.677(x-2)}}{1.750 \text{ x } 0.677} - \frac{e^{0.756(x+2)} - e^{0.765(x-2)}}{2.075 \text{ x } 0.756}), \; x \le -2$$

$$D = \frac{0.25}{0.4-0.25} \cdot \frac{2160}{300x5.4} \cdot ((\frac{e^{-0.1845(x+2)} - 1}{1.750(-0.1845)} - \frac{e^{0.677(x-2)} - 1}{1.750x0.677}) - (\frac{e^{-0.264(x+2)} - 1}{2.075(-0.264)} - \frac{e^{0.756(x-2)} - 1}{2.075x0.756})), \; -2 \le x \le 2$$

$$D = \frac{0.25}{0.4-0.25} \cdot \frac{2160}{300x5.4} \cdot (\frac{e^{-0.1845(x+2)} - e^{-0.1845(x-2)}}{1.750(-0.1845)} - \frac{e^{-0.264(x+2)} - e^{-0.264(x-2)}}{2.075x0.756}), \; x \ge 2$$

Location of max. deficit (Table 5-_)

$$s = \frac{e^{2x2(-0.1845)} - 1}{1.750} - \frac{e^{2x2(-0.264)} - 1}{2.075} = + 0.016$$

Since s > 0, Xc > a = 2 and xc satisfies (Table 5-9)

$$\frac{e^{-0.1845(x_c+2)} - e^{-0.1845(x_c-2)}}{1.750} = \frac{e^{-0.264(x_c+2)} - e^{-0.264(X_c-2)}}{2.075}$$

from which X_c = 2.66 miles.

	PS		NPS		Total		PS		NPS		Total
MP	xp	Conc	x	Conc.	Conc	MP	xp	Conc.	x	Conc	Conc.
60	-8	0.02	-6	0.05	0.07	49.63	2.37	1.98*	4.37	0.90	2.88
58	-6	0.08	-4	0.15	0.23	49	3	1.96	5	0.87	2.83
56	-4	0.26	-2	0.40	0.66	48	4	1.88	6	0.80	2.68
55	-3	0.45	-1	0.58	1.03	46	6	1.60	8	0.64	2.24
54	-2	0.73	0	0.75	1.48	44	8	1.29	10	0.50	1.79
53	-1	1.59	2	0.95	2.54	37	15	0.47	17	0.17	0.64
51.34	0.66	1.80	2.66	0.96*	2.76	32	20	0.21	22	0.01	0.22
51	1	1.87	3	0.95	2.82	27	25	0.09	27	0.00	0.09
50	2	1.97	4	0.92	2.89*						

*Max. Conc.

Dissolved Oxygen Deficit - Low Flow

Q = 50 cfs, U = 0.164 mi/day,

$m_1 = 18.59$, $m_1 = 8.681$, $g_1 = 0.3969$/mi, $j_k = -0.3149$/mi

$n_2 = 29.74$, $m_2 = 10.953$, $g_2 = 0.4901$/mi, $j_2 = 0.4081$/mi

Point Source: x_c = 0.2861 miles, D_c = 2.56 mg/l

Non-point Source: None

distributed intermittent source with characteristics similar to a separate sewer system (Table 2-11,pg. 2-58). Total coliform concentrations are calculated for a summer storm condition typical of a major storm system with a duration of several days. Dissolved oxygen deficits are calculated for summer average and low flow conditions. Estimates of the non-point flows are consonant with the municipal area served and detailed calculation of these values would be performed using methods in Chapter 3.

Using the equations in Table 5-11 for both point and non-point sources, tabular solutions for given estuarine milepoints are set up. The point source solutions are generated for the coordinate system x_p, with origin at Milepoint 52, and the non-point concentrations use an x - abscissa with origin at Milepoint 54. Following the detailed equations and tabular solutions for both total coliform and dissolved oxygen deficit, plots of the concentration profiles are presented for the combined impact of both sources and the effect of the point source alone in Figure 5-17.

The profiles for total coliform indicate that the separate sewer system is causing over 5,000 MPN/100 ml in the estuary from Milepoint 50 to Milepoint 56 whereas the peak concentration due to the STP is approximately 130 MPN/100 ml. The results for the intermittent separate sewer discharge are reasonably accurate since the bacteria are highly reactive and only require one to several days to approach the equilibrium (steady state) concentration. Thus, it could be concluded that the sewer system is a major source of bacterial contamination and further efforts should be devoted to quantifying its impact on a statistical basis.

The peak dissolved oxygen deficit occurs between Milepoint 49 and Milepoint 50 for the summer average condition. The STP contributes 2.0 mg/l and the separate sewer 0.9 mg/l to the total deficit of 2.9 mg/l. The deficit of 0.9 mg/l from the intermittent separate sewer discharges may be a significant overestimation since the more slowly reactive UOD may require several days to several weeks to attain a steady state concentration. In the low flow analysis without separate sewer discharges, the STP causes a peak deficit of 2.6 mg/l. If one assumes an estuarine chloride concentration of 7500 mg/l (30% seawater) and a water temperature

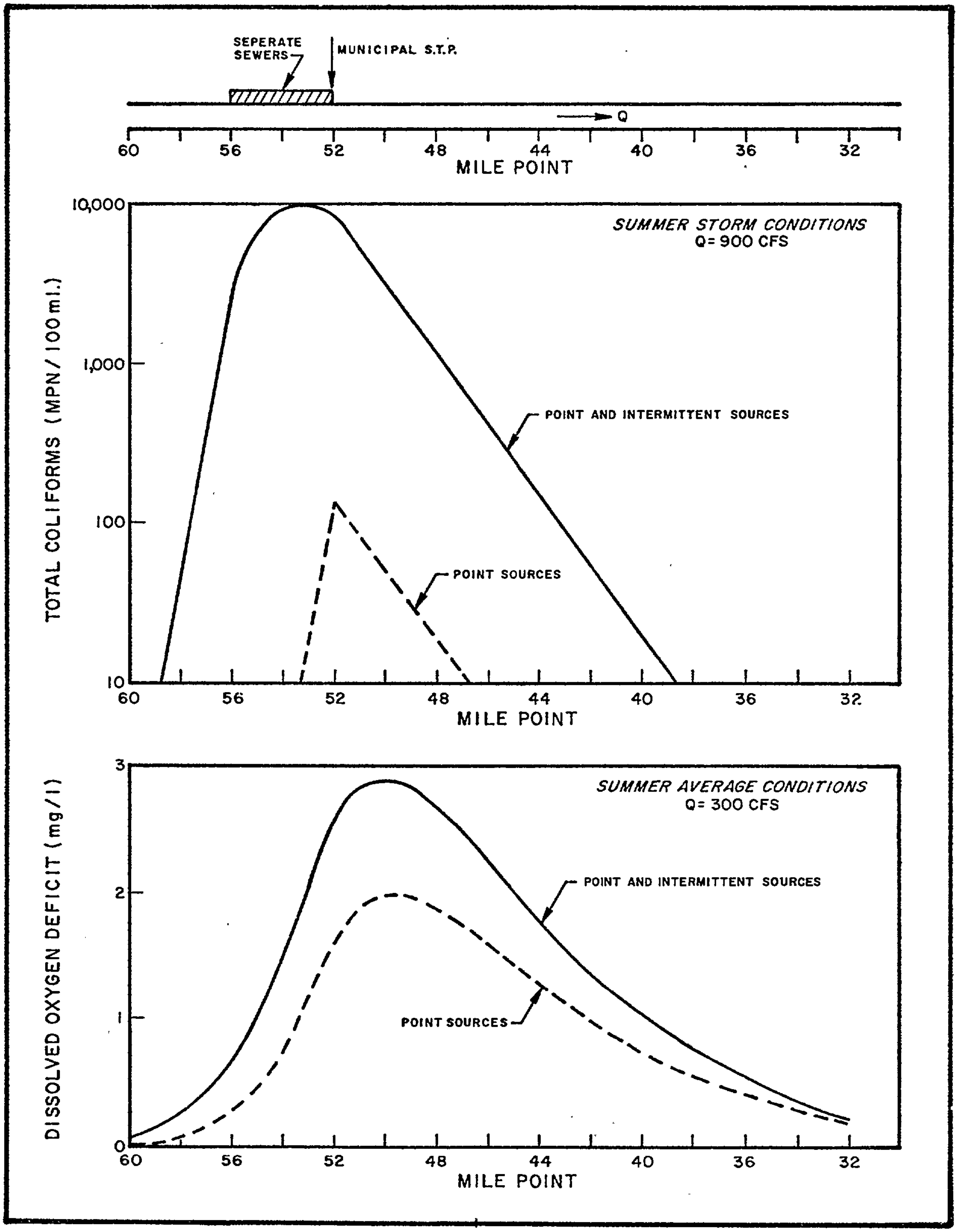


FIGURE 5-17
CONCENTRATION PROFILES-ESTUARY WITH MULTIPLE SOURCES

of $25^{\circ}C$, the dissolved oxygen saturation concentration is 7.7 mg/l (Figure 2-19(b)). Therefore, minimum dissolved oxygen concentrations of 4.6 mg/l and 5.1 mg/l occur in the estuary for summer average and low flow conditions respectively. Depending on the water use and classification, water quality standards may or may not be contravened by these results. In any case, the dominant cause of the deficit is the continuous STP discharge and solution of any dissolved oxygen problem would emphasize alternatives involving the STP.

5.3.4 Methods For Estimating Water Quality Variability in Estuaries

The steady state water quality analysis for estuaries described in the previous sections is appropriate for estimating receiving water responses for annual average loads, or as shown in Section 5.3.3, for water quality analyses of average conditions during critical average or high flow (storm) periods. Another method of evaluating variability in receiving water response is briefly presented in this section.

In cases where the steady state response can be developed using the equations in Table 5-11 or a numerical water quality model, it is often useful to obtain an estimate of the random (at least for purposes of this discussion) variability of that response. Consider the observed and computed water quality profiles shown in Figure 5-18. In this case, the computed response, which adequately represents the mean observed water quality data, is shown to be within the water quality standard. However, the observed variability in dissolved oxygen indicates that frequent measurements are below the standard. An important question is: if mean water quality can be maintained above a standard, as in Figure 5-18, what is the estimated frequency with which the standard will be violated due to random variations about the mean?

The task facing the 208 planner is often one of evaluating the frequency with which water quality objectives will be violated under a wide range of control options, including the 'no action' alternative. Steady state mean responses are generally limited in their interpretation within this context. However, a simple method is presented in this section for

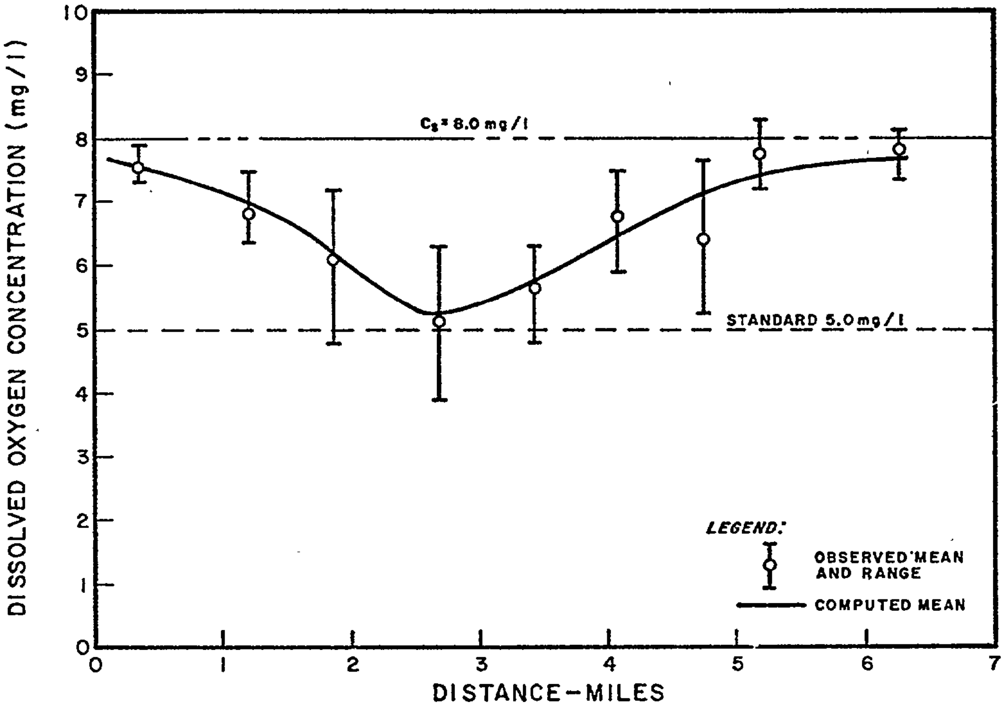


FIGURE 5-18
EXAMPLE SUMMARY OF DISSOLVED OXYGEN DATA
SHOWING VARIABILITY

employing steady state model results in concert with water quality data to estimate the frequency of compliance with water quality objectives.

5.3.4.1 Method of Analysis

In many receiving waters the variability of a water quality indicator increases as the stress on the area increases. The greatest variability around the mean dissolved oxygen response in a stream is expected to occur in the region of the maximum deficit(10). Similar responses are observed in estuaries(11). However, the analytical techniques to evaluate such responses are not straightforward for estuaries.

An empirical approach to analyzing the problem is suggested in Figure 5-19(a) which presents a cross plot of the long term mean and the standard deviation of dissolved oxygen deficit concentrations for a number of sampling stations in South San Francisco Bay. The probability density function at each station must be known and must be the same at all stations. The display indicates a trend in this study area toward increasing variability in deficit concentration in areas of large mean dissolved oxygen deficit concentrations. In this particular case the statistics were shown to be associated with normally distributed deficits at each station.

Figure 5-20 displays a mean computed dissolved oxygen concentration profile for the study area. Also shown in the figure is the estimated lower 90 percentile dissolved oxygen concentration computed in the following manner.

$$\sigma_{def} = 0.4 + .25\ \bar{Def}\ \text{(from Figure 5-19)}$$

$$Def_{90} = 90\ \text{percentile deficit} = \bar{Def} + 1.27\ (0.4 + .25\ (\bar{Def})) \qquad (5\text{-}7)$$

where:

$\bar{Def}$ = mean computed dissolved oxygen deficit concentration = $(C_s - \bar{DO})$

1.27 = 90% ordinate of the normal cumulative probability curve, β

C_s = Dissolved oxygen saturation

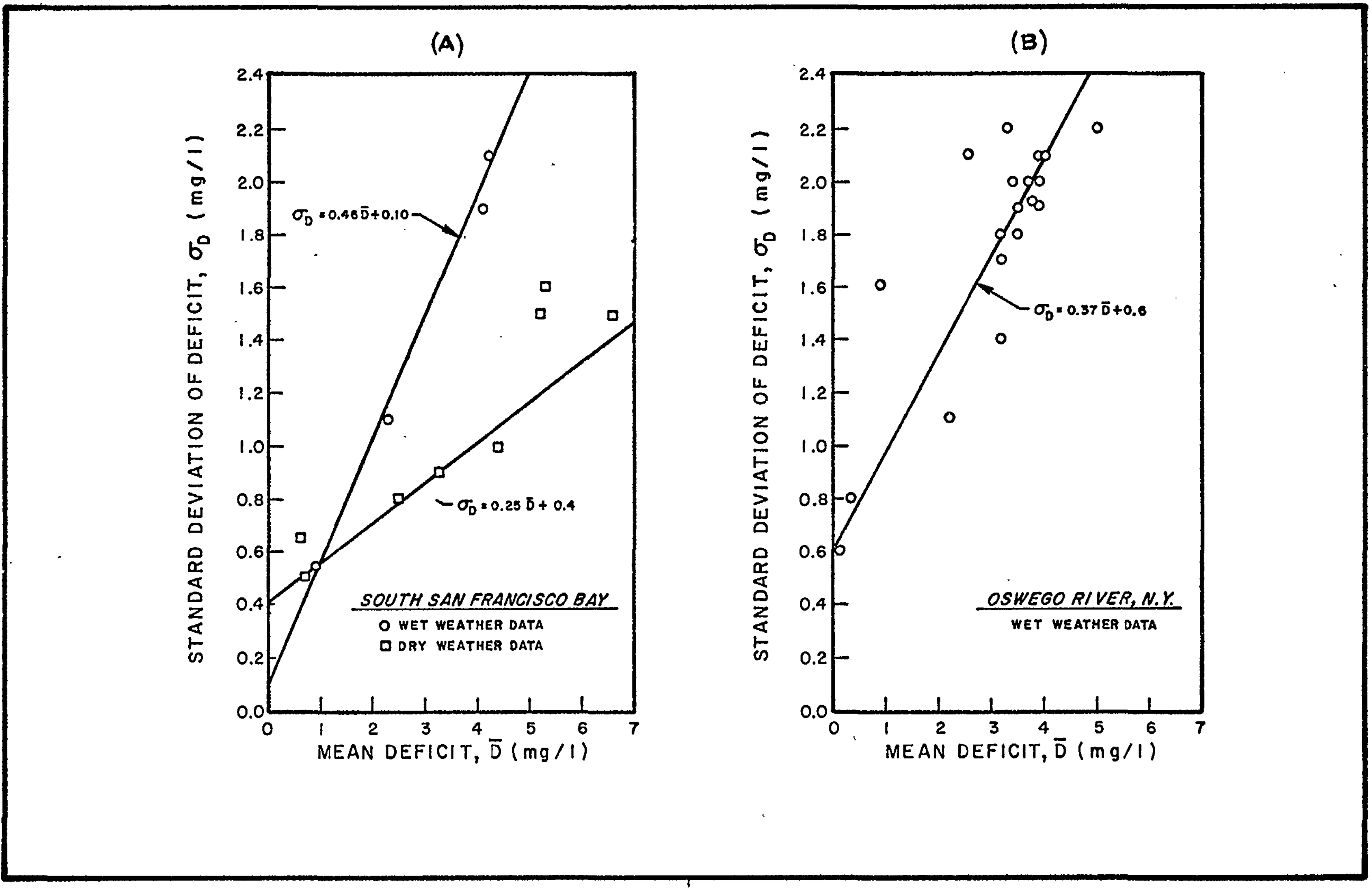


FIGURE 5-19

MEAN VS. STANDARD DEVIATION OF DISSOLVED OXYGEN DEFICIT CONCENTRATION

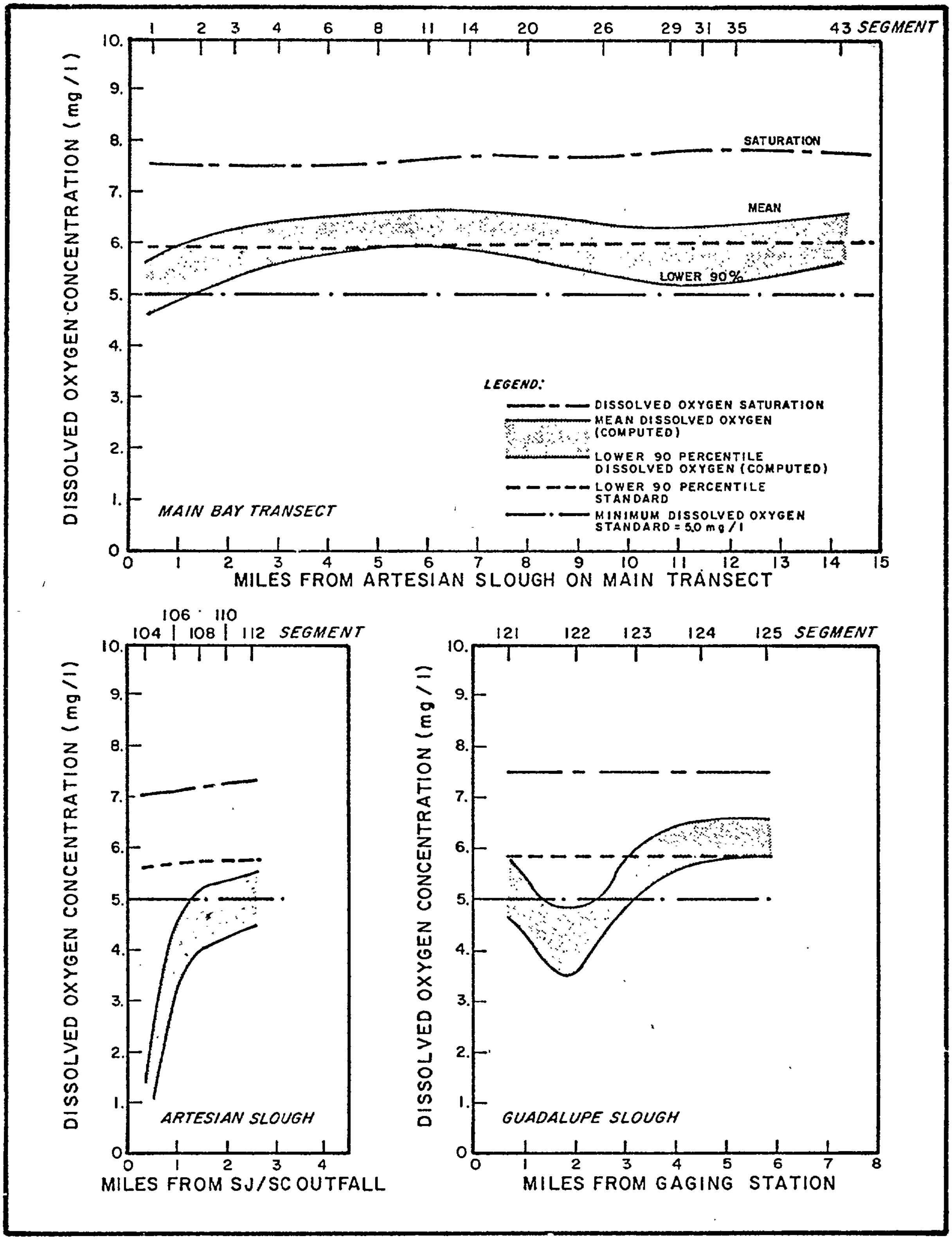


FIGURE 5-20
EXAMPLE OF ESTIMATING VARIABILITY
AROUND STEADY STATE MEAN WATER QUALITY

$\overline{DO}$ = mean computed dissolved oxygen concentration

Def_{90} = 90 percentile deficit concentration

For example, if the mean dissolved oxygen deficit concentration at a location in the South Bay is computed to be 1.2 mg/1 and saturation is 7.5 mg/1, the lower 90 percentile dissolved oxygen concentration can be estimated as:

$\bar{Def}$ = 1.2 mg/1

Def_{90} = $1.2 + 1.27 \cdot (0.4 + .25(1.2)) = 2.09$ mg/1

DO_{90} = $C_s - Def_{90} = 7.5 - 2.09 = 5.41$ mg/1

where:

DO_{90} = 90 percentile dissolved oxygen concentration.

Conversely, if a relevant planning question is to respond to the frequency with which the 5.0 mg/1 standard is expected to be violated in the above example, equation 5-7 can be solved for the ordinate, β, on the normal cumulative probability curve. The formula for this computation is:

$$\beta = \frac{DO_\beta - (C_s - \bar{Def})}{0.4 + 0.25\ \bar{Def}}$$

$$\beta = \frac{5.0 - (7.5 - 1.2)}{.4 + .25(1.2)} = \frac{-1.3}{0.7} = -1.86$$

Cumulative normal distribution tables, found in most statistical texts or (17) can be used to determine that 1.86 corresponds to a 97% event. Therefore, 3% of the dissolved oxygen concentrations at that location might fall below the 5.0 mg/1 standard due to random background factors not included in the modeling analysis.

Application of this method need not be limited to dissolved oxygen concentration alone. Relationships similar to those shown in Figure 5-19 can be used to develop background variabilities in specific water quality constituents for purposes of making probablistic statements using steady state water quality model results. Such methods are particularly useful in estimating the frequency with which standards might be violated in streams or estuaries. The 208 planner is cautioned to pay particular attention to the frequency distribution of the concentrations (i.e., normal, lognormal, gamma) in applying this analysis and also to the sample size used to generate each point in a figure such as Figure 5-19. A sample size greater than 10 observations at each station should be adequate to indicate reliable trends in the data.

5.4 Coastal Areas

The disposal of wastewaters in open water bodies is frequently an attractive wastewater discharge alternative because of the large quantities of available dilution water. The discharge is accomplished using an underwater outfall conveyance line to the disposal site. Good engineering practice usually necessitates a multiport diffuser structure at the terminus of the outfall to facilitate dilution-mixing of the wastewater in the overlying water column. Diffuser design is expensive and requires a comprehensive analysis of many water quality constituents. Caution must be exercised in final design considerations to justify such a structure. Analyses of this type consider such factors as: BOD-dissolved oxygen, nutrients and biostimulation, bioinhibition, sanitary water quality, and aesthetic concerns.

Analysis of water quality in coastal areas and large open water bodies is often viewed as a multi-dimensional problem, the principal features of which are:

1. Plume rise and initial dilution
2. Two dimensional spreading of diluted waste constituents within a mixing zone
3. Advective transport of dilute wastewater mixtures toward shorelines and other vulnerable areas.

The analysis proceeds through separate analysis steps which focus on these three scales of problems (at least until a no effect level of the pertinent water quality constituents is reached). That is, if the analysis indicates that the concentration of pertinent water quality constituents meets objective levels after initial dilution, a conservative estimate of water quality in the mixing zone or at shoreline areas is that it also meets the objective levels.

Section 5.4 presents first approximation analysis techniques and guidelines which are appropriate for analyzing this class of wastewater discharges. The methods can be applied using data collected in the study area in combination with estimated model coefficients which find rather wide application in engineering practice. The methods are also applicable to study areas other than coastal areas. For example open water bodies such as large lakes or large estuaries can be analysed using methods presented here. Guidelines for these analyses are presented with each analysis description. Dispersion coefficients in large lakes can be found in Section 5.5.3.3.

The analysis methods for coastal areas are only for preliminary assessments of water quality impacts. While the methodologies are useful in making level 1 decisions they can and in many cases should be supplemented by more detailed numerical modeling techniques. The 208 planner should exercise engineering judgement in deciding when an analysis indicates the existence of water quality problems which warrant detailed analysis either because of the magnitude of the water quality problem or the costs to implement controls suggested by the preliminary analysis. Guidance in this regard is presented in Section 5.6.

5.4.1 Initial Dilution

Initial dilution is the term applied to the ratio of seawater to wastewater mixed by port discharge of effluent from a diffuser manifold into seawater. Initial dilution takes place during the rise of the effluent plume toward the water surface or to some intermediate trapping level. Momentum of the discharging jet together with the buoyancy associated with density and temperature differences produce shear stresses resulting in boundary turbulence as the plume rises toward the surface. This turbulence facilitates seawater entrainment and dilution of the effluent. The relationships between buoyancy, shear, and dilution are complex and are dependent to a large degree on the ambient density structure in the receiving water. Existence of a moderate salinity gradient or a moderate thermocline may preclude the effluent plume reaching the water surface. This specific case will not be discussed in this manual. However the user is referred to reference (12) for a detailed discussion of nomographic methods for analysing the stratified ocean case. In general the effect of a vertical density gradient is to reduce initial dilution and also the height of rise of the effluent plume. Some guidance in evaluating this case is presented later in this section.

The assumption of a uniform density structure implies vertical uniformity in the receiving water in so far as temperature and salinity are concerned.

In this regard, Rawn, Bowerman, and Brooks (13) using dimensional analysis reevaluated the earlier work of Rawn and Palmer (14), for hydraulic model discharges of submerged jets. The result is an empirical correlation between observed initial dilutions and dimensionless hydraulic parameters which are appropriate to full scale design. Short term dilution varies as a function of distance along the centerline of the effluent plume.

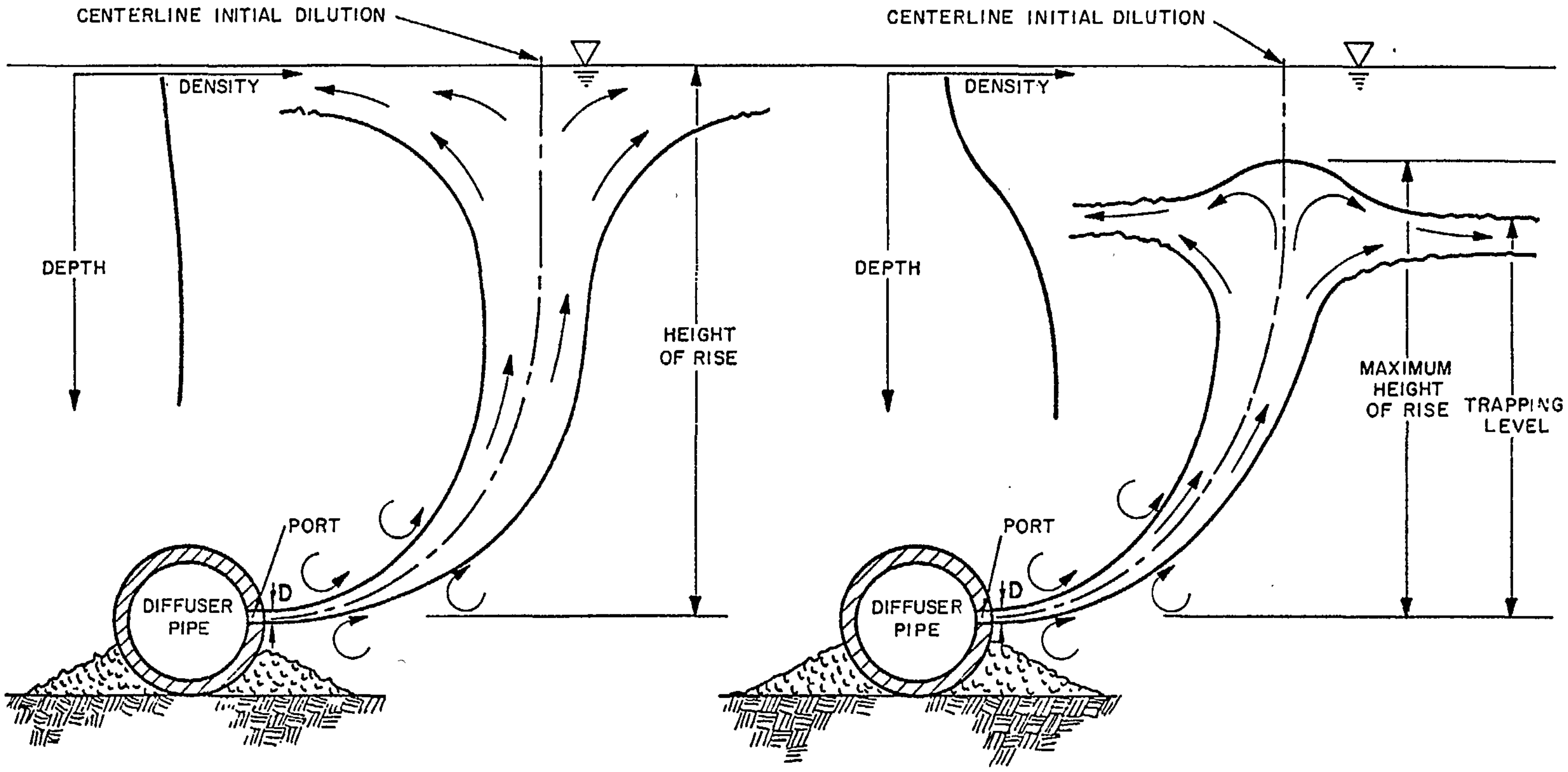
Figure 5-21 schematically shows the effluent plume resulting from a single port discharge as it rises toward the water surface in a stratified and non-stratified environment. Assuming no stratification in the water mass receiving the effluent discharge, S_o, the initial dilution at the center of the surfaced plume, is considered a function of five independent variables: y_o is the total depth from center of outlet to the surface; D is the initial diameter of jet (approximated by the port diameter); V is the jet velocity; g' is the apparent acceleration due to gravity; and γ is the kinematic viscosity of the sewage.

g' may be expressed as:

$$g' = g\frac{\Delta s}{s} \qquad (5\text{-}8)$$

by balancing of forces acting on a buoyed body. In this equation s is the specific gravity of the wastewater, Δs the difference in specific gravity of seawater and wastewater and g the gravitional constant.

It is then possible, to express S_o as a function of three independent dimensionless parameters:



BUOYANT PLUME RISE IN A MILDLY STRATIFIED WATER COLUMN

BUOYANT PLUME RISE IN A HIGHLY STRATIFIED WATER COLUMN

FIGURE 5-21

BUOYANT PLUME CHARACTERISTICS IN STRATIFIED AND NON-STRATIFIED ENVIRONMENT

$$S_o = f\left(\frac{y_o}{D}, F, R\right) \qquad (5\text{-}9)$$

where:

The Froude number, $F = \frac{V}{\sqrt{g'D}}$ and the Reynolds number, $R = \frac{VD}{\gamma}$.

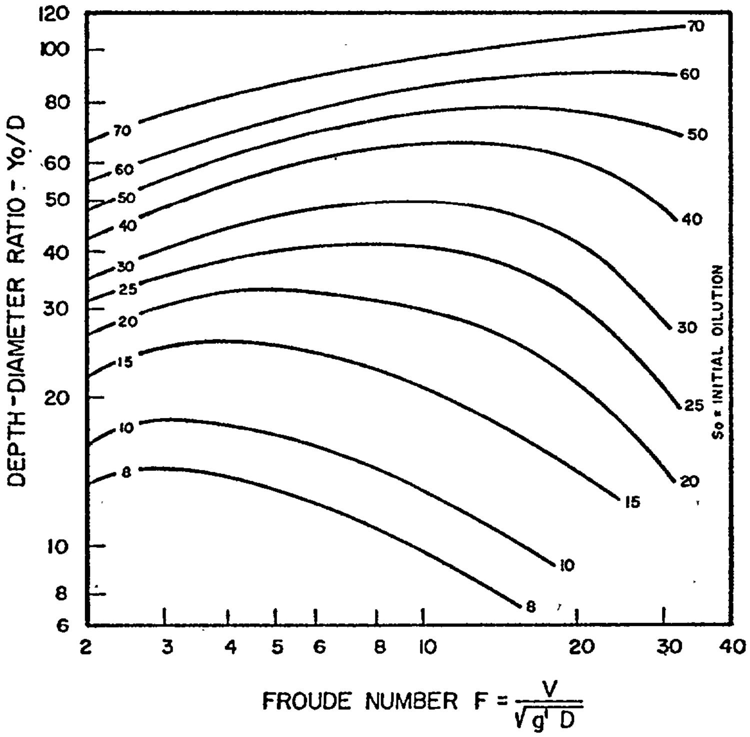
The magnitude of the Froude number reflects the path of jet discharge and plume rise as influenced by gravity and by density differences between seawater and sewage. The Reynolds number is indicative of the effect of inertia and viscosity as a measure of turbulence. Several investigators (13, 15) have presented evidence that, for ranges of turbulent flow, the Reynolds number has little influence on the initial dilution. Apparently, once significant turbulent flow is developed, other hydrodynamic forces are of more consequence to dilution.

Eliminating R as a factor once turbulent flow has developed reduces Equation (5-9) to:

$$S_o = f\left(\frac{Y_o}{D}, F\right) \qquad (5\text{-}10)$$

An illustration of the relationship implied in equation(5-10) is presented graphically with smooth curves in Figure 5-22. This diagram indicates that for a given Froude number, the initial dilution is directly related to the depth of water above the discharging port and inversely related to the diameter of the port. The greater the depth of receiving water and the smaller the port, the greater is the initial dilution. These curves, based on observed initial dilution data, agree reasonably with other theoretical developments, and, with a minor modification (empirical) consisting of multiplication of S_o by 1.15 to account for a zone of flow establishment, are applicable to field conditions. With the above proviso, the curves indicate the minimum initial surface dilution expected from a single round buoyant jet for various conditions of submergence, port diameter, port flow, and density difference.

As an example, assume that the required initial dilution is 50 to 1 (50:1). The question is what water depth is required to achieve this



REFERENCE: AFTER RAWN, BOWERMAN AND BROOKS
PROC. SED, ASCE, VOL. 86, NO SA 2, 1960

FIGURE 5-22
INITIAL DILUTION FOR SUBMERGED HORIZONTAL JET DISCHARGE

objective using a 500 foot diffuser consisting of 40 three inch ports. The design flow is 15 MGD.

An assumption regarding the flow distribution through the ports is required first. A good engineering diffuser design normally distributes the flow uniformly along the outfall. Therefore, the port discharge can be approximated by:

$$Q_p = 15 \text{ MGD} \times 1.55 \frac{\text{cfs}}{\text{MGD}} \times \frac{1}{40} = 0.58 \text{ cfs};$$

The port area is: $A_p = (\frac{3}{12})^2 \frac{\pi}{4} = 0.049 \text{ ft}^2$;

and the port velocity is 11.83 feet per second.

The density of seawater in coastal areas is normally between 1.020 and 1.040 and depends upon nearness to major freshwater discharges from rivers and approaches 1.000 from fresh water areas at the upstream end of tidal rivers. Reference (16) presents methods for computing seawater density from salinity and temperature data. For purposes of the present example, the density of sewater is selected as 1.025, a typical value in many coastal areas. The density of the effluent can normally be taken as 1.00 unless data indicates significant concentrations of total dissolved solids (greater than 1000 ppm).

The Froude number, F, is therefore computed as:

$$F = 11.83 \times \frac{1}{\sqrt{32.2 \times \frac{1.025 - 1.000}{1.000} \times .25}} = 26.4$$

Figure 5-22 yields a required Y/D ratio of 70 to accomplish the required initial dilution. Therefore, the diffuser must be located in 18 feet of water. At low water slack:

$$\frac{Y}{D} = 70 = \frac{Y}{.25}; \ Y = 17.5 \text{ feet}$$

If the outfall discharge rate or effluent quality characteristics are expected to vary between wet and dry weather conditions as might be expected if the treatment plant load is from a combined sewer system, the analysis should be performed for wet weather conditions. In this case the analysis would be repeated for higher flow conditions (i.e., Q = 40 MGD) and the required initial dilution would reflect wet weather effluent quality.

In cases where a non-uniform denisty gradient exists, the foregoing analysis is not supported by experimental data. In those cases the 208 planner can develop an estimate of the range of initial dilutions to be expected in one of two ways, both of which require some experimental data on stable vertical density gradients in the study area. The first method is a very simplified analysis, but one which is generally acceptable for preliminary assessment purposes. Assume a maximum height of rise of the effluent plume by inspecting the vertical density gradients in the study area. The density gradient shown in Figure 5-21(b) is typical of many coastal areas. That is there is a sharp break in the density structure at an intermediate depth. This depth can be used in Figure 5-22 to estimate the maximum initial dilution. For most situations this estimate should not be in error by more than 20 - 30%.

An alternate method for performing more reliable estimates of initial dilution in uniformly stratified coastal areas is presented in Reference (12). This solution to a complex set of experimentally derived relationships yields estimates of the expected height of rise and the expected initial dilution. The reference presents detailed examples of the calculation procedure.

5.4.2 Analysis of Far Field Effects

Ocean disposal of treated effluent necessitates consideration of several factors affecting the final concentrations of contaminants in the nearshore ocean environment. These include physical, chemical, and biological factors, each of which contributes to the total dilution achieved by outfall disposal of wastewaters.

Physical factors include the dilution of effluent constituents with surrounding seawater and dispersion, which spreads the effluent field. In the specific case of bacteria, aftergrowth phenomenon and the natural die-away and predation are additional factors effecting total dilution.

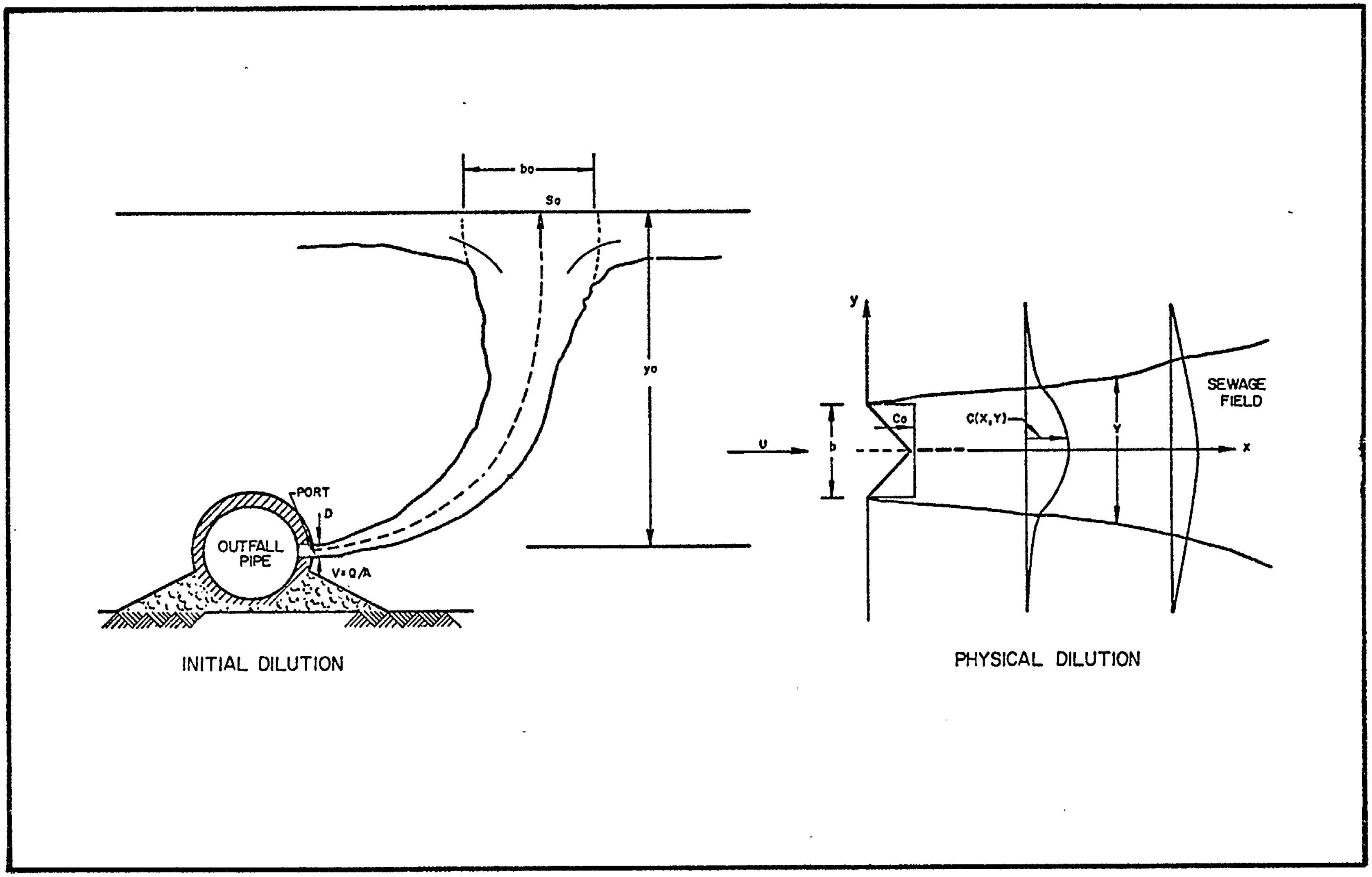
The emphasis in Section 5.4.2 is on analyzing the transport and distribution of coliform bacteria and the consequent impacts on sanitary water quality. The analysis techniques are however appropriate for other water quality indicators such as suspended solids, acute and chronic toxicants, total nutrients and, with some modifications, color.

Bacterial levels are presently the major criteria for evaluating the acceptability of recreational bathing waters and areas of shellfish culture. Therefore, a prime concern in the design of ocean outfall and diffuser systems is the maintenance of coliform objectives at beaches and shellfish beds. Four basic factors are considered in forecasting coliform levels from the discharge of treated wastewater effluent in the marine environment: a) initial concentration in the treated effluent, b) initial plume dilution, c) physical dilution, and d) bacterial aftergrowth and dieoff. Initial concentrations are normally developed from treatment plant effluent records and from land side simulation techniques similar to those presented in Chapter 3. Initial dilution was treated in the previous section of this chapter.

5.4.2.1 Physical Dilution

After the initial dilution of the effluent field is completed, tidal motion and other larger scale phenomenon such as wind and ocean currents spread the field away from the diffuser zone. Longitudinal and vertical mixing are generally assumed to be negligible. Therefore, lateral dispersion is the principal factor responsible for far field dilution known as physical dilution.

Figure 5-23 exhibits an idealized surface effluent field as it is translated away from a diffuser device by an ocean current. The diagram indicates the concentration distribution of material in the effluent field as it moves along the longitudinal axis, x, from the diffuser device at current velocity, U. The initial concentration C_o, in a field width, b, is almost uniform in the immediate vicinity of the diffuser structure. However, as the field is translated and is dispersed along the lateral axis, concentration gradients develop. The maximum concentration along the centerline of the effluent field decreases and



INITIAL DILUTION

PHYSICAL DILUTION

FIGURE 5-23
DIAGRAM OF INITIAL AND PHYSICAL DILUTION

is gradually reduced toward the limits of the effluent field. The concentration of any effluent material is related therefore, to longitudinal distance, x, and the lateral spread, y, of the effluent field.

A mathematical representation of this phenomenon predicts the short-term steady state concentration of an effluent field produced by discharge from a multiport diffuser.

The partial differential equation governing this phenomenon is discussed by Brooks (17). The equation is solved for the case of variable dispersion coefficients. It is assumed that lateral dispersion, expressed by the coefficient E_y, varies with distance along the longitudinal axis of the effluent field as a function of the field width. Based on empirical evidence and theoretical development, oceanic dispersion coefficients have been related to scale by the following expression:

$$E = \varepsilon L^{4/3} \qquad (5\text{-}11)$$

where:

E = the lateral dispersion coefficient

L = the width of the effluent field,

ε = an empirical constant.

A value of $\varepsilon = 0.01$ for cgs units is suitable for surface effluent fields. No information is presently available in the literature for the submerged field case; that is, the case in which the effluent plume remains submerged below the density gradients shown in Figure 5-21(b).

By applying the pertinent boundary conditions, and assuming that the lateral dispersion coefficient varies with scale according to equation (5-11) where L is taken as the projected width of the effluent field normal to the current, the concentration of an effluent constituent at various distances from the outfall diffuser is:

$$C = C_o \operatorname{erf} \left[\frac{3/2}{\left(1 + \frac{8E_o x}{U b^2}\right)^3 - 1} \right]^{1/2} \qquad (5\text{-}12)$$

in which:

C = concentration along x-axis of the effluent field direction of current

C_o = concentration at $x = 0$.

x = distance along x-axis of effluent field

U = current velocity along the x-axis

b = initial width of effluent field above the diffuser and normal to the current

E_o = lateral dispersion coefficient at $x = 0$ as defined by equation (5-11) where $L = b$

erf(z)= error function $\frac{2}{\sqrt{\pi}} \int_o^z e^{-\nu^2} d\nu$

Once an outfall and diffuser system has been designed on a preliminary basis from initial dilution considerations, equation (5-12) is normally used to determine the pattern and concentration of the resultant effluent field and physical dilution. The lateral dispersion coefficient is assigned on the basis of diffuser orientation, diffuser length and initial field width. Current velocity information may be obtained from Tide and Current Tables and Tidal Current Charts prepared by the National Oceanographic and Atmospheric Administration. This information can be obtained by writing to: National Ocean Survey, Rockville, Maryland 20852 or a local sales office (normally located at marinas or marine supply sales centers).

5.4.2.2 Biological Factors

Discussion to this point has been limited to those physical factors which affect dilution and dispersion of the effluent. All effluent constituents are subject to physical dilution phenomena. However, certain constituents, notably organics and bacteria are subject to reactions which result in a concentration decay.

Bacterial populations undergo growth and death in a natural environment. To incorporate this phenomenon into equation (5-12), a first-order reaction rate, K, is assumed. The resulting equation is:

$$C = C_o e^{\frac{-kx}{U}} \text{ erf} \left[\frac{3/2}{(1 + \frac{8E_o x}{U\ b^2})^3 - 1} \right]^{1/2} \qquad (5\text{-}13)$$

where t is the reaction time, and may be expressed as x/U.

Currently available information on the behavior of bacteria in the marine environment suggests the possibility of an aftergrowth phenomena after discharge into a receiving water. The aftergrowth is followed by a rapid die-away of the bacteria. Sequential aftergrowth and die-away have been postulated as first-order reactions and are normally included in bacterial analysis using equation 5-13. There are two methods of treating aftergrowth phenomenon in equation(5-13). The first is to apply an aftergrowth factor to the initial concentration, C_o. Thus: $C_1 = A\ C_o$, where A is the aftergrowth factor expressed as a multiple of the initial concentration in the effluent field. The magnitude of the aftergrowth is strongly dependent upon water temperature and incident ultraviolet solar radiation. Typically aftergrowth factors range between 1.0 and 5.0. The lower end of the range is indicative of warm waters in sunny conditions such as might occur in the southern portions of the United States. The higher aftergrowth factors are normal in areas where the sunlight intensity is low or where water clarity is poor.

A second method of estimating aftergrowth is through a modification of equation(5-13)to include both the die-away and aftergrowth processes. The resulting equation is similar to equation(5-13)and reflects a separation of the overall reaction rate, K, into its two components: K_1, a die-away coefficient, and K_2, an aftergrowth coeficient.

$$C = C_o\ e^{-K_1 t}\ e^{K_2 t} \text{ erf} \left[\frac{3/2}{(1 + \frac{8E_o x}{Ub^2})^3 - 1} \right]^{1/2} \qquad (5\text{-}14)$$

Where K_1 is treated as a constant in time and K_2 is a positive aftergrowth rate in the time interval $t = 0$ to $t = T_A$. T_A is defined as the time required to realize the aftergrowth fraction A as defined above. Thus:

$$A = e^{K_2 T_A}; \quad T_A = \frac{\ln A}{K_2} \qquad (5\text{-}15)$$

An aftergrowth factor of 3 in 12 hours is typical of marine systems. Therefore: $K_2 = \frac{\ln A}{T_A} = \frac{24}{12} \times \ln(3) = 2.2$ per day.

Knowledge of the physical and geometric parameters of the diffuser and oceanic systems, along with estimates of the appropriate reaction rates, allows the calculation of the probable concentration of substances at any distance from the point of discharge (for various current velocities and directions) on a short-term basis.

5.4.2.3 Application of Analysis Methods

An example of the methods for evaluating water quality at locations remote from the diffuser are demonstrated in this section. A diffuser with characteristics described in Section 5.4.1 is used. In the example, only sanitary (bacterial) water quality is evaluated. Other parameters can be evaluated in a similar manner. The basic data is:

Required initial dilution(S_o)	=	60:1
Length of Diffuser (1)	=	500 feet
Discharge Depth	=	18 feet
Mean Current Velocity (U)	=	50 fpm (0.83 fps)
Oceanic Diffusion (E_o)	=	0.01 $L^{4/3}$(cgs units)
Effluent coliform conc. (C_e)	=	10^4 MPN/100 ml
Coliform Die-away rate (K_1)	=	1.0 day^{-1}
Coliform Aftergrowth rate (K_2)	=	2.2 day^{-1} (12 hours).

The maximum concentration of coliform organisms at a shellfish harvesting location 10,000 feet away from the diffuser is estimated as follows:

$$C_o = C_e/S_o = \frac{10200}{60} = 170 \text{ MPN/100 ml}$$

$$E_o = .01\ (500 \text{ ft} \times 30.5 \text{ cm/ft})^{4/3} = 3782 \frac{\text{cm}}{\text{sec}} = 4.1 \frac{\text{ft}^2}{\text{sec}}$$

$$t = 10{,}000/50 = 200 \text{ Min} = .14 \text{ days}$$

Therefore, aftergrowth is appropriate for the entire spatial interval.

$$C = (167)e^{-1.0(0.14)}e^{2.2(0.14)} \operatorname{erf}\left[\frac{3/2}{\left(1 + \frac{8(4.1)(10,000)}{0.83(500)^2}\right)^3 - 1}\right]^{1/2}$$

$$C = 197 \operatorname{erf}(0.306)$$

The error function is evaluated using standard error function tables (18) to be 0.893, and:

$$C = 176 \text{ MPN/100 ml}$$

Therefore, the coliform concentration within the shellfish area is expected to be 176 MPN/100 ml under the assumed conditions. Conditions at other current velocities can be similarly computed and compared to water quality standards for sanitary water quality for shellfishing within the study area. The computed concentration, 176 MPN/100 ml at mean current velocities normally indicates a high probability of a problem in a shellfishing area. Local standards should be consulted before evaluating the problem further.

5.5 Lakes and Impoundments

Information presented in this section is taken in large part from reference (19). Lakes and impoundments are characterized by physical features which result in a special class of water quality problems. The relatively long detention times in these systems permit relatively slow reaction processes to proceed to completion; response times to changes in loadings are longer than for river or estuary systems; and transport characteristics in three-dimensions are frequently required to adequately analyse the system. The procedures and methods presented in this section are designed to permit the 208 planner to analyse lake systems in a preliminary assessment framework. The methods focus on major problems and yield fundamental relationships between loads and water quality which must be interpreted prudently. The planner is cautioned to exercise critical judgement in using Section 5.5, and utilize qualified persons and consultants in analysing problem frameworks which are beyond the scope of these analyses.

Lake systems have historically been analysed in numerous frameworks, from simple empirical analysis methods to sophisticated numerical models of transport and kinetic reactions. This section of the manual deals with lake water quality in terms of the simpler methods and relatively straight forward numerical methods which can be performed with an electronic calculator. Stratification in lakes is described in terms of a dimensionless parameter so that first cut assessments of the vertical dimension of water quality problems can be determined. Empirical methods for assessing nutrient loadings are then reviewed to place eutrophication problems in perspective. Methods suggested by Vollenweider and Dillon (20,21) are employed for this purpose. Finally, closed-form solutions to equations for analysing a wide spectrum of special water quality problem types are presented and demonstrated. Where appropriate, assumptions and limitations are clearly spelled out.

5.5.1 Stratification in Lakes and Impoundments

A method for classifying lakes as deep stratified systems, weakly stratified, or vertically mixed is fundamental to defining the vertical dimension of water quality problems in lakes. The classification system is based on the ratio of the inflow volume to the storage volume in the impoundment. Three classes are defined as follows:

a. Deep stratified lake (low flow to volume ratio). Reservoirs in this class are large and have detention times greater than one year. They are characterized by periods of strong vertical stratification.

b. Weakly stratified lakes (medium flow to volume ratio). Large lakes with detention times in the order of four months to a year are normally in this class.

c. Vertically mixed (large flow to volume ratio). This class generally includes run-of-the-river impoundments with detention times less than four months. Vertical temperature gradients are not present; however, longitudinal temperature variations may exist.

Determination of the impoundment classification is made by computing its densimetric Froude number (F_D). This number is the ratio of the inertial force of the horizontal flow to the gravitational forces within the impoundment. Consequently, it is a measure of the ability of the horizontal flow to alter the internal density structure of the reservoir from its static equilibrium condition. The densimetric Froude number is given by:

$$F_D = \frac{LQ}{DV} \quad \frac{\rho}{ge} \tag{5-16}$$

where:

F_D = densimetric Frounde number

L = reservoir length (ft)

D = mean reservoir depth (ft)

Q = flow through the reservoir (cfs)

V = reservoir volume (ft^3)

g = gravitational constant (ft/sec^2)

e = average normalized vertical density gradient (5.25×10^{-2} lbs/ft^3-ft)

ρ = reference density.

Substituting values for the constants g, ρ, and e, the equation simplifies to:

$$F_D = 320 \frac{L}{D} \frac{Q}{V} \tag{5-17}$$

The use of this equation is demonstrated in Table 5-19. In a deep lake where the isotherm is horizontal, the inertia of the longitudinal flow will be insufficient to disturb the overall gravitational (or density) static equilibrium state of the lake. Local disturbances may occur at points near the tributary inflow and the lake outlet. Thus, the F_D for such an impoundment is small. For the vertically mixed reservoir, the inertia of the inflow is sufficient to completely upset the density structure, leaving the impoundment non-stratified. F_D for reservoirs of this class is large. Between these two extremes lies the "gray area" which includes the weakly-stratified reservoirs. Here the longitudinal inflow has enough inertia to disrupt the gravitational static equilibrium configuration but is not sufficient to completely mix the reservoir.

Theoretical and experimental work in stratified flow indicates that flow separation or stratification occurs in systems having F_D values in the order of $1/\pi$ or 0.32. For F_D values less than 0.32, the water flow will be longitudinal or stratified, while for F_D values greater than 0.32, the water column is more mixed in the vertical direction. A general rule to follow is:

F_D values $>> 0.32$ - completely mixed lake

F_D values $<< 0.32$ - deep, stratified lake

F_D values $\simeq 0.32$ - weakly stratified lake.

Table 5-19 presents the Froude number for a selected group of reservoirs and demonstrates the classification system presented above. The value of the specific Froude number which would distinguish the transition from one classification to another has not yet been clearly determined.

5.5.2 Eutrophication Assessment Methods

Assessments of the eutrophication potential in an impoundment is of concern because of the long detention times of these systems. Eutrophication is very complex and not easily adapted to simplified analysis techniques. Various approaches have been used to empirically evaluate the conditions in impoundments. The most widely used one is discussed here. Several other techniques are also available. The applicability of these techniques to a site specific study area must be determined on an individual impoundment basis. The manual user selecting such a method should become familiar with the various assumptions involved in its development and determine if these assumptions are valid for the impoundment under study.

TABLE 5-19

CALCULATION OF FROUDE NUMBER F_D FOR LAKE CLASSIFICATION

Reservoir	Length (feet)	Average Depth (feet)	Discharge to Volume Ratio (sec^{-1})	Froude Number F_D	Class
Lake Roosevelt (Wash)	2.0×10^5	70	5.0×10^{-7}	0.46	Weakly Stratified
Priest Rapids (Wash)	2.9×10^4	18	4.6×10^{-6}	2.4	Vertically Mixed
Wells (Wash)	4.6×10^4	26	6.7×10^{-6}	3.8	Vertically Mixed
Detroit (Wash)	1.5×10^4	56	3.5×10^{-8}	0.003	Deep
Hungry Horse (Wash)	4.7×10^4	70	1.2×10^{-8}	0.0026	Deep
Lake LBJ (Tx)	34×10^3	7	3.3×10^{-7}	0.51	Weakly Stratified
Lake Livingston (Tx)	3.8×10^4	6.5	1.0×10^{-7}	0.51	Weakly Stratified

One of the earliest efforts to relate external nutrient loadings to eutrophication was accomplished by Vollenweider (20). Vollenweider plotted the phosphorus loading to a number of lakes as a function of the mean lake depth to provide a basis for classifying the eutrophic status of the water body. Dillon (21) has expanded this approach to include the consideration of the effect of hydraulic detention time on nutrient loadings and nutrient retention. Consideration of the hydraulic detention time provides an improvement over the Vollenweider approach. This improvement is realized because two impoundments with the same mean depth may have considerably different detention times. For example, one might be a run-of-the-river type impoundment while the other is a storage reservoir. The eutrophic potential of the run-of-the-river impoundment will be less due to the higher washout rate of nutrients.

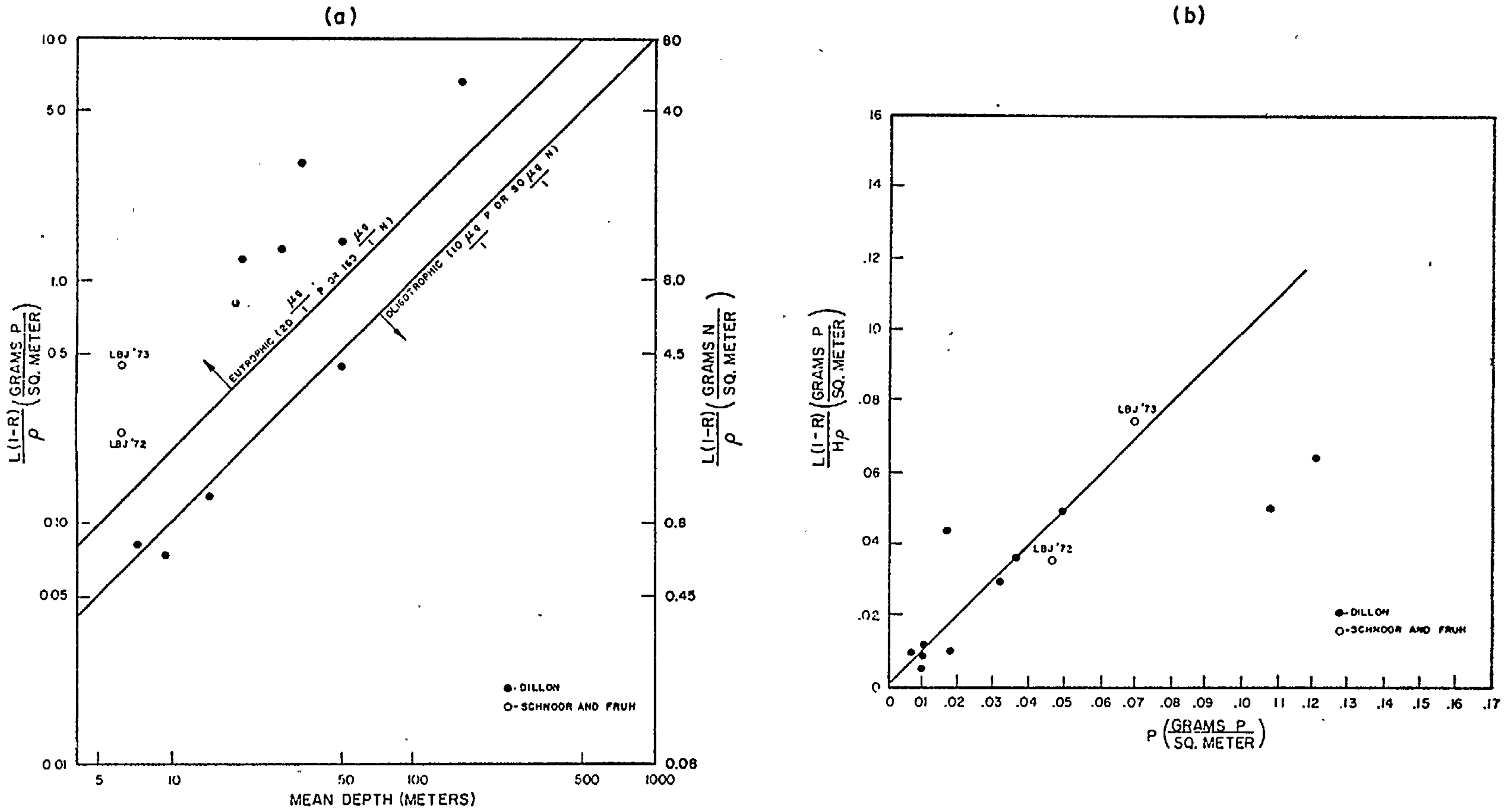
The empirical analysis developed by Vollenweider and expanded upon by Dillon for impoundment trophic classification assumes steady state and completely mixed conditions. The equation used to develop the model relates to the hydraulic flushing time, the phosphorus loading, the phosphorus retention ratio, the mean depth, and the phosphorus. concentration of the impoundment as follows:

$$\frac{L(1 - R)}{\rho} = HP \qquad (5\text{-}18)$$

where:

L = phosphorus loading ($lbs/ft^2/yr$)
R = fraction of phosphorus retained
ρ = hydraulic flushing rate (per year)
H = mean height (ft)
P = phosphorus concentration (lbs/ft^3).

The graphical solution is presented as a log-log plot of $\frac{L\,(1-R)}{\rho}$ versus H. Figure 5-24(a) is a reproduction of Dillon's work on several lakes in Canada. Lakes or impoundments which fall above the 20 μg/1 concentration line tend to be eutrophic while those below the 10 μg/1 concentration line tend to be oligtrophic.



GRAPHICAL SOLUTION TO THE DILLON APPROACH

TEST OF IMPOUNDMENT
STEADY STATE AND COMPLETE MIXING

FIGURE 5-24

RELATIONSHIPS FOR EVALUATING THE STATE OF A LAKE

In order to check the assumptions of steady state and complete mixing made in the Dillon approach, a plot of $\frac{L(L-R)}{H\rho}$ versus the P concentration should be developed. This plot, shown in Figure 5-24(b), is for various impoundments examined by Dillon. If the lake under study plots near the line shown on Figure 5-24(b) it appears that the assumptions associated with the Dillon techniques are valid. This indicates that the phosphorus levels in the impoundment are in equilibrium with the phosphorus inputs to the system. Satisfaction of this criteria would support the application of the Dillon technique to estimate the eutrophic status of the impoundment.

The application of the Dillon equation requires information on the physical characteristics of the impoundment and observed or calculated phosphorus loading data. The fraction of the phosphorus retained by an impoundment can be determined either empirically or theoretically. The empirical approach involves a mass balance calculation of the total inflow phosphorus, the total outflow phosphorus and the amount that remains in the impoundment. This difference may be estimated directly by collecting sinking material at the bottom of the impoundment and monitoring the phosphorus in the water column, or indirectly by monitoring the inflow and outflow phosphorus. This empirical approach requires data obtained through a comprehensive monitoring program which may not be feasible within the planning effort.

In general, lakes which appear to be in the eutrophic stage on the basis of the Vollenweider - Dillon analysis are candidates for more comprehensive analysis aimed at determining the principal cause of the nutrient enrichment so that alternative controls strategies can be developed. Chapters 2, 3 and 4 discuss various methods for determining nutrient sources in this case.

In situations where the Vollenweider - Dillon analysis would lead to the classification of the lake as oligotrophic, the analyst should be guided by other data in drawing conclusions regarding the status of the lake. Simplified assessments of impoundment water quality conditions such as this technique provide information having a level of reliability appropriate only for gaining a perspective of water quality conditions.

Results from these procedures should not be utilized for the design and implementation of control strategies.

5.5.3 Analysis Methods for Water Quality Evaluations in Lakes and Impoundments

An assessment of the expected level of pollutant concentrations in impoundments using a number of initial estimating techniques are presented in this section. The applicability and accuracy of these techniques are dependent upon how closely the characteristics of the actual impoundment fit the simplifying assumptions of the approach.

Three basic problem cases will be addressed. The first framework is applicable to run-of-river impoundments having small detention times. The system is assumed to be at steady state with one-dimensional plug flow and no vertical stratification. The second approach is applicable to large reservoirs and lakes wherein the impoundment is assumed to be completely mixed in the horizontal plane. Calculations for both vertically mixed and vertically stratified systems will be presented. The third set of estimates addresses pollutant concentrations in a two-dimensional, localized area. These are for impoundment areas where there is insignificant advective transport due to flow-through, wind currents, or density gradients, and where there may be significant concentration gradients around the point of discharge. Water quality estimates for a cove area adjacent to an impoundment will also be presented. These latter methods may also be used in analyzing estuaries and tidal rivers discussed in Section 5.4.2 of this manual.

For each system, analyses will be presented for conservative substances such as suspended solids, reactive substances such as coliforms and BOD, and coupled reactions such as BOD-dissolved oxygen deficit.

5.5.3.1 One-Dimensional Plug Flow Systems

With run-of-the-river type impoundments, the same analysis that is used to examine steady state water quality in free flowing streams and rivers may be applied to the impounded portion of the river. The basic reactions and equations are the same; the major difference is the decreased velocity and the correspondingly increased detention time in the impoundment.

The water is assumed to enter the impoundment with a flow Q, and conservative, reactive or coupled pollutant concentrations, C_o, L_o, and D_o, respectively. A constant loading rate, W, enters the impoundment at the upstream end and a distributed (nonpoint) load, ω, may enter throughout the length of the impoundment. For the analysis of the upstream point source, a constant or an expanding cross-sectional area may be used to characterize the geometry of the impoundment, whereas the analysis of the distributed load is limited to a constant cross-sectional area.

The solutions for pollutant concentrations as a function of distance, X, in the impoundment are shown in Table 5-20. The reaction rate of the first system is K_r, the reaction rate of the second system is K_a, and the rate describing the interaction between the two systems is K_d. For a BOD-DO deficit system where all of the BOD is removed by oxidation (rather than settling), $K_d = K_r$. Typical reaction rates are shown in Table 2-19.

For the BOD-DO deficit system, K_a is the reaeration rate of dissolved oxygen in the river. It is best estimated in impoundments by using equation (5-6). Temperature correction of these values is accomplished using methods described in Section 2.6.3.4.

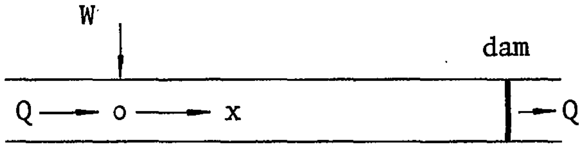
For the BOD-DO analysis, the magnitude of the ultimate oxygen demand (W_1) is entered in the BOD system and, as appropriate, the deficit loading is included in the DO deficit system (W_2). The ultimate oxygen demand is comprised of both carbonaceous and nitrogenous components which may be estimated to be 1.5 x W(BOD_5) and 4.57 x W(NH_3-N).

For the point source solutions of Table 5-20, f(x) will depend upon the distance-area relationship assumed for the impoundment. The relationship between cross-sectional area and distance downstream may have many functional forms. The most common for free-flowing streams is the constant cross-section, particularly when relatively short distances and small drainage areas are considered. A constant area is designated as

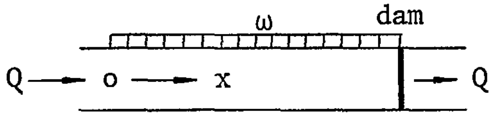
TABLE 5-20

SUMMARY OF STEADY STATE SOLUTIONS FOR POLLUTANT CONCENTRATIONS IN A FLOW-THROUGH IMPOUNDMENT

POINT SOURCE



DISTRIBUTED SOURCE



	POINT SOURCE	DISTRIBUTED SOURCE
Conservative	$c = c_o + W/Q$	$c = c_o + \omega x/Q$
Reactive	$L = L_o e^{-K_r f(x)/U_o} + W/Q\ e^{-K_r f(x)/U_o}$	$L = L_o e^{-K_r x/U} + \frac{\omega}{A.K_r} (1 - e^{-K_r x/U})$
Coupled	$D = D_o e^{-K_a f(x)/U_o}$ $+ L_o \cdot \frac{K_d}{K_a - K_r} (e^{-K_r f(x)/U_o} - e^{-K_a f(x)/U_o})$ $+ W/Q \cdot \frac{K_d}{K_a - K_r} (e^{-K_r f(x)/U_o} - e^{-K_a f(x)/U_o})$	$D = D_o e^{-K_a x/U}$ $+ L_o \cdot \frac{K_d}{K_a - K_r} \cdot (e^{-K_r x/U} - e^{-K_a x/U})$ $+ \frac{\omega}{AK_r} \cdot \frac{K_d}{K_a - K_r} (\frac{K_r}{K_a} e^{-K_a x/U} - e^{-K_r x/U} + \frac{K_a - K_r}{K_a})$

A_o. For greater distances, larger drainage areas, or impounded rivers, the exponential and linear forms may be represented:

$$A(x) = \frac{A_o}{X_o} x \qquad (5\text{-}19)$$

$$A(x) = A_o e^{ax} \qquad (5\text{-}20)$$

In these equations, A is the cross-sectional area at distance x, which is measured from the origin of the coordinate system. In equation (5-19) x_o is the distance from a hypothetical origin to the location of A_o. In equation (5-20) A_o is located at $x = 0$. In both cases, the location of A_o may be arbitrarily assigned. Examples of constant, linearly and exponentially increasing representations of the area are shown in Figure 5-25. The appropriate expression for f(x) for use in Table 5-20 will depend upon the area-distance relationship assumed and is shown below.

Expression for f(x)

Area-Distance Relationship

	Constant	Linear Increase	Exponential Increase
$f(x) =$	x	$\frac{x^2 - x_o^2}{2x_o}$	$\frac{e^{ax} - 1}{a}$

where: a is an empirically developed constant.

Note that U_o, the velocity at the beginning of the impoundment, is used in the solutions presented in Table 5-20 and that for the linearly expanding case, $x = 0$ at the location which is a distance x_o before the beginning of the impoundment. To apply the solution equations for a distributed source, a constant cross-sectional area must be assumed. If the area varies considerably, the impoundment may be divided into several constant-area segments and the analysis performed within each segment.

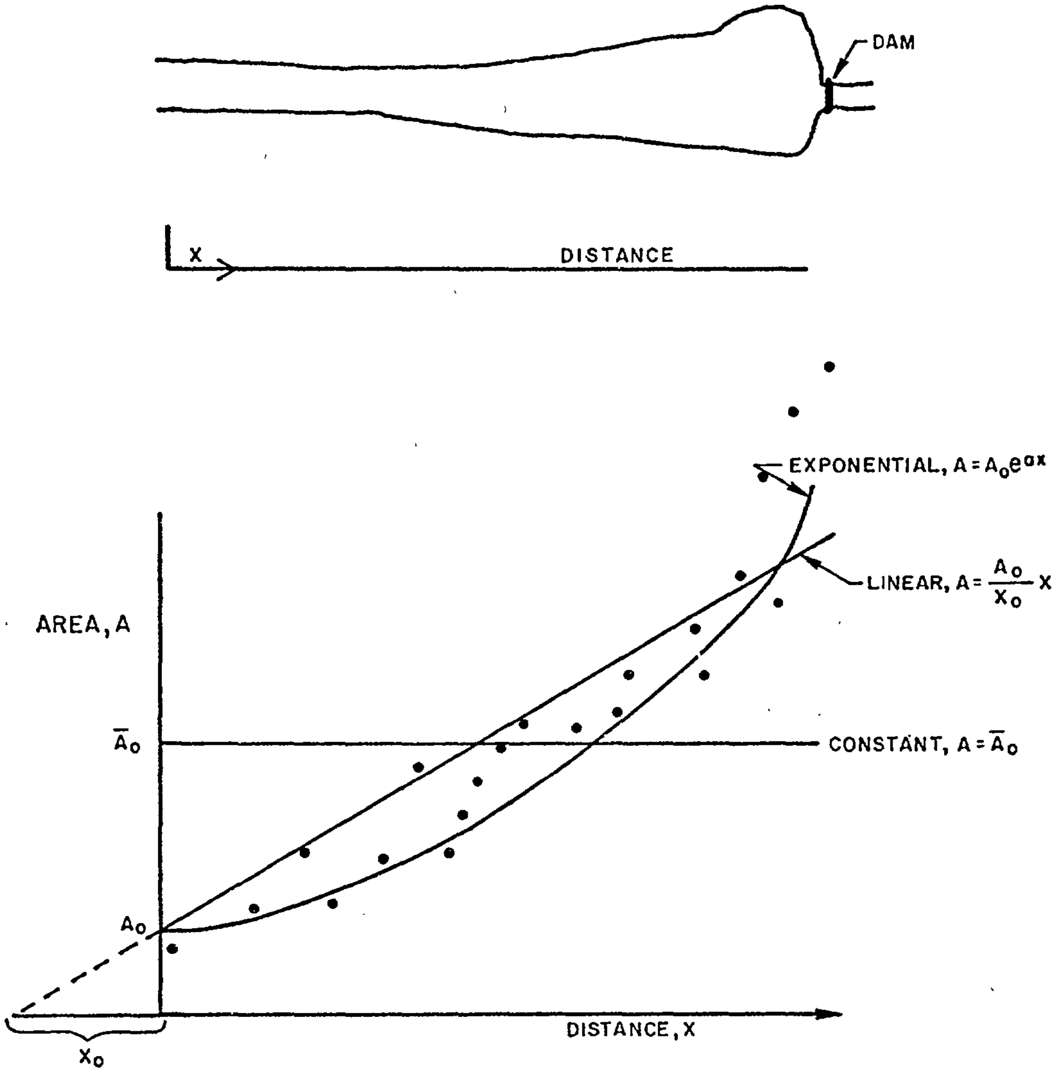


FIGURE 5-25
FUNCTIONAL AREA REPRESENTATIONS

5.5.3.1.1 Dissolved Oxygen Deficit Determination

The solutions for dissolved oxygen deficit obtained from Table 5-20 include only the effects of BOD loads and upstream BOD and DO deficit concentrations. They do not include the effects of the oxygen demand imposed on an impoundment by benthic material and algal respiration, and the input of dissolved oxygen from algal photosynthesis.

The components of the benthic material which exert an oxygen demand include inflow organic material, settled biomass, and chemical constituents which undergo oxidation-reduction at the sediment-water interface. Except for impoundment areas that receive direct discharges of settleable wastes, the benthal oxygen rate, S_B, may be estimated to be on the order of 0.3 to 3.0 gm O_2/m^2/day. Site specific data is normally required to refine the estimate of S_B. The monitoring appendix provides guidance in developing this type of data.

When algae are present in streams, there is a diurnal variation of dissolved oxygen due to varying photosynthetic oxygen production during the day. The primary factor that governs the photosynthetic oxygen production of algae is the quantity of solar radiation the algae receive. The average relative oxygen production may be estimated as follows:

$$\frac{P_{av}}{P_s} = \frac{2.718\ f}{K_e H\ T_p}\left(e^{-\alpha I_a/I_s} - e^{-I_a/I_s}\right) \qquad (5\text{-}21)$$

where:

$\alpha = e^{-K_e H}$

P_s is the light saturated rate of oxygen production

I_a is the average light intensity during the daylight portion of the day

I_s is the light saturated intensity

f is the number of hours of daylight

T_p equals twenty-four hours

K_e is the extinction coefficient

H is the river depth.

P_{av} in equation (5-21) is the average algal oxygen production over the entire day. The extinction coefficient ranges from 0.1 - 0.5 m^{-1} for very clear impoundments, from 0.5 to 2.5 m^{-1} for moderately turbid waters, and greater than 2.5 m^{-1} for very turbid waters.

The ability to calculate the absolute oxygen production rate depends upon the estimation of P_s, the light saturated rate of oxygen production. A correlation between P_s and the concentration of chlorophyll "a" taken as a measure of the algae population density may be used to estimate P_s:

$$P_s = 0.25\ Chl_a \tag{5-22}$$

where:

P_s = mg oxygen produced/1/day

Chl_a = Chlorophyll "a" concentration in µg/1.

Thus, by measurement of the chlorophyll "a" concentration, the incident solar radiation I_a, the length of daylight f, the extinction coefficient k_e and the depth H, the average daily rate of photosynthetic dissolved oxygen production may be estimated.

Although algae produce oxygen by photosynthesis, they also utilize oxygen for respiration. The respiration rate, R, has also been related to chlorophyll "a" concentration:

$$R = r(Chl_a) \tag{5-23}$$

where:

R = mg oxygen utilized/1/day

Chl_a = chlorophyll "a" concentration in µg/1

r = constant ranging from 0.005 to 0.030 with 0.025 a common value (22) corresponding to a 10 to 1 ratio of P_s to R.

The respiration rate is known to vary considerably and depends on the nutrient concentration and age of the culture. Hence the average daily algae respiration calculated from equation (5-27) is subject to some degree of uncertainty.

The estimates of the benthal oxygen demand, algal photosynthesis and respiration may be incorporated into the estimate of the average DO deficit by the following equation:

$$D = \frac{1}{K_a} (\frac{S_B}{H} - P_{av} + R) (1 - e^{-K_a f(x)/U_o}) \qquad (5\text{-}24)$$

This indicates the average in the impoundment, but not the diurnal variation around the average. The diurnal range of dissolved oxygen, Δ_o (mg/1), due to algal effects may be estimated (23) as:

$$\Delta_o = P_{av} (1 - \frac{f}{T_p}) \qquad (5\text{-}25)$$

Equation(5-25) requires that K_a be less than about 0.2 day^{-1} to be applicable. This should be true in most impoundments due to their depth and low velocities.

5.5.3.1.2 Location of Maximum DO Deficit Due to Point Source

Guidelines have been presented for estimating the steady state, spatial distribution of dissolved oxygen deficit in a run-of-the-river impoundment. The DO deficit due to the various sources may be added together and plotted as a function of distance, x, along the impoundment to determine the location and magnitude of the maximum DO deficit. A simpler approach is available for estimating the maximum DO deficit response due to a single point source. The maximum deficit, D_c, is:

$$D_c = (\frac{W}{Q}) \frac{K_d}{K_a} (e^{-\frac{K_r \ln(K_a/K_r)}{(K_a - K_r)}}) \qquad (5\text{-}26)$$

The determination of the location of the maximum DO deficit, x_c, depends upon the distance-cross-sectional area relationship assumed, and may be estimated for each as:

$$x_c = \frac{U_o}{K_r - K_a} \ln(\frac{K_r}{K_a}), \text{ for constant area} \qquad (5\text{-}27)$$

$$x_c = \sqrt{x_o^2 + \frac{2U_o x_o}{(K_r - K_a)} \ln(\frac{K_r}{K_a})}, \text{ for linearly expanding area} \tag{5-28}$$

$$x_c = \frac{\ln(1 + \frac{aU_o}{(K_r - K_a)} \ln(\frac{K_r}{K_a}))}{a}, \text{ for exponentially expanding area} \tag{5-29}$$

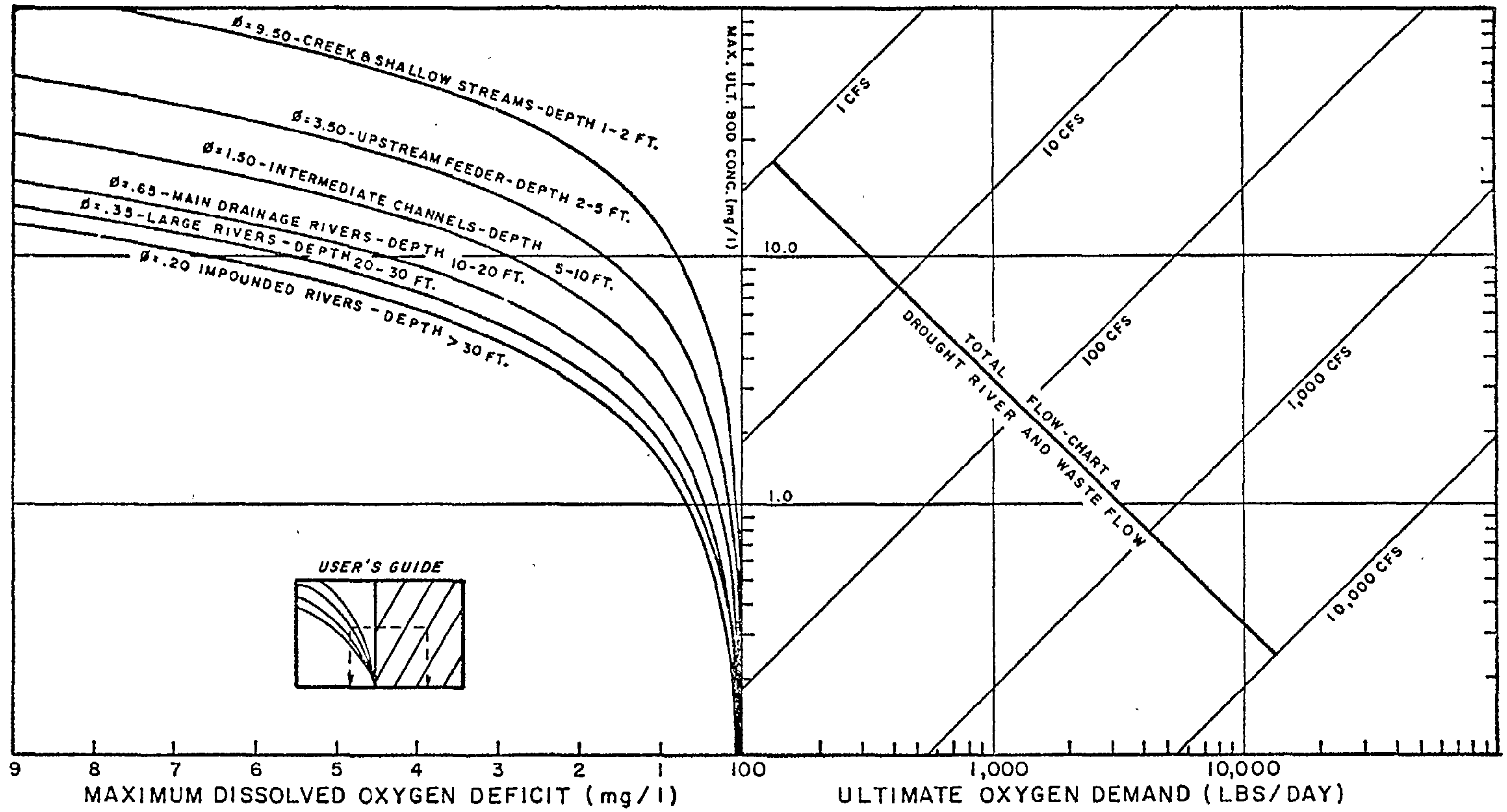
A graphical estimate of D_c due to the point source, W, may also be made using Figure 5-26 (8). The ultimate oxygen demand load is input on the upper right axis and the solution is determined by moving counterclockwise from the flow, to the maximum BOD concentration, to ϕ, where $\phi = \frac{K_d}{K_a}$, (this analysis assumes $K_r = K_d$), to the maximum DO deficit. The location of the maximum deficit should still be calculated, however, to insure the maximum deficit occurs within the impoundment.

5.5.3.1.3 Determination of DO Concentration

Once the dissolved oxygen deficit has been determined, the DO concentration may be calculated. The dissolved oxygen saturation concentration, C_s, may be determined in streams, impoundments and marine systems from Figure 2-19(b) (pg. 2-92). Chloride concentrations may generally be assumed to be near zero in lakes and impoundments. The DO concentration, C, then equals C_s-D. Applications of the foregoing estimating technique are shown in Table 5-21.

5.5.3.2 Completely Mixed in the Horizontal Dimensions

The second set of impoundment systems analyzed are the large reservoirs and lakes having long detention times. These impoundments are assumed to be completely mixed in the horizontal directions. Estimating procedures are presented for vertically mixed and vertically stratified systems. The impact of external pollutant loads can only be estimated for conservative or slowly reacting substances, as more rapidly reacting constitutents will not have a chance to be mixed throughout the impoundment. Estimates for dissolved oxygen concentrations are presented for assumed uniform rates of benthal demand, reaeration, algal photosynthesis, and respiration throughout the lake. The algal effects



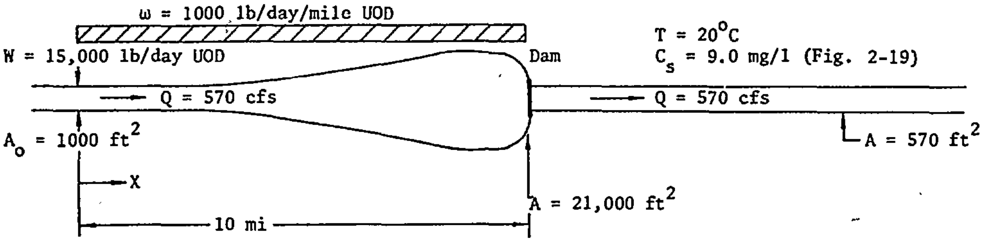
SOURCE:(8)

FIGURE 5-26

SIMPLIFIED ASSESSMENT OF DISSOLVED OXYGEN DEFICIT IN IMPOUNDMENTS AND RIVERS

TABLE 5-21

EXAMPLE OF DO DEFICIT PROBLEM: RUN-OF-THE-RIVER IMPOUNDMENT AND DOWNSTREAM EFFECTS



Impoundment

Average H = 12 ft = 3.6 m
Average A = 5700 ft^2
Average U = 0.1 ft/sec = 1.64 mi/day
$K_r = K_d = 0.25$ day^{-1}
$K_a = 0.16$ day^{-1}
P_{av} = 1.3 mg 0_2/1/day
f = 14 hr
R = 0.4 mg 0_2/1/day
S_B = 2 g 0_2/m^2/day
Assumed Upstream UOD and DO Deficit = 0

Downstream

Height Water Falls over Dam = H_d = 10 ft
Average H = 7 ft = 2.1 m
Average A = 570 ft^2
Average U = 1 ft/sec = 16.4 mi/day
$K_r = K_d = 0.25$ day^{-1}
$K_a = 0.75$ day^{-1}
$P_{av} = R = S_B = 0$

Check Location of Maximum Deficit in Impoundment due to Point Source.

1) Assume Constant Area = 5700 ft^2 (Eq. 5-27)

$$X_c = \frac{1.64}{(0.25-0.16)} \ln \left(\frac{0.25}{0.16}\right) = 8.1 \text{ mi } (<10 \text{ mi, OK})$$

2) Assumed Exponentially Expanding Area from A_o = 1000 ft^2 to A = 21,000 ft^2.
Therefore, a = 0.30 (Fig. 5-25), $U_o = \frac{570}{1000}$ = 0.57 ft/sec = 9.35 mi/day.

$$X_c = \ln \; 1 + \ln \left(\frac{0.25}{0.16}\right) x \frac{0.30 \text{ x } 9.35}{(0.25 - 0.16)} \div 0.30 = 9.0 \text{ mi } (10 \text{ mi, OK}) \quad \text{(Eq. 5-29)}$$

Maximum Deficit In Impoundment From Point Source (Eq. 5-26)

$$D_c = \frac{15,000 \text{ (lb/day)}}{570\text{(cfs) x } 5.4\left(\frac{\text{lb/day}}{\text{cfs . mg/l}}\right)} \text{ x } \frac{0.25}{0.16} \left(e^{\frac{-0.25}{(0.16-0.25)} \ln \left(\frac{0.16}{0.25}\right)}\right) = 2.20 \text{ mg/l}$$

TABLE 5-21
(Continued)

EXAMPLE OF DO DEFICIT PROBLEM: RUN-OF-THE-RIVER IMPOUNDMENT AND DOWNSTREAM EFFECTS

<table>
<tr><th>X_c = 8.1 mi
(At location of maximum point source deficit)</th><th>X = 10 mi
(In impoundment, at dam)</th></tr>
<tr><td colspan="2">a) Deficit due to Point Source</td></tr>
<tr><td>See preceding sheet
$D = D_c = 2.20$ mg/l</td><td>See Table 5-20
$D = \frac{15,000}{570 \times 5.4} \times \frac{0.25}{(0.16 - 0.25)} (e^{\frac{-0.25 \times 10}{1.64}} - e^{\frac{-0.16 \times 10}{1.64}})$
= 2.16 mg/l</td></tr>
<tr><td colspan="2">b) Deficit due to Distributed Source (Table 5-20)</td></tr>
<tr><td>$D = \frac{1000 \times 3.0 (\frac{mg/l}{lb/ft^3\ mi})}{5700 \times 0.25} \times \frac{0.25}{(0.16 - 0.25)}$
$\times \frac{0.25}{0.16} e^{\frac{-0.16 \times 8.1}{1.64}} - e^{\frac{-0.25 \times 8.1}{1.64}}$
$+ \frac{(0.16 - 0.25)}{0.16} = 0.8$ mg/l</td><td>$D = \frac{1000 \times 3.0}{5700 \times 0.25} \times \frac{0.25}{(0.16 - 0.25)}$
$\times \frac{0.25}{0.16} e^{\frac{-0.16 \times 10}{1.64}} - e^{\frac{-0.25 \times 10}{1.64}} + \frac{0.16 - 0.25}{0.16}$
= 1.1 mg/l</td></tr>
<tr><td colspan="2">c) Deficit due to Benthal Demand (Eq. 5-24)</td></tr>
<tr><td>$D = \frac{1}{0.16} (\frac{2}{3.6}) (1 - e^{\frac{-0.16 \times 8.1}{1.64}})$
= 1.9 mg/l</td><td>$D = \frac{1}{0.16} (\frac{2}{3.6}) (1 - e^{\frac{-0.16 \times 10}{1.64}}) = 2.2$ mg/l</td></tr>
<tr><td colspan="2">d) Deficit due to Algae (Eq. 5-24)</td></tr>
<tr><td>$D = \frac{1}{0.16} (-1.3 + 0.4) (1 - e^{\frac{-0.16 \times 8.1}{1.64}})$
= -3.1 mg/l</td><td>$D = \frac{1}{0.16} (-1.3 + 0.4) (1 - e^{\frac{-0.16 \times 10}{1.64}})$
= -3.5 mg/l</td></tr>
<tr><td colspan="2">e) Dissolved Oxygen Concentrations without Algal Effects</td></tr>
<tr><td>$DO = C_s - \Sigma D = 9.0 - (2.2 + 0.8 + 1.9)$
= 4.1 mg O_2/l</td><td>DO = 9.0 - (2.2 + 1.1 + 2.2)
= 3.5 mg O_2/l</td></tr>
<tr><td colspan="2">f) Dissolved Oxygen Concentrations with Algae</td></tr>
<tr><td colspan="2">Range of diurnal variation = $\Delta_o = P_{av} (1 - f/24) = 1.3 (1 - 14/24) = 0.54$ mg/l (Eq. 5-25)</td></tr>
<tr><td colspan="2">f.1) Daily Average Concentration</td></tr>
<tr><td>$DO_{av} = 9.0 - (2.2 + 0.8 + 1.9 - 3.1)$
= 7.2 mg O_2/l</td><td>$DO_{av} = 9.0 - (2.2 + 1.1 + 2.2 - 3.5)$
= 7.0 mg O_2/l</td></tr>
<tr><td colspan="2">f.2) Minimum Daily Concentration</td></tr>
<tr><td>$DO_{MIN} = DO_{av} - \Delta_o/2 = 7.2 - 0.54/2$
= 6.9 mg O_2/l</td><td>$DO_{MIN} = 7.0 - 0.54/2$
= 6.7 mg O_2/l</td></tr>
</table>

may be qualitatively related to external nutrient loads using the Vollenweider and Dillon techniques previously presented.

The first case analyzed is the completely mixed (horizontally and vertically) lake with a conservative or slowly reacting (in relation to the detention time) substance. Examples of slowly reacting substances which may be relevant for this type of analysis include pesticides and radioactive materials. The equations for the lake concentration are shown in Table 5-22. A constant pollutant load, W_o is assumed to enter the lake. The lake has a volume, V, an outflow, Q, initial conservative and reactive pollutant concentrations, C_o and L_o, and for the reactive substance, a reaction rate K_r. If the inflow is estimated from the upstream drainage area, the net evaporation should be subtracted to determine Q. The impoundment does not reach a steady state rapidly if the detention time is long, and the time variable equations of Table 5-22 may be used to estimate the buildup over time. Also shown is the lake response to a load increasing or decreasing linearly with time. Since this loading condition never reaches a constant value, the receiving water concentrations never reach a steady state concentration. Application of these equations are shown in Table 5-23.

5.5.3.2.1 Vertical Stratification

Stratification occurs when the flow through an impoundment is insufficient to overturn the temperature and density gradients discussed in Section 5.5.1, due to sources and sinks of heat, such as solar radiation, evaporation and conduction. The temperature patterns in a lake follow a seasonal pattern, and prediction of the temperature profile requires analysis techniques beyond the scope of this manual.

In impoundments with long detention times, concentrations of solids-associated pollutants vary with depth due to settling. This process may be characterized for conservative substances by the following equation(24):

$$C(Z) = C_o e^{ZV_s/E_V} \qquad (5\text{-}30)$$

TABLE 5-22

CONCENTRATIONS IN LARGE, COMPLETELY MIXED IMPOUNDMENT

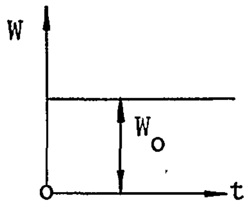
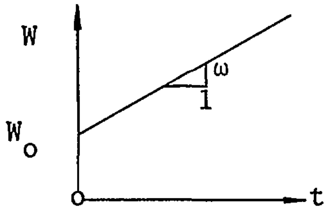
	Constant Load	Linearly Increasing or Decreasing Load
	 $W = W_o$	 $W = W_o + \omega t$
Conservative - Concentration vs. Time	$C_o e^{-\frac{Q}{V}t} + \frac{W_o}{Q}(1 - e^{-\frac{Q}{V}t})$	$C_o e^{-\frac{Q}{V}t} + \frac{W_o}{Q}(1 - e^{-\frac{Q}{V}t}) \pm \frac{\omega V}{Q^2}(\frac{Q}{V}t + e^{-\frac{Q}{V}t}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

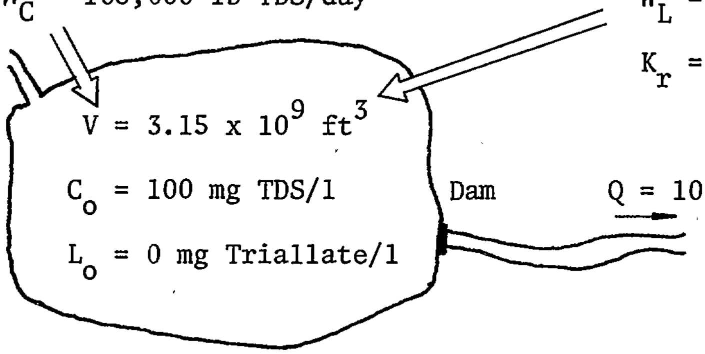
TABLE 5-23

EXAMPLE, COMPLETELY MIXED IMPOUNDMENT
(See Table 5-22)

Conservative, TDS
W_C = 108,000 lb TDS/day

Slow Decaying Pesticide, Triallate
W_L = 1080 lb Triallate/day

$K_r = 0.23$ year^{-1} $= 6.3 \times 10^{-4}$ day^{-1}



Assume constant loads beginning at time t = 0

1) Equilibrium TDS Concentration, $C_e = \frac{W_c}{Q}$

$$C_e = \frac{108,000}{100 \times 5.4} = 200 \text{ mg/1}$$

2) Buildup over time of TDS

$$Q/V = \frac{1}{\text{detention time}} = 1 \text{ year}^{-1}$$

$$C = C_o e^{-Qt/V} + \frac{W_c}{Q} (1 - e^{-Qt/V}) = 100\ e^{-t} + 200\ (1-e^{-t})$$

t (years)	C (mg TDS/1)	t (years)	C (mg TDS/1)
0	100	2.0	187
0.5	139	3.0	195
1.0	163		

3) Equilibrium Triallate Concentration, $L_e = \frac{W_L}{\alpha V}$

$$\alpha = (1 + 0.23) = 1.23 \text{ year}^{-1}$$

$$L_e = 1080/(1.23 \times 3.17 \times 10^{-8} \text{ year/sec} \times 3.15 \times 10^9 \times 5.4)$$

= 1.6 mg Triallate/1

where C(Z) is the concentration at a depth Z, C_o is the concentration at the surface, V_s is the settling rate, and E_V is the vertical diffusion coefficient. Settling rates for suspended solids range from 0.1 to 30 m/day depending on the density and particle size (25) while E_V will range from 0.1 to 10 m^2/day with 1 m^2/day frequently used (26). Examples of exponentially increasing concentrations with depth are shown for Lake Livingston, Texas in Figure 5-27.

Large impoundments also tend to be vertically stratified with respect to dissolved oxygen concentrations, because the lake surface is a source of oxygen from reaeration, the bottom is a sink of oxygen due to benthal demand, the production of oxygen from algal photosynthesis varies with depth due to decreasing light penetration with depth, and there is limited vertical exchange of oxygen in the water column. If the lake is considered to have one zone (that is, a constant E_V), the dissolved oxygen deficit profile may be estimated by the following (23):

$$D(Z) = \frac{S_B}{K_L} \left(1 + \frac{K_L Z}{E_V}\right) + \frac{RH}{K_L} \left(1 + \frac{K_L Z}{E_V} \left(1 - \frac{Z}{2H}\right)\right) - \frac{P_s f}{K_L K_e T_p} \left[1 - e^{-K_e H} + \frac{K_L}{E_V} \left(\frac{1 - e^{-K_e Z}}{K_e} - Z e^{-K_e H}\right) \right] . \qquad (5\text{-}31)$$

Guidelines for estimating S_B, R, P_s, $\frac{f}{T_p}$, K_e, and E_V have been previously presented, and may also be applied here. Note this simplification assumes $\frac{I_a}{I_s} = 1$, (see equation (5-21); and there is some error when it does not. K_L is the oxygen transfer coefficient ($K_L = HK_a$) and may be estimated as follows for the horizontally mixed impoundments (27):

$$\begin{aligned} K_L &= 0.362\ s_w^{1/2} && \text{for } s_w < 5.5 \text{ m/sec} \\ K_L &= 0.0277\ s_w^{2} && \text{for } s_w \geq 5.5 \text{ m/sec} \end{aligned} \qquad (5\text{-}32)$$

where K_L is measured in m/day and s_w is the wind speed in m/sec. An application of this technique is shown in Table 5-24. Note that this is only recommended for an initial order-of-magnitude estimate, and more sophisticated time variable models are required for a more accurate estimate.

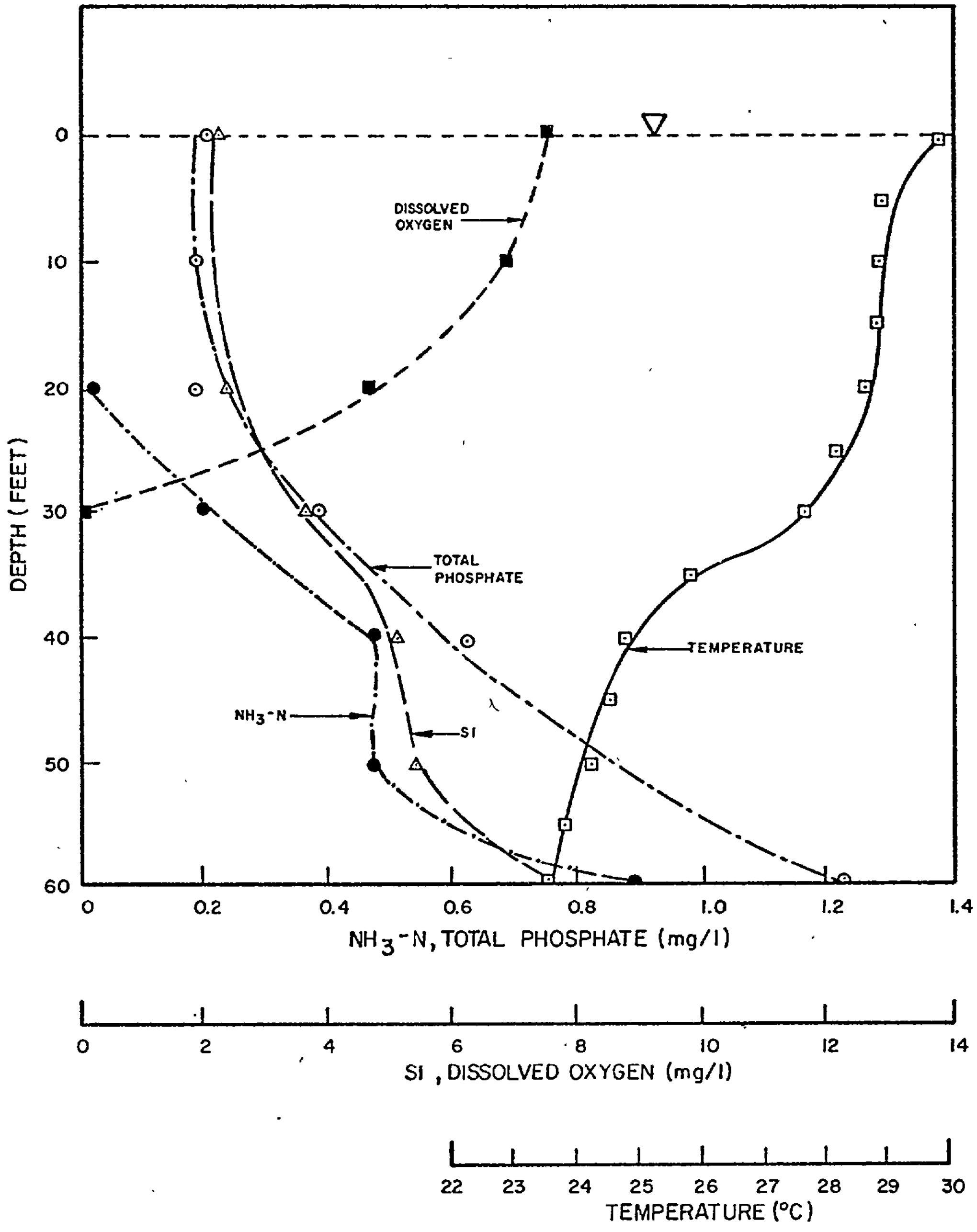
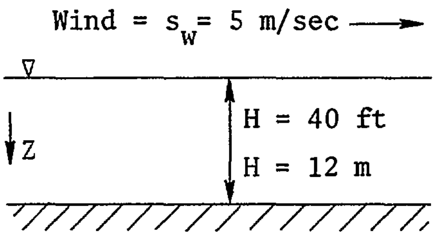


FIGURE 5-27
WATER QUALITY VARIATION WITH DEPTH IN LAKE LIVINGSTON, TEXAS

TABLE 5-24

EXAMPLE HORIZONTALLY MIXED IMPOUNDMENT
DO DEFICIT PROFILE



$f = 14$ hr $S_B = 0.5$ g O_2/m^2/day

$T_p = 24$ hr $E_V = 1.0$ m^2/day

$Chl_a = 20$ µg/l

$K_e = 0.2$ $ft^{-1} = 0.67$ m^{-1}

$T = 20^{\circ}C$ (Assume Uniform), $C_s = 9.0$ mg/l

---

1) Estimate K_L (Eq. 5-32)

$K_L = 0.362\ (5)^{1/2} = 0.81$ m/day

2) Estimate P_s (Eq. 5-22)

$P_s = 0.25\ (20) = 5.0$ mg O_2/l/day

3) Estimate R (Eq. 5-23)

$R = 0.025\ (20) = 0.50$ mg O_2/l/day

---

4) Determine DO Deficit at Different Depths (Eq. 5-31)

$$D(Z) = \frac{0.5}{0.81}\left(1 + \frac{0.81\ Z}{1.0}\right) + \frac{0.50 \times 12}{0.81}\left(1 + \frac{0.81\ Z}{1.0}\left(1 - \frac{Z}{2 \times 12}\right)\right)$$

$$- \frac{5.0 \times 14}{0.81 \times 0.67 \times 24}\left[1 - e^{-0.67 \times 12} + \frac{0.81}{1.0}\left(\frac{1 - e^{-0.67Z}}{0.67} - Ze^{-0.67 \times 12}\right)\right]$$

$$D(Z) = -3.84 + 6.5Z - 0.25Z^2 + 6.5e^{-0.67Z} \quad \text{(Z in meters)}$$

Z (meters)	Def (mg/l)	Z (meters)	Def (mg/l)
0	2.7	3	14.3(anaerobic)
1	5.7	>3	(anaerobic)
2	9.9		

The equation presented for the DO deficit profile assumes one zone in the lake. Often the stratification becomes so severe that two zones are formed when a thermocline divides the upper warmer layer and the lower colder layer of the lake. When this occurs, the lower layer may become anaerobic, resulting in regeneration of inorganic nitrogen and phosphorus from the sediment. The DO profile shown for Lake Livingston, Texas, in Figure 5-27 demonstrates the increasing deficit with depth, leading to anaerobic conditions in the lower portion of the water column. In the lake, the thermocline occurs at a depth of approximately 30 feet.

5.5.3.3 Localized Pollutant Concentrations

The third set of problems analyzed deal with the pollutant concentrations around a localized load to an impoundment. The impoundment is assumed vertically mixed in the vicinity of the waste source and mass is assumed to be transported in the impoundment only by dispersion (insignificant effect from flow-through or currents). The load, W, enters the impoundment over a width, 2a, and a depth, H, the depth of the impoundment in the vicinity of the waste source. Figure 5-28 depicts the problem and the curve (28) for estimating the concentration at the shortline of the impoundment at the midpoint of the load, C(0,0). The load may be due to a tributary, a cove where a treatment plant discharges, or a distributed source such as septic tanks or a marsh area. If the load is assumed to be distributed, that is equivalent to ω(lb/day/mile) in Table 5-20 for the run-of-the-river impoundment, W may be calculated as W = $\omega 2a$. The dispersion coefficient, E, is for dispersion in the horizontal directions. E will range generally from 10^4 to 10^7 m^2/day in lakes, and the lower values are typical of the near-shore region. Once C(0,0) has been determined, the concentration at points along the shore may be estimated from Figure 5-29. The concentration at a given distance perpendicular to the shore (along the x axis) will be less than the concentration at the same distance along the shore (along the y axis). Therefore, Figure 5-29 may also be used for a conservative estimate of pollutant concentration profiles perpendicular to the shoreline.

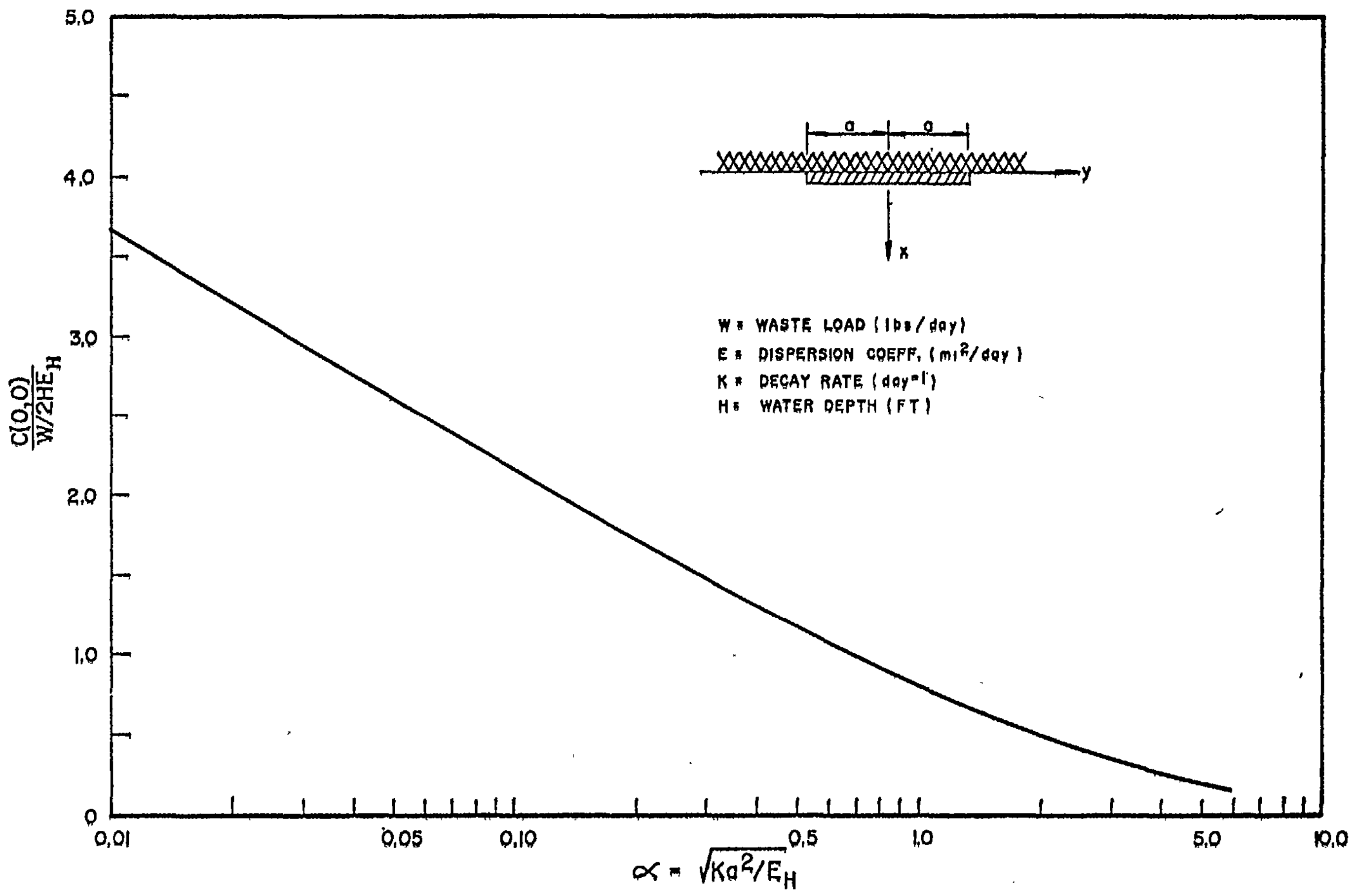


FIGURE 5-28
MAXIMUM ALONGSHORE CONCENTRATION

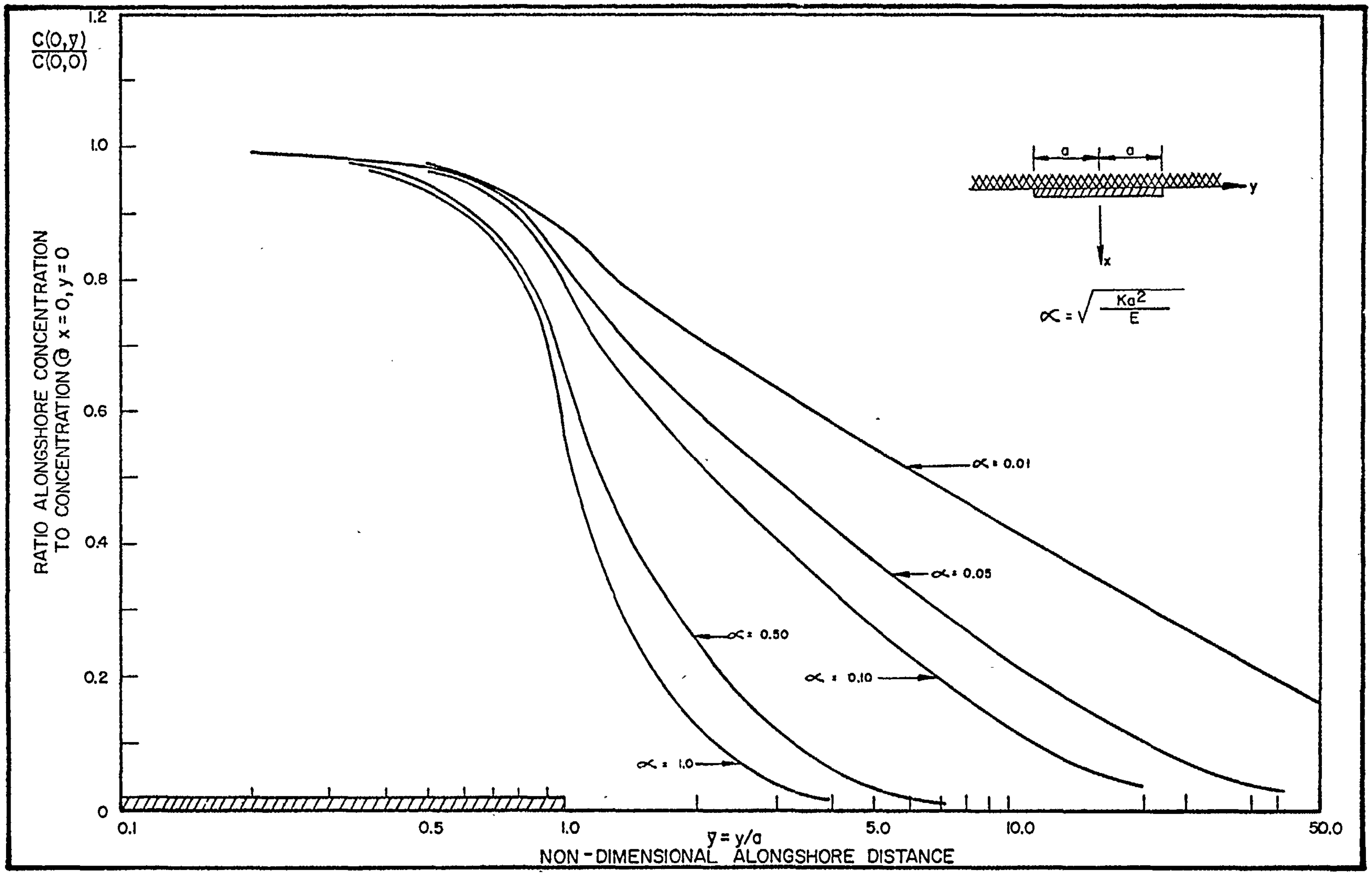


FIGURE 5-29
ALONGSHORE CONCENTRATION ESTIMATE

To determine the concentration of a coupled substance, such as DO deficit, the following equation may be used (29):

$$D = \frac{K_d}{K_a - K_r} (C_1 - C_2) \quad (5\text{-}33)$$

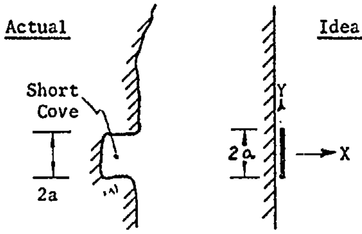
where D is the deficit at the point of interest, C_1 is the concentration of BOD calculated at the point of interest using Figures 5-28 and 5-29, and C_2 is the BOD that would be calculated at the point of interest with $K_r = K_a$. Applications of this technique are shown in Table 5-25.

If a wasteload is discharged into a short cove near the interface of the impoundment, the previous analysis may be used. If the cove is long and the discharge is far from the impoundment interface, however, significant reaction may occur in the cove. In this case, the following procedure should be employed.

The waste load is assumed to enter the water at the upstream end of the cove as shown in Table 5-26. The equations for each case for the reactive and sequentially reactive concentrations in the cove are shown in the respective Tables. Note that, as before, BOD and DO deficit loads may both be analyzed. The pollutant is assumed to be transported within the cove by advection (Q) and dispersion (E). The dispersion coefficient should be of approximately the same magnitude as within the impoundment: $E = 10^4$ to 10^7 m^2/day with the higher values appropriate for coves with significant advective flow. The concentration from the load is assumed to be reduced to nearly zero (5 percent of the maximum cove concentration) at the cove-impoundment interface, and thus the load has a negligible effect on the main portion of the impoundment. For this assumption to be true the cove must be "long", that is, the distance from the load to the impoundment, b, must be greater than $3/|j_1|$ for the reactive analysis (j_1 is defined in Table 5-26) and b must satisfy the requirements indicated in Figure 5-30 for the coupled DO deficit analysis.

The reaction rates may be taken from Table 2-18 and the reaeration may be estimated from the wind speed in equation (5-32) for coves with an advective velocity ($U = \frac{Q}{A}$) less than 0.1 ft/sec, and from quation (5-6)

TABLE 5-25
EXAMPLE OF SHORT COVE ANALYSIS



Short cove assumed well mixed with concentration approximately equal to that calculated at (0,0).

a = 100 ft   Coli: $K = 1/day$

H = 5 ft   BOD-DO: $K_r = K_d = 0.25/day$

$E = 0.1\ mi^2/day.$ Reaer: Wind Speed = 5 m/sec, $K_L = 0.81$ m/day = 2.66 ft/day (Eq. 5-32)

$= 32.3\ ft^2/sec$ $K_a = K_L/H = 2.66/5 = 0.53/day$

Coliform Analysis   Use W = 10 cfs x 100,000 MPN/100 ml

$$\alpha = \sqrt{Ka^2/E} = \sqrt{\frac{1/day \times (100\ ft)^2}{0.1\ mi^2/day \times \frac{(5280\ ft)^2}{mi^2}}} = 0.060$$

∴ C(0,0)/(W/2HE) = 2.50 (Fig. 5-28)

$$C(0,0) = C_{MAX} = 2.50 \times \frac{(10\ cfs \times 100,000\ MPN/100\ ml)}{2 \times 5\ ft \times 32.3\ ft^2/sec} = 7740\ \frac{MPN}{100}\ ml$$

For a level of 1000 MPN/100 ml,

$C(0,\bar{Y})/C(0,0) = 1000/7740 = 0.13$

& $\bar{Y} = Y/a \approx 15$ (Fig. 5-29, for $\alpha = 0.06$)

∴ Alongshore, beyond approximately $Y \doteq \pm 1500$ ft, coliform levels would be less than 1000 MPN/100 ml. In the offshore (x) direction, coliform concentrations would decrease even more rapidly than in the y-direction.

Dissolved Oxygen Deficit Analysis

Use $W_{UOD} = 10\ cfs \times 165\ mg\text{-}UOD/l \times 5.4\ \frac{lb/day}{cfs\text{-}mg/l} = 8910\ lb/day$

$W_{DEF} = 10\ cfs \times 5\ mg\text{-}DEF/l \times 5.4 = 270\ lb/day$

At x = 0, Y = 0: Max UOD concentration is:

$\alpha(K_r) = \sqrt{K_r a^2/E} = \sqrt{0.25 \times 100^2/(0.1 \times 5280^2)} = 0.030$

∴ $L(0,0)/(W_{UOD}/2HE) = 2.95$ (Fig. 5-28)

L(0,0) = 2.95 x 8910/(2 x 5 x 32.3 x 5.4) = 15.1 mg-UOD/l

At x = 0, y = 0:

a) : Max DO Def due to UOD is:

$D_{UOD} = (K_d/(K_a - K_r)) \cdot (L(K_r) - L(K_a))$ (See Eq. 5-33)

As above, $L(K_r) = 15.1$ mg/l

$\alpha(K_a) = \sqrt{K_a a^2/E} = \sqrt{0.53 \times 100^2/(0.1 \times 5280^2)} = 0.044$

∴ $L(K_a)/(W_{UOD}/2HE) = 2.72$ (Fig. 5-28)

$L(K_a) = 2.72 \times 8910/(2 \times 5 \times 32.3 \times 5.4) = 13.9$ mg/l

$D_{UOD} = (0.25/(0.53 - 0.25)) \cdot (15.1 - 13.9) = 1.1$ mg DEF/l

b) : Max DO DEF due to DEF Load is:

$\alpha\ (K_a) = 0.044$ as above & $D_{DEF}/(W_{DEF}/2HE) = 2.72$ as above

∴ $D_{DEF} = 2.72 \times 270/(2 \times 5 \times 32.3 \times 5.4) = 0.4$ mg DEF/l

c) Total DO Def is 1.1 + 0.4 = 1.5 mg/l

At X = 0, y = 1500 ft ($\bar{Y}$ = 15)

$L(K_r) = 15.1 \times 0.22 = 3.32$ mg/l; $L(K_a) = 13.9 \times 0.17 = 2.36$ mg/l

$D_{UOD} = (0.25/(0.53 - 0.25)) \cdot (3.32 - 2.36) = 0.96$ mg DEF/l

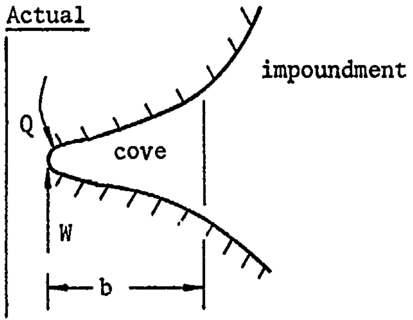
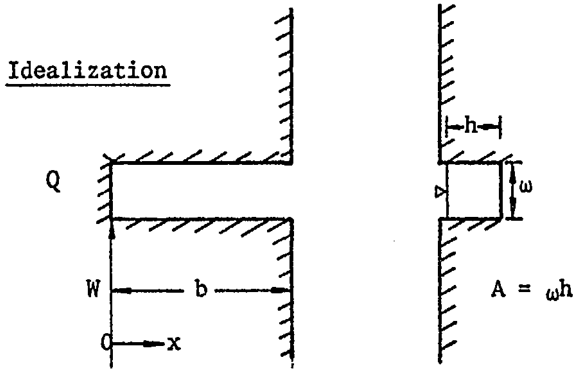
$D_{DEF} = 0.4 \times 0.17 = 0.07$ mg DEF/l

Tot. DO DEF is 0.96 + 0.07 = 1.0 mg/l

Note: At $\bar{y}$ = 15, although coliform conc. have decreased to 13% of its peak, the DO Def has only decreased to 67% to its peak value.

TABLE 5-26

EQUATIONS FOR LONG COVE ANALYSIS

"long" cove	Actual  Idealization 
Reactive $b \geq \frac{3}{\lvert j_1 \rvert}$	$L = L_{\frac{1}{4}} e^{j_1 x}$, where $L_{\frac{1}{4}} = \frac{W_1}{Q} \cdot \frac{2}{1+m_1}$
Sequentially Reactive $b \geq b_{MIN}$ see Fig. 5-30	$D = D_o e^{j_2 x} + \frac{K_{12}}{K_2 - K_1} \cdot L_o e^{j_1 x} - (\frac{1+m_1}{1+m_2}) e^{j_2 x}$, where $D_o = \frac{W_2}{Q} \cdot \frac{2}{1+m_2}$ $D = D_{MAX}$ occurs at $x = x_c = \ln (\frac{1-m_2}{1-m_1}) (\frac{1+m_1}{1+m_2} - \frac{D_o}{L_o} \cdot \frac{K_2 - K_1}{K_{12}}) \div \frac{U}{2E} (m_2 - m_1)$

Description of symbols with typical units

Q=flow (cfs)

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

A=cross-sectional area (ft^2)

W=mass discharge (lb/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})

$m_i = \sqrt{1 + 4K_i E/U^2}$

$j_i = (U/2E)(1-m_i)$

E = dispersion coeff. (mi^2/day)

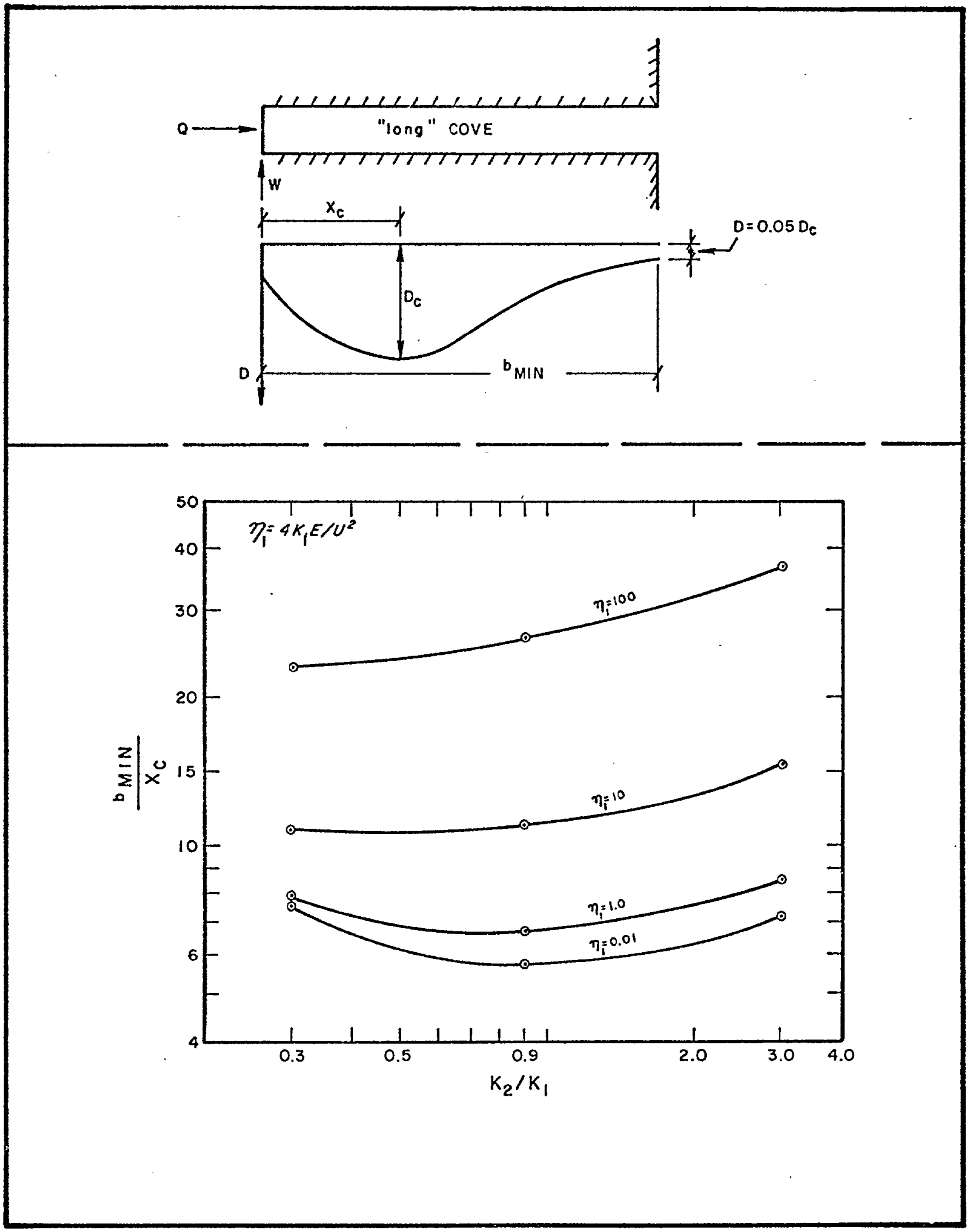


FIGURE 30
MINIMUM LENGTH FOR DISSOLVED OXYGEN DEFICIT ANALYSIS IN "LONG" COVE

when the advective velocity is greater than 0.1 ft/sec. Applications of the "long" cove analysis are shown in Table 5-27.

5.5.3.4 Water Quality Downstream of Impoundments

The degree to which impoundment releases impact downstream water quality depends upon the quality of the impounded water at the dam and the characteristics of the downstream channel. The release is assumed to have one-dimensional plug flow, and the equations for conservative, reactive and coupled concentrations in Table 2-17 may be applied. Another consideration in the downstream analysis is the manner of release from the impoundment. If the water flows over a dam, there will be reaeration which decreases the dissolved oxygen deficit. This effect may be estimated by the following equation:

$$D_a - D_b = 0.037 H_d D_a \tag{5-34}$$

where D_a is the deficit above the dam (in the impoundment), D_b is the deficit below the dam, and H_d is the height the water falls (ft). This formulation was developed for the Mohawk River and Barge Canal, and is valid for dams up to 15 feet high and temperatures in the range of 20° to 25°C. For example, if the deficit concentration above the dam is 2.0 mg/1 and the dam height is 8 feet, the dissolved oxygen deficit concentration below the dam is estimated to be:

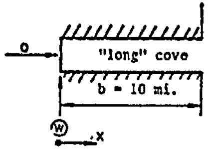
$$D_b = 2.0 - 0.037(8)(2.0)$$

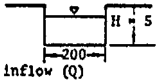
$$D_b = 1.4 \text{ mg/1}$$

If the impoundment release is from the bottom of the dam, other factors must be considered. As previously indicated, the deeper layer of impoundments, particularly larger stratified reservoirs, tend to be colder, lower in dissolved oxygen, higher in solids related pollutants, and may possibly contain anaerobic by-products. Releases of this type may result in serious downstream water quality problems, and should be carefully monitored.

TABLE 5-27

EXAMPLE OF LONG COVE ANALYSIS





Tributary inflow (Q)
50 cfs = av. summer flow
10 cfs = design low flow

$E = 0.2\ mi^2/day$; $K_{coli} = 1/day$, $K_r = K_d = 0.25/day$, $K_n = 0.5/day$

$W_{coli} = 1\ cfs \times 100{,}000\ MPN/100\ ml$, $W_{UOD} = 2700\ lb/day$, W_{DEF} negligible

COLIFORM ANALYSIS

For Q = 10 cfs

$U = 10\ cfs/(5\ ft \times 200\ ft) = 0.01\ ft/sec = 0.1636\ mi/day$

$m = \sqrt{1 + 4\ K_{coli}\ E/U^2} = \sqrt{1 + 4 \times 1 \times 0.2/(0.1636)^2} = \sqrt{1 + 29.89} = 5.558$

$j = (U/2E)(1-m) = \frac{0.1636\ mi/day}{2 \times 0.2\ mi^2/day}(1-5.558) = -1.864/mi$

Check: $b = 10\ mi \geq 3/|j| = 3/1.864 = 1.6\ mi$ OK (Fig. 5-26)

Max. conc. @ $x = 0$ $= \frac{W}{Q} \cdot \frac{2}{1+m} = \frac{1\ cfs \times 100{,}000\ MPN/100\ ml}{10\ cfs} \cdot \frac{2}{1 + 5.558}$

$= 3050\ \frac{MPN}{100\ ml}$

For Q = 50 cfs Using same procedure as above, $U = 0.818\ mi/day$, $m = 1.482$, $j = -0.9857/mi$, $3/|j| = 3\ mi$, max. conc = 1610 MPN/100 ml

DISSOLVED OXYGEN ANALYSIS

For Q = 10 cfs

UOD: $m_1 = \sqrt{1 + 4 \times 0.25 \times 0.2/(0.1636)^2} = \sqrt{1 + 7.472} = 2.911$

$j_1 = \ !0.1636/(2 \times 0.2)1\ (1-2.911) = -0.7816/mi$

Check: $b = 10\ mi > 3/0.7816 = 3.8\ mi$ OK

Max. UOD conc. $= \frac{2700\ lb/day}{10\ cfs \times 5.4\ \frac{lb/day}{cfs\text{-}mg/l}} \cdot \frac{2}{1 + 2.911} = 25.6\ mg\text{-}UOD/l$

DO DEF: $m_2 = \sqrt{1 + 4 \times 0.5 \times 0.2/(0.1636)^2} = \sqrt{1 + 14.94} = 3.993$

$j_2 = (0.1636/(2 \times 0.2))\ (1 - 3.993) = -1.224$

$X_c = \ln\left(\frac{1-m_2}{1-m_1} \cdot \frac{1+m_1}{1+m_2}\right) \div (U/2E(m_2-m_1))$ (Table 5-26)

$= \ln\left(\frac{1-3.993}{1-2.911} \cdot \frac{1+2.911}{1+3.993}\right) \div \left(\frac{0.1636}{2 \times 0.2}\ (3.993-2.911)\right) = 0.4619\ mi$

Check b_{min}: For $\eta_1 = 7.47$, $K_2/K_1 = 2$,

$b_{min}/X_c = 12$ and $b_{min} = 12 \times 0.4619 = 5.5\ mi < 10\ mi$ OK (Fig. 5-30)

Max DO DEF conc @ $X = X_c$

$= \frac{0.25}{0.5-0.25} \times 25.6\ \left(e^{-0.7816 \times 0.4619} - \left(\frac{1 + 2.911}{1 + 3.993}\right)e^{-1.224 \times 0.4619}\right)$

$= 6.45$ mg-DO Def/l

For Q = 50 cfs Using same procedure as above:

UOD: $\eta_1 = 0.2989$, $m_1 = 1.140$, $j_1 = -0.2863/mi$, $3/0.2863 = 10.4\ mi \simeq 10\ mi$, OK

Max. UOD conc. = 9.34 mg-UOD/l

DO DEF: $m_2 = 1.264$, $j_2 = -0.5399/mi$, $X_c = 2.279\ mi$, $b_{min} = 16\ mi$.

Since $b = 10\ mi < b_{min} = 16\ mi$, "long" cove analysis is inappropriate and would lead to inaccurate results in the cove. A higher level analysis is required which incorporates the cove and impoundment simultaneously.

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Areawide Assessment Procedures Manual
Chapter 6
Evaluation and Selection of Control Alternatives

Prepared for
Municipal Environmental Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

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CHAPTER 6

EVALUATION AND SELECTION OF CONTROL ALTERNATIVES

6.1 Introduction

Chapter 6 provides an approach for developing cost-effective water quality management plans for the 208 planning area. This chapter extends the analysis procedures of Chapter 5 by presenting guidance for establishing water quality objectives, for developing strategies for waste load reductions and allocations, and for developing and evaluating alternatives for controlling pollutant sources. The discussion presented in this chapter is intended to provide guidance for these activities. However, where other techniques for load reduction or cost optimization are available to the 208 planner or engineer, they may be substituted in whole or in part for the methods presented in Chapter 6.

Water quality objectives are defined from consideration of water quality standards and water use objectives within the 208 planning area. For certain parameters, it will be difficult to meet desired water quality objectives through urban source controls because of the contribution of upstream non-point source loadings, both within and outside ("background") the 208 planning area. In such cases, the 208 planner will have to investigate combinations of urban and non-point source control practices which achieve water quality objectives.

Techniques for developing rational load-reduction strategies in the 208 planning area are presented. These include controls for both dry- and wet-weather water quality problems, through combinations of municipal and industrial point source controls and storm water controls. Methods are presented for determining the levels of control required at each source in order to attain the relevant water quality objectives.

A methodology is provided for developing and evaluating methods for controlling individual pollutant sources in a 208 planning area. For purposes of this manual, these methods will be referred to as control alternatives. Examples of control alternatives are: an upgraded wastewater treatment plant to handle a point source problem; a storage basin and treatment unit to reduce loadings from combined sewer overflows; and a street-sweeping program to reduce runoff loadings from urbanized areas. (Although not discussed in detail, control techniques for non-urban sources are mentioned in Chapter 4.)

In developing methods of control for wastewater sources, the capability of various control alternatives to achieve specified performance requirements is considered. Performance requirements are stated in terms of percentage reductions of wastewater source loadings necessary to achieve the identified water quality objectives for the receiving water. Alternative combinations of percentage load reductions from the various sources are developed through the water quality impact analysis procedures in Chapter 5. These alternative combinations are referred to as load-reduction strategies. Control alternatives which meet the performance requirements stated in the load-reduction strategies are then evaluated to determine the monetary cost of implementing the alternatives.

The monetary cost of control alternatives is only one of the factors upon which a final selection of an alternative is based. Other important factors include: technical reliability; economic, social and environmental impact; implementation feasibility; and public acceptability. In this manual, however, the evaluation of alternatives is based only on monetary cost; the other considerations, and the final selection of control alternatives, are beyond the scope of this manual. Other EPA 208 guidance documents address the consideration of other factors needed to determine the control alternatives most desirable from an economic, environmental, and social point of view.

6.2 Water Quality Objectives

Water quality standards are the water quality objectives most frequently utilized in wastewater management programs. State agencies establish standards to satisfy State and Federal water quality objectives. These standards are generally based on scientific or empirical evidence that indicate enhancement of water uses when water quality is within prescribed limits. Information regarding local water quality standards can be obtained from the State water quality agencies listed in Table 2-7 (pg. 2-43).

Standards are established to achieve objectives which include but are not limited to:

1. Protection of public health where waters are to be used for recreation, public water supply, or commercial harvesting of fin and shell fish.
2. Protection of the integrity and diversity of the aquatic and marine biology, including valuable commercial and sport fisheries.
3. Insurance of safety to recreational and commercial navigation.
4. Protection of industrial and agricultural water supply.
5. Maintenance of publicly acceptable levels of aesthetic water quality.

In general, the 208 planner should consider water quality standards specific to the planning area as the minimum acceptable water quality objectives.

In certain cases, there are no water quality standards for variables which are at problem levels in a specific study area. For example, local standards might not prescribe maximum levels of nitrogen and phosphorus to protect against nuisance algal blooms. In these instances, the 208 planner should develop target levels of water quality parameters that will insure a desirable level of water quality protection in the study area.

One basis for defining water quality objectives not specifically dealt with in existing standards is contained in Chapter 2 -- Tables 2-22 (pg. 2-100), 2-23 (pg. 2-104), 2-24 (pg. 2-107), and 2-25 (pg. 2-111). These tables, however, are presented for guidance purposes only and should be modified to reflect local experience. Additional information which will prove to be valuable in setting water quality objectives in the absence of specific standards is contained in reference (1).

Finally, there will be cases where water quality contravenes standards or other water quality objectives are attributable to upstream effects which are either outside of the planning agencies jurisdiction or are uncontrollable using present technology. These cases should be addressed by assigning reasonable objectives which involve consideration of opportunities to meet water quality goals at some future time if upstream controls can be implemented, and which recognize the practical water use benefits of partial control of existing problems.

6.3 Load-Reduction Analysis

An overall aim of water quality analysis in a 208 plan is to describe waste-load reductions which result in compliance with water quality objectives. This aim is achieved through a waste-load allocation analysis which specifies alternative combinations of allowable loads to satisfy water quality objectives. This section provides a methodology for development of the required load-reduction strategies.

The procedure for identifying the needs for water quality improvement (outlined in Figure 6-1) summarizes many of the analyses made in Chapter 5 and provides the receiving-water-oriented basis for making allocations for waste-load reduction. The methods of Chapters 2, 3, and 5 normally precede the load-reduction strategy. In particular, the analysis methods of Chapter 2 provide an initial assessment of water quality problems and their associated loads. More detailed loads are then generated in Chapter 3 and applied in detailed water quality analyses using methods presented in Chapter 5. Chapter 6 then provides a systematic approach to developing load-reduction strategies to achieve water quality objectives. The methodology is described briefly as follows.

Projections are developed for each relevant water quality constituent for selected river-flow conditions (Section 2.7, pg. 2-97). The summer low flow (normally described by the 7-consecutive-day/once-in-ten-year low flow) is usually considered the critical case for analyzing dissolved oxygen concentrations, as well as nutrients and other point source pollutants from municipal and industrial sources, and should be examined first (Section 2.7.1.3, pg. 2-101). Storm-related problems are then examined during mean or design-storm conditions (Section 2.7.1.2, pg. 2-101). Pollutants which have longer-term impacts are analyzed on a long-term average basis (Section 2.7.3, pg. 2-107).

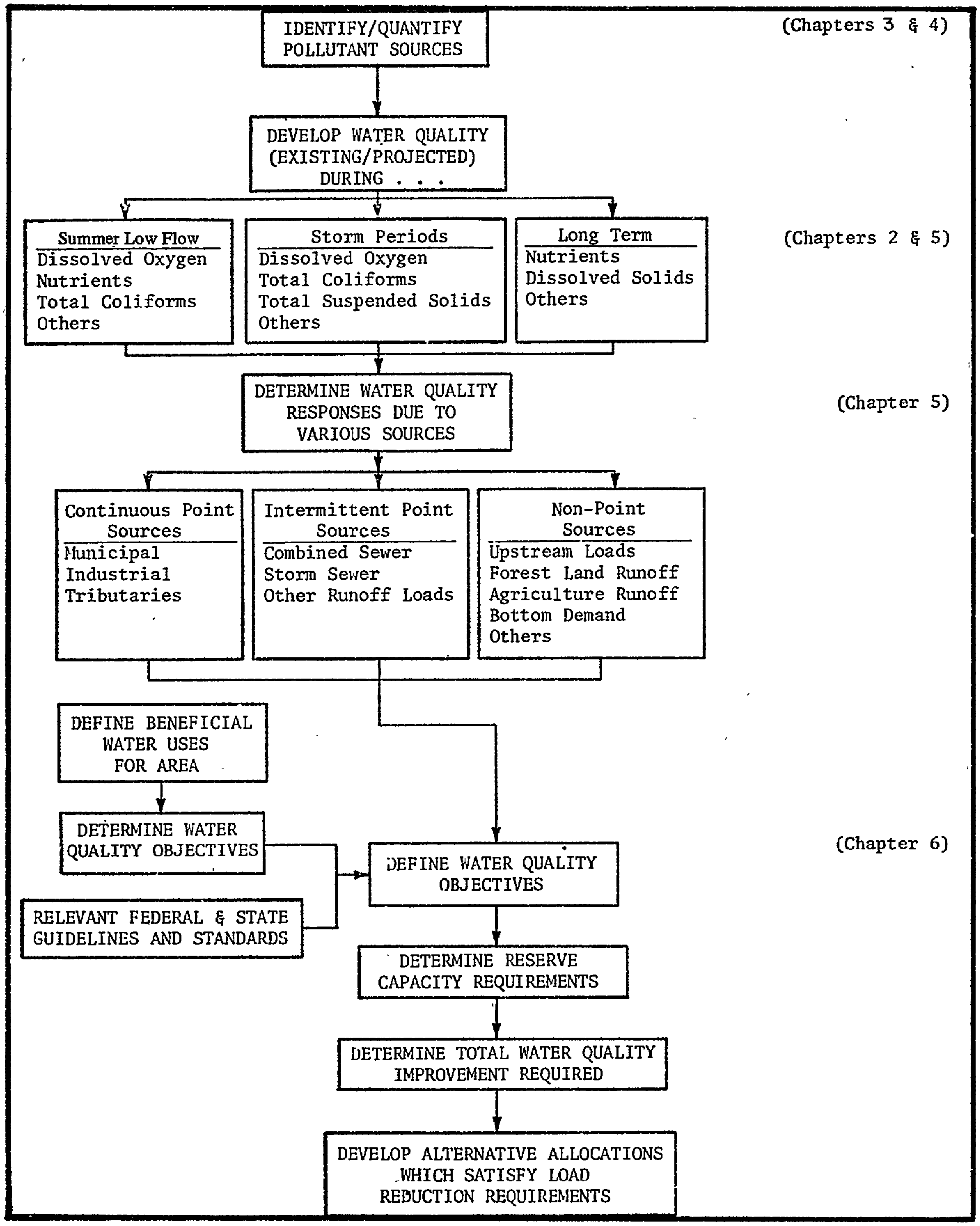


FIGURE 6-1
PROCEDURE FOR DETERMINING WATER QUALITY IMPROVEMENT REQUIREMENTS

Each of these analyses is employed in identifying water quality impacts of the waste loads at critical locations in the stream. The results are then used in determining the degree to which present and projected loads contribute to water quality problems, and, furthermore, the degree of load reduction required to meet water quality goals. The final element of the allocation analysis is a stepwise procedure which develops acceptable load-reduction strategies.

Various water quality problems in the hypothetical South River are identified and analyzed in Chapter 5. In Chapter 6, combinations of point source, urban non-point source, storm sewer and combined sewer load reductions necessary to meet water quality objectives are identified, and control alternatives to improve water quality to desired levels are analyzed.

6.3.1 Sample Problem

The objectives used for the hypothetical South River are presented in Table 6-1. Water quality standards set the objective levels for dissolved oxygen, while objectives for other variables are defined in the 208 planning process from consideration of background water quality and site-specific water uses. The Dissolved Oxygen (D.O.) standard indicates that the D.O. concentration should be above 5 mg/l from M.P. 0 to M.P. 20, and above 4 mg/l from M.P. 20 to M.P. 35. To evaluate this objective effectively, analyses are demonstrated in Chapter 5 for low-flow conditions and for conditions during the "average" storm. Figures 5-10 (pg. 5-41) and 5-11 (pg. 5-42) indicate that the D.O. standards are currently violated under these conditions.

Water quality objectives are also developed for total coliform organisms, total suspended solids, total nitrogen, and total phosphorus. The objectives may still result in problematic conditions, but they appear to be reasonable as initial objectives due to the current level of pollutants from upstream non-point and background sources. The objectives for total coliform organisms and for total suspended solids are defined in terms of the concentration during the average storm, while the objectives for nitrogen and phosphorus are defined in terms of the average (long-term) summer concentration (including storm and non-storm periods) because of the longer

TABLE 6-1

SOUTH RIVER
DISSOLVED OXYGEN OBJECTIVES

1.	Dissolved Oxygen - 1 (DO-1)	Dissolved oxygen minimum of 5 mg/1 from MP 0 to MP 20 and Dissolved Oxygen minimum of 4 mg/1 from MP 20 to MP 35 during 7 day-10 year low flow
	Dissolved Oxygen - 2 (DO-2)	Dissolved oxygen of 5 mg/1 from MP 20 and Dissolved oxygen minimum of 4 mg/1 from MP 20 to MP 35 during mean storm.

Goals For Other Water Quality Variables

2.	Total Coliform Organisms - 1 (TC-1)	Total Coliform Organisms not to exceed 5,000 MPN/100 ml during mean storm
	Total Suspended Solids - 1 (TSS-1)	Total suspended solids not to exceed 200 mg/1 during mean storm
	Nitrogen - 1 (N-1)	Average total Nitrogen not to exceed 2.5 mg/1 as a long term summer average
	Phosphorus - 1 (P-1)	Average total Phosphorus not to exceed 0.4 mg/1 as a long term summer average

temporal scale of impacts associated with nutrients. The rationale for describing these as critical time periods for each water quality variable are described in Chapter 2 and summarized in Chapter 5 (pg. 5-43).

The water quality constituents of concern in the hypothetical South River and the locations of the maximum concentrations are summarized in Table 6-2. Preliminary estimates of the load reductions required to meet water quality objectives at these locations are also summarized in Table 6-2. In this regard, the contributions to water quality problems attributable to each source and the total reductions required to meet the water quality objectives are estimated in Table 6-2.

Using this data, the allocation proceeds through the steps outlined in Figure 6-2. Section (a) of the diagram describes the procedures for developing waste-load allocations for dry-weather conditions, and section (b) describes the procedures for wet-weather conditions. The steps in this dry-weather allocation analysis are described and illustrated sequentially in the following paragraphs.

6.3.2 Dry-Weather Allocation Analysis

Figure 5-11(a) (pg. 5-42) indicates two critical locations at which Dissolved Oxygen concentration is in violation of water quality standards: Milepoints 19.5 and 23.5. The unit response presented in Figure 5-11(b) suggests that one alternative which could satisfy the water quality standards while maintaining a reserve of 0.5 mg/l of Dissolved Oxygen at the critical location is treatment at STP #2 alone. This conclusion is developed as follows:

Critical deficit at M.P. 23.5:	8.1 mg/l
Allowable deficit at M.P. 23.5:	8.17 mg/l (saturation)
	-4.00 mg/l (standard)
	4.17 mg/l
	-0.50 mg/l (reserve)
	3.67 mg/l (allowable)

TABLE 6-2

HYPOTHETICAL SOUTH RIVER WATER QUALITY SUMMARY
(20 YEAR PROJECTIONS)

Constituent:	Dissolved Oxygen Deficit			Total Coliform	Total Suspended Solids	Total Nitrogen	Total Phosphorus
Flow Condition:	Low Flow	Low Flow	Mean Summer Flow	Mean Summer Flow	Mean Summer Flow	Average Summer Flow	Average Summer Flow
Location of Peak Concentration (Milepoint)	19.5 (Critical Location in Upper Reach)	23 (Critical Location in Lower Reach)	33	19	15	24	24
Components of Impact at Critical Location	(mg/1)	(mg/1)	(mg/1)	MPN/100 ml	(mg/1)	(mg-N/1)	(mg-P/1)
a) STP I	-	3.10	0.55	-	-	0.90	0.45
b) STP II	5.25	5.00	1.00	350	-	1.60	0.80
c) Industry	-	-	0.15	-	-	-	-
d) Storm Sewer Runoff	-	-	1.10	53,000	260	-	-
e) Combined Sewer Overflow	-	-	2.00	1,150,000	-	-	-
f) Other Background Loads	0.05	0.05	0.50	330	110	1.45	0.15
g) Total	5.30	8.15	5.30	1,204,000	370	3.95	1.40
Criteria/Objective	DO=5 C_s=8.17 Allowable DEF=3.17	DO=4.0 C_s=8.17 Allowable DEF=4.17	DO=4.00 C_s=8.17 Allowable DEF=4.17	5,000	200	2.50	0.40
Preliminary Estimate of Required Reduction	2.13 +0.50(Reserve) 2.63 $\frac{2.63}{5.30}$ = 50%	3.98 +0.50(Reserve) 4.48 $\frac{4.48}{8.15}$ = 45%	1.13 +0.50(Reserve) 1.63 $\frac{1.63}{5.30}$ = 31%	1,199,000 (99.6%)	170 (46%)	1.45 (37%)	1.90 (72%)

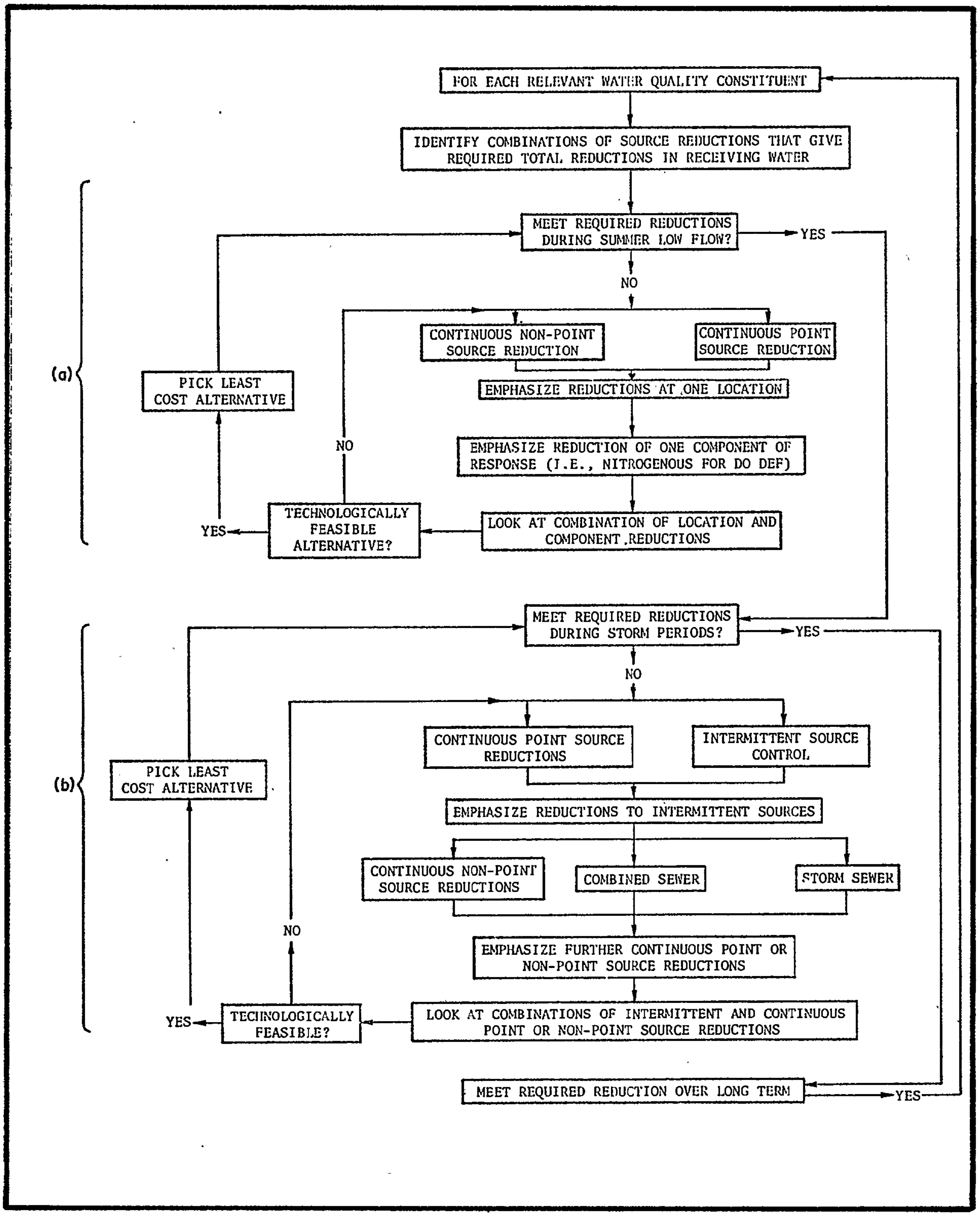


FIGURE 6-2
METHODOLOGY FOR DETERMINING
LOAD REDUCTION REQUIREMENTS

The total deficit due to STP #1 and the industry is 3.3 mg/l at Milepoint 23.5. Thus the allowable deficit due to STP #2 at Milepoint 23.5 is:

Allowable Total Deficit (M.P. 23.5)	3.67 mg/l
Deficit (other sources)	3.30 mg/l
Allowable STP #2 deficit (M.P. 23.5	0.37 mg/l

Thus if the deficit from the STP #2 discharge can be reduced to 0.37 mg/l, the D.O. water quality standards can be maintained. The required additional UOD (Ultimate Oxygen Demand) reduction in the secondary effluent is computed as follows:

Deficit due to STP #2 (M.P. 23.5)	8.10 mg/l (total deficit)
	3.10 mg/l (STP #1 Deficit)
	5.00 mg/l (STP #2 Deficit)

Present reduction required = $\frac{5.00-0.37}{5.00}$ = 92.6 percent UOD removal

The required reduction must be accomplished through a combination of carbonaceous and nitrogenous BOD removal which satisfies the 92.6 percent UOD reduction criteria.

Clearly, other combinations of treatment at STP #1 and STP #2 can also be considered. For example, the minimum treatment above secondary treatment at STP #2 which satisfies the water quality standards considers reduction in the UOD load at that plant such that compliance with the standard is achieved at Milepoint 19.5 (the critical point in the STP #2 D.O. deficit profile). The computation proceeds as follows:

Total deficit at M.P. 19.5	5.3 mg/l
Allowable deficit at M.P. 19.5	8.17 mg/l (saturation)
	-5.00 mg/l (standard)
	3.17 mg/l
	-0.50 mg/l (reserve)
	2.67 mg/l (allowable)

The percent UOD reduction is computed as:

Total deficit due to STP #2 (M.P. 19.5)	5.3 mg/l

Percent reduction required = $\frac{2.67}{5.30}$ = 50 percent UOD removal

Therefore, the minimum additional treatment of the secondary discharge required at STP #2 is 50 percent reduction of UOD, which can be accomplished through various combinations of carbonaceous and nitrogenous BOD removal. The resulting D.O. profile in the South River is illustrated in Figure 6-3. The figure indicates the need for additional removals of UOD at STP #2. Computation of this load reduction is accomplished as follows:

Deficit due to STP #1 (M.P. 23.5)	3.3 mg/1
Allowable deficit at M.P. 23.5	8.17 mg/1 (saturation)
	-4.00 mg/1 (standard)
	4.17 mg/1
	-0.50 mg/1 (reserve)
	3.67 mg/1 (allowable)

A 50 percent reduction in UOD at STP #2 reduces the deficit due to STP #2 at Milepoint 23.5 from 4.1 mg/1 to 2.05 mg/1 (0.5 x 4.1 = 2.05). The total deficit at M.P. 23.5 is calculated to be:

Deficit due to STP #1 (M.P. 23.5)	3.30 mg/1 (Figure 5-10)
Deficit due to STP #2 (M.P. 23.5)	2.05 mg/1 (above)
Total deficit (M.P. 23.5)	5.35 mg/1

and the load reduction at STP #1 is:

Total deficit (M.P. 23.5)	5.35 mg/1
Allowable deficit (M.P. 23.5	3.67 mg/1
Difference	1.68 mg/1
Percent reduction at STP #1	$\frac{1.68}{3.30}$ = 51% UOD removal

The resulting D.O. profile based on 50 percent UOD removal at STP #2 and 51 percent UOD removal at STP #1 is shown in Figure 6-3. Note that the final allocation results in a 0.5 mg/1 reserve to allow for modeling uncertainties and for future growth. The selection of a reserve capacity is normally based on a local knowledge of future growth projections and on an understanding of uncertainties in the water quality modeling framework. A reserve of 0.5-1.0 mg/1 is typical.

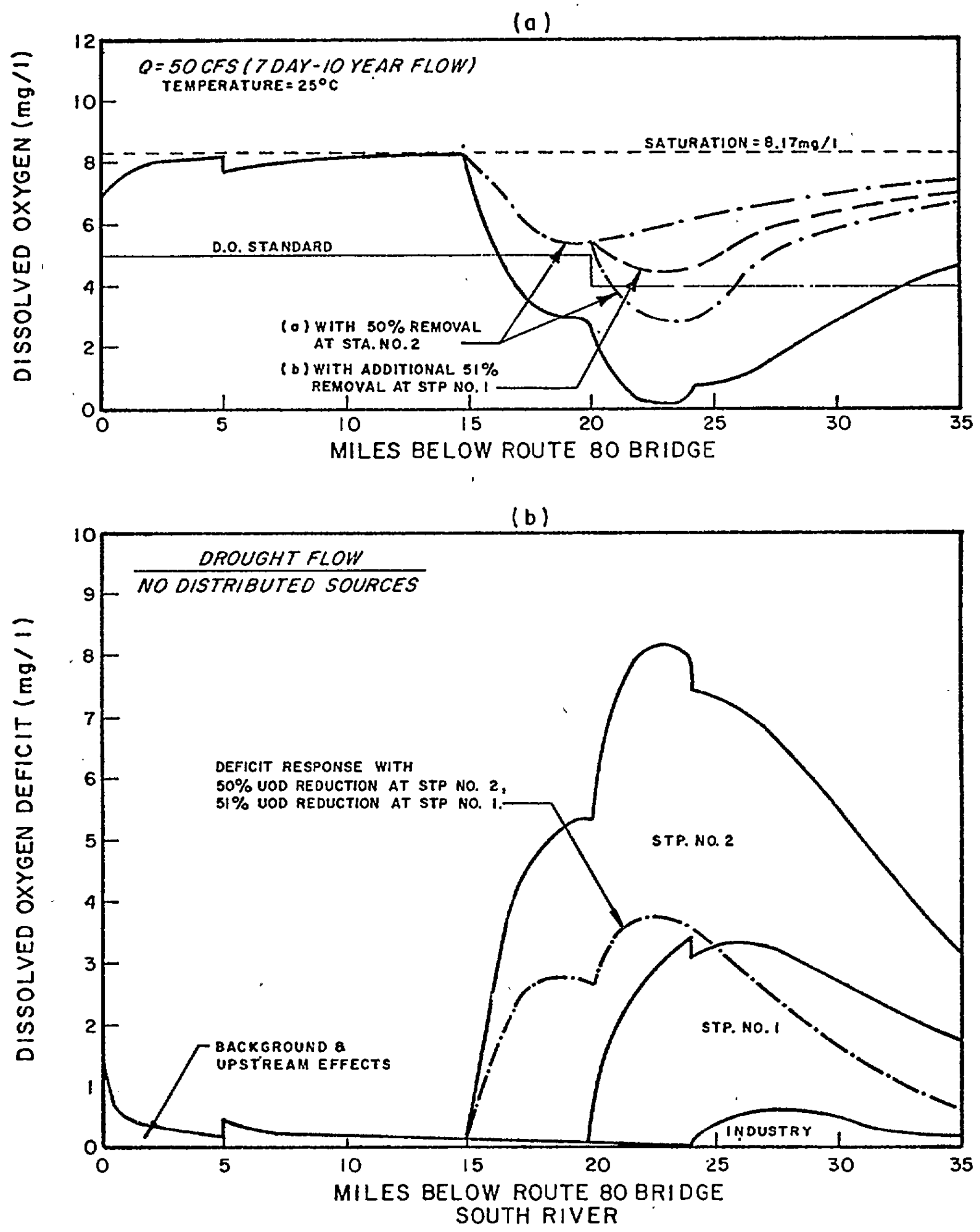


FIGURE 6-3
EFFECT OF WASTE LOAD REDUCTION ON DRY WEATHER DISSOLVED OXYGEN CONCENTRATION

The user should be aware that EPA guidelines do not provide specific recommendations for how much reserve capacity (if any) should be included in wasteload allocation for future growth (whether for possible industrial activities whose wastewaters will not be treated by municipal facilities or for "ultimate development"). In many cases, there will be strong pressure for full utilization of stream assimilative capacity in order to hold down treatment costs. In stream segments where assimilative capacity is already or expected to be constrained during the planning period, local communities will be faced with difficult wasteload allocation decisions. Such decisions should be supported by local growth policies, other local policies, and related regulatory programs (i.e., local ordinances and pricing structures designed to encourage flow reduction or pretreatment). While discussion of these non-structural management techniques is beyond the scope of this manual, the user should be cognizant of their potential application.

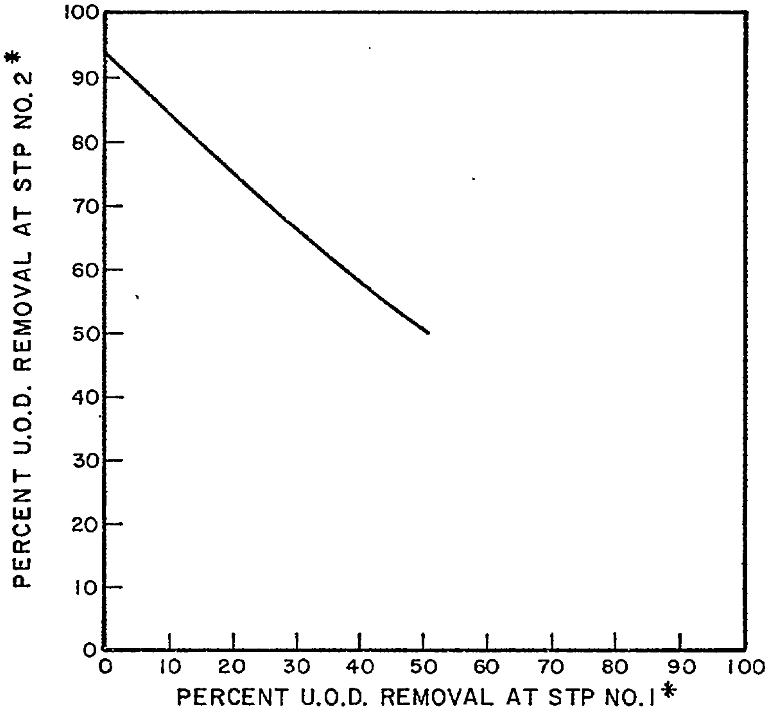
The conclusions that can be drawn from the analysis to this point are:

1. Compliance with D.O. water quality standards during critical-flow, dry-weather conditions requires treatment at least at STP #2. The upper limit on that treatment is an additional 92.6 percent UOD removal. The lower limit on treatment at STP #2 requires an additional 50 percent UOD removal at STP #2 and an additional 51 percent UOD removal at STP #1.

2. There is a continuum of allocations which can be developed for both plants to attain additional UOD reductions of between 50 percent and 92.6 percent at STP #2 and between 51 percent and 0 percent at STP #1.

3. Additional UOD load reductions at either plant can be accomplished through carbonaceous and/or nitrogenous removal.

At this point, the analyst is faced with the problem of determining the most cost-effective treatment option which effects compliance with water quality objectives. A convenient way of viewing the options is presented in Figure 6-4, which displays combinations of UOD removals at the two treatment plants which result in satisfactory water quality. For example, 75 percent UOD removal at STP #2 and 20 percent removal at STP #1 also results in compliance



NOTE:
* PERCENT UOD REDUCTIONS REPRESENT ADDITIONAL REDUCTIONS BEYOND PRESENT TREATMENT.

FIGURE 6-4
ALTERNATIVE ALLOCATIONS AT TWO POINT SOURCES
HYPOTHETICAL SOUTH RIVER EXAMPLE

with the water quality standards under critical-flow, dry-weather conditions. The relationship in Figure 6-4 is not necessarily linear and must be determined by computing several independent allocations. Also, this formulation is not appropriate where the carbonaceous and nitrogenous BOD decay rates are significantly different. Similar allocation procedures apply to other water quality variables which contribute to dry-weather water quality problems.

6.3.3 Wet-Weather Load Reductions

Once the dry weather allocations have been completed for each of those water quality parameters for which problems are projected to exist, the analysis proceeds to wet-weather load allocations. The specific techniques presented here are limited to wet-weather allocations based on treatment of a design-storm load only. The reason for this restriction is that the state-of-the-art technology regarding the effect of storm-water-control structures on the statistical properties of the storm load (probability density function and coefficient of variation) is limited at present to statements regarding the mean-load from the control device. As more information regarding the behavior of these devices is developed from prototype units, present theories regarding the relationships between the input and output storm-load statistics of control devices may be verified or modified such that full characterization of the frequency distribution of treated storm loads can be made.

In practical terms, this places limitations on the use of a statistical approach for wet-weather allocations until additional information on the behavior of storm treatment and control devices becomes available, as previously described. To further illustrate this limitation, consider Figure 5-10 (pg.5-41). The figure shows that the mean storm results in a minimum D.O. deficit of 2.8 mg/l at Milepoint 34; the standard is 4.0 mg/l at this point. Therefore, the load reduction developed in this section considers treatment of the mean load such that the minimum D.O. concentration during the mean storm is raised from 2.8 mg/l to 4.0 mg/l. Since further evaluations using the probability density function or the variability of the treated load cannot be made, the frequency distribution of water quality for treated loads cannot be defined as it was for untreated loads in Chapter 5.

It should be noted, however, that if one knows the probability density function and the variability (coefficient of variation) of the treated load, the frequency distribution of water quality responses due to treated loads could be calculated in exactly the same manner as presented in Chapter 5 for untreated storm loads. Two ways to overcome this problem are under development. First, theoretical studies of treatment-device effect on the probability density function, and of variability around a treated mean storm load are now in the research stage, and possibly will be developed for application purposes within a year or two. Second, the statistical properties of the treated storm load may be empirically determined through the use of treatment-device simulators on a long sequence of treated storm loads.

The output from such studies can be used to develop the statistics of treated storm water loads for inclusion in a water quality analysis such as that presented in Chapter 5. When such work is completed, the planner will be able to determine storm treatment requirements necessary to prevent violation of water quality standards a given percentage of the time. Conversely, if a storm control device is planned for an area, this method of analysis will permit estimates of the reduced frequency with which standards will be contravened with the control in operation. In subsequent sections of Chapter 6, a more generalized load-reduction methodology, which considers procedures for evaluating various storm water control options, is developed.

6.3.3.1 Dissolved Oxygen

Table 6-2 indicates that the critical location for Dissolved Oxygen during the mean storm is at Milepoint 33, where a 1.63 mg/l reduction in the deficit is required. A portion of this reduction will be met by the 50 percent UOD removal at STP #1 and STP #2 necessary to meet objectives during low flow. The average storm deficit response at Milepoint 33 from STP #1 is 0.50 mg/1, and from STP #2 it is 1.00 mg/1. The 50 percent reduction, therefore, results in 0.50 (0.55 + 1.00) = 0.77 mg/1 deficit reduction during the average storm. An additional reduction of 0.86 mg/1 (1.63-0.77) is required to meet standards under the average storm condition. The average storm deficit response at Milepoint 33 from the combined sewer overflow is 2.00 mg/1. An effective load-reduction plan therefore requires a 43 percent (100 x 0.86/2.00) reduction in the combined sewer UOD load for the average storm event.

The storm sewer runoff load contributes 1.10 mg/l of deficit at Milepoint 33 during the average storm. If the storm-related UOD reductions are obtained equally from the combined and storm sewer loads, a 28 percent (0.86/(2.00 + 1.10)) UOD load reduction is needed at both sources.

6.3.3.2 Total Coliform Organisms

Allocation of wet-weather loads of total coliform organisms from the urban area is developed in a manner similar to that for Ultimate Oxygen Demand (UOD) allocations. Figure 5-14 indicates that the projected total coliform concentrations in the South River are dominated by storm water loadings. Two allocations are developed here to illustrate the load-reduction methodology.

The first emphasizes load reduction at the combined sewer system overflow points. In this regard, a reduction is required in the storm sewer system to reduce the peak coliform concentrations in the region between Milepoints 14 and 19 to less than the 5,000 MPN/100 ml storm-period objective. The required storm sewer reduction is:

$$\text{Percent Reduction (Storm Sewer)} = \frac{140{,}000 - 5{,}000}{140{,}000} = 96.4 \text{ percent}$$

In addition to the 96.4 percent reduction of the storm sewer load, further reductions in total coliform loads from the combined sewer system are required to meet the objective in the region below Milepoint 19. Note that background and point sources are expected to contribute 800 MPN/100 ml to the water quality condition; the combined sewers will contribute 1.15 x 10^6 MPN/100 ml. With the previously indicated reduction in the storm sewer load of 96.4 percent, that source will contribute about 2,500 MPN/100 ml at Milepoint 19. The reduction in total coliforms from the combined sewer system is then computed to be:

$$\text{Percent Reduction (CSO)} = \frac{(1.15 \times 10^6 + 800 + 2{,}500 - 5{,}000)}{1.15 \times 10^6}$$

$$\text{Percent Reduction (CSO)} = 99.85 \text{ percent}$$

An alternative load-reduction strategy that involves equal removals at both storm and combined sewer systems can also be developed. In this case, the computation is as follows:

CSO Impact (M.P. 19)	1.15×10^6 MPN/100 ml
Storm Sewer Impact (M.P. 19)	70,000 MPN/100 ml
Other Sources (M.P. 19)	800 MPN/100 ml
Total	1.2208×10^6 MPN/100 ml
Objective (Storm Periods)	5,000 MPN/100 ml
Percent Reduction (Storm Periods)	$\frac{(1.22 \times 10^6 - 5 \times 10^3)}{1.22 \times 10^6}$ = 99.65 percent

Therefore, control of storm-related discharges sufficient to achieve 99.65 percent removal at both storm system discharges and combined sewer system overflows will also meet the required objective.

6.3.3.3 Total Suspended Solids

The water quality objective for suspended solids concentration during summer storm periods is 200 mg/l; this reflects the high background concentrations coming from non-point sources upstream of the urban area. The total background effect (Figure 5-13, pg. 5-45) is expected to be 185 mg/l at Milepoint 15. The storm sewer system which contributes the largest single component to the impact downstream of Milepoint 19 contributes an additional 280 mg/l at this point. The load reduction for suspended solids control is therefore applied to the storm sewer load as follows:

$$\text{Percent Reduction (SS)} = \frac{(185 + 280 - 200)}{280} = 94.6 \text{ percent}$$

Similarly, an objective of 250 mg/l of total suspended solids would require 77 percent removal.

$$\text{Percent Reduction (SS)} = \frac{(185 + 280 - 250)}{280} = 77 \text{ percent}$$

6.3.3.4 Nitrogen and Phosphorus

The nutrient water quality objectives in the South River study area also reflect elevated background concentrations. The objectives are set at 2.50 mg of nitrogen per liter and 6.40 mg of phosphorus per liter. Additional modeling efforts would normally be required to determine the impact of these target nutrient levels on phytoplankton productivity and weed growth in downstream areas. This would normally involve detailed numerical modeling analyses of nutrient-phytoplankton-zooplankton interactions and/or weed-growth modeling frameworks. Both are sophisticated analytical procedures that require specialized tools and personnel thoroughly familiar with the technical issues involved in this type of modeling.

In the absence of such modeling, objectives are normally set merely to reduce nutrient levels, thereby reducing the probability of existing or potential problems. Nutrient reductions are not always required for both macronutrient species, nitrogen and phosphorus. That is, a limitation of either nutrient will normally limit the productivity of plant systems in the downstream area. Cost effectiveness of removing one nutrient preferentially over the other normally governs the load-reduction analysis in this case. Therefore, in the South River example both nutrients are allocated, but only one allocation is ultimately implemented.

The load reduction for nitrogen can be accomplished by requiring nitrogen removal at one or both plants. Figure 5-11 indicates that, if load reduction is required at only one plant, the allocation must be total (100 percent) removal at STP #2 (removal of 100 percent of the nitrogen load at STP #1 would not accomplish the objective).

Combinations of nutrient removal at both plants, however, are feasible. For example, equal percent removals at both plants is computed to be 55 percent removal of total nitrogen, a reasonable treatment level which could be achieved by some flow splitting in both plants. The allocation is computed as follows for a critical location (Milepoint 20 in Figure 5-12(a), pg. 5-44):

STP #2 Impact = 1.65 mg N/l
STP #1 Impact = $\underline{0.90}$ mg N/l
Total STP Impact = 2.55 mg N/l
Background Impact = 1.35 mg N/l
Objective = 2.50 mg N/l

$$\text{Percent Reduction (STP 1 and 2)} = \frac{(2.55 + 1.35 - 2.50)}{2.55} = 55 \text{ percent}$$

Similarly, the phosphorus reduction required to meet the objective (0.40 mg of phosphorus per liter) can be computed as follows:

STP #2 Impact = 0.80
STP #1 Impact = 0.50
Total STP Impact = 1.30
Background Impact = 0.10
Objective = 0.40

Percent Reduction (STP 1 and 2) $= \frac{(1.30 + 0.10 - 0.40)}{1.30} =$ 77 percent

The load reduction computed in this manner requires 77 percent phosphorus reduction at both treatment plants. An alternative scheme would be to remove 90-95 percent of the phosphorus at one plant, with an additional allocation to remove the remainder of the phosphorus overload from the other plant.

6.4 Methodology for the Development and Evaluation of Control Alternatives

6.4.1 Introduction

6.4.1.1 Role of the Methodology in the 208 Planning Process

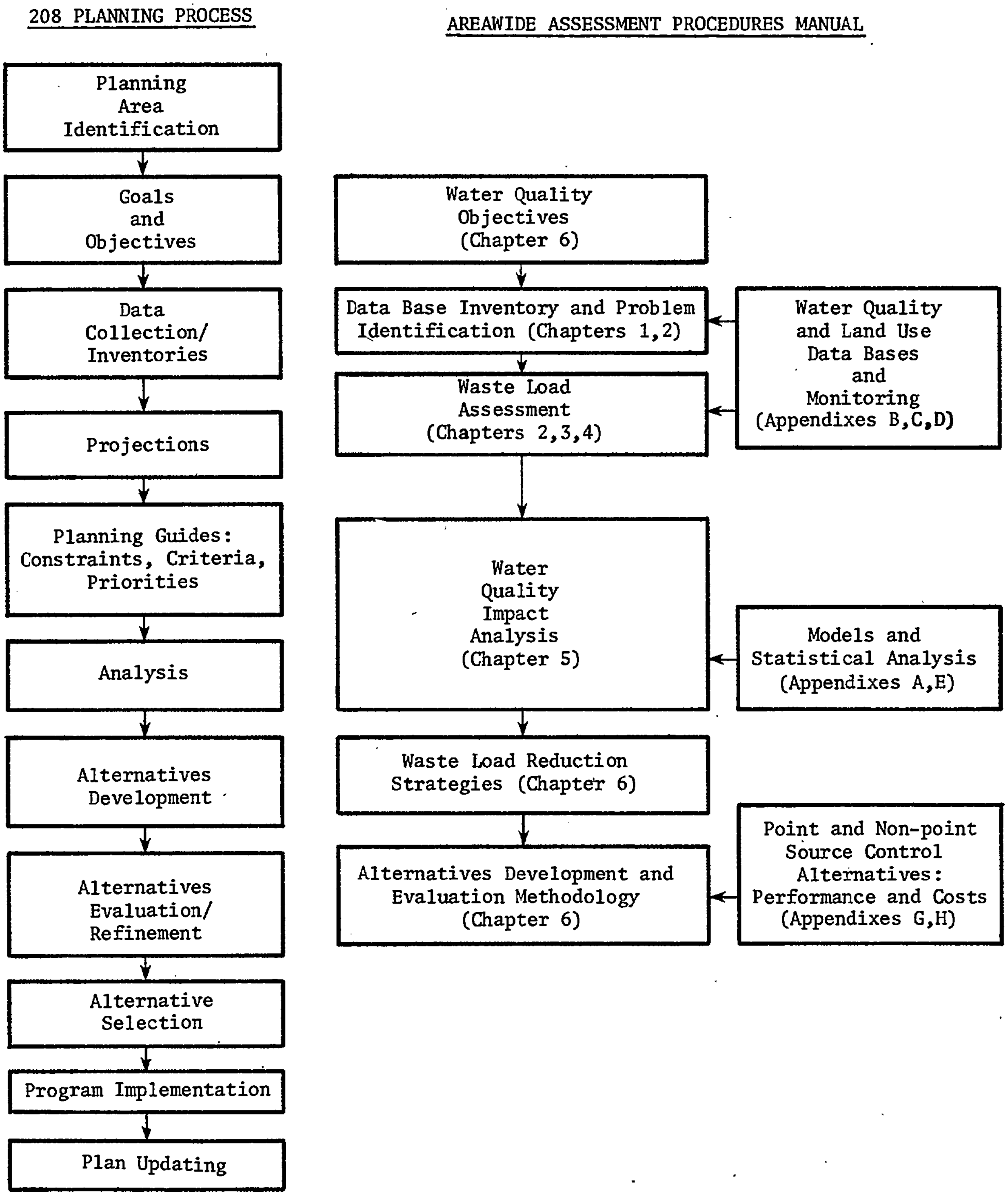
This Areawide Assessment Procedures Manual provides guidance only for certain steps of the 208 planning process. Figure 6-5 illustrates how the Chapter 6 methodology and topics addressed in other chapters in this manual fit into the overall 208 planning process. The steps of the general 208 planning process are presented as a flow chart at the left, in approximate order of their occurrence. The right side of Figure 6-5 lists the specific topics addressed in this manual, with the horizontal position of each manual topic in this flow chart corresponding to the position of the step or steps in the planning process which the topic represents. The general planning steps which have no corresponding topics in the manual are covered by other EPA guidance documents.

Figure 6-5 also shows the relationships among different parts of this manual. Chapters 1 through 4 cover problem identification and load assessment, Chapter 5 addresses analysis of the water quality impact of the wastewater loads, and Chapter 6 presents guidance for three key areas: water quality objectives, load-reduction strategies, and control alternatives. The selection of water quality objectives forms the basis for determining the type and degree of wastewater source control. Various load-reduction strategies for meeting the objectives are formulated, as explained earlier in this chapter. These strategies deal with the number and types of sources to be controlled and the amount of load reduction to be achieved at each source. Finally, control alternatives are developed on the basis of performance capabilities, and are evaluated for their monetary costs. The result is an identification of the cost of control alternatives which can achieve the desired water quality objectives.

The appendixes to the manual support various chapters, as shown in Figure 6-5. For example, Appendixes G and H contain performance and cost data for the development and evaluation of control alternatives, which are covered in Chapter 6. Specifically, Appendix G contains information on urban non-point source control alternatives, including land management and collection system

FIGURE 6-5

RELATIONSHIP OF CHAPTER 6 METHODOLOGY
TO 208 PLANNING PROCESS



controls. Appendix H contains performance data and cost curves for continuous and intermittent point source control alternatives, including wastewater and residuals treatment systems, and wet-weather storage and treatment units.

The appendixes may be used as sources of performance and cost data to support calculations and alternatives evaluation based on methodologies outlined in this chapter. Before applying the methodologies, the user should study carefully the introductions to the appendixes to understand the assumptions and design basis upon which the cost curves were developed. Only certain costs have been specifically included in the cost curves. In order to develop total cost estimates, the user must add allowances for the factors, such as engineering design and contingencies.

Of course, sources of cost information other than Appendixes G and H may be utilized for any of the necessary determinations. Here again, the user must be careful to investigate which factors have been included in the estimate being used so that it will be consistent with other cost information.

In the Chapter 6 methodology, the performance data are used to determine the technological feasibility of a particular control alternative and the size or number of control devices necessary to achieve the required load reduction. The cost curves or equivalent sources are then utilized to determine the associated monetary cost.

6.4.1.2 Monetary Cost

The Federal Water Pollution Control Act Amendments of 1972 (P.L. 92-500) specify cost-effectiveness as the principal criterion for the planning and development of wastewater management programs as those programs relate to municipal treatment works and to the control of combined sewer overflows and storm sewer discharges. EPA has defined cost-effectiveness analysis as a systematic comparison of alternatives to identify the solution which minimizes total costs to society over a defined planning period to meet given goals and objectives in a reliable manner. Section 208(b)(2)(e) specifies that the plan, in determining the total cost to society, should document the economic, social, and environmental costs as well as the capital, operating, administrative, and maintenance costs of implementing the control alternatives. These latter costs can usually be quantified in monetary terms, but the economic, social, and environmental costs are more difficult to quantify and may require description and evaluation using more subjective techniques.

Monetary costs tend to receive most of the planner's attention in cost-effectiveness analysis. Also, decision-makers tend to be oriented toward cash outlays and toward the financial aspects of 208 planning implementation; in fact, many public advisors and government officials are not at all accustomed to dealing with and making decisions based on non-monetary cost factors. Nevertheless, the 208 engineer or planner has the responsibility to identify and present all significant costs (both monetary and non-monetary) when considering control alternatives.

Other 208 guidance documents provide guidelines for social, economic, and environmental impacts and cost analysis. This manual deals only with the determination of the monetary cost associated with implementing the various techniques presented. However, this should not be construed to mean that the user should orient his cost-effectiveness analysis so closely to monetary costs that the other cost considerations are overlooked or obscured. Also, alternatives should not be eliminated from further evaluation solely because of a high monetary cost relative to other alternatives. The consideration of non-monetary or non-quantifiable factors may result in a low monetary-cost alternative actually having a high total cost to society. Conversely, further analysis may render an alternative with high monetary cost very attractive because of public sensitivities or other localized factors. For these reasons, <u>the Chapter 6 methodology is not intended to serve as the only guide to the user for selecting or eliminating a control alternative from further consideration</u>.

In some cases, pre-screening of specific applications of a control alternative will be made on the basis of least monetary cost. For example, the most promising (least-cost) sites among a number of potential sites will be chosen in the initial evaluation of land application as a feasible control alternative. If the land application approach is still attractive after other factors have been evaluated, a more detailed analysis procedure may be employed to assure that the most cost-effective land application sites have been identified. As far as this methodology is concerned, all control alternatives which are found to be feasible from a performance point of view will be evaluated for monetary cost, unless the user chooses not to do so for other reasons.

6.4.1.3 Relative Reliability of Performance and Cost information

Monetary-cost comparisons should not be the sole basis for selecting control alternatives, but at some point in the selection process the relative costs of various alternatives must be considered. Therefore, it is very important for the engineer or planner to understand the reliability of the information upon which the cost figures are based.

The reliability of performance data must also be considered, because this type of information frequently is the basis for determining the size of treatment units or other control devices needed for a particular control alternative. This section covers the use of the performance and cost information of Appendixes G and H, and provides a method for assuring adequate consideration of the relative reliability of the various inputs of information (performance and cost) involved in comparison of.the cost of various control alternatives.

The performance data and cost information in Appendixes G and H are utilized to determine the monetary cost of feasible control alternatives for addressing water quality problems in 208 planning areas. Performance data for a particular control alternative are compared to a required standard to assess the alternative's capability to meet the standard. Then the monetary costs of those alternatives which meet the performance requirements are determined by utilizing the cost curves. Since performance and cost information for one alternative may be based on much more extensive data and experience than the information for another alternative, the reliability of both the cost information and the performance information should be taken into account when considering relative cost and performance capabilities.

The reliability of information is especially important in developing, evaluating, and selecting control alternatives for areawide water quality management, because there are highly varying degrees of experience with the various control alternatives. For example, an abundance of cost figures and estimates are available to substantiate performance and cost curves for an activated sludge treatment plant. Less data and experience are available to substantiate performance and cost relationships for street sweeping as an alternative in control of pollution from urban runoff. Even less data and experience are available to substantiate performance and cost estimates for various land management alternatives such as zoning. Although each of these three alternatives is known to be effective in reducing pollutant levels, determination of the best combination will require careful deliberation, good judgment, and full recognition of the reliability of each type of information at the time the decision is made.

The concept of "relative reliability" is presented here to aid the user of this manual in comparing the monetary cost of control alternatives. Five levels of relative reliability are used to identify the nature and extent of the experience and data upon which the cost and performance information is based:

- Level A indicates estimates based on detailed breakdowns of all pertinent cost elements and is supported by detailed engineering data. This level of reliability is always based on site-specific information. The relative reliability of information in this level is ± 15%. For example, facilities-planning estimates (Section 201 of P.L. 92-500) represent Level A information reliability.
- Level B indicates that the data and experience on a particular control alternative are sufficient only to establish a relationship, as expressed by a table of data or a single curve or family of curves. The relative reliability of information at this level is ± 30%. For example, general cost curves such as the wastewater treatment systems curves and process curves presented in Appendix H represent Level B information reliability.
- Level C indicates that the data and experience are sufficient only to establish a range of values for cost or performance. The relative reliability of information at this level is ± 50%. For example, street sweeping estimates, such as those in Appendix G, represent Level C information reliability.

- Level D indicates that the data and experience are sufficient only to establish the relative order of magnitude of the cost and performance characteristics.
- Level E indicates that the data or experience is insufficient to establish any level of cost or performance estimate, or that site-specific factors are so critical to the performance and cost that a general estimate should not be made.

This manual does not present guidelines on the application of the relative reliability concept for particular situations. Rather, the application by the user will be a function of the control alternatives that are being compared, the closeness of the cost or performance estimates, the background of the user, the consequences of error, and other factors.

The "relative reliability" concept is introduced to emphasize to the user that comparisons of cost or performance estimates prepared using this manual are only as reliable as the lowest level of reliability assigned to the control alternatives being considered. The concept is particularly well suited to compare more traditional engineering approaches to load reduction with "emerging" non-structural control techniques whose costs and relative effectiveness have not been satisfactorly evaluated or sufficiently documented.

6.4.1.4 Organization

The methodology consists of: 1) a framework methodology for directing the user through the methodology operations; and 2) a number of component methodologies for investigating specific control alternatives. The framework methodology guides the user through the entire process of developing and evaluating control alternatives, including the determination of one or more feasible alternatives for controlling each source under each load-reduction strategy. The component methodologies are designed to facilitate the determination of the feasibility and monetary cost of implementing a particular control alternative for a wastewater source of concern.

Both the framework and the components are presented in Section 6.4.2. An example illustrating the use of the methodology in a hypothetical planning area is presented in Section 6.5 as a further aid to the user.

6.4.1.5 Methodology Characteristics

In order that the user may better understand and use this methodology, certain characteristics of the methodology are discussed in the following paragraphs.

The methodology is a logical approach to addressing various pollution problems. It shows the relationship between various types of pollution problems and feasible control alternatives, and illustrates a logical sequence for addressing the problems.

The methodology involves a level of detail which is consistent with the 208 level of analysis. Assumptions and simplifications are made throughout the methodology to facilitate the development of cost estimates with a minimum of site-specific data gathering or use of sophisticated analytical techniques and calculations.

The calculations, engineering assumptions, and judgments used in the methodology allow the user to arrive at a particular determination or answer. The methods presented herein should not be interpreted as the only possible approach. New data, additional information, or advanced techniques may be substituted at any point for those suggested in the methodology. The function of the methodology is to illustrate the interfacing and logical timing of determinations, rather than to present a rigid or all inclusive list of control alternatives. The user should not hesitate to use other data or techniques if he has the necessary information and expertise.

The methodology has been designed for ease of understanding and application. Both the framework methodology and the component methodologies include: 1) logic summary, 2) a detailed operations flowchart, 3) worksheets, and 4) notes on specific operations in order to facilitate understanding and use. Also, the methodology is designed so that the major determinations can be accomplished by hand calculations, with a minimal requirement for outside data or special expertise.

The methodology is comprehensive in that it deals with point (continuous and intermittent) and non-point sources of pollution, and with structural and non-structural alternatives for controlling these sources. However, it does not provide all the detailed analytical techniques or background information needed to analyze all aspects of the implementation of a specific control alternative, such as information on funding, siting, or staging of facilities, and other detailed engineering determinations. These considerations can be explored separately if they are found to be critical to the development and evaluation of alternatives. However, the methodology does suggest where the considerations should be injected into the analysis and how they relate to other parts.

The methodology provides for several levels of analysis. Depending on the complexity of the planning area (i.e., the numbers and types of sources, the degree and complexity of water quality problems, the size of the area, population, etc.), the methodology can be used to identify the most effective general approaches to the problems, or can be used to perform a more detailed evaluation of individual sources for a specific water quality problem. For example, if a number of point sources are in close proximity, their wastewater loads may be aggregated to simplify both the water quality impact analysis and the investigation of control alternatives. The methodology may be used to investigate the necessity for an improvement in the level of treatment of the aggregated load or the elimination of that load by application to the land, or other appropriate alternatives. In addition, the methodology can be used to refine this analysis by segregating loads and investigating the treatment process alternatives at a particular site, when this is of interest. Thus, the methodology is flexible in that the level of analysis can be adjusted to the level of complexity of the area and to the particular problems or control alternatives of interest.

6.4.2 Methodology

6.4.2.1 Use of the Methodology

Content

The presentation of the framework and component methodologies includes:

1. Introduction - relates in general terms the objectives of the procedure or the control alternative.

2. Logic Summary - summarizes the basic logic on which the procedure is based, by listing the major steps involved.

3. Flowchart - presents the detailed steps involved in carrying out the procedure. (These steps are expansions of steps listed in the foregoing logic summary and include references to the Worksheets.)

4. Worksheet - suggests one of several methods for recording the determinations, calculations, comparisons, and assumptions called for in the flowchart. There is a cross-reference between each flowchart step requiring an operation and the corresponding item in the worksheet. The Logic Summary and Flowchart can be used with other types of worksheets or guidelines for calculations. The Worksheets included herein are a suggested approach to aid the user in performing the evaluations.

5. Notes - cover assumptions, reference information and sources, and explanations related to specific flowchart or worksheet items.

The worksheets for each procedure are designed to be copied for repeated use. They can be filled out and stored as the user proceeds through the framework and the component methodologies to document the decisions, calculations, and cost determinations. The FRAMEWORK METHODOLOGY WORKSHEET, Table 6-3, is used to summarize the monetary cost and reliability information for all control alternatives considered for each source, and for monitoring progress in considering load-reduction strategies and sources. Worksheets for the components should be placed behind the framework worksheet, Table 6-3, in the order in which they are considered. The illustrative example in Section 6.5 includes a sample set of worksheets used in the development and evaluation of control alternatives for the hypothetical South River planning area.

The Logic Summary, Flowchart, and Worksheets offer increasing levels of sophistication and detail in the evaluation of control alternatives. The user should use the level of detail which best fits his particular situation and needs. In addition, other techniques or approaches to specific operations within any of the component methodologies should be substituted where the user desires or has more up-to-date or site-specific information.

Level of Analysis

Difficulties may arise when determining the appropriate level of analysis. These difficulties arise from the complexity of many planning areas, the dissimilarity of the alternatives being compared, the various degrees of detail necessary to adequately characterize various alternatives, and the variability in the quantity and quality of information available on various alternatives.

The level of analysis for a particular planning area will be a function of the complexity of the area, the types of pollution sources, and the amount of information readily available on these sources. Highly complex areas will require the use of simplifying assumptions (e.g., aggregation of loads); this tends to decrease the usefulness of the performance and cost determinations if specific sites or sources are of concern.

However, the objective of the overall process must be kept in mind, i.e., to obtain a preliminary evaluation of various approaches to areawide problems. Also, depending on the particular needs and/or background of the planner or engineer using the manual, the methodology determinations may be based on site-specific information, actual data in place of assumptions, or more sophisticated techniques if these are available. In addition, if the area is complex and if loads have been aggregated and assumptions made, the methodology may be applied a second time in order to examine in more detail the sources or alternatives of particular interest.

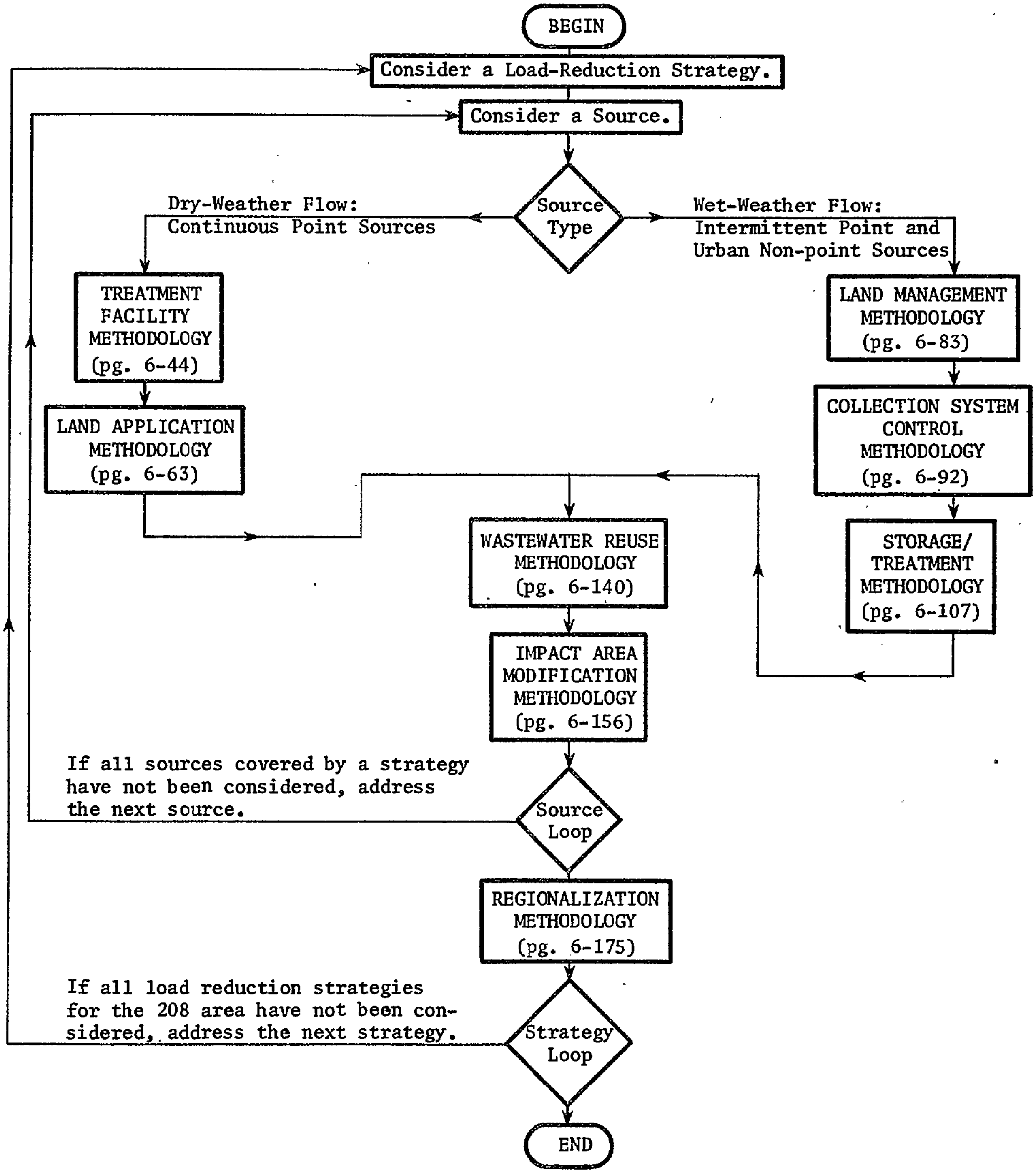
6.4.2.2 Framework Methodology

Discussion

The FRAMEWORK METHODOLOGY is a guide for the entire process of developing and evaluating control alternatives. Within this framework, the user considers each source under each load-reduction strategy, and investigates various control alternatives for that source. The investigation of control alternatives is accomplished by using component methodologies applicable to the wastewater sources involved. An overview of the FRAMEWORK METHODOLOGY is presented in Figure 6-6, which shows the position of the component methodologies in the framework.

FIGURE 6-6

FRAMEWORK METHODOLOGY
OVERVIEW



The FRAMEWORK METHODOLOGY logic considers two basic types of sources: wet-weather sources and dry-weather sources. As shown in Figure 6-6, wet-weather sources of interest fall into two general categories: 1) intermittent point sources (such as separate storm and combined sewer overflows); and 2) non-point sources (such as runoff from construction sites, landfill sites, and urbanized areas). Dry-weather sources of greatest concern are continuous point source discharges, such as municipal and industrial wastewater effluents. Figure 6-6 shows the component methodologies for investigating control alternatives for the two types of wastewater flows.

Wet-weather wastewater flows can be handled at several points. The LAND MANAGEMENT METHODOLOGY covers control alternatives which can be used to reduce the quantity of runoff or the runoff loadings at the source. The COLLECTION SYSTEM CONTROL METHODOLOGY covers alternatives which can be applied to flows after they enter the collection system to reduce the quantity of flow or pollutant load that reaches the stream as an overflow or bypass. Finally, the STORAGE/TREATMENT METHODOLOGY covers alternatives to store and treat overflows at the end of the collection system to reduce the pollutant loadings.

Dry-weather wastewater flows from continuous point sources are generally managed by treatment and discharge to a receiving water, or by treatment and application to the land. The TREATMENT FACILITY METHODOLOGY covers new-plant construction or plant expansion and upgrading, and the LAND APPLICATION METHODOLOGY covers application of wastewater to the land after some level of pretreatment at a facility.

Control alternatives common to both wet- and dry-weather flows are considered in WASTEWATER REUSE METHODOLOGY, IMPACT AREA MODIFICATION METHODOLOGY, and REGIONALIZATION METHODOLOGY. Several component methodologies which are utilized by other of the component methodologies are not shown in Figure 6-6 because they are not employed separately. These are the TRANSPORTATION METHODOLOGY, RESIDUALS DISPOSAL METHODOLOGY, and PRESENT-WORTH METHODOLOGY.

A component methodology guides the user in determining if the performance capability of a control alternative for a particular source meets the requirements of the load-reduction strategy. Then, the monetary cost of implementing the control alternative is determined.

Methodology Logic

A summary of the logic of the FRAMEWORK METHODOLOGY is presented in Figure 6-7. An expanded flowchart, Figure 6-8, lists the steps to be taken in determining performance and costs. The worksheets for recording the operations are presented as Table 6-3. Notes on specific steps or worksheet items are presented after the worksheets.

FIGURE 6-7

FRAMEWORK METHODOLOGY
LOGIC SUMMARY

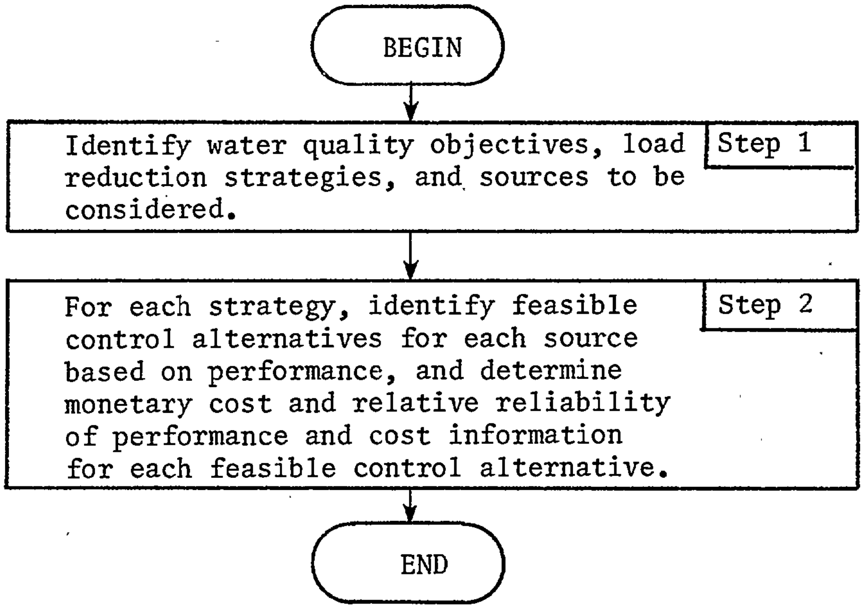


FIGURE 6-8

FRAMEWORK METHODOLOGY
FLOWCHART

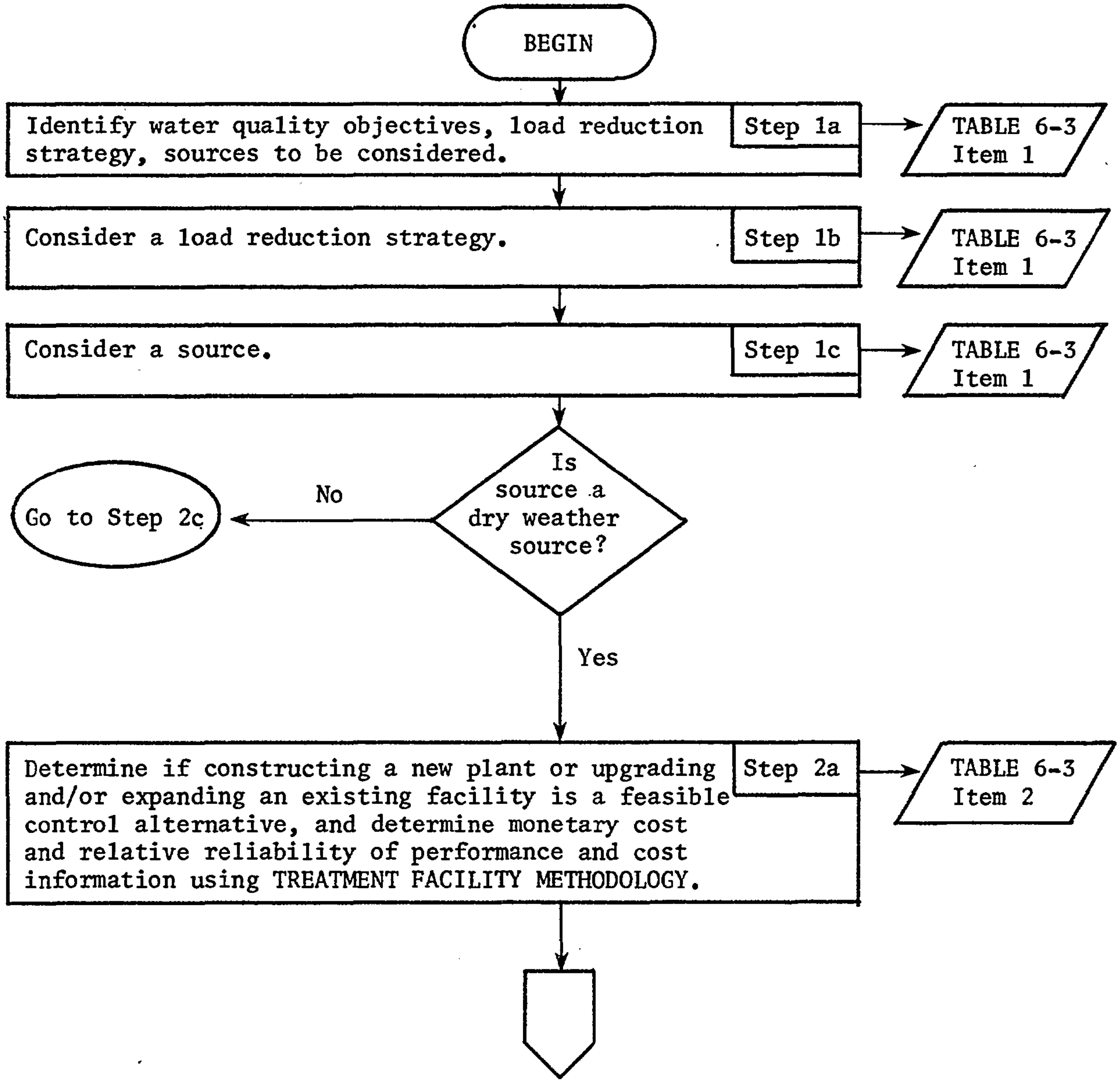
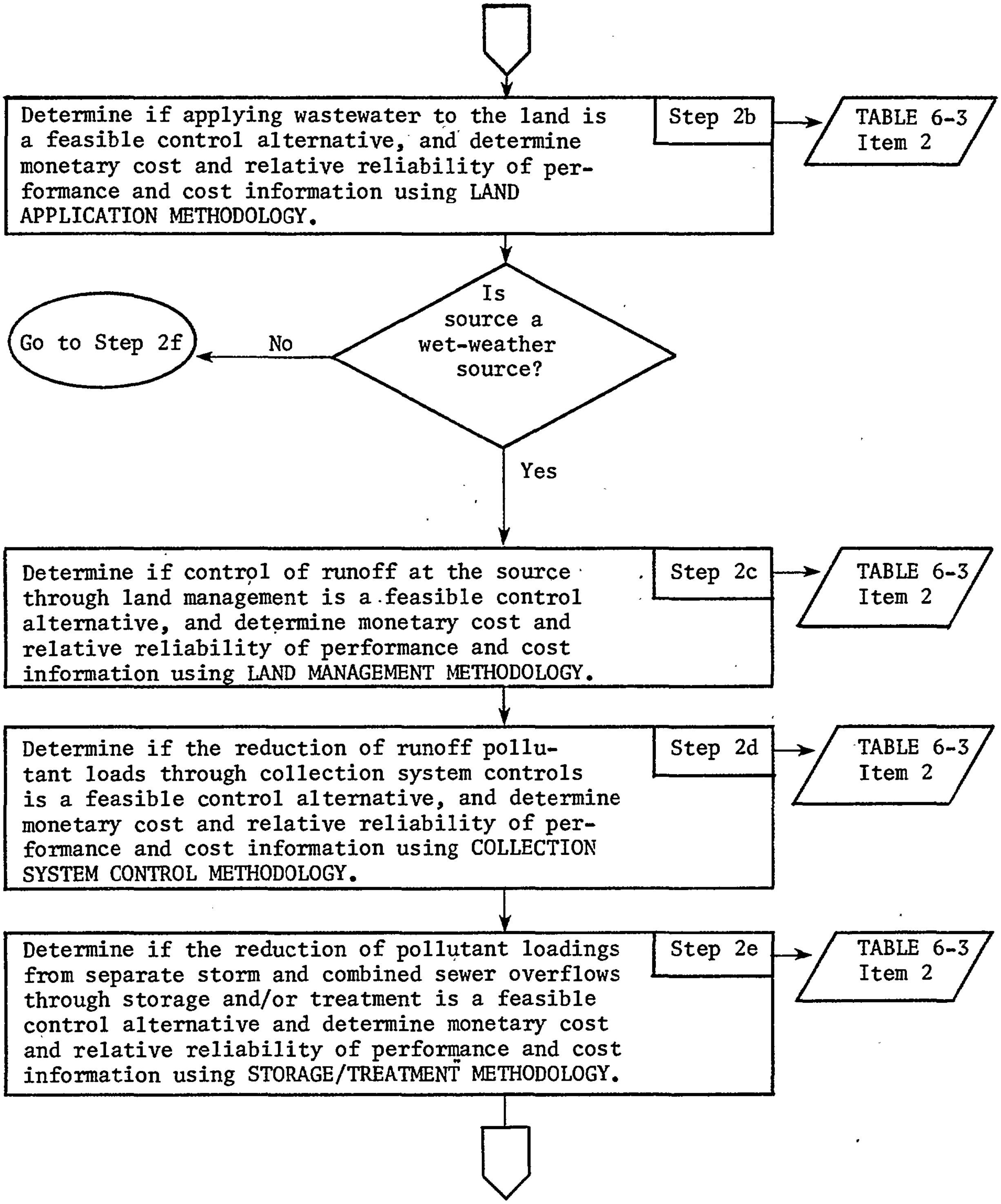


FIGURE 6-8 (CONTINUED)

FRAMEWORK METHODOLOGY
FLOWCHART



Determine if applying wastewater to the land is a feasible control alternative, and determine monetary cost and relative reliability of performance and cost information using LAND APPLICATION METHODOLOGY.

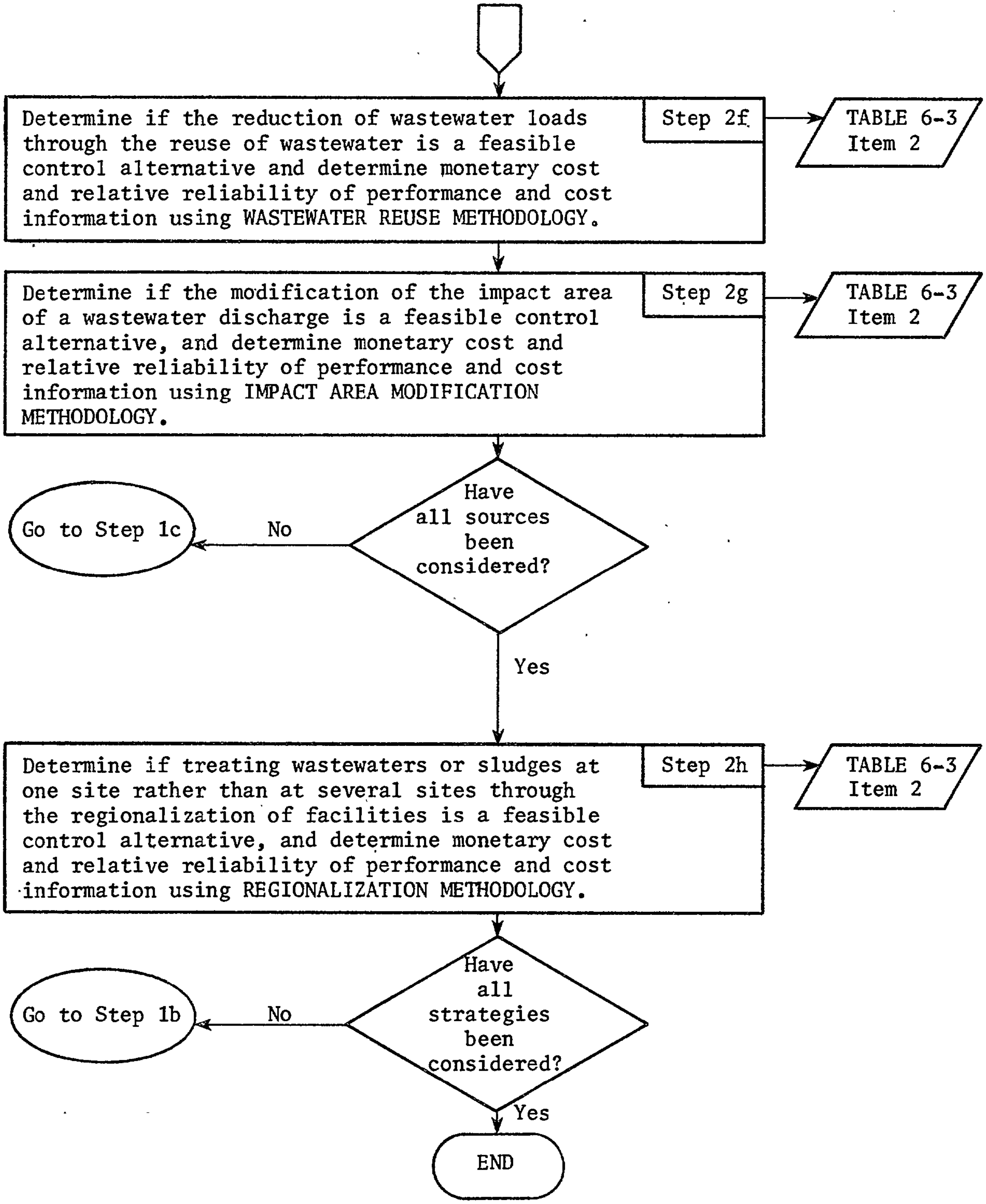
Determine if control of runoff at the source through land management is a feasible control alternative, and determine monetary cost and relative reliability of performance and cost information using LAND MANAGEMENT METHODOLOGY.

Determine if the reduction of runoff pollutant loads through collection system controls is a feasible control alternative, and determine monetary cost and relative reliability of performance and cost information using COLLECTION SYSTEM CONTROL METHODOLOGY.

Determine if the reduction of pollutant loadings from separate storm and combined sewer overflows through storage and/or treatment is a feasible control alternative and determine monetary cost and relative reliability of performance and cost information using STORAGE/TREATMENT METHODOLOGY.

FIGURE 6-8 (CONTINUED)

FRAMEWORK METHODOLOGY
FLOWCHART



Determine if the reduction of wastewater loads through the reuse of wastewater is a feasible control alternative and determine monetary cost and relative reliability of performance and cost information using WASTEWATER REUSE METHODOLOGY.

Determine if the modification of the impact area of a wastewater discharge is a feasible control alternative, and determine monetary cost and relative reliability of performance and cost information using IMPACT AREA MODIFICATION METHODOLOGY.

Have all sources been considered?

Determine if treating wastewaters or sludges at one site rather than at several sites through the regionalization of facilities is a feasible control alternative, and determine monetary cost and relative reliability of performance and cost information using REGIONALIZATION METHODOLOGY.

Have all strategies been considered?

TABLE 6-3
FRAMEWORK METHODOLOGY
WORKSHEET

> The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1

i. Identification of water quality objectives, load reduction strategies, and sources.

a. Define water quality objectives by number and parameters to be controlled.

	Receiving Water Constituents to be Controlled					
Water Quality Objective #	D.O. (Dry Weather)	D.O. (Wet Weather)	Total Suspended Solids (Wet Weather)	Total Nitrogen	Total Phosphorus	Total Coliforms (Wet Weather)

b. Load reduction strategies represent differing percentage reductions in load at the various sources of interest for a particular water quality objective. These strategies are identified by a letter (a, b, c, etc.) where more than one strategy is proposed for a particular water quality objective.

c. Identify sources by number:

Source #	Source Type (Wet or Dry)	Source Description

By ________________ Date ________________ Strategy No. ________________

Checked by ________________ Date ________________ Source No. ________________

Remarks: ______________________________________ Page ________________

TABLE 6-3 (continued)
FRAMEWORK METHODOLOGY
WORKSHEET

Item 1 (continued)

ii. Record of load reduction strategies and sources considered.

- Check (x) the sources to be considered under each load-reduction strategy.
- Circle the checks in the matrix after all appropriate control alternatives have been considered for a source.
- Go to next load reduction strategy when all sources have been considered for that strategy.
- End when all strategies have been considered.

Load Reduction Strategy	Source Number					
	1	2	3	4	5	6

By ____________ Date ____________ Strategy No. ____________
Checked by ____________ Date ____________ Source No. ____________
Remarks: ________________________________ Page ____________

TABLE 6-3 (continued)
FRAMEWORK METHODOLOGY
WORKSHEET

Item 2 - Feasible Control Alternatives.

i. Record the Present-Worth cost of control alternatives determined using the component methodologies.

ii. Record the worksheet page number (from lower right corner) where the present worth is recorded in the appropriate component methodology.

iii. Record the relative reliability of the performance and cost information for the control alternative as identified in Appendixes G and H or at the discretion of the user.

					Information Reliability	
Load Reduction Strategy	Source	Control Alternative	Present-Worth $	Page	Performance	Cost

By ______________ Date ______________ Strategy No. ______________

Checked by ______________ Date ______________ Source No. ______________

Remarks: ______________ Page ______________

Notes on Methodology Logic

Step 1 (Item 1i) - The determination of water quality objectives and the development of load-reduction strategies to achieve these objectives should take place before the development and evaluation of control alternatives.

The water quality constituents to be controlled in order to achieve each objective are identified in Item 1i. As explained in the worksheet, a load-reduction strategy states the degree to which various sources are to be controlled in order to reduce pollutant loadings. There may be several strategies for achieving each water quality objective. For this manual, any reference to a load-reduction strategy should be taken as a reference to the strategy water quality objective combination. Also, the sources of interest in the study area are numbered for ease of reference.

Step 1 (Item 1ii) - The framework methodology is employed as an iterative technique to investigate feasible control alternatives for each source within each load-reduction strategy. Step 1 guides the user through the selection of the particular load-reduction strategy and source that he will be looking at for any particular iteration. Later steps in the framework methodology bring the user back to these steps if there is another source to be considered for one particular load-reduction strategy, or for other load-reduction strategies to be considered for the planning area.

Also, Item 1ii of the control methodology worksheet provides a place for the user to keep track of exactly where he is in the iterations.

The control box just after Step 1c directs the user to the appropriate steps in the flowchart, depending on whether he is considering a wet-weather source or a dry-weather source.

Step 2 (Item 2) - This step sends the user to the appropriate component methodology in order to test the feasibility of the control alternative and to determine the monetary cost. The cost and information reliability (from Appendixes G and H) for each control alternative considered is recorded in Item 2.

6.4.2.3 Treatment Facility Methodology

Discussion

The purpose of this methodology is to identify the monetary costs incurred in treating a wastewater to meet a specific effluent quality. This methodology will identify the cost for constructing new facilities, expanding existing facilities, upgrading existing facilities, or expanding and upgrading existing facilities.

The TREATMENT FACILITY METHODOLOGY provides the user with an approach that will develop the facility costs in the desired detail. It should be noted, however, that in the 208 planning process, generalized treatment levels such as those present in the treatment system curves in Appendix H (e.g., secondary treatment, advanced waste treatment, etc.) will usually be sufficient to provide ample cost information for making decisions in the overall management picture. However, in many cases, the user may have site-specific facility data (e.g., 201 facilities plans) which will provide cost estimates of a higher confidence level.

The TREATMENT FACILITY METHODOLOGY facilitates consideration of phased construction for a control alternative capable of meeting the effluent criteria based on population growth characteristics within the planning area. Therefore, the facility cost that is being developed will be based on a more realistic consideration of the time-value of project costs. The methodology also encourages and facilitates the evaluation of existing facility utilization. It should be noted, however, that upgrading and expanding existing facilities which are old, outmoded, etc., often represents a financial drawback. For example, upgrading or expanding an existing facility, and working in and around existing equipment, often requires non-optimal construction techniques. A correlation is presented in Appendix H, Figure H-1, that shows a relation between the additional cost associated with expansion as a function of the expansion flow. This can serve as a rough guide for estimating the increased capital associated with expansions.

Also, provisions are included to identify the costs for replacing worn-out equipment at the end of its service life, and also for crediting value to salvageable equipment at the end of the planning period. This should ensure that proper consideration has been given to all major factors that influence economical comparisons of facilities.

While not specifically addressed in the TREATMENT FACILITY METHODOLOGY, the user should consider other factors such as siting requirements and site characteristics (e.g., flood hazards, surrounding land uses, utilities, and rail and highway access) in evaluating treatment alternatives. These factors will reflect not only differential costs, but also comparative site suitability. Consideration of these factors should be coordinated with the REGIONALIZATION METHODOLOGY. For additional guidance in facilities planning, the user should consult references (12) and (13).

Methodology Logic

A summary of the logic of the TREATMENT FACILITY METHODOLOGY is presented in Figure 6-9. An expanded flowchart, Figure 6-10, lists the steps to be taken in determining performance and costs. The worksheets for recording the operations are presented as Table 6-4. Notes concerning the methodology steps and worksheet items are presented after the worksheets.

FIGURE 6-9

TREATMENT FACILITY METHODOLOGY
LOGIC SUMMARY

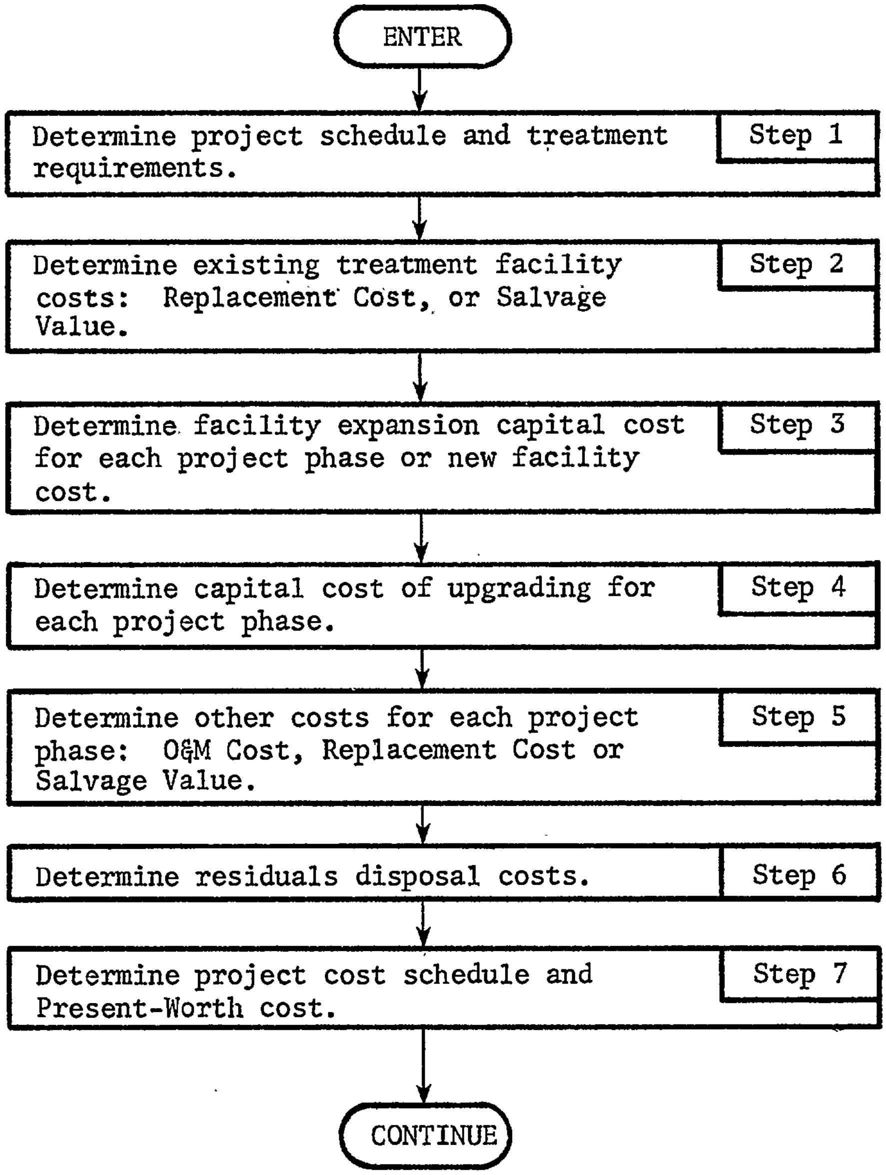


FIGURE 6-10

TREATMENT FACILITY METHODOLOGY FLOWCHART

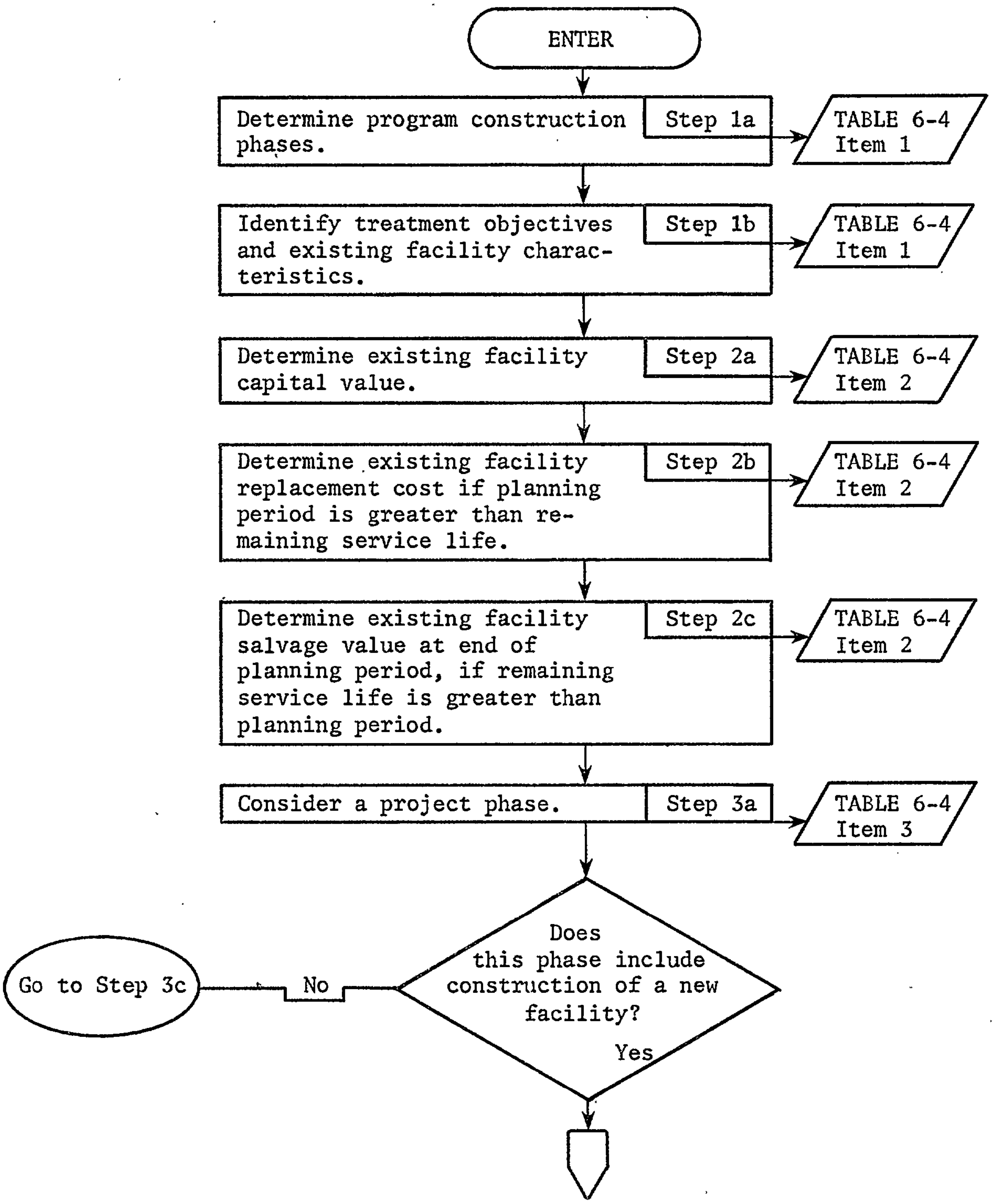


FIGURE 6-10 (CONTINUED)

TREATMENT FACILITY METHODOLOGY
FLOWCHART

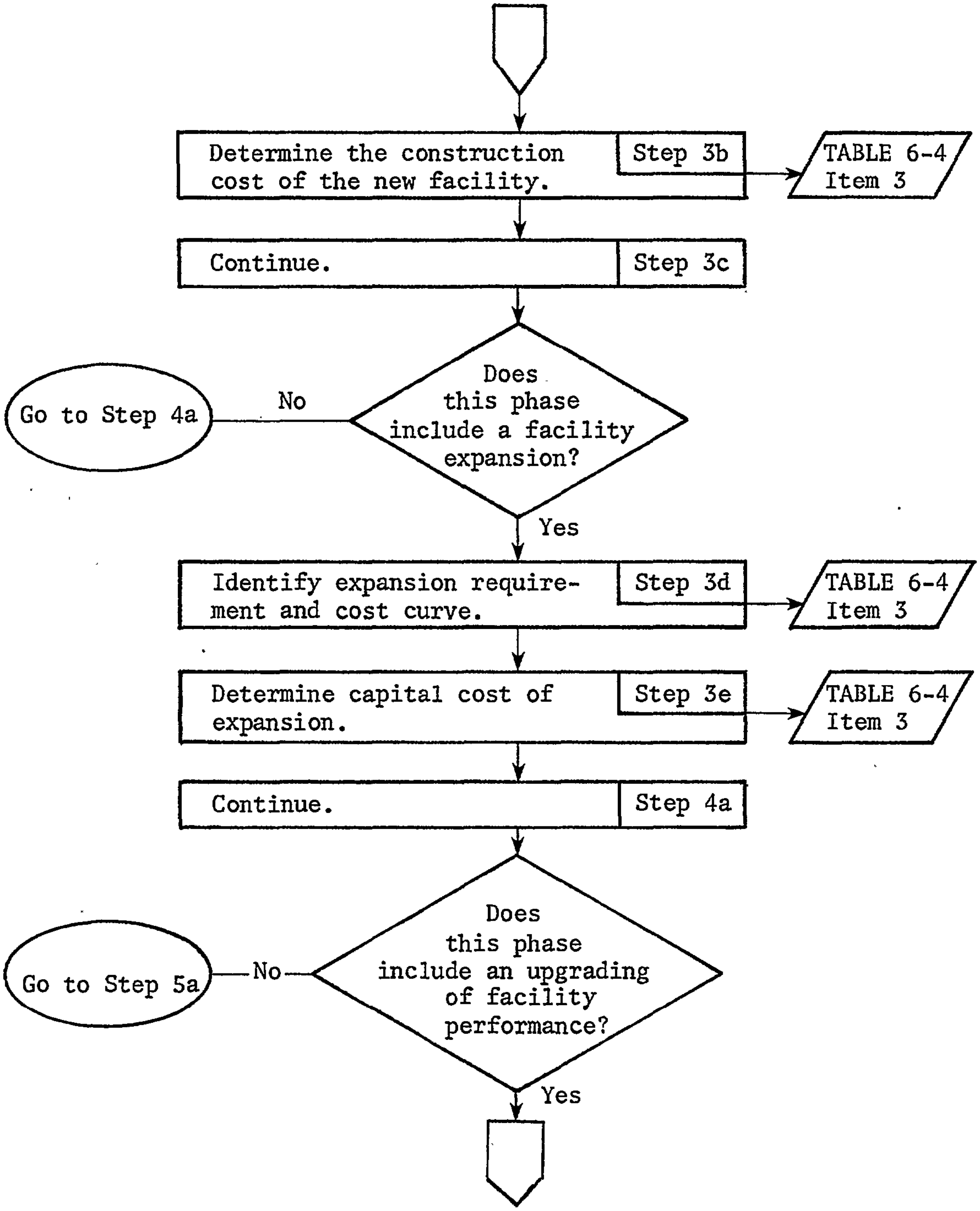


FIGURE 6-10 (CONTINUED)

TREATMENT FACILITY METHODOLOGY
FLOWCHART

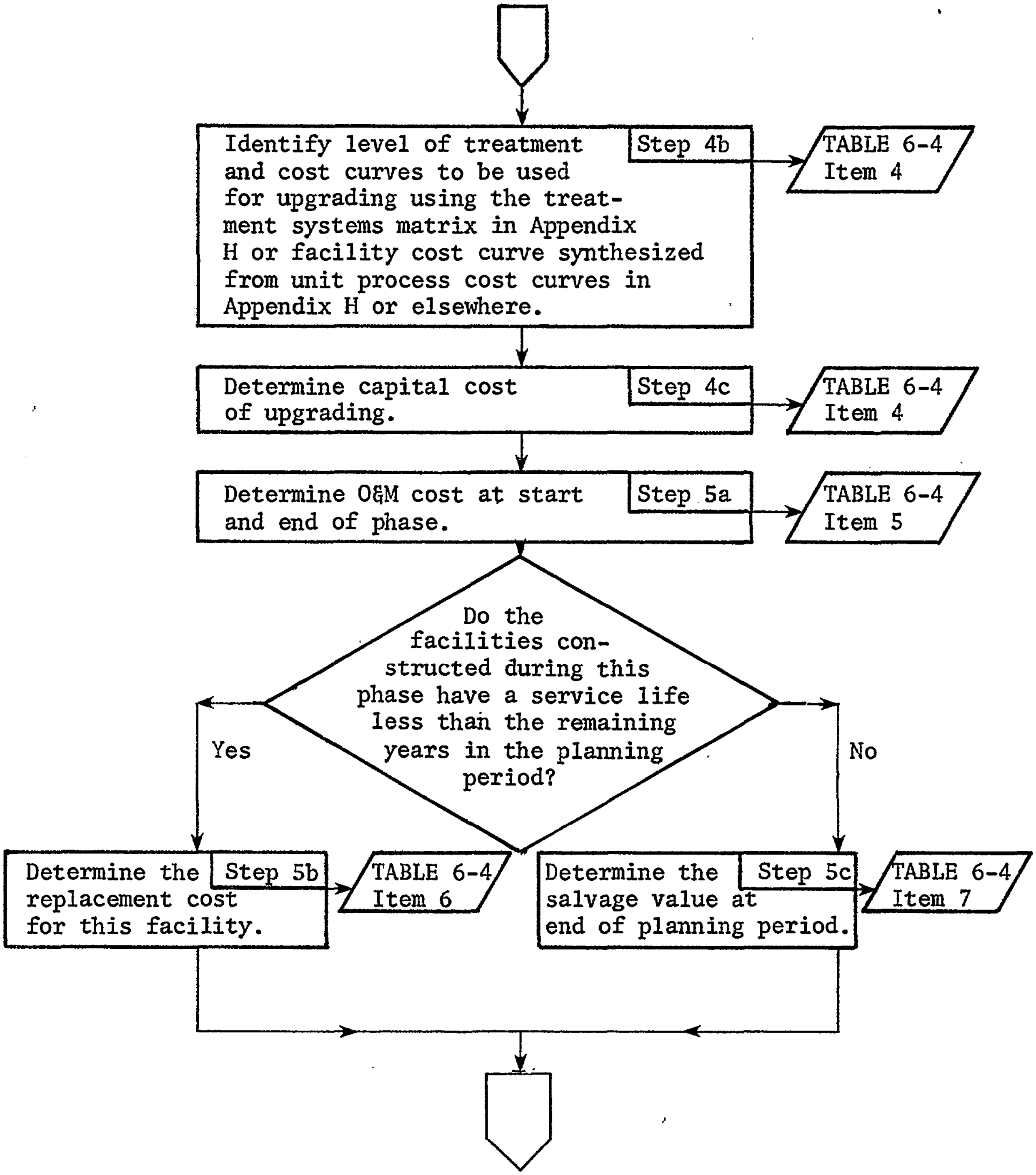


FIGURE 6-10 (CONTINUED)

TREATMENT FACILITY METHODOLOGY FLOWCHART

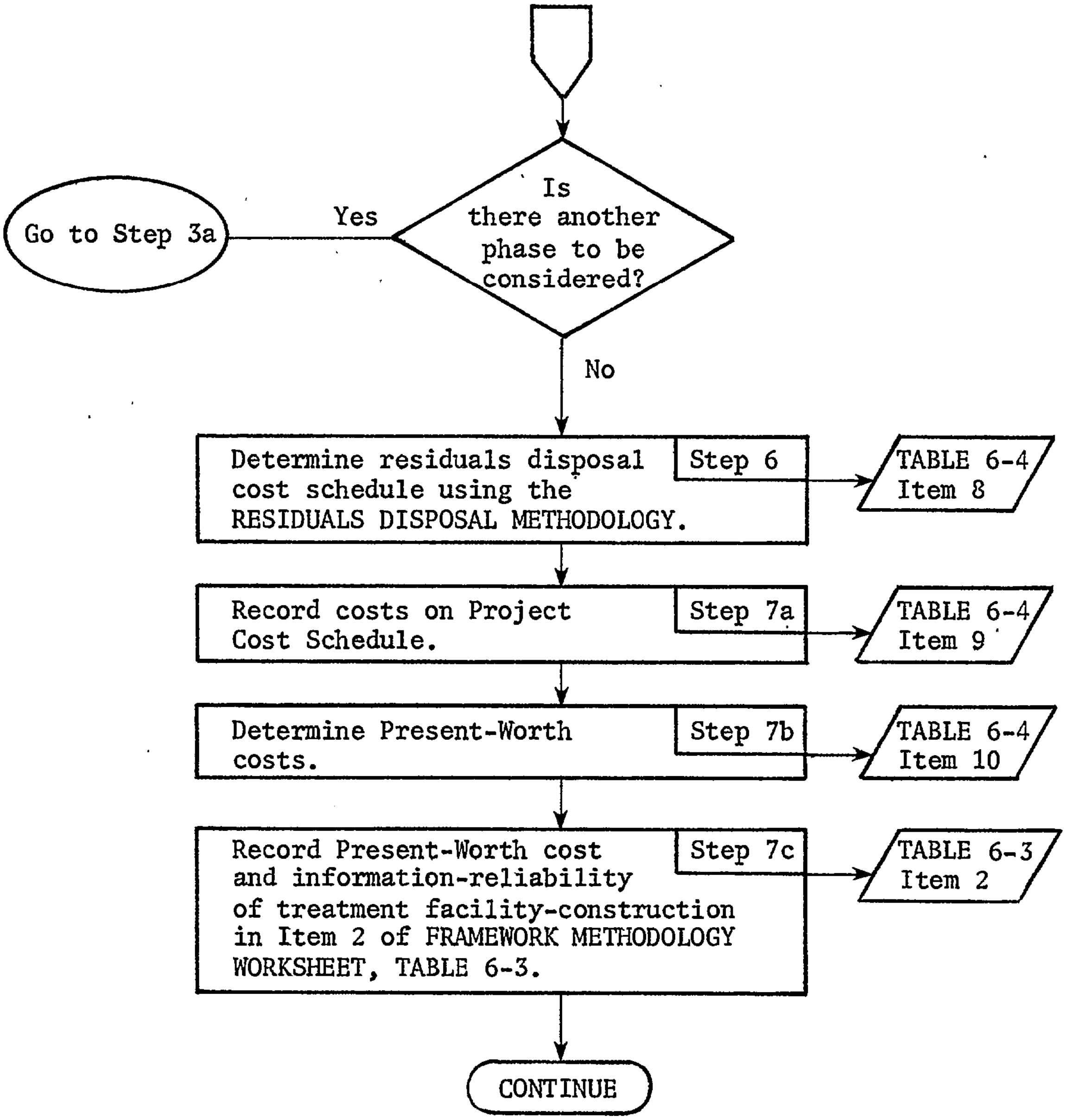


TABLE 6-4
TREATMENT FACILITY METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - Program Implementation Schedule.

i. Planning Period: 20 years

ii. Construction phases: ________

Phase	Timing			Flow Projection (mgd)		Design Flow (mgd)
	Year	to	Year	Start	End	
1*	Present	to	____	____	____	____
2	____	to	____	____	____	____
3	____	to	____	____	____	____
4	____	to	____	____	____	____
n	____	to	____	____	____	____

*Existing facility not utilized at full capacity.

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TABLE 6-4 (continued)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

iii. Treatment Objectives.

Note: Dissolved oxygen deficits use ultimate oxygen demand inputs (Table 6-3). These must be reconverted back to CBOD and NBOD (NH_3) concentrations to determine discharge limitations (See Appendix H discussion of Treatment Systems Performance Matrix).

	Effluent Quality								
Phase	Reference Cost Curve**	BOD mg/1	COD mg/1	TSS mg/1	T-P mg/1	NH_3-N mg/1	NO_3-N mg/1	T-N mg/1	T-C #/100ml
Existing Facility									
1									
2									
3									
n									

**Treatment System curve number (Appendix H, Figures H-2 to H-15) or reference number for synthesized system cost curve developed from unit process curves (Appendix H).

iv. Existing Facility Characteristics.

Design Capacity: __________mgd

Service Life: __________years

Years in Service: __________years

Remaining Service: __________years

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 2 - Existing Facility Cost.
Note: For the first phase of new facility construction, Items 2i, 2ii, and 2iii will equal zero since there is no existing facility.

i. Capital Value (i.e., construction cost plus add-ons).

a. Design Q = ____________mgd

b. Level of Treatment: Reference Cost Curve __________
Service Life __________

c. Construction Cost (Curve $) = __________

d. plus Piping - Curve $ x 15% = __________
Electrical - Curve $ x 12% = __________
Instrumentation - Curve $ x 8% = __________
Site Preparation - Curve $ x 5% = __________
Miscellaneous Structures = __________

e. Sub-Total 1, Construction Cost (c+d) __________

f. plus Sub-Total 1 x Engineering and Construction 15% = __________
Sub-Total 1 x Contingencies 15% = __________
____

g. Sub-Total 2: Capital Cost (e+f) __________

h. CAPITAL VALUE OF EXISTING FACILITY = Sub-Total 2 x $\frac{\text{ENR (Current)}}{2475^*}$ = __________
* ENR = 2475, September, 1976.

ii. Replacement Cost.
(compute only if planning period is greater than remaining service life)

$$\text{Replacement Cost} = \frac{\text{Planning Period - Remaining Service Life}}{\text{Planning Period}} \times \text{Capital Value}$$

= ____________at year______________

iii. Salvage Value.
(compute only if remaining service life is greater than planning period)

$$\text{Salvage Value} = \frac{\text{Remaining Service - Years to Planning End}}{\text{Remaining Service}} \times \text{Capital Value}$$

= ____________at end of planning period.

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 3 - Expansion Program or New Facility Construction.

Phase Number ____________

i. Existing Capacity = __________ mgd (previous phase or existing facility; zero if new facility)

ii. Expanded or New Facility Capacity = __________ mgd (design capacity of next phase)

iii. Level of Treatment: Reference Cost Curve __________
Service Life __________

iv. Construction cost of expanded or new facility - enter cost curve at expanded or new facility at capacity (ii) __________

v. Construction cost of existing facility - enter cost curve at existing facility at capacity (i) __________

vi. Sub-Total 1: Expanded or New Facility Construction Cost (iv-v) = __________

vii. plus Sub-Total 1 x Piping 15% = __________
Sub-Total 1 x Electrical 12% = __________
Sub-Total 1 x Instrumentation 8% = __________
Sub-Total 1 x Site Preparation 5% = __________

viii. Sub-Total 2: Construction Cost (vi + vii) __________

ix. plus Sub-Total 2 x Expansion/Upgrading Factor____ = __________
Sub-Total 2 x Engineering and Construction 15% = __________
Sub-Total 2 x Contingencies 15% = __________

x. Sub-Total 3: Capital Cost (viii + ix) __________

xi. CAPITAL COST OF EXPANSION OR OF NEW FACILITY $= \text{Sub-Total 3} \times \frac{\text{ENR (Current)}}{2475^*}$ = __________

* ENR (Engineering News Record) = 2475, September, 1976.

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 4 - Upgrading Program.

Phase Number ____________

i. Existing Level of Treatment: Reference Cost Curve ____________
(previous phase or existing facility)

ii. Required Level of Treatment: Reference Cost Curve ____________
(for the identified phase) Service Life ____________

iii. Q = ____________ mgd (design capacity)

iv. Construction Cost at required level of treatment - curve from ii ____________

v. Construction Cost at existing level of treatment - curve from i ____________

vi. Sub-Total 1: Construction Cost of Upgrading (iv-v) = ____________

vii. plus Sub-Total 1 x Piping 15% = ____________
Sub-Total 1 x Electrical 12% = ____________
Sub-Total 1 x Instrumentation 8% = ____________
Sub-Total 1 x Site Preparation 5% = ____________

viii. Sub-Total 2: Construction Cost (vi + vii) ____________

ix. plus Sub-Total 2 x Expansion/Upgrading Factor ___ = ____________
Sub-Total 2 x Engineering and Construction 15% = ____________
Sub-Total 2 x Contingencies 15% = ____________

x. Sub-Total 3: Capital Cost (viii + ix) ____________

xi. CAPITAL COST OF UPGRADING = Sub-Total 3 x $\frac{\text{ENR (Current)}}{2475^*}$ = ____________

* ENR = 2475, September, 1976.

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 5 - O&M Constant and Variable Cost.

Phase _____

Level of Treatment: Reference Cost Curve __________

Timing		Design Flow		O&M Cost	
Start (yr.)	End (yr.)	Start (mgd)	End (mgd)	Start	End
____	____	_____	_____	$_____	$_____

Item 6 - Phase ____ Replacement Costs (Upgraded and/or Expanded Portion) (Compute if planning period is greater than phase service life) Replacement Cost Schedule.

Expansion		Upgrading		Total	
Year	Cost	Year	Cost	Year	Cost
_____	_____	_____	_____	_____	_____
_____	_____	_____	_____	_____	_____

Replacement Cost for Phase ___ =

$$\frac{\text{Years from Time of Replacement to End of Planning Period}}{\text{Service Life}} \text{ X Capital}$$

Item 7 - Phase ____ Salvage Value at End of Planning Period. (Compute if phase service life is greater than years to planning period end)

$$\text{Salvage Value} = \frac{\text{(Service Life - Years to Planning End)}}{\text{Service Life}} \text{ x Capital}$$

Expansion S.V. = $__________

Upgrading S.V. = $__________

Total Phase S.V. $__________

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 8 - Residual Disposal Cost, using RESIDUALS DISPOSAL METHODOLOGY.

i. Residual Disposal Technique.

Solids Nature ____________________

Residual Type ____________________

Disposal Method ____________________

Transportation ____________________

ii. Residual Disposal Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Land Cost								
1								
2								
3								
n								

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 9 - Project Cost,Schedule (Summary of costs developed in TREATMENT FACILITY METHODOLOGY).

Phase	Year to Year	Item No.	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
1		3 (Expand/New) 4 (Upgrade 5/6/7 8 (Residual)					
	Total Phase 1						
2		3 (Expand/New) 4 (Upgrade) 5/6/7 8 (Residuals)					
	Total Phase 2						
3		3 (Expand/New) 4 Upgrade) 5/6/7 8 (Residuals)					
	Total Phase 3						
n		3 (Expand/New) 4 (Upgrade) 5/6/7 8 (Residuals)					
	Total Phase n						

Replacement Schedule

Year	Cost

Item 10 - Present-Worth Cost, using PRESENT-WORTH METHODOLOGY

Interest ________ % (from Water Resources Council 18 CFR 704.39, Discount Rate, published annually)

Present-Worth Cost $____________

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Notes on Methodology Logic

Step 1a (Item 1i) - This step identifies the program implementation schedule. The flow projection represents the anticipated wastewater flow at the beginning and end of each phase. The design flow, however, represents the desired treatment facility capacity to be provided for the duration of the phase. Therefore, facility construction costs are defined by an increase in the design flow, while the gradual increase of O&M costs will be identified by the flow projection.

Step 1b (Item 1ii) - This step identifies the treatment objectives to be met during the various phases of construction throughout the planning period. Included are design flows and effluent characteristics for each of the phases. Also identified are the cost curves applicable to the identified treatment levels which can be either the systems curves included in Appendix H, a special curve synthesized from the unit process cost curves, or other acceptable cost curves. The condition of the existing facility is identified in terms of effluent characteristics and useful life (remaining service life).

Step 2a (Item 2i) - This step determines the value of the existing treatment facility, using the cost-curve information in Appendix H. This information is adjusted to reflect installed costs based on the identified percentage factors or on better information (e.g., site-specific data) the user may possess. The capital value does not actually represent the Present-Worth of the existing facility because it is not adjusted for the remaining service life. However, the capital value does represent the cost to build the existing facility under present conditions, and is useful in defining the phase out cost, replacement cost, or salvage value.

Step 2b (Item 2ii) - This step identifies the replacement cost associated with an existing facility which is utilized in the project but which has a remaining service life shorter than the planning period. This computation assumes that the replacement cost is a lump sum that occurs at the end of the service life and has no salvage value at the end of the planning period.

Step 2c (Item 2iii) - This step identifies the salvage value at the end of the planning period when the existing facility is utilized and has a remaining (current) service life that exceeds the planning period. This service life can be identified using the information presented in Appendix H for the treatment systems or by another acceptable method.

Step 3a (Item 3) - In this step, a project phase is considered for detailed cost evaluation. Note that Steps 3a through 5c (Items 3 to 7) will be repeated for each project phase.

Step 3b (Item 3) - This step determines the construction cost for a new facility included in the phase under construction. The times in Worksheet Item 3 pertaining to the value of existing facilities and the upgrade/expansion factor are not relevant for new facilities and should be set to zero.

Step 3c (Item 3) - No discussion.

Step 3d (Item 3) - In this step, the phases that include a treatment-plant expansion are identified by the desired increase in capacity and the level of treatment for the expansion.

Step 3e (Item 3) - In this step, the construction cost associated with the expansion is determined using the Appendix H cost curves or an equivalent synthesized curve and the identified construction factors. The upgrading/expansion factor in Appendix H, Figure H-1, or equivalent, can be utilized, when necessary, to refine this cost estimate to reflect the costs associated with construction which adjoins existing structures.

Step 4a (Item 4) - No discussion.

Step 4b (Item 4) - In this step, the project phases that include upgrading of a treatment facility are identified by the old (previous phase) treatment level and the desired (upgraded) treatment level.

Step 4c (Item 4) - In this step, the cost associated with a treatment upgrading is determined using the cost curves in Appendix H and the identified construction factors. The upgrading/expansion factor can be included, if necessary, to reflect the costs associated with construction which adjoins existing structures.

Step 5a (Item 5) - In this step, the O&M costs are identified for the phase. Since, in general, the wastewater flow will increase during the phase under consideration, the time pattern of O&M costs has been approximated by considering constant and variable O&M costs. The constant O&M cost will be incurred throughout the phase and will be determined by the initial flow rate. The variable O&M reflects the flow increase and is computed by multiplying the average increase in the annual O&M cost during the phase

$$\frac{\text{Final O\&M Cost - Initial O\&M Cost}}{\text{Elapsed Years}}$$

by the gradient series present-worth factor, which is described in the PRESENT-WORTH METHODOLOGY. The required O&M costs are included in the systems curves in Appendix H or can be developed using the process curves or their equivalent.

Step 5b (Item 6) - This step is used to identify the replacement cost schedule for the upgraded or expanded facilities. These values are developed for facilities having a service life shorter than the years remaining in the planning period.

Step 5c (Item 7) - This step identifies the salvage value for the upgraded or expanded facilities. This is computed for the expansion/upgrading or new facilities for each phase, with the total project salvage value then computed from the sum of the salvage values identified for the facilities constructed during each phase.

Step 6 (Item 8) - This step identifies the residual-disposal technique, involving nature of solids, types of residuals, disposal method, and transportation. This item also includes a residual-disposal cost schedule. It is important to note that the RESIDUALS DISPOSAL METHODOLOGY and TRANSPORTATION COST METHODOLOGY will be used to develop the cost schedule in Item 8.

Step 7 (Item 9) - The project cost schedule for each alternative is summarized in this step. A phase-by-phase schedule allows orderly consolidation of the cost values determined throughout this methodology. The item and number indicated on this schedule correspond to the item numbers of the TREATMENT FACILITY METHODOLOGY. It will generally be easiest to combine the identified start and end O&M costs (e.g., expansion plus residuals) and compute the variable O&M costs for each phase by the method outlined in Step 5a. The data in this schedule are utilized in the PRESENT-WORTH METHODOLOGY.

Step 9a (Item 10) - This step records the interest rate and the computed Present-Worth of the alternative. The present worth should also be entered on the FRAMEWORK METHODOLOGY Worksheet.

6.4.2.4 Land Application Methodology

Discussion

Although land application of a treated wastewater effluent can take many forms, this methodology is designed to evaluate two types of systems: 1) the underdrained spray irrigation site which involves a point source discharge of the treated effluent; and 2) the undrained spray irrigation site. The former would serve as a treatment system and might be evaluated as an alternative to existing or additional wastewater treatment unit processes. The latter case would actually describe a disposal technique and might have the added benefit of groundwater recharge or no discharge of pollutants to the receiving stream.

Design criteria to be considered when selecting potential sites and, in particular, when evaluating a specific disposal site, are well documented in EPA guidance documents on land application systems. See references (14) through (18). The criteria for identifying potential sites should be developed for the specific areas of interest. The following items, while not all inclusive, should be addressed during site identification:

- Land use patterns (e.g., land use restrictions, buffer zones).
- Socio-economic impacts (e.g., density of dwellings and other structures, health risks, and traffic patterns).
- Physical characteristics of the site (e.g., soil groups, topography, geologic conditions, ground-water conditions).
- Restricted areas (e.g., historical sites, sensitive environmental areas).

It is advisable in the identification of potential sites to establish the evaluation criteria in conjunction with local officials; this will insure that local concerns are properly considered.

The LAND APPLICATION METHODOLOGY describes the evaluation process for a particular land-application/point-source combination. Factors that are considered in this evaluation include the water quality impact of the land disposal alternative, the wastewater transportation cost, and the land application site cost. The methodology computes these costs for several alternative land application sites.

It is assumed that all wastewater flow from the point source is applied to the land application site being evaluated. However, the methodology logic could be readily modified by the user to handle specific situations where wastewater from one point source is sent to several sites, or where only a portion of the point source discharge is sent to a land-application site.

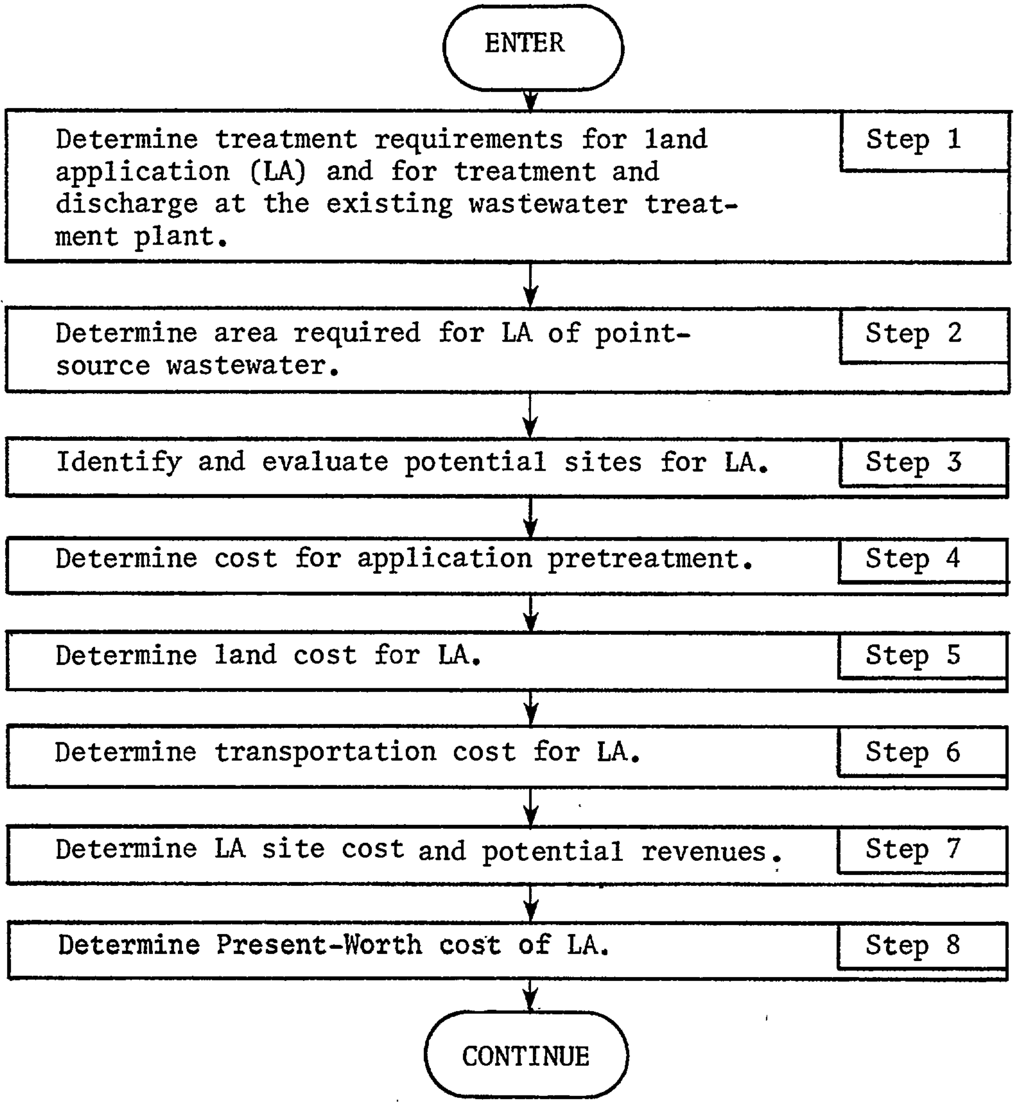
Finally, the user should be aware that much of the information, particularly with respect to potential land-application sites, should be readily available from earlier evaluations for the planning area. The user should utilize any such existing data to simplify the evaluation process.

Methodology Logic

A summary of the logic of the LAND APPLICATION METHODOLOGY is presented in Figure 6-11. An expanded flowchart, Figure 6-12, lists the steps to be taken in determining performance and costs. The worksheets for recording the operations are presented as Table 6-5. Notes concerning the methodology steps and worksheet items are presented after the worksheets.

FIGURE 6-11

LAND APPLICATION METHODOLOGY
LOGIC SUMMARY



Determine treatment requirements for land application (LA) and for treatment and discharge at the existing wastewater treatment plant.

Determine area required for LA of point-source wastewater.

Identify and evaluate potential sites for LA.

Determine cost for application pretreatment.

Determine land cost for LA.

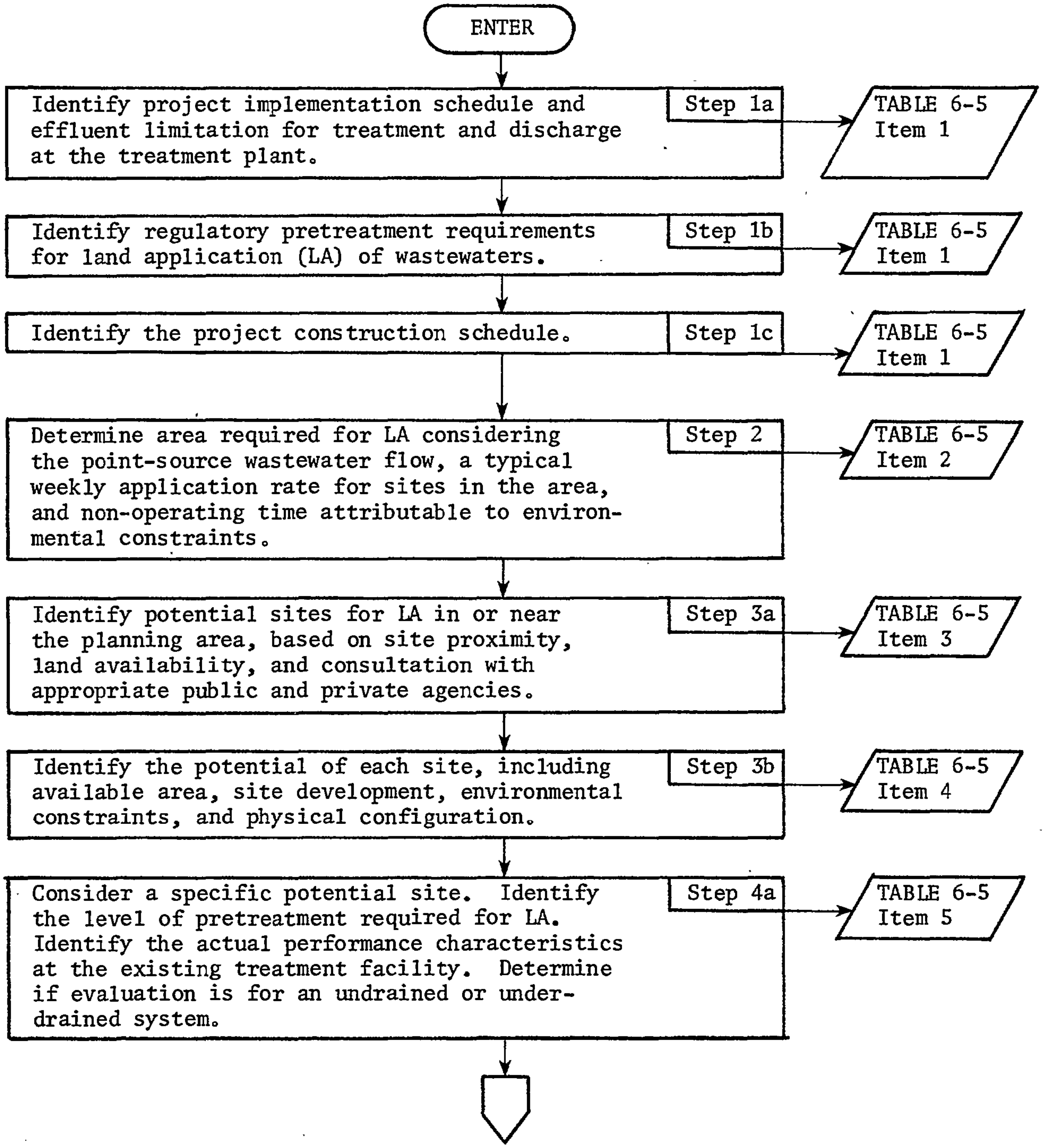
Determine transportation cost for LA.

Determine LA site cost and potential revenues.

Determine Present-Worth cost of LA.

FIGURE 6-12

LAND APPLICATION METHODOLOGY FLOWCHART



Identify project implementation schedule and effluent limitation for treatment and discharge at the treatment plant.

Identify regulatory pretreatment requirements for land application (LA) of wastewaters.

Identify the project construction schedule.

Determine area required for LA considering the point-source wastewater flow, a typical weekly application rate for sites in the area, and non-operating time attributable to environmental constraints.

Identify potential sites for LA in or near the planning area, based on site proximity, land availability, and consultation with appropriate public and private agencies.

Identify the potential of each site, including available area, site development, environmental constraints, and physical configuration.

Consider a specific potential site. Identify the level of pretreatment required for LA. Identify the actual performance characteristics at the existing treatment facility. Determine if evaluation is for an undrained or underdrained system.

FIGURE 6-12 (CONTINUED)

LAND APPLICATION METHODOLOGY FLOWCHART

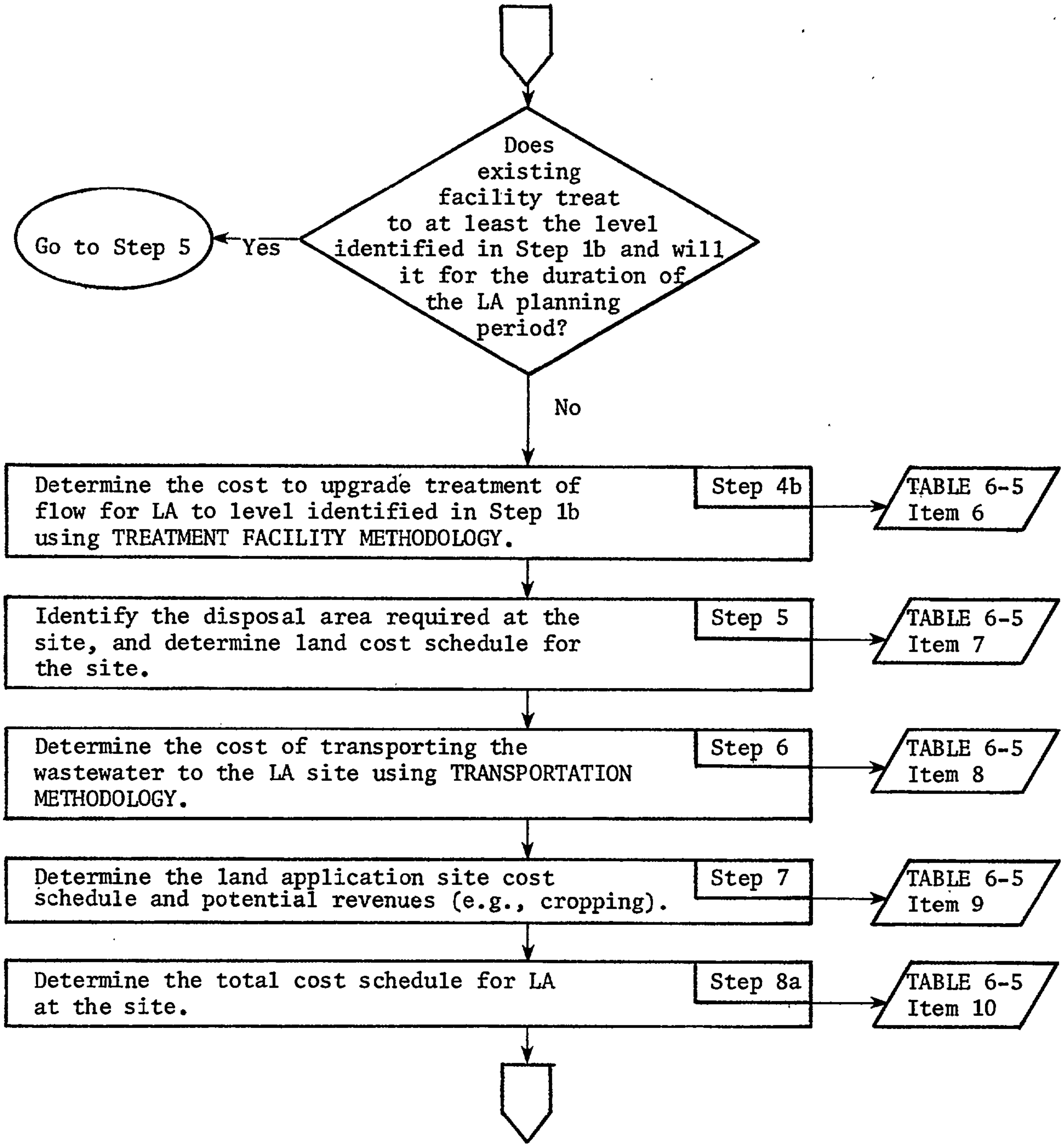


FIGURE 6-12 (CONTINUED)

LAND APPLICATION METHODOLOGY
FLOWCHART

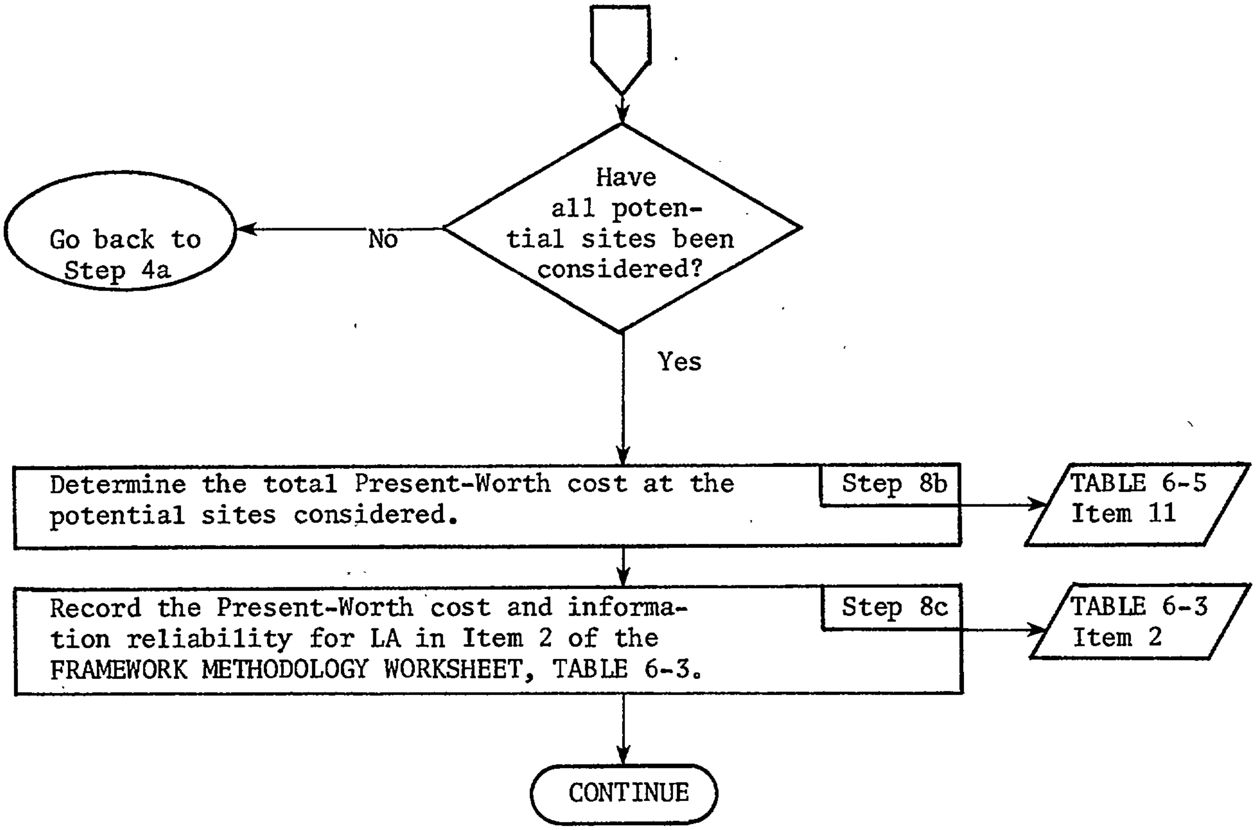


TABLE 6-5

LAND APPLICATION METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approach used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

PROJECT SCHEDULE

Item 1 - Program Implementation Schedule.

i. Planning Period: 20 years

ii. Existing Facility Conditions.

Design Capacity: ________ mgd
Years of Service: ________ years
Remaining Service: ________ years

iii. Treatment Levels.

Note: Dissolved oxygen deficits use ultimate oxygen demand inputs (Table 6-3). These must be reconverted back to CBOD and NBOD (NH_3) concentrations in order to determine discharge limitations (See Appendix H discussion of Treatment Systems Performance Ratios).

Level	Reference Cost Curve	Parameter Control Levels							
		BOD mg/l	COD mg/l	TSS mg/l	T-P mg/l	NH_3-N mg/l	NO_3-N mg/l	T-N mg/l	T-C #/100ml
Existing Facility									
Pretreatment									
Discharge Limitations									

iv. Construction Phases: ________________

		Design Flow (mgd)						
	Timing	Flow Projection		Pretreatment		Transportation	Site	
Phase	Year to Year	Start	End	Start	End		Start	End
1								
2								
3								
n								

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

GENERAL SITE EVALUATION

Item 2 - Land Application Ultimate Area Requirement.

i. Maximum Annual Flow Rate at end of Planning Period = ________mgd

ii. Application Rate[1] = ________in./week

iii. Non-operating time = ________weeks/year

iv. Area Required = ________acres, without buffer zone (includes area for roads, buildings, etc.)

Gross Area Required (with 200 ft buffer zone)[2]= ________acres

(Use Nomograph "Total Land Requirement", Figure 6-13, or equivalent)

---

[1] Items 2ii and 2iii can be used for determining maximum capacity for potential LA sites in Item 5.

[2] Use a more stringent buffer zone limitation if indicated by applicable Federal, State, or local regulations or site conditions.

By ______________ Date ______________ Strategy No. ______________
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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

Item 3 - Potential LA Sites - Location.

(Attach USGS Quad Sheet or equivalent with potential sites outlined and identified.)

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY WORKSHEET

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Item 4 - Potential LA Sites Data Sheet. (Sample of factors to be considered. This is not an inclusive list; see LA references for additional considerations.)

Site	Approximate Available area (acres)	Estimated treatment capacity (mgd)	Area of site presently irrigated (percent)	Distance from plant (feet)	Elevation difference from plant (feet)	Homes onsite (No.)	Other buildings onsite (No.)	Roads onsite (miles)	Comments-Major problems or advantages

TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

SPECIFIC SITE EVALUATION

SITE ____________

Item 5 - Site Implementation Schedule.

i. Planning Period: 20 years

ii. Construction Phases: __________
(If different from Project Schedule, then describe.)

iii. Pretreatment Requirements: Reference Cost Curve _____
(If different from Project Schedule, then describe.)

iv. Performance Characteristics - existing facility: Reference Cost Curve _____ (From Item 1iii)

Item 6 - Pre-application Treatment Cost.

i. Use Treatment Facility Methodology.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

Item 7 - Land Cost.

Application Rate = __________in./week
Curve Rate = __________in./week
Factor = Application Rate/Curve Rate = __________

Phase	Design[1] Flow	Adjusted[2] Flow	Land Cost	Salvage Value
1				
2				
3				
n				

[1] Design Flow is the desired application site daily capacity addition for the project Phase.

[2] Adjusted Flow = Design Flow x Factor

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

Item 8 - Transportation Cost.

i. Use Table 6-18, TRANSPORTATION COST METHODOLOGY to complete following schedule:

Phase	Capital	O&M		Salvage Value
		Start	End	
1				
2				
3				
n				

Replacement Schedule

Year	Cost

Item 9 - Application Site Costs.

i. Use cost curve in Appendix H, Figure H-16, or equivalent method:

Curve No. ________

Service Life ______

Phase	Timing	Flow Rate, mgd		Design, mgd	Capital Cost	O&M		Salvage[2] Value	Revenue[3]	
	Yr to Yr	Start	End			Start	End		Start	End
1										
2										
3										
n										

Replacement Schedule

Year	Cost

[1] Adjust curve cost to reflect installed cost.

[2] Develop Salvage Value = $\frac{\text{Service Life - Years to Planning End}}{\text{Service Life}}$ x Capital which reflects the remaining Phase value at the planning period end.

[3] Include crop revenues, etc.

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

Item 10 - Monetary Cost Evaluation

i. Cost Schedule.

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value	Revenues Start End
1		#6						
		#7						
		#8						
		#9						
	TOTAL PHASE 1							
2		#6						
		#7						
		#8						
		#9						
	TOTAL PHASE 2							
3		#6						
		#7						
		#8						
		#9						
	TOTAL PHASE 3							

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

Item 10 - Monetary Cost Evaluation (Continued).

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value	Revenues Start End
4		#6	______	______	______	______	______	______
		#7	______	______	______	______	______	______
		#8	______	______	______	______	______	______
		#9	______	______	______	______	______	______
			______	______	______	______	______	______
	TOTAL PHASE 4							

Replacement Schedule

Item	Year	Cost

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TABLE 6-5 (CONTINUED)

LAND APPLICATION METHODOLOGY WORKSHEET

PRESENT-WORTH COST EVALUATION

Item 11 - Present-Worth Cost.

Site	Present-Worth Cost	Reference Sheet

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Notes on Methodology Logic

Step 1a (Item 1i, ii, iii) - The project implementation schedule is developed by considering the projected wastewater quantity estimates for the planning area, existing facilities, state and Federal requirements, and local factors. In many cases, the user will wish to identify a phased program that is identical to that developed for the wastewater treatment facility evaluation, since this will allow a direct comparison of the monetary costs associated with each alternative during each phase. The effluent limitation for treatment and discharge should be available from a previous evaluation; if not, the user can develop the required control level using the water quality impact analysis techniques described in Chapter 5. The required level of treatment can then be determined: from the Treatment System Performance Matrix (Table H-3), from a synthesis of the unit process curves, or their equivalent.

Step 1b (Item 1iii, iv) - In this step, the requirements for land-application pretreatment are defined. The user should identify any regulations concerning the minimum acceptable pretreatment for land application in the planning area. Where appropriate, the characteristics of the land in the planning area should be considered in the determination of the required pretreatment level. In the general case, secondary treatment or equivalent should be considered the minimum that will insure adequate site performance.

Step 1c (Item 1iv) - This step identifies the construction program to be evaluated for the land application alternative. The various aspects of the project (pretreatment, transportation, application site) can have different design flows during the project, since these are evaluated individually. However, each of the individual project considerations should be keyed to the same phase timing to facilitate the development of the project schedule.

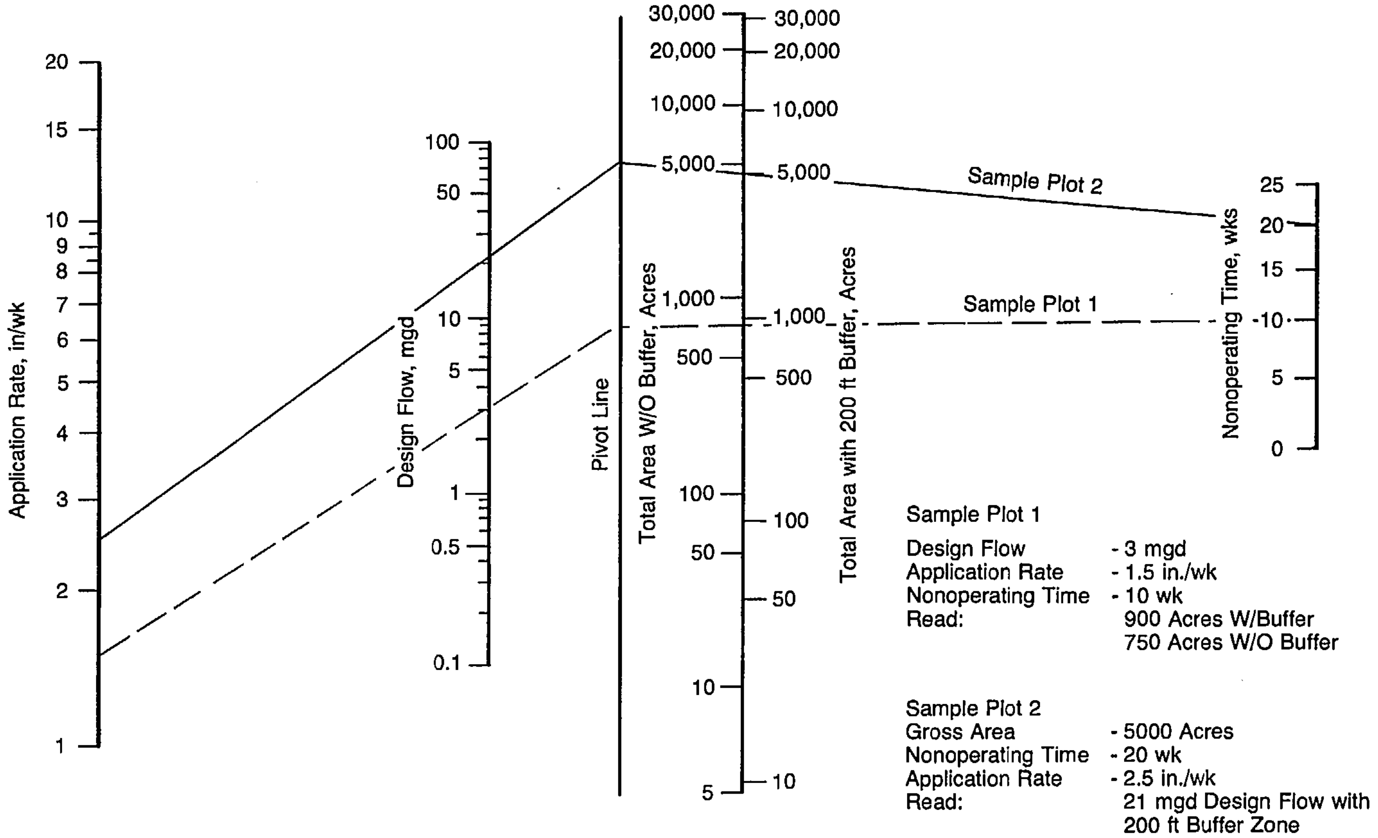
Step 2 (Item 2) - The purpose of this step is to identify the area requirements for a land-application site. The factors that should be considered for sites in the planning area include the design flow rate of the wastewater source, the allowable application rate (in terms of applied inches per week), and the required non-operating time. The design flow rate should be determined as an average flow, because storage facilities are usually provided at the land-application site. The user should identify any Federal, state, or local regulations concerning allowable application rates at land-application sites or siting requirements. The user should also consider the allowable application rate at existing land-application sites in the planning area. The non-operating time can be determined by considering the climate in the immediate area.

In addition, the user should identify buffer zone requirements for land application sites, as well as the area required for roads, buildings, and so forth. To assist the user in determining the area required for land application of the wastewater, a nomograph, Figure 6-13, has been provided. The required area, both with and without a buffer zone, should be identified, so that the user can identify the maximum capacity at a potential land-application site.

Step 3a (Item 3) - In this step, potential land-application sites are identified. There are numerous factors to be considered in identifying the site, and these are well documented in EPA guidance documents on land application. Characteristics of the potential sites include proximity to the wastewater source and adequate land availability, and should be evaluated utilizing the recommendations of appropriate public and private agencies. The potential sites should be shown on USGS quad sheets or equivalent topographic maps.

Step 3b (Item 4) - The potential sites identified in the previous step are further evaluated in this step. The maximum wastewater capacity at each site is determined using the nomograph "Total Land Requirement" (Figure 6-13) and the application rate and non-operating time previously identified (Item 2). However, the user can modify these typical numbers for any sites for which they would not be applicable. The additional information requested for each potential site should be available from an accurate topographic map, and should be useful in identifying the most feasible sites for a particular wastewater source.

Step 4a (Item 5) - In this step, one of the potential sites is selected for a cost evaluation. The planning period and project phases are evaluated and described in detail when they are different from the project schedule previously identified in Step 1a (Item 1). The application pretreatment level



Sample Plot 1

Design Flow	- 3 mgd
Application Rate	- 1.5 in./wk
Nonoperating Time	- 10 wk
Read:	900 Acres W/Buffer 750 Acres W/O Buffer

Sample Plot 2

Gross Area	- 5000 Acres
Nonoperating Time	- 20 wk
Application Rate	- 2.5 in./wk
Read:	21 mgd Design Flow with 200 ft Buffer Zone

Reference: Costs of Wastewater Treatment by Land Application,
EPA 430/9-75-003, June 1975

FIGURE 6-13 TOTAL LAND REQUIREMENT

should be identified and the Treatment System Performance Matrix (Table H-3) or a comparable method utilized to determine required treatment levels. The user should identify whether an underdrained or undrained site is to be evaluated based upon the site conditions, results of water quality impact analysis in Chapter 5, and other site-specific factors. If desired, the evaluation could be performed for both types of systems.

Step 4b (Item 6) - In this step, the user will identify the cost schedule for wastewater pretreatment prior to land application, where the existing facility is currently inadequate or will become inadequate (due to increased wastewater flow, etc.) during the planning period. The TREATMENT FACILITY METHODOLOGY should be utilized to develop this cost schedule.

Step 5 (Item 7) - In this step, the user will identify the area required at the disposal site for each project phase. The cost of land can be determined by the user on the basis of the average land cost in the area, which, if necessary, can be obtained from the tax assessor. This step can be eliminated when the total cost curve of Appendix H, Figure H-16, (which includes land cost) is used, but only if the assumptions used to develop that curve are valid for the system.

Step 6 (Item 8) - The cost to relocate the wastewater from the existing discharge point to the proposed land-application site is determined in this step. This can be determined using the TRANSPORTATION COST METHODOLOGY which will identify the pipeline and pumping cost for transporting the wastewater.

Step 7 (Item 9) - In this step, the cost schedule for developing and expanding the land-application site is determined in accordance with the desired project phasing. When evaluating land application as a control alternative, the cost associated with the underdrain system should also be developed for the site. (Note: Figure H-16 in Appendix H is for undrained systems only.) Also, potential revenues should be identified, including cash crops, etc. Although the land-application cost curve includes an assumed storage requirement and land-application rate, the user can modify the capital cost estimate to reflect the conditions at a specific site. In general, this adjustment is proportional to the ratio of the site condition to the design condition. For example, the cost for a 2.5 inch/week application site would be 2.0/2.5 times the cost described in the construction cost curve, which assumes a 2.0 inch/ week rate. The construction cost developed from the curve does not include estimated costs for piping, electrical, instrumentation, construction, engineering, or contingencies; these are figured separately using assumptions in the introduction to Appendix H or site-specific data. Typically, the user can identify these costs using a method similar to that described in the TREATMENT FACILITY METHODOLOGY.

Step 8a (Item 10) - In this step, the cost schedules identified in Steps 4, 5, 6, and 7 (Worksheet Items 6, 7, 8, 9) are combined to represent the project cost schedule of the land-application site.

Step 8b (Item 11) - The Present-Worth cost of this site is then determined using the PRESENT-WORTH METHODOLOGY. Note that revenues represent a negative cost and should be entered into this calculation as such.

6.4.2.5 Land Management Methodology

Discussion

Land management control alternatives have great potential for cost-effective water pollution control. However, the implementation of these control alternatives has been severely hindered because of the lack of information both in regard to documentation of effective performance and in regard to cost. Furthermore, implementation of land management decisions often has an impact on or requires close and continuing interface with local institutions and social customs, and the reactions and/or resistance are difficult to anticipate.

Quantitative data on performance and cost for the few land management control alternatives for which such information is available are included in Appendix G. Additional data on these alternatives and data on other alternatives not currently included in Appendix G will become available when further development of the relatively new field of land management will provide better understanding and documentation of its concepts and applications.

Methodology Logic

A summary of the logic of the LAND MANAGEMENT METHODOLOGY is presented in Figure 6-14. An expanded flowchart, Figure 6-15, lists the steps to be taken in determining performance and cost. The worksheets for recording the operations are presented as Table 6-6. Notes concerning the methodology steps and worksheet items are presented after the worksheets.

FIGURE 6-14

LAND MANAGEMENT METHODOLOGY
LOGIC SUMMARY

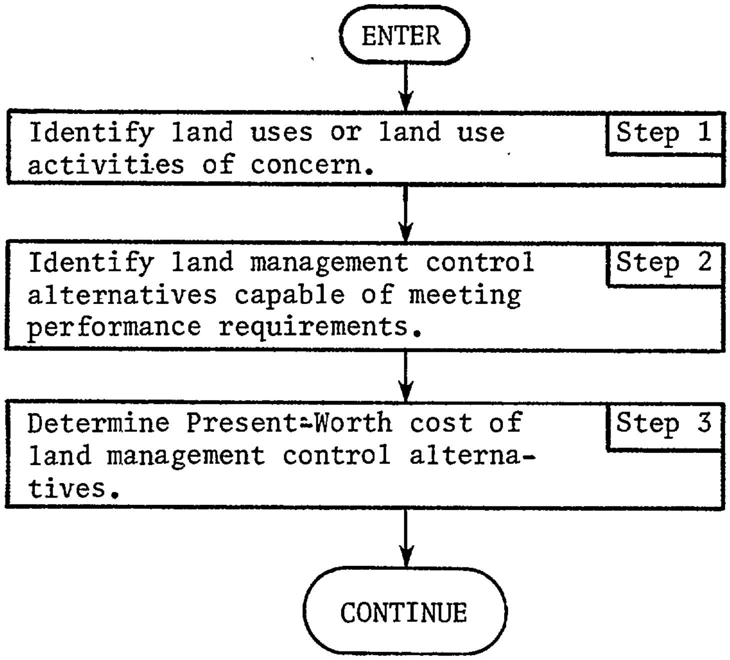


FIGURE 6-15

LAND MANAGEMENT METHODOLOGY
FLOWCHART

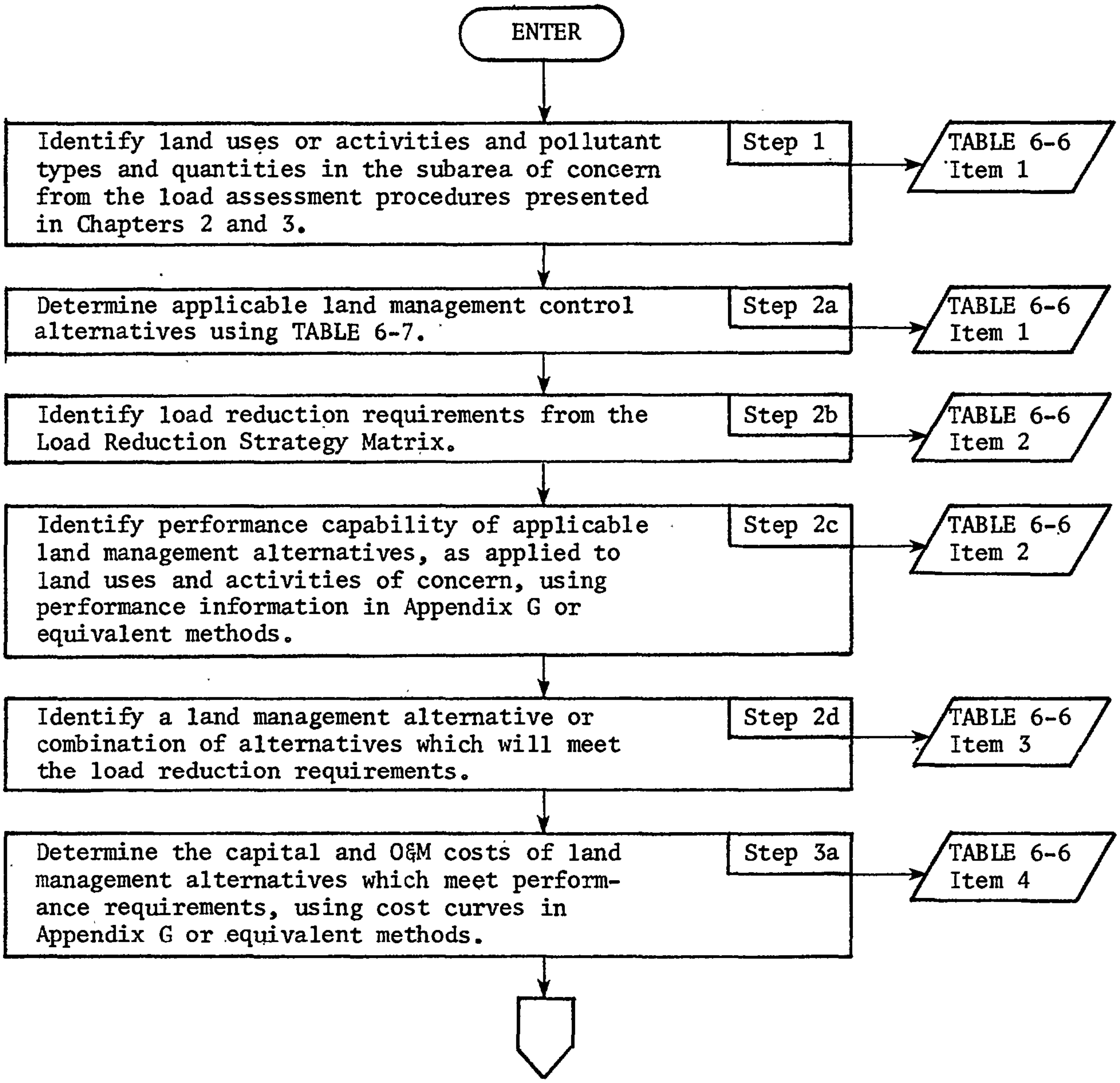
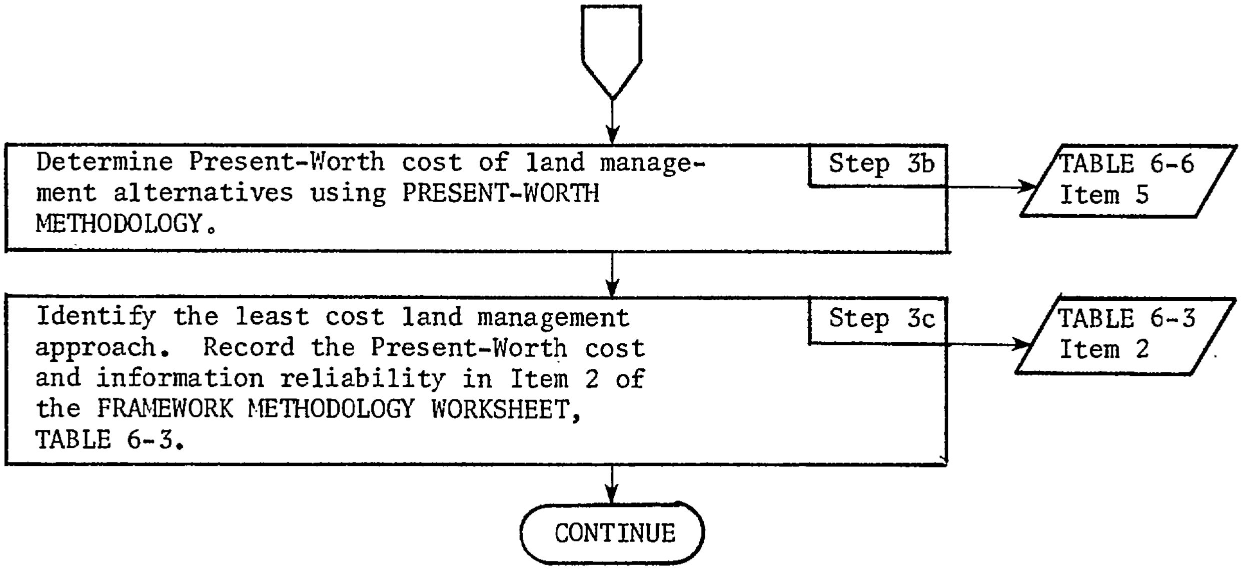


FIGURE 6-15 (CONTINUED)

LAND MANAGEMENT METHODOLOGY
FLOWCHART



Determine Present-Worth cost of land management alternatives using PRESENT-WORTH METHODOLOGY.

Identify the least cost land management approach. Record the Present-Worth cost and information reliability in Item 2 of the FRAMEWORK METHODOLOGY WORKSHEET, TABLE 6-3.

TABLE 6-6
LAND MANAGEMENT METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgements presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 Land uses and land use activities of concern.

Land Uses and Land Use Activities[1]	Wasteload					Applicable Land Management Alternatives
	BOD_5	N	P	TSS	TC[2]	

Item 2

	Percent Reduction				
	BOD_5	N	P	TSS	TC
i) Load reduction requirements:					

ii) Land management alternative performance capability.

Land Uses and Land Use Activities	Applicable Land Management Alternative	Performance Range-Percent Reduction				
		BOD_5	N	P	TSS	TC

[1] See Table 6-7.

[2] Most probable number/100 ml

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TABLE 6-6 (continued)
LAND MANAGEMENT METHODOLOGY WORKSHEET

Item 3 Identification of alternatives which will achieve required reduction.

Land Uses and Land Use Activities	Land Management Alternatives	Parameter	[Waste-load] x	[Performance Capa.] =	[Waste-load Red.]

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TABLE 6-6 (continued)
LAND MANAGEMENT METHODOLOGY WORKSHEET

Item 4 Capital and O&M Cost (From Appendix G).

Land Management Alternative	Affected Land Use/Activity (Acres)	Capital Cost	O&M Cost

Item 5 Present-Worth cost of land management alternatives.

Land Management Alternatives	Present-Worth Cost	Information Reliability[1]	
		Performance	Cost

[1] From Appendix G

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Notes on Methodology Logic

Step 1 - The various land uses and land use activities, and the pollutant loads from these land uses and activities are identified using the Load Assessment Methodology of Chapters 2 and 3.

Step 2a - Land Management control alternatives applicable for controlling land uses and activites of interest are listed in Table 6-7. The pollutants which are significantly affected by the application of these control alternatives to the land uses and activities listed are also presented in the table. Further explanation of the nature, effectiveness, etc., of the various techniques is presented in Appendix G, along with performance and cost information.

Step 2d (Item 3) - In this step, the actual pollutant reductions which can be expected by application of Land Management control alternatives to the land uses and activities of concern are calculated. Based on the load-reduction requirements recorded in Item 2, the user can identify a land use activity for potential control. Based on the amount of reduction achieved by the various Land Management control alternatives applied, if one alternative does not produce the required reduction for any particular parameter, various combinations of alternatives can be considered. A combination which achieves the required reductions for all parameters specified in the load-reduction strategy should be selected. If more than one alternative or combination of alternatives could be used to control a particular parameter or parameters, both should be carried forward to the cost-determination steps in order to choose the control alternative that would be most effective on the basis of monetary cost.

TABLE 6-7
LISTING OF LAND MANAGEMENT CONTROL ALTERNATIVES APPLICABLE TO DIFFERENT LAND USES AND LAND USE ACTIVITIES[1]

Land Use or Land Use Activity	Applicable Land Management Control Alternatives	Principal Pollutants Affected				
		BOD_5	N	P	TSS	TC
High Density Residential	Street Sweeping	X			X	
Medium Density Residential	Street Sweeping	X			X	
	Sediment and Erosion Control	X	X	X	X	
	Septic Tank Management	X	X	X		X
Low Density Residential	Sediment and Erosion Control	X	X	X	X	
	Septic Tank Management	X	X			X
Other Developed Areas	Sediment and Erosion Control	X	X	X	X	
Commercial Areas	Street Sweeping	X			X	
Industrial Sites	Site Runoff Controls	X	X	X	X	
Landfill Sites	Operation Regulations	X	X			
	Design Practices	X	X			X
New Development	Sediment and Erosion Control	X	X		X	
	Zoning/Subdivision Regulation	X	X	X	X	X
	Site Design Restrictions	X	X	X	X	

---

[1] Limited to consideration of urban non-point sources.

6.4.2.6 Collection System Control Methodology

Discussion

The purpose of investigating collection system control alternatives is to determine whether such controls can reduce the runoff-pollutant load sufficiently to meet the desired water quality objectives. These controls may be employed to reduce the quantity of storm runoff which overflows to a stream during a storm event, or to reduce the pollutant loading of this runoff. Maximizing the use of the existing sewer system for storage of runoff will reduce the volume of overflow which must be stored or treated external to the system during the storm event. Also, this will enable the plant to handle the storm runoff over a longer period of time, and thus reduce the occurrence of overflows. Increasing the conveyance capability of the collection system can reduce the frequency of overflows by making the system better able to handle storm flows. Regular flushing of the sewer system helps to minimize the accumulation of material which settles out of the dry-weather flow and then is subject to scouring and discharge during the higher wet-weather flows.

Existing data show that maximizing in-line storage and conveyance along with the use of selective sewer flushing are effective techniques in reducing pollution attributable to sewer overflows. In addition, their costs are minimal compared to other control alternatives, and they should be seriously considered in situations where control of combined or storm sewer overflows is desirable.

Other collection-system control alternatives are available. In some (e.g., catch-basin cleaning, improved sewer design and maintenance, inflow control), the impact on alleviating overflow pollution is less certain than with the techniques already mentioned. Others (sewer separation, sewer rehabilitation) are partially effective, but are also much more costly. Information concerning these control alternatives, as well as on those already discussed, is included in Appendix G; both cost and performance data are presented where available.

The intent of the COLLECTION SYSTEM CONTROL METHODOLOGY is to guide the user in selecting control alternatives which have the best chance of providing significant pollution control at a minimum cost. Much of the cost and performation on these alternatives is still being developed and thus cannot be presented in this document. Where this is the case, or where assessment or implementation methodologies are presented in current literature, the highlights of the approach are presented here, and references are made to specific documents for further information.

Methodology Logic

A summary of the logic of the COLLECTION SYSTEM CONTROL METHODOLOGY is presented in Figure 6-16. An expanded flowchart, Figure 6-17, lists the steps to be taken in determining performance and costs. The worksheets for recording the operations are presented as Table 6-8. Notes concerning the methodology steps and worksheet items are presented after the worksheets.

FIGURE 6-16

COLLECTION SYSTEM CONTROL METHODOLOGY
LOGIC SUMMARY

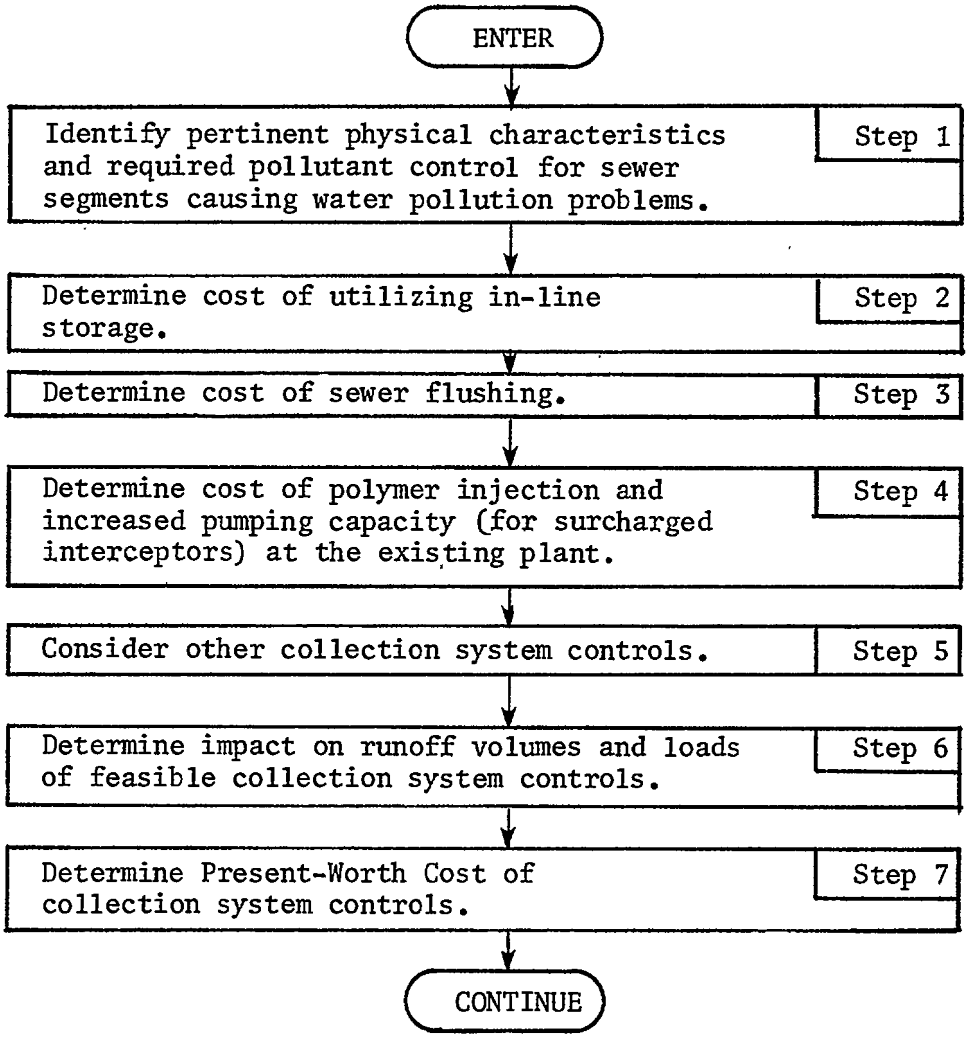
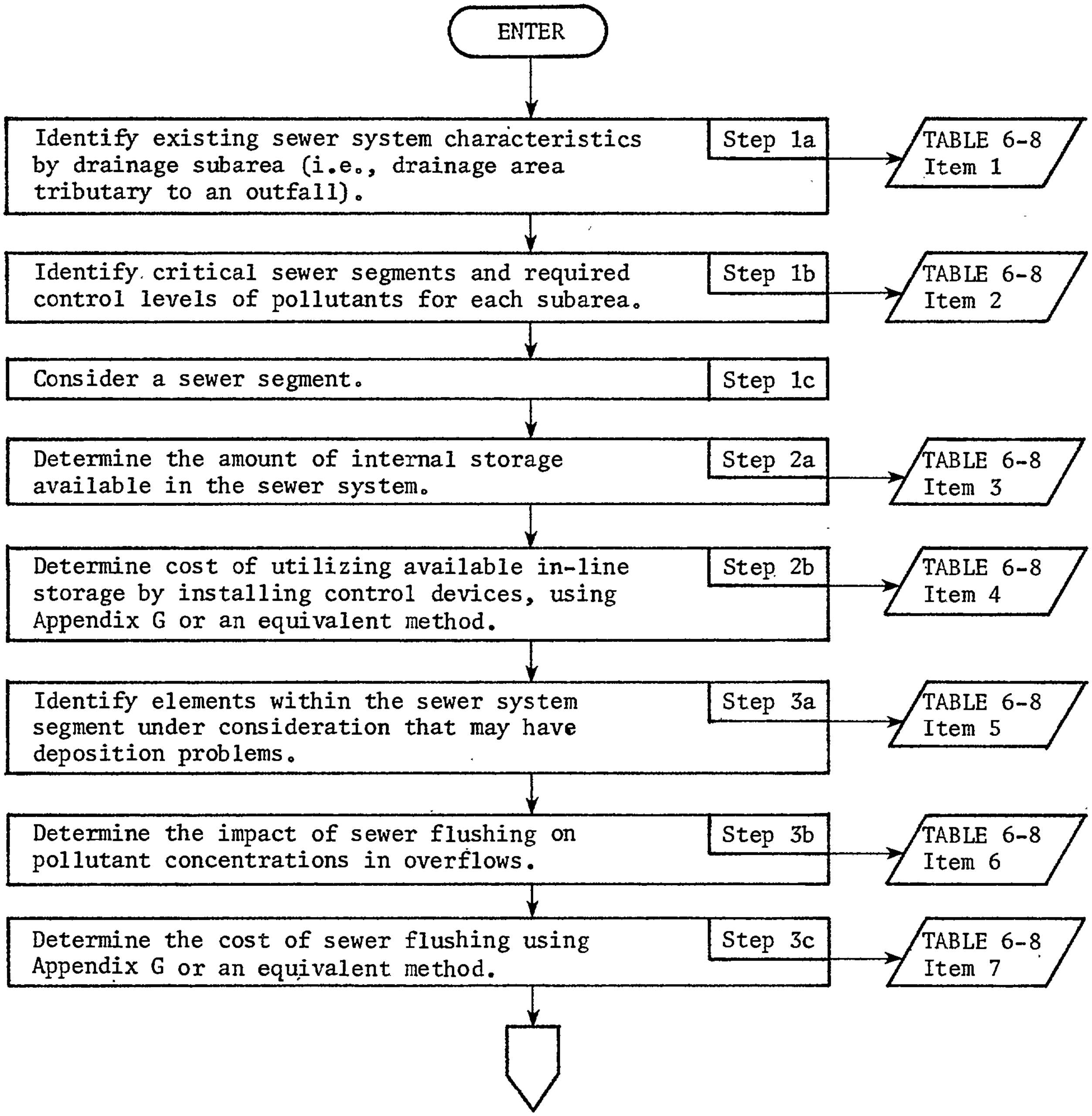


FIGURE 6-17

COLLECTION SYSTEM CONTROL METHODOLOGY FLOWCHART



Identify existing sewer system characteristics by drainage subarea (i.e., drainage area tributary to an outfall). Step 1a

Identify critical sewer segments and required control levels of pollutants for each subarea. Step 1b

Consider a sewer segment. Step 1c

Determine the amount of internal storage available in the sewer system. Step 2a

Determine cost of utilizing available in-line storage by installing control devices, using Appendix G or an equivalent method. Step 2b

Identify elements within the sewer system segment under consideration that may have deposition problems. Step 3a

Determine the impact of sewer flushing on pollutant concentrations in overflows. Step 3b

Determine the cost of sewer flushing using Appendix G or an equivalent method. Step 3c

FIGURE 6-17 (CONTINUED)

COLLECTION SYSTEM CONTROL METHODOLOGY
FLOWCHART

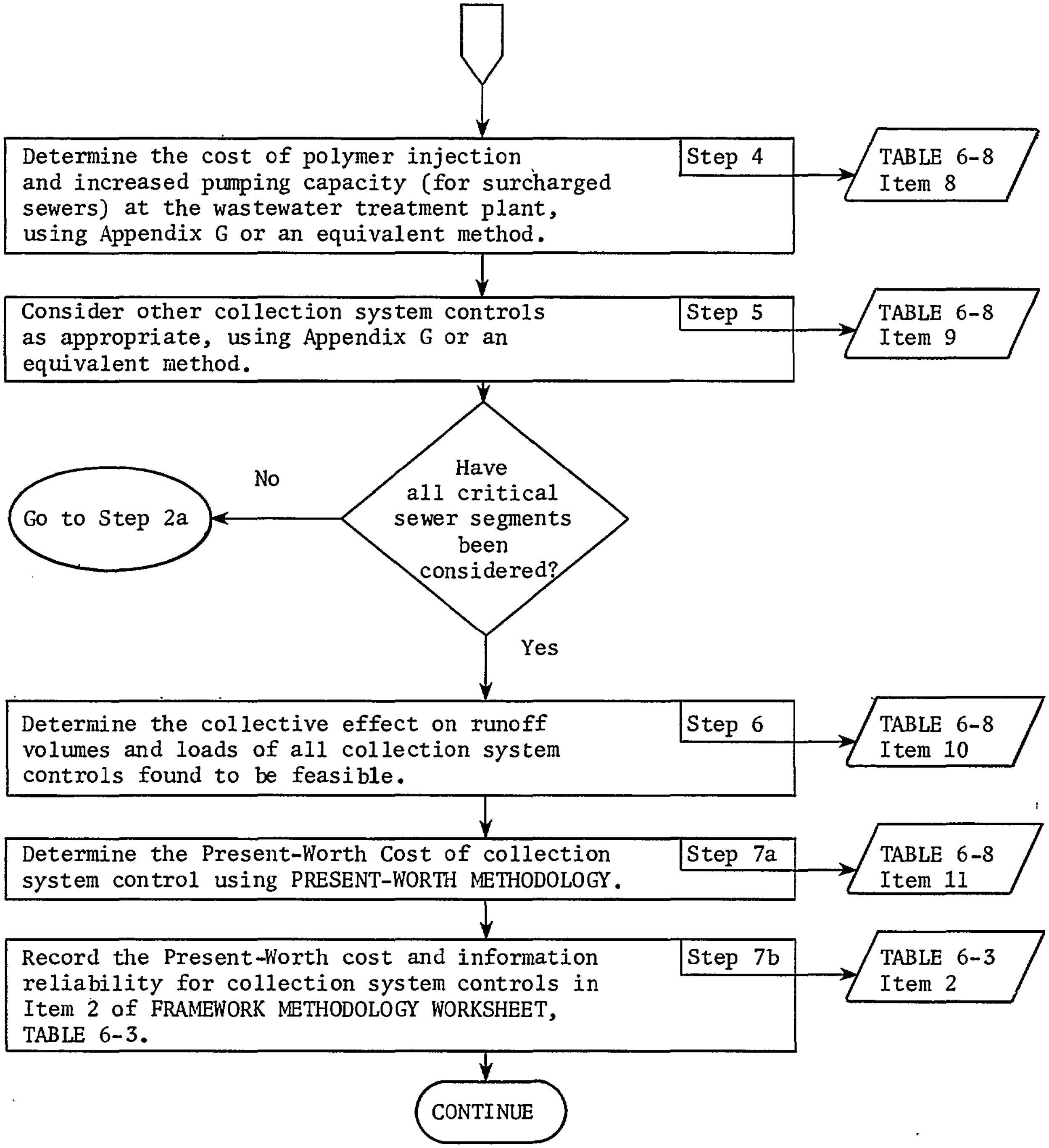


TABLE 6-8
COLLECTION SYSTEM CONTROL METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State and local guidelines and regulations.

Item 1 Sewer System Characteristics.

Outfall No.	Subarea Location[1]	Sewer Segment Type[2]	Type of Overflow Control Device[3]
1			
2			
3			
4			
5			

[1] Locations should be referenced to a map using outfall and subarea numbers.

[2] Combined sewer, storm sewer or unsewered.

[3] Swirl separator or conventional regulator.

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 2 Pollutants (BOD_5, TSS, TC, P, N) to be controlled (from Load Reduction Strategy Matrix developed in Chapter 6.

Segment No.	Pollutant Parameter	Required % Reduction

Item 3 In-line storage volume.

Segment No.	Hydraulic Capacity	Dry Weather Flow	Internal Storage Capacity

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 4 Cost of utilizing available in-line storage from cost information information in Appendix G, or an equivalent method.

Segment No.	Type of Control[1]	Construction Costs	O&M Cost	Total Present Worth Costs[2]

Item 5 Sewer segments with deposition problems.

Segment No.	Extent of Problem	Source of Problem[3]

[1] Weir, gate, etc.

[2] Using PRESENT-WORTH METHODOLOGY

[3] Obstructions, slack velocity, etc.

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY WORKSHEET

Item 6 Impact of sewer flushing on pollutant concentrations in overflows.

Outfall No.	Segment No.	Pollutant Parameter	Concentration	
			Before	After

Item 7 Cost of sewer flushing using cost information in Appendix G, or an equivalent method.

Segment No.	O&M	Total Present-Worth Costs

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 8 Cost of measuring conveyance capability using cost curves in Appendix G or an equivalent method.

i. Polymer injection costs.

Segment No.	Construction Costs	O&M Costs	Total Present-Worth Costs[1]

ii. Increased pumping capacity at the wastewater treatment plant where influent interceptor is surcharged.

Existing Capacity	Increased Capacity	Construction Cost	O&M Costs	Total Present-Worth Costs[1]

---

[1] Using PRESENT-WORTH METHODOLOGY

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 9 Cost of other collection system controls as appropriate, using Appendix G or an equivalent method.

Segment No.	Type of Control	Construction Costs	O&M Costs	Total Present-Worth Costs[1]

[1] Using PRESENT-WORTH METHODOLOGY

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 10 Collective effect on runoff volumes and loads of all collection system controls found to be feasible.

Segment No.	Control Alternative	% Runoff Controlled	Pollutant Parameter	Load Reduction Achieved

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 11 Summary of feasible collection system control alternatives.

Collection System Control	Present-Worth Costs

Total ____________________

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Notes on Methodology Logic

Step 1a (Item 1) - Pertinent sewer-system and drainage subarea characteristics are discussed in Section 3.4.4 (Chapter 3), including presentation of a simulation technique for the assessment of storm loads and the effect of control measures. The technique described in Section 3.4.4 is a summary of a methodology presented in reference (2). The user is referred to this reference for more information on this simulation technique.

Step 1b (Item 2) - Pollutants to be controlled and the degree of reduction required are specified in the Load-Reduction Strategy Matrix developed in Chapter 6.

Step 2a (Item 3) - It is often possible to use the available internal storage capacity of sewer systems more effectively by installing flow-regulation devices to retain storm runoff or by routing runoff flows through the sewer system so as to maximize detention time. The first step in attempting to reduce the overflows by using these methods is to determine the hydraulic capacities of existing sewer interceptors and the magnitude of the dry-weather flow. The difference between these two values is an indication of the capacity in the interceptor which might be available for internal storage. The planner can use this information, along with information on slopes of the sewer lines, to determine where the control devices could be placed to best utilize the existing storage capacity. Detailed application of this alternative depends on the specific area being examined and, thus, this methodology must remain in general terms. Users are referred to references (2), (3), (4), (5), (6), (7).

Step 2b (Item 4) - In most cases, the cost of in-line storage will be the cost of installing regulator devices within the sewer system to utilize existing available capacity. For sophisticated systems, costs will also include remote-control instrumentation and automation.

Step 3a (Item 5) - Certain parts of a sewer system may be subject to the settling of the solids contained in the dry-weather flow. In most cases, this is because the slope is insufficient to maintain a velocity which will keep the solids in suspension. These are candidates for a sewer-flushing program which will remove the deposited material during dry-weather periods.

This will help prevent later scouring of these solids, which adds to the pollutant loading of the first flush during a storm.

Step 4 (Item 8) - The conveyance capability of a sewer system can be improved by several methods, including polymer injection and increasing pumping capacity of the influent pump station where the interceptor to the plant is surcharged. Polymer injection during storms into sewer segments subject to surcharging can reduce hydraulic friction thus increasing carrying capacity and possibly eliminating the surcharge condition. Also, increased pumping rates at the head of a treatment plant in anticipation of a storm event will create more capacity within the sewer system for conveyance and storage of storm flows.

6.4.2.7 Storage/Treatment Methodology

Discussion

The storage and treatment of combined and storm sewer overflows has received much attention in recent years. New approaches and new technologies are being developed on a continuing basis. The feasibility, cost, and performance of storage/treatment alternatives are in some cases unproven or uncertain, and usually are highly dependent on site-specific factors, such as location of overflow points in relation to existing interceptors and treatment facilities, available capacity of wastewater treatment units and sludge handling facilities, and rainfall and runoff characteristics.

The following methodology is presented: to aid the user in recognizing the aspects of the problem that must be addressed; to provide available information in some of the key factors to be considered; and to suggest alternative approaches for handling sewer overflow problems by using storage and treatment. Again, the user should keep in mind that the alternatives suggested are not necessarily the only ones that might prove effective.

Storage/treatment control alternatives are the last resort in controlling storm overflows. Because they usually involve the construction of facilities, their cost is higher than the other wet-weather control alternatives. However, it is often necessary to resort to them, especially in large basins, because it is difficult to control the large volumes of water and high pollutant loads using land management or collection system controls alone.

For an understanding of the state-of-the-art in urban runoff pollution control technology and programs currently underway, the user should consult pertinent EPA documents on urban runoff and urban stormwater management, including references (8) and (9).

Methodology Logic

A summary of the methodology logic is presented in Figure 6-18. An expanded flowchart, Figure 6-19, lists the steps to be taken in determining performance and costs. The worksheets for recording the operations are presented as Table 6-9. Notes concerning the methodology steps and work sheet items are presented after the worksheets.

FIGURE 6-18

STORAGE/TREATMENT METHODOLOGY
LOGIC SUMMARY

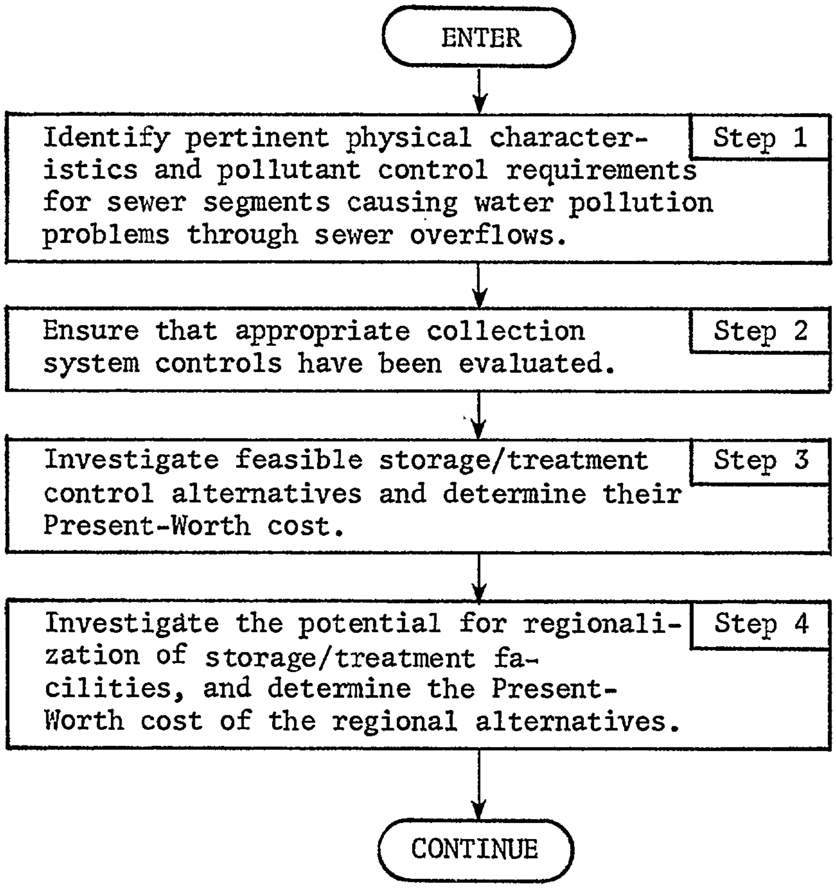
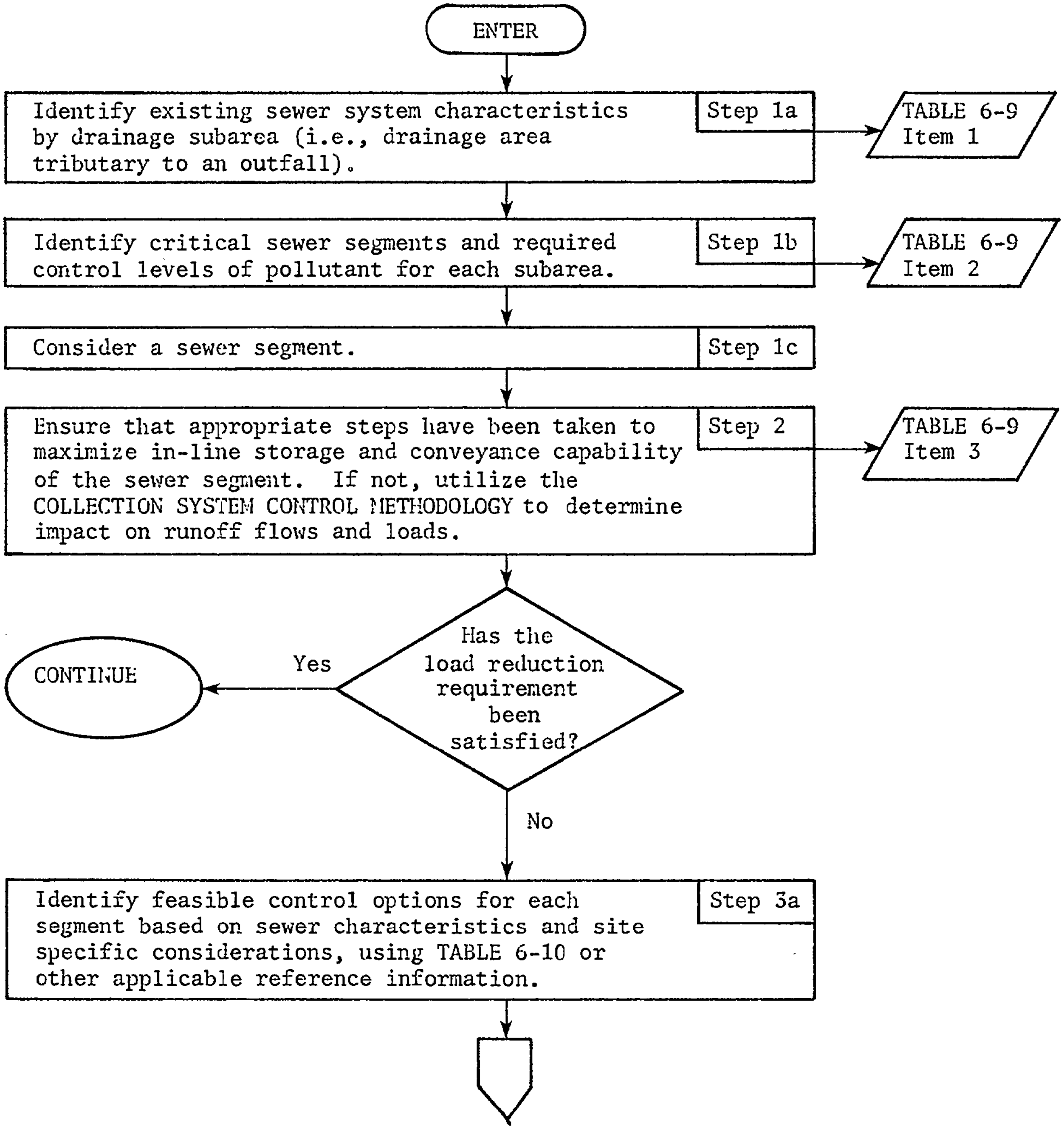


FIGURE 6-19

STORAGE/TREATMENT METHODOLOGY
FLOWCHART



Identify existing sewer system characteristics by drainage subarea (i.e., drainage area tributary to an outfall).

Identify critical sewer segments and required control levels of pollutant for each subarea.

Consider a sewer segment.

Ensure that appropriate steps have been taken to maximize in-line storage and conveyance capability of the sewer segment. If not, utilize the COLLECTION SYSTEM CONTROL METHODOLOGY to determine impact on runoff flows and loads.

Has the load reduction requirement been satisfied?

Identify feasible control options for each segment based on sewer characteristics and site specific considerations, using TABLE 6-10 or other applicable reference information.

FIGURE 6-19 (CONTINUED)

STORAGE/TREATMENT METHODOLOGY
FLOWCHART

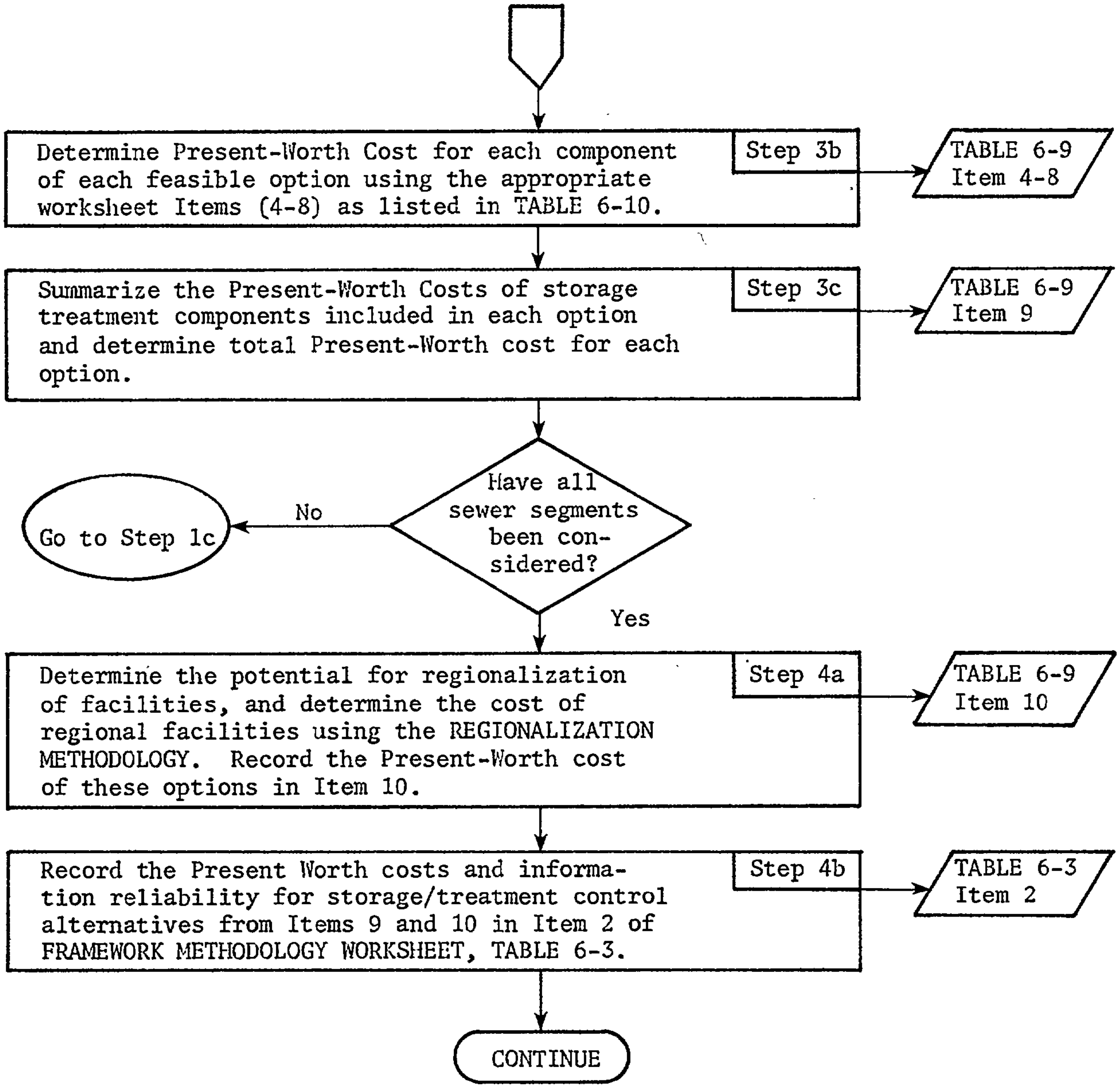


TABLE 6-9
STORAGE/TREATMENT METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 Sewer System Characteristics.

Outfall No.	Subarea Location[1]	Sewer Segment Type[2]	Type of Control Device[3]

[1] Locations should be referenced to a map using Outfall and Subarea numbers.
[2] Combined sewer, storm sewer, or unsewered.
[3] Swirl separator or conventional regulator.

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 2 Pollutant Parameters (BOD_5, TSS, TC, P, N).

Segment No.	Pollutant Parameter	Required % Reduction
________	________	________
________	________	________
________	________	________
________	________	________
________	________	________

Item 3 Results of collection system controls.

i. Quantity of design storm runoff volume stored in internal storage of collection system ________ mg.

ii. Remaining runoff volumes and load:

Volume: ________ mg.
Flow: ________ mgd.
Load: ________ mg/1 BOD_5
________ mg/1 TSS
________ mg/1 P
________ mg/1 N
________ #/100 ml

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 4 i. Cost of Regulator, from information in Appendix G, or equivalent method.

Construction Cost $ ____________

O&M Cost $ ____________

Total Present-Worth Cost $ __________________
(using PRESENT-WORTH METHODOLOGY)

ii. Cost of swirl separator, using Table 6-11, or equivalent method.

Design flow ____________

Construction Cost $ ____________

O&M Cost $ ____________

Total Present-Worth Cost $ __________________
(using PRESENT-WORTH METHODOLOGY)

Item 5 Cost of storage tanks, from Table 6-11, or equivalent method.

Type of storage: Settling ____________ Complete mix ____________

Construction Cost $ ____________

O&M Cost $ ____________

Total Present-Worth Cost $ __________________
(using PRESENT-WORTH METHODOLOGY)

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6

i. Design Storm Characteristics.

Intensity ____________ in/hour

Duration ____________ hour

Frequency ____________ /year

ii. Inlet hydrograph(s) for design storm (storm runoff entering the sewer system).
Method ________________________________

Sub-area ____		Sub-area ____		Sub-area ____	
TIME	FLOW	TIME	FLOW	TIME	FLOW

NOTE: Plot hydrographs for each subarea on separate sheets of graph paper.

By ______________ Date ______________ Strategy No. ______________
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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

iii. Inflow hydrograph to overflow·control structure.

Routing Procedure ____________________

TIME	FLOW

iv. Overflow hydrograph from the control structure.

(Attach rating curve for specific structure)

TIME	FLOW

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

v. Mass Curve from Overflow hydrograph.

TIME	INCREMENTAL VOLUME	CUMULATIVE VOLUME

NOTE: Plot Mass Curve on separate sheet of standard graph paper.

vi. Storage/Treatment requirements.

% MAXIMUM TREATMENT RATE	TREATMENT RATE	STORAGE VOLUME
100	________	0
75	________	________
50	________	________
25	________	________
0	0	________

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY WORKSHEET

Item 6 - Continued

vii. Cost of storage, using cost functions from Table 6-11, or equivalent method, PRESENT-WORTH METHODOLOGY.
Type of storage ________________________.

Volume to be Stored	Construction Cost	O&M Cost	Present Worth Cost

NOTE: Plot Volume to be stored and Present Worth Cost.

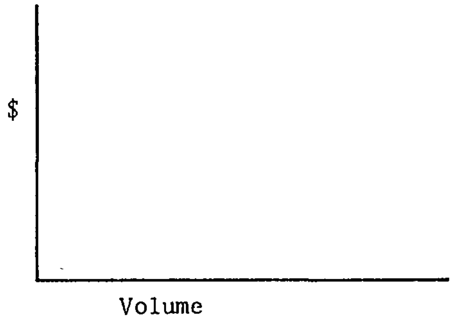

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

viii. Cost of on-site treatment, using cost functions from Table 6-11, or equivalent method, and PRESENT-WORTH METHODOLOGY.

Treatment Unit ____________________

Treatment Rate	Construction Cost	O&M Cost	Present-Worth Cost

Treatment Unit ____________________

Treatment Rate	Construction Cost	O&M Cost	Present-Worth Cost

Treatment Unit ____________________

Treatment Rate	Construction Cost	O&M Cost	Present-Worth Cost

NOTE: Plot a cost curve for each type of treatment unit on the same set of axes.

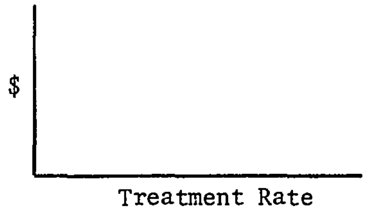

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

ix. Cost of treatment at the existing plant.

  a) Cost of discharging effluent to the interceptor and treating at existing plant.

    i. If interceptor is adjacent to overflow control device (as in combined sewer), no cost is associated with discharge to the interceptor.

    ii. If interceptor is not adjacent to the overflow control device, the cost to construct a sewer to transport the effluent can be determined using the TRANSPORTATION COST METHODOLOGY and PRESENT-WORTH METHODOLOGY.

Flow to be Transported and Treated	Sewer Construction		Present-Worth Cost
	Construction Cost	O&M Cost	

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

b) Costs of upgrading/expanding existing treatment facility using TREATMENT FACILITY METHODOLOGY and PRESENT-WORTH METHODOLOGY.

Treatment Rate	Construction Cost	O&M Cost	Present-Worth Cost

c) Total Present-Worth cost of treatment at existing facility.

Treatment Rate	Total Present-Worth Cost of Transportation and Treatment

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

x. Least-cost combination of storage and treatment.

Treatment Rate	Least-cost Treatment Unit	Present-Worth Cost of Treatment	Storage Volume	Present-Worth Cost of Storage	Total Present-Worth Cost

NOTE: Plot toal Present-Worth cost of storage and treatment versus treatment rate.

Least-cost combination of storage and treatment.

$ ____________

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 7

i. Cost of laying pipe to connect new regulator to existing outfall pipe (if significant), or cost of laying a new or larger outfall pipe, from cost curve in Appendix H, Figure H-84.

Construction Cost $ ____________

O&M Cost $ ____________

Total Present-Worth Cost $ ____________
(using PRESENT-WORTH METHODOLOGY)

ii. Cost of Disinfection (where required) from curve in Appendix H, Figure H-26.

Construction Cost $ ____________

O&M Cost $ ____________

Total Present-Worth Cost $ ____________
(using PRESENT-WORTH METHODOLOGY)

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 8 Cost of sludge handling.

i. On-site sludge handling.
  a. Sludge treatment (using cost curves in Appendix H).
    1) Organic sludges

Lime stabilization (Figure H-79)
Construction Cost $ ___________
O&M Cost $ ___________

Vacuum Filtration (Figure H-68)
Construction Cost $ ___________
O&M Cost $ ___________

2) Inorganic sludges

Vacuum Filtration (Figure H-69)
Construction Cost $ ___________
O&M Cost $ ___________

3) Subtotal $ ___________

b. Sludge transport (using cost curves in Appendix H, Figures H-86 through H-90).

Method: ____________________
Construction Cost $ ___________
O&M Cost $ ___________

c. Sludge disposal (using cost curves in Appendix H, Figures H-81 through H-83).

Method: ____________________
Construction Cost $ ___________
O&M Cost $ ___________

d. Total cost for on-site sludge handling.

Construction Cost $ ___________
O&M Cost $ ___________

e. Total Present-Worth Cost $ ____________________
(using PRESENT-WORTH METHODOLOGY)

ii. Sludge handling at existing wastewater treatment facility.
  a. Sludge transport to existing facility

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Remarks: ______________________________ Page ______

TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 8 - Continued

1) If sewer is storm sewer, determine cost to construct sewer to connect with sanitary sewer interceptor to treatment plant, using TRANSPORTATION COST METHODOLOGY.

   Construction Cost $ ___________
   O&M Cost $ ___________

2) If sewer is combined, existing interceptor capacity should be sufficient to transport sludges to the existing wastewater treatment plant.

b. Sludge treatment.

Determine if there is capacity available in existing sludge handling facilities to accept additional sludge volumes from the treatment of storm overflows. If not, determine cost to provide additional sludge handling capacity at the existing facility, using TREATMENT FACILITY METHODOLOGY.

Construction Cost $ ___________
O&M Cost $ ___________

c. Sludge disposal (using cost curves in Appendix H, Figures H-81, 82, or 83).

Method: ___________
Construction Cost $ ___________
O&M Cost $ ___________

d. Total cost for sludge handling at existing wastewater treatment facilities.

Construction Cost $ ___________
O&M Cost $ ___________

e. Total Present-Worth Cost $ ___________
(using PRESENT-WORTH METHODOLOGY)

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY WORKSHEET

Item 9 Total Present Worth Cost.

Sewer Segment ______________________ Sewer Type ______________________

i. For each storage/treatment option, record the Present-Worth costs of each component determined using Items 4-8.

ii. Determine the total Present-Worth cost of each option.

Present-Worth Cost (in $1,000)

Option #	Regulator	Swirl	Storage w/ Settling	Storage w/ mixing	On-site WW Treatment	Treatment at Existing Plant	Interceptor Construction	Disinfection	On-site Sludge Handling	Sludge Handling at Existing Plant	Total Present Worth Cost

By ______________ Date ______________ Strategy No. ______________
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Remarks: ____ Page ______________

TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 10 Regionalization of storage/treatment components, using REGIONALIZATION METHODOLOGY.

Components regionalized:

Regional Facility:

Construction Cost $ ____________

O&M Cost $ ____________

Present-Worth Cost $ ____________________
(using PRESENT-WORTH METHODOLOGY)

By ____________ Date ____________ Strategy No. ____________
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Notes of Methodology Logic

Step 1a (Item 1) - Pertinent sewer system and drainage area characteristics are discussed in Section 3.4.4 (Chapter 3), along with the presentation of a simulation technique for the assessment of storm loads and of the effect of control measures. The technique described in Section 3.4.4 is a summary of a methodology presented in the EPA document on Development and Application of a Simplified Stormwater Management Model (EPA-600/2-76-218). The user should consult this reference for more information on this simulation technique.

Step 1b (Item 2) - Pollutants to be controlled and the percentage reductions required are specified in the Load-Reduction Strategy Matrix (illustrated for the South River hypothetical example in Table 6-19).

Step 2a (Item 3) - The maximizing of storage and conveyance capability, as well as other collection system controls such as sewer flushing, was described in the COLLECTION SYSTEM CONTROL METHODOLOGY. It is mentioned again here to emphasize the importance of this alternative in controlling wet-weather flows. These alternatives are so effective in reducing sewer overflows, and have such relatively minor cost, that any consideration of storing and treating sewer overflows should be considered in conjunction with these collection-system controls. This step insures that the user has considered these alternatives, and is using a load which has been modified to take into account the effect of the collection system. If the total load-reduction requirement specified in the Load-Reduction Strategy Matrix has been achieved using the collection system controls, the user need go no further. If not, the user should continue with the STORAGE/TREATMENT METHODOLOGY and consider various storage/treatment control alternatives.

Step 3a - The identification of feasible control options for storage and/or treatment of sewer overflows should be based on the specific situation under investigation. Choices must be made concerning: the type of regulator to be used; the type and capacity of the storage device; the type, degree, and location of treatment required; the type, capacity, and location of sludge-handling facilities; and numerous other factors. The following paragraphs are provided to aid the user in addressing some of these decisions. This discussion should be viewed as a guide only, and is not intended to be prescriptive in any way. Table 6-10 presents feasible combinations of the storage/treatment components already mentioned, and lists the worksheet item number which can be used to obtain a Present-Worth cost of the components based on the flow and load information from Steps 1 and 2. The user should consult the following discussion and other applicable references to aid in determining which control option is applicable to his situation.

Flow Regulation - In determining the type of flow regulation to provide at a sewer overflow location, it will be necessary first to determine the types of regulators already in the system. Of course, storm sewers will not have regulators, because the total flow in a storm sewer is discharged to the receiving water. Combined sewers, in most cases, will have a conventional type of regulator device of varying degrees of sophistication, ranging from a simple weir to a dynamic device which has moving parts and can be controlled from remote locations. However, a significant number of unattended regulators regularly malfunction because parts are frozen in an undesirable position, or pieces are broken off, often allowing a continual dumping of dry-weather flow to the receiving water. Thus, it is probable that a new regulator device or repair of existing regulators will be necessary in many cases.

The user may consider installing either a conventional regulator (which will provide flow regulation alone) or a swirl regulator/separator device (which will provide flow regulation plus solids removal). The swirl device can be designed to provide solids separation up to a certain design flow and then to bypass additional flows.

As far as the subsequent storage or treatment components are concerned, the basic difference between the swirl separator and the conventional regulator is that the swirl separator provides both an effluent flow (light fraction) and a more concentrated underflow, one or both of which may be handled by the storage/treatment components. The conventional regulator merely bypasses those flows which are in excess of the hydraulic capacity of the existing interceptor and/or wastewater treatment plant.

Storage - Off-line storage of the sewer overflow in the vicinity of the overflow location can be either of two basic types: storage with settling,

TABLE 6-10
WETWEATHER FLOW STORAGE/TREATMENT CONTROL OPTIONS

Storage/Treatment Components		Worksheet Item No.	Control Options																								
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Sewer Type	Combined Sewer	1	X	X	X	X	X	X	X	X	X	X	X														
	Storm Sewer	1												X	X	X	X	X	X	X	X	X	X	X	X	X	X
Regulation	Conventional Regulator	4							X	X	X	X	X										X	X	X	X	X
	Swirl Separator	4	X	X	X	X	X	X						X	X	X	X	X	X	X	X	X					
Storage	Storage with Settling	5		Es	Es				Or	Or					Ss				Es	Es			Or	Or			
	Storage with Complete Mix	5				Es	Es	Es			Or	Or	Or			Ss	Es	Es			Es	Es			Or	Or	Or
Wastewater Treatment	On-Site High Rate Treatment	6					Em	Em				Em	Em				Em	Em	Et	Et						Em	Em
	Interceptor to Existing Plant And Treatment	6				Em					Em				Et*	Em*					Em*	Em*			Em*		
Overflow	Disinfection**/ Outfall	7	Es	Et	Et		Ed	Ed	Et	Et		Ed	Ed	Es	Es	Es	Ed	Ed	Ed	Ed			Et	Et		Ed	Ed
Sludge Handling	On-Site Treatment	8			St			Sd		St			Sd		St		Ss, Sd		Ss, Sd			Ss		St			Sd
	Interceptor to Existing Plant and Treatment	8	Ss	Ss, St	Ss	Ss	Ss, Sd	Ss	St			Sd		Ss*				Ss*, Sd*		Ss*, Sd*	Ss*		St*			Sd*	

Or = Regulator Overflow
Es = Swirl Separator Light Fraction
Et = Effluent from Settling Tank
Em = Effluent from Mix Tank
Ed = Effluent from Hi-Rate TMT Device
Ss = Underflow from Swirl Separator

St = Settled Sludge
Sd = Sludge from High-Rate Treatment
* May involve transportation of effluent or sludge if storm overflow location is not near a sanitary sewer interceptor.
** Disinfection is optional based on site specific requirements.

and storage with complete mix. Any storage of storm overflows without specific provisions to keep the solids suspended will result in some settling of the solids in the storage basin. The basin can be designed to remove the solids and thus act as a primary settling tank. Alternatively, the solids may be allowed to settle and other means of removing the solids may be employed, such as washing down the basin after the effluent is pumped out. Depending on the load-reduction requirements, storage with settling may provide treatment sufficient to allow the effluent from the settling tank to be discharged directly to the receiving water, with disinfection as the only other treatment required. If not, the effluent must receive additional treatment.

Storage with complete mix is usually accomplished through mechanical mixers or aerators. In most cases, the effluent from the complete-mix tank will require additional treatment before discharge to the receiving water. Occasionally the storage tank will need washing down after emptying to avoid odor and solids-accumulation problems. This will depend in part on the effectiveness of the mixers.

These two techniques can be used in conjunction. For example, a facility in Milwaukee, Wisconsin was designed to allow settling during the storm event, since there was a reasonable chance that the total storm runoff volume entering the tank would exceed the tank's volumetric capacity. By allowing settling during the storm flow condition, excessive flow that might overflow the storage tank at least receives primary treatment by sedimentation. After the storm, mechanical mixers are used within the tank to resuspend settled solids as the tank contents are pumped out.

A benefit of having off-line storage facilities for wet-weather flows is that these facilities may also be used for equalization of dry-weather flows. Storage volumes provided for wet-weather flows will probably be more than adequate, since only approximately 20 percent of the average dry-weather flow is needed to provide equalization. The tank could be compartmentalized to handle the smaller dry-weather storage in order to prevent solids from this flow from spreading out over the whole tank.

Wastewater Treatment - Effluents from storage tanks may receive further treatment in facilities provided at the overflow site, or may be transported through an interceptor to the existing wastewater treatment plant. The location of treatment is selected by determining the least-cost mix of transport and treatment. The tradeoff is between the size of the treatment units plus cost of transportation, if necessary, and the size of the storage device. A treatment unit could be provided at the overflow site or at an existing plant that would treat the full flow expected during the storm event. A smaller unit could be utilized if storage were also made available. Similarly, the provision of storage capacity on-site might make the available capacity at the existing plant sufficient to handle the storm flows. A simplified approach to the storage/treatment optimization is provided in worksheet Item 6. The user can utilize this approach in the preliminary stages of planning, but more detailed evaluation should be made as key decision points are reached.

In determining the optimum mix of storage and treatment from a cost standpoint, cost functions for both storage and treatment must be utilized. Storage may be provided in the sewer lines themselves, or in a tank external to the sewer system. Treatment may be provided at the overflow site by utilizing a high-rate treatment unit, or at the existing treatment plant by utilizing existing facilities or by expanding those facilities. The most attractive combination of these options must be determined for each specific situation. The cost of these options will depend on the degree of treatment required, the excess capacity at the existing wastewater plant, the availability of land at the overflow site for the construction of treatment facilities, the availability of land at the existing wastewater treatment plant site for expanding existing facilities, the sludge handling ability at both sites, the opportunity to build an overflow treatment facility which would serve more than one outfall, and, in the case of the storm sewer, the length of the connecting interceptor which must be constructed to transport storm flows to the sanitary sewer interceptor leading to the existing plant.

As shown in Table 6-11, the swirl separator provides a degree of primary treatment. The settling efficiency of a subsequent storage tank therefore will be less for the swirl light fraction than for the overflow from a conventional regulator because of the partial treatment already provided. Other

TABLE 6-11

SUMMARY OF CONCENTRATION REDUCING TREATMENT ALTERNATIVES FOR ON-SITE OVERFLOW TREATMENT DEVICES[1,2]

	Typical Pollutant Removals[3] (%)					Hydraulic Loading Rate (gpm/ft^2)	Detention Time (min)	Cost Functions Amortized Capital (CA)[5] CA=lT^m or CA=lS^m		(Annual Cost: $/yr)[4] O & M OM=p$T^q$		TOTAL COST (TC) TC=sT^z or TC=sS^z	
TREATMENT	BOD$_5$	TSS	N	P	FC	Range	Range	l	m	p	q	s	z
1. Microscreens	40-60	70				20		7,343.8	0.76	1,836.0	0.76	9,179.8	0.76
2. Screening/Dissolved Air Flotation	50-60	80				2.5		8,161.4	0.84	2,036.7	0.84	10,198.1	0.84
3. Swirl Separator	25-60	50				60		1,971.0	0.70	584.0	0.70	2,555.0	0.70
4. High-Rate Filtration	60-80	90				24							
5. Disinfection					99.9			(See Appendix H, Figure H-26)					
STORAGE													
1. Storage with settling[6] (i.e., sedimentation)	25-40	55				0.5		32,634.7	0.70	8,157.8	0.70	40,792.5	0.70
2. Storage (high density areas, 15 persons per acre)												51,000.0	1.00
3. Storage (low density areas, 5 persons per acre)												10,200.0	1.00

Notes: [1] T = Wet-Weather Treatment Rate in mgd; S = Storage Volume in mg.

[2] 100 mgd. No economies of scale beyond 100 mgd.

[3] Reference (9).

[4] Reference (10).

[5] Amortized at 7% over 20 years.

[6] ENR = 2000. Includes land costs, chlorination, sludge handling, engineering, and contingencies. Additional cost information for wet-weather storage and treatment devices can be found in Appendix H, Figures H-17 through H-27.

high-rate treatment units which may be considered for on-site treatment include those listed in Table 6-11, which gives ranges of treatment for various pollutant parameters for each unit. Table 6-11 also includes cost information for these high-rate processes or devices. In the case where an existing plant is to be expanded or upgraded to handle storm flows, the TREATMENT FACILITY METHODOLOGY may be utilized to cost out the expansions/upgradings. The on-site treatment units listed in Table 6-11 do not include biological processes because of the difficulty of maintaining such a system with intermittent flows. However, biological treatment of storm flows by expanding existing biological plants is feasible, if an arrangement is made to provide necessary activated sludge from the dry-weather flow units, or by routing dry-weather flow through standby storm-flow units.

The user should consider providing the capability of increasing the loading on the primary settling tanks at existing wastewater treatment facilities. This may involve resetting the hydraulic regulators to allow more flow to enter the treatment plant, or increasing the pumping capacity at the head end of the treatment plant. This would accomplish several things. It would provide the capability of working off the wet-weather flows stored in upstream storm flow storage tanks more rapidly. It would also allow the collection system to be drawn down in order to provide more capacity for in-line storage, as was discussed under the COLLECTION SYSTEM CONTROL METHODOLOGY. The treatment efficiency of the primary settling tanks would be decreased, but the overall increase in loading to the stream could be less than if overflows were made necessary because of storage capacity (in the lines or in tanks) that was not available when the storm event occurred. Of course, it would also be necessary to provide bypasses after the primary tanks in case the secondary processes could not handle the increased flow.

Overflow - An overflow to the receiving water may come from a swirl separator, a storage (with settling) tank, or an on-site high-rate treatment unit. The user will want to make sure that the existing overflow line will handle the quantity of overflow anticipated. If state or local standards require disinfection, the cost of providing this disinfection must be included in the overall cost of the alternative.

Sludge Handling - An underflow or sludge is generated at the overflow site by a swirl separator, by storage with settling, or by on-site storm runoff treatment. The sludge may be treated on-site or may be transported via an interceptor to the existing wastewater treatment facility. Several sludge treatment processes may be used to treat the sludges on-site, as listed in worksheet Item 8. If this alternative is selected, provision must be made for transporting the treated sludge to a final disposal site. If the swirl separator or storage tank wastewater effluents are treated at the existing plant, then, of course, the sludges will also be handled at the plant.

If the sludges generated on-site are discharged to the interceptor in order to be treated at an existing wastewater treatment plant, several problems may arise. The first is related to the transportation of the sludges to the plant. The solids in storm flows are heavier than those in dry-weather flows, because the velocity of the stormwater in the sewer is usually greater than dry-weather velocities and will result in a scouring of previously deposited heavy solids. These solids will tend to settle out unless sufficient velocity is maintained in the interceptor to keep them suspended. Therefore, sections of the interceptor with marginal slopes might be subject to settling problems. Another potential problem associated with either on-site or at-plant treatment is the solids-handling capacity at the existing plant. For example, it has been estimated by EPA that the sludge load at a particular facility would be approximately doubled if combined sewer overflow sludges are directed to the plant. Therefore, the alternative of discharging sludges from sewer overflow storage and treatment components may include expanding the sludge-handling capacities at the existing treatment facility. The excess capacity of these units should be investigated when costing out this alternative. In addition, in the case of storm sewers it may be necessary to construct an interceptor from the overflow site to the nearest sanitary sewer interceptor in order to transport the sludges to the treatment plant.

A combination of the on-site and at-plant approaches is also feasible in some cases. Grit removal can be provided on-site to alleviate the heavy-solids transportation problem, and the sludge dewatering and additional treatment can take place at the existing wastewater treatment facility.

Step 3b (Item 4) - If the existing regulator in a combined sewer is to be replaced, or if a control device is to be installed in a storm sewer, the cost of a conventional regulator or swirl separator is calculated by using Table 6-11 or an equivalent method, and recorded in Item 4.

Step 3b (Item 5) - If the control option under consideration calls for on-site storage and direct discharge to the receiving water (without treatment either on-site or at the existing plant), Item 5 may be used for recording the cost of settling. If treatment is to be provided, Item 6 should be utilized in order to determine the least-cost combination of storage and treatment.

Step 3b (Item 6i) - Rainfall and stream flow are the driving forces behind all storm flow investigations. Since storm patterns and rainfall characteristics vary with geographic location, alternative methods of control must be considered.

Storms of high intensity and short duration may be controlled effectively by using storage facilities, whereas storms of low intensity and long duration may be controlled more effectively through increased treatment capacity or surface runoff deterrents. Intervals between storms are significant, because they may dictate dewatering requirements and in turn control treatment rates in a system clean-up between storms. Therefore, when dealing with storm events, the important characteristics are the intensity and type of precipitation, and the magnitude, frequency, and duration of the storm. These parameters may be determined by an investigation of rainfall records in the area of interest, by utilizing parameters already determined for nearby areas, or by any other appropriate method (see Chapter 3).

In order to determine the optimum (i.e., the least Present-Worth cost) mix of high-rate treatment and storage of storm flows, it is necessary to develop a hydrograph for the overflow from the control structure. Development of this overflow hydrograph involves identification of a design storm frequency, development of inlet (to the sewer system) hydrographs for the subarea tributary to an overflow structure, and use of a routing procedure to develop a composite hydrograph of the inflow to the control structure.

The design storm may be designated by a local drainage ordinance, or it may be determined by means of an engineering analysis. The choice of a design storm for a particular study area involves the following considerations:

- Timing of the rainfall and interval between events.
- Source data available.
- Scope and objectives of the investigation.
- Limitations of the physical system.

Engineering judgments relative to the most significant events for planning purposes can usually be made from available data.

Step 3b (Item 6ii) - Using the design storm event and the physical characteristics of the drainage area, the user must develop an inlet hydrograph for runoff from the subarea tributary to each outfall. Numerous hand and computer techniques are available for generating these hydrographs. The actual method applicable in a specific situation is left to the user's discretion.

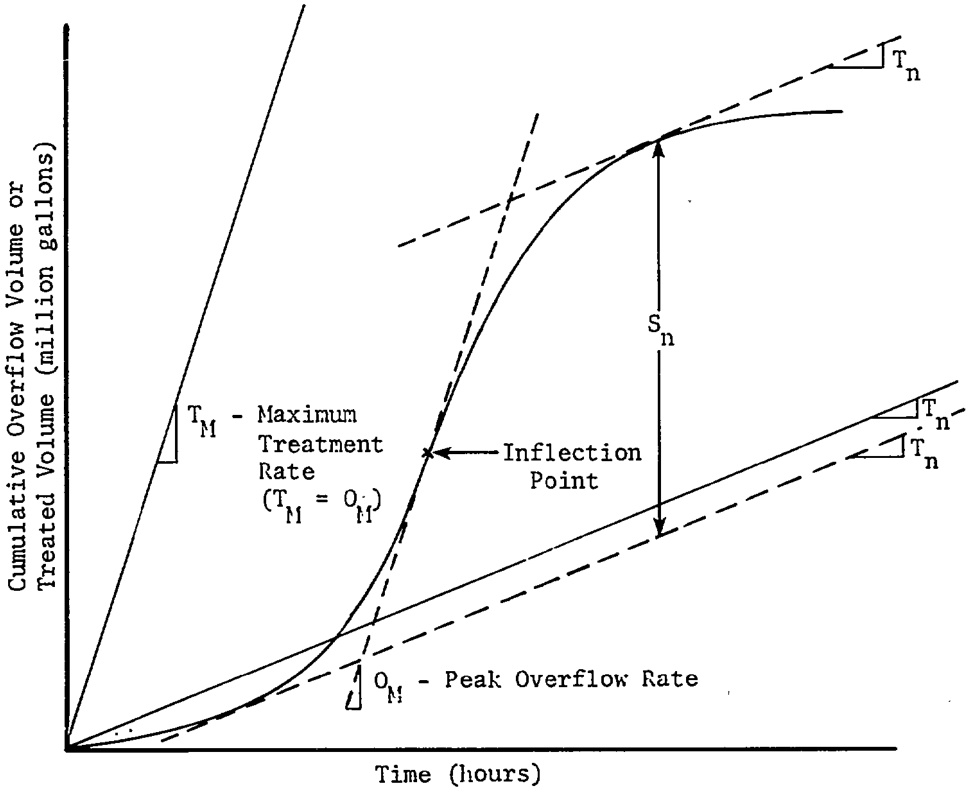
Step 3b (Item 6iii) - Having developed individual inlet hydrographs for each subarea, the user must now route the time-dependent flows to the overflow control structure in order to obtain a composite inflow hydrograph. During the routing procedure, the user should take into account available in-line storage capacity. The location and amount of available in-line storage can be determined by using the COLLECTION SYSTEM CONTROL METHODOLOGY.

Step 3b (Item 6iv) - Using this inflow hydrograph and a rating curve (i.e., head versus discharge) for the structure, the user can generate an overflow hydrograph.

Step 3b (Item 6v) - Assuming that all the overflow must be treated or stored for subsequent treatment, the user can identify storage requirements necessary for various treatment rates by using a mass curve developed directly from the overflow hydrograph. The time-specific flows of the hydrograph are converted to equivalent time-specific volumes and accumulated over the entire time span of the hydrograph. The cumulative volume at time T is obtained by taking the area under the overflow hydrograph to the left of time T. The cumulative volumes are plotted versus time as shown in Figure 6-20.

FIGURE 6-20

TYPICAL MASS DIAGRAM



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Step 3b (Item 6vi) - Using the mass curve, the user can determine the storage requirements for various treatment rates. The slope of the mass curve at any time is a measure of the overflow rate. Treatment rates can be represented by straight lines drawn from the origin. The storage volume required for a specific treatment rate can be obtained by drawing two tangents to the mass curve parallel to a desired treatment rate line and then measuring the vertical distance between the two lines.

The peak overflow rate (O_M) occurs at the inflection point of the mass diagram, which corresponds graphically to the maximum point on the inflow hydrograph. A treatment rate equal to the peak overflow rate would require no storage facilities. This rate is represented by the line with slope T_M ($T_M = O_M$). Lesser treatment rates would require storage of excess flows in order to prevent overflows. For example, for treatment rate T_N ($T_N < T_M$), storage S_N is required to prevent overflow.

The user can use the mass diagram to determine the storage requirements for treatment rates ranging from TM to no treatment, in convenient increments. These combinations of storage and treatment are the bases for identifying the least cost mix of storage and treatment.

Step 3b (Item 6vii) - The cost of the various levels of storage can be determined using the appropriate equation from Table 6-11, or an equivalent method. A plot should be made on standard graph paper of the volume to be stored and of the Present-Worth cost of storing that volume.

Step 3b (Item 6viii) - Table 6-11 can be used to determine the high-rate treatment units that can achieve the required load reductions. The cost of these units for the various treatment rates can be calculated by using the cost functions in Table 6-11. A plot of the cost of each type of unit for the range of treatment rates should be made on standard graph paper.

Step 3b (Item 6ix) - The user must also determine the costs associated with using existing wastewater treatment facilities in lieu of construction of on-site facilities. The various options to consider are discussed under Step 3a. The costs associated with treatment at the existing facility must be determined for each rate of treatment, using the TREATMENT FACILITY

METHODOLOGY when expansions or upgradings are necessary. The cost of transportation and treatment at each rate of treatment should be plotted on the same graph with the costs of on-site treatment (Item 6viii).

Step 3b (Item 6x) - The cost curves plotted in Items 6viii and 6ix can be used to determine the least-cost combination of storage and treatment rate in the following manner:

- For each treatment rate, identify the least-cost treatment unit (high-rate unit or existing plant expansion/upgrading from Item 6viii), and record the treatment cost.
- Identify the necessary storage volume for each rate of treatment (from Item 6vi), and record the cost of storage (from Item 6vii).
- Determine the total Present-Worth cost of storage and treatment for each treatment rate.
- Plot total Present-Worth Cost versus Treatment Rate on standard graph paper.

The minimum point on the curve will be the least-cost combination of storage and treatment.

Step 4a (Item 10) - The user should investigate the potential advantages of regionalization of storage/treatment facilities. Overflow locations are often in reasonably close proximity.

Depending on the specific situation, overflow points may be close enough to make the use of one large storage tank (or treatment unit, or both) an economical alternative.

6.4.2.8 Wastewater Reuse Methodology

Discussion

The purpose of this component methodology is to identify and evaluate potential wastewater reuse opportunities for a specific point source. The initial step is to identify whether there is a reuse potential in the planning area. When reuse seems feasible, the potential reuse candidates and their water quality requirements are identified. The additional cost to achieve a treatment level beyond that required for discharge, if necessary, is evaluated by means of other component methodologies included in this manual or equivalent methods. The wastewater transportation cost is also identified, and the total cost for treatment and reuse is determined. The reuser's cost for the existing water supply or the development cost for a new supply is then determined and compared with the additional cost incurred by the reuser for wastewater reuse. This evaluation will indicate if an economic incentive exists for wastewater reuse, as indicated by a potential saving to the reuser.

This methodology does not deal with pricing policies for wastewater reuse. This is a specific consideration for any potential reuse application, and is complex because of the interaction of supply and demand. However, the determination of the potential economic incentive for reuse accomplished in this methodology should be adequate for identifying the potential for wastewater reuse revenues.

Finally, this methodology can be readily modified to handle the case where more than one reuser exists for a point source. The 208 planner or engineer can do this by beginning his evaluation with the potential reuser who has the most stringent water quality requirement. The wastewater from this alternative could then be used by any subsequent reuser in the planning area, so that only the transportation cost would have to be computed. Another approach would be to combine several potential reusers for one evaluation. By utilizing these concepts, the 208 planner or engineer can readily develop the cost associated with several reuse schemes and at the same time minimize his effort in identifying treatment facility costs.

Methodology Logic

A summary of Methodology Logic is presented in Figure 6-21. An expanded flowchart, Figure 6-22, lists the steps to be taken in determining performance and costs. The worksheets for recording the operations are presented as Table 6-12. Notes on specific steps or worksheet items are included after the worksheet.

FIGURE 6-21

WASTEWATER REUSE METHODOLOGY
LOGIC SUMMARY

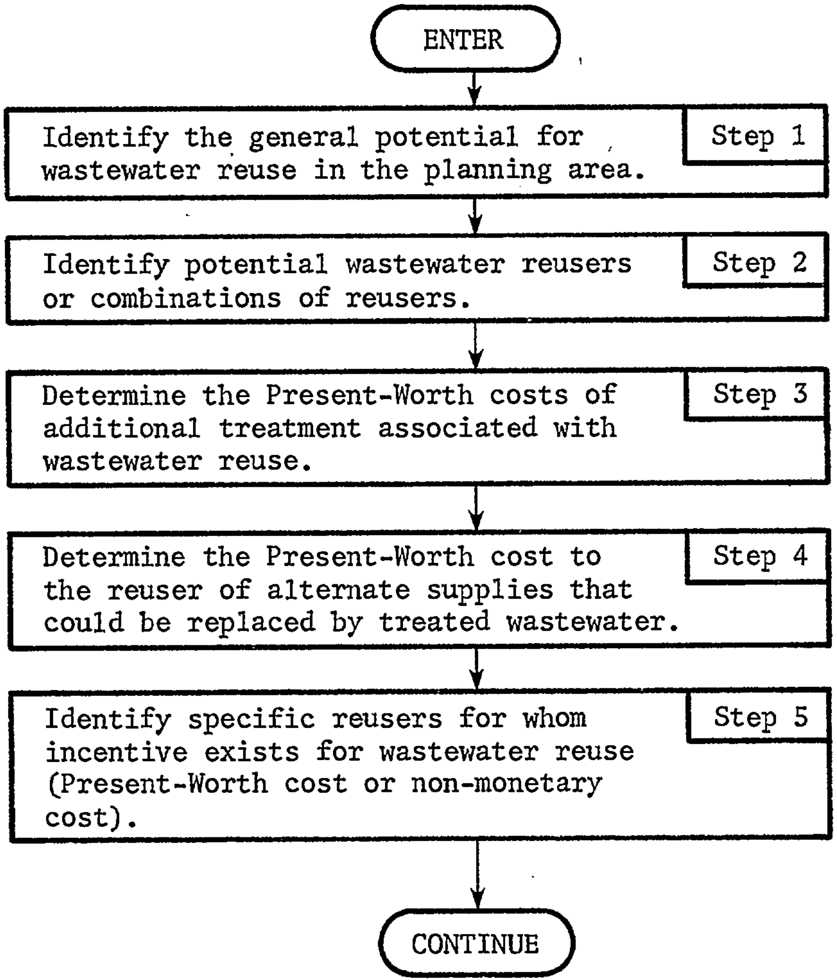

FIGURE 6-22

WASTEWATER REUSE METHODOLOGY FLOWCHART

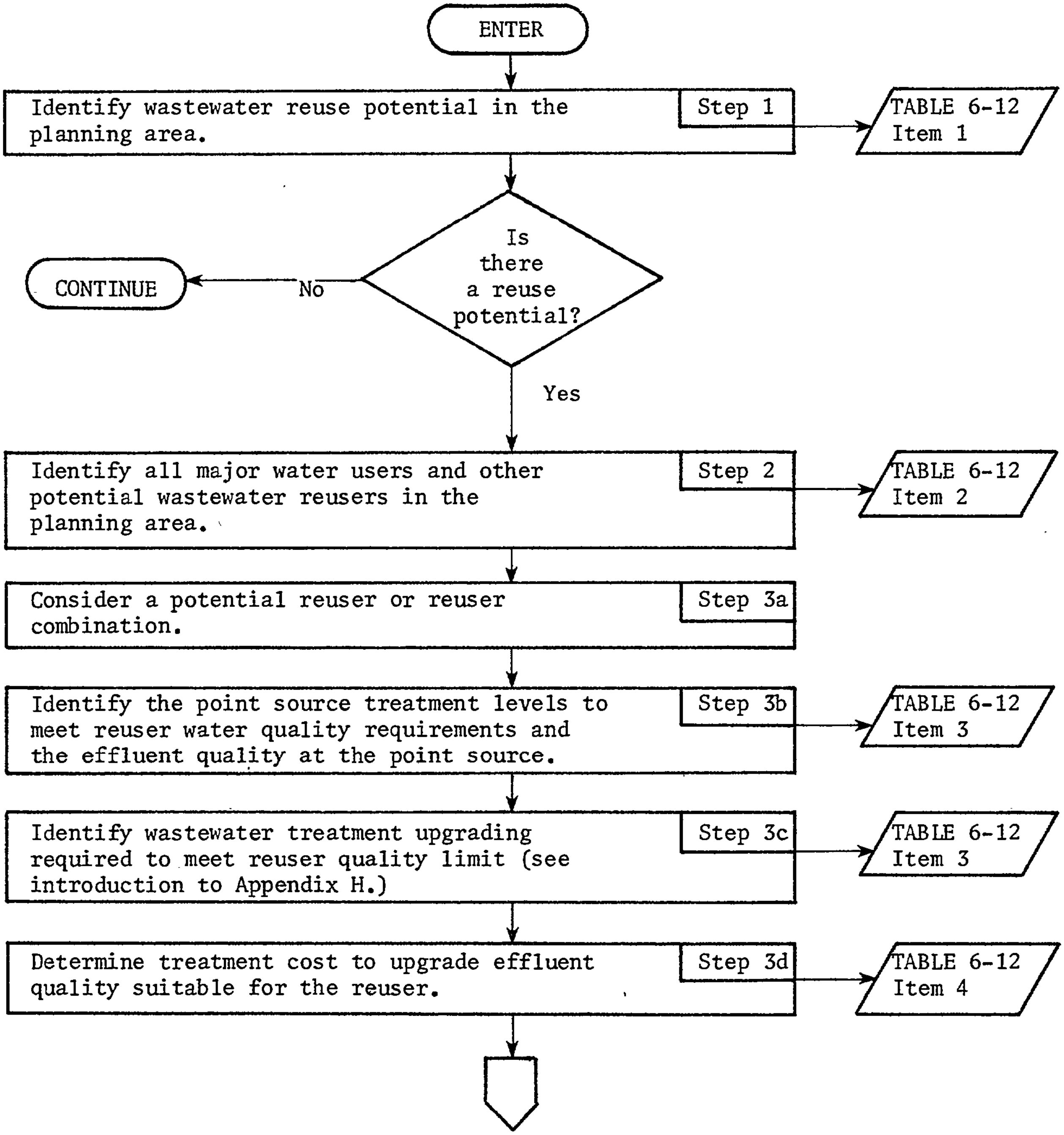

FIGURE 6-22 (CONTINUED)

WASTEWATER REUSE METHODOLOGY
FLOWCHART

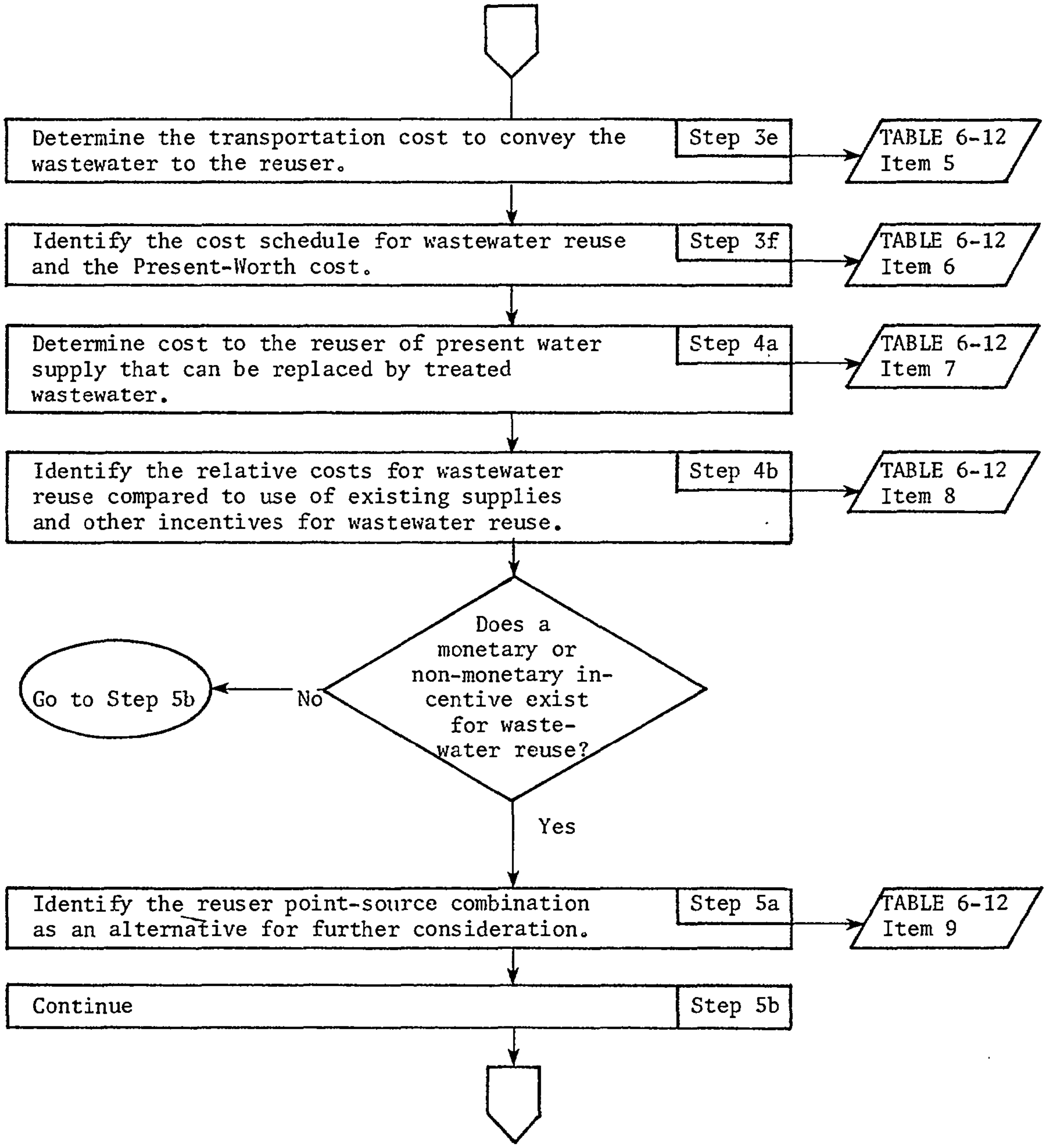

FIGURE 6-22 (CONTINUED)

WASTEWATER REUSE METHODOLOGY FLOWCHART

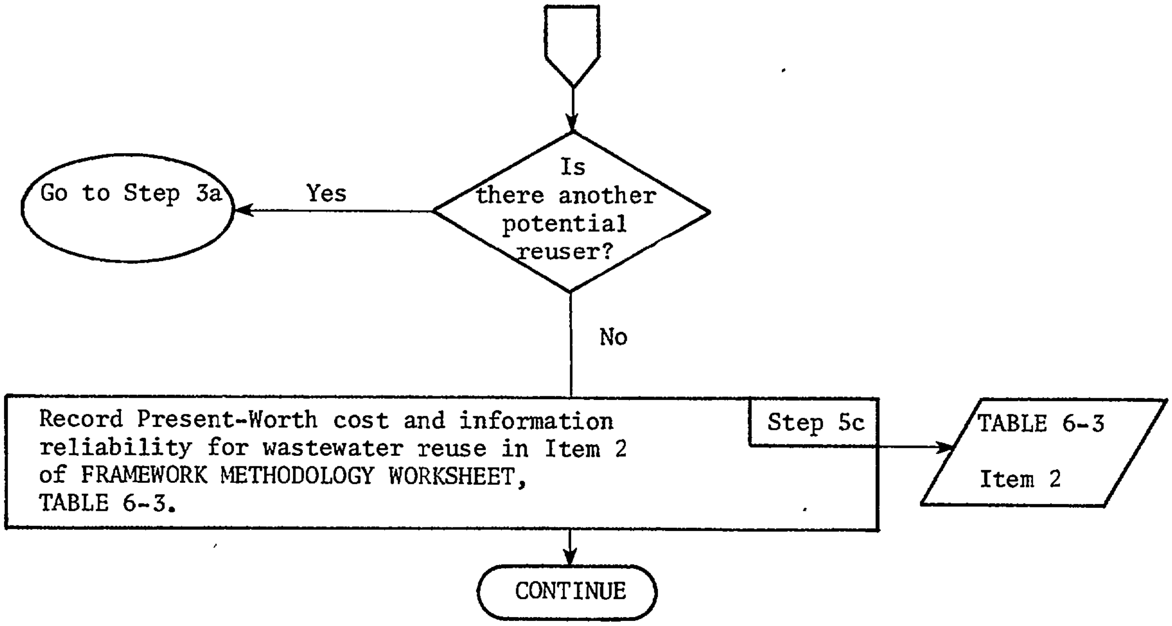

TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approach used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - General Reuse Criteria.

i.	Total municipal water demand approaching existing water supply?	YES	NO
ii.	Existing water supplies unavailable or insufficient for new uses (e.g., irrigation, industry)?	YES	NO
iii.	Existing water supply subject to environmental degradation (e.g., salt water intrusion, aquifer drawdown)?	YES	NO
iv.	Point source wastewater effluent available in sufficient quantity to satisfy potential new needs?	YES	NO
v.	Point source disposal technique involves unusually high costs?	YES	NO
vi.	Planning area includes any large uses of non-potable water?	YES	NO
vii.	Absence of restrictions (riparian rights) or related water law?	YES	NO
viii.	Expressed interest on the part of potential wastewater reusers?	YES	NO

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TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

Item 2 - Potential Reuse Source Identification

Potential Water Reuser Identification	Water Source/ Treatment	Water Use	Present Water Cost	Projected Water Cost	Quantity		Quality		Reliability	Distance
					Current	Design Yr	Desired	Minimum		
EXAMPLE:										
1. Georgia Power Plant, Mitchell. Mr. John Phillips Plant Engineer	Flint River/ screened, settled.	Cooling Water	$0.12/1000 gal	$0.24	1.4 mgd (0.6 mgd firm minimum)	1.4 mgd (1985)	76°F pH, 7.2	84°F pH 7.0-7.8	complete	3 miles, across Flint River
2. Blue Valley Pecan Farm. Mr. Tom Young Owner	Private well, no treatment	Irrigation	$.08/1000 gal pumping cost	$0.22	400,000 gpd (May-Nov)		No toxic metals, low dissolving solids, normal pH	same	low	2 miles
Albany County Club	Private well, no treatment	Irrigation	$.14/1000 gal pumping cost	$0.38	300,000 gpd (May-Nov)	700,000 gpd (1980)	same		low	8 miles

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TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

POTENTIAL REUSER EVALUATION

Reuser: ______________________

Item 3 - Point Source Treatment to Meet Reuse Criteria.

	Reference Cost Curve	Reuser/Point Source Critical Parameters							
		Q	BOD	TSS	N	P	T		
Reuser Water Quality Requirements	N/A								
Existing Wastewater Effluent Quality									
Existing Wastewater Effluent Quality Control Criteria									
Upgrade Technique[1]									

[1] Identify the cost curve for upgrading each identified parameter to the reuser criterion; identify the required system under "Reference Cost Curve" from individual curves.

Item 4 - Treatment Requirement Cost Schedule to Meet Reuser Criteria.

(Use TREATMENT FACILITY METHODOLOGY to define costs for treatment above that required for discharge)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

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TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

Item 5 - Reuse Transportation Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

Item 6 - Wastewater Reuse Project Costs.

i. Project Cost Schedule.

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
1		#4					
		#5					
	TOTAL PHASE 1						
2		#4					
		#5					
	TOTAL PHASE 2						
3		#4					
		#5					
	TOTAL PHASE 3						

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TABLE 6-12
WASTEWATER REUSE METHODOLOGY WORKSHEET

Item 6 - Wastewater Reuse Project Costs (continued).

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
n		#4					
		#5					
			______	______	______	______	______

TOTAL PHASE n

Replacement Schedule Year	Replacement Schedule Cost
______	______
______	______
______	______

ii. Present-Worth Cost (using PRESENT WORTH METHODOLOGY).

Interest ________%

Present-Worth Cost $________

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TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

Item 7 - Reuser Replaceable Water Costs.
(Cost that would be incurred for replaceable water supply without wastewater recycle)

i. Cost Schedule.

Phase	Timing	Recycle Water Use[1], gpd		Unit Water Cost[2]	Annual Water Cost[3], $/year	
	Yr to Yr	Start	End	$/1000 gal	Start	End
1						
2						
3						
n						

[1] Recycle Water Use represents the portion of total reuser water requirement that could be satisfied by treated wastewater.

[2] Unit Water Cost represents projected cost for existing supply if adequate, or the unit cost for development and treatment of a new supply.

[3] Annual Water Cost = (gpd) ($/gal) (365 days/yr)

ii. Reuser Replaceable Water Present-Worth Cost.
(using PRESENT-WORTH METHODOLOGY): $______________________

Item 8 - Wastewater Reuser Relative Costs.

i. Replaceable Water Cost: $________________
(Alternative Present-Worth cost for reuser water needs that can be satisfied during the planning period by treated wastewater; from Item 7 ii)

ii. Reused Wastewater Cost: $________________
(Present-Worth cost to utilize treated wastewater; from Item 6 ii)

iii. Other Factors: (describe factors other than cost that will affect further evaluation)

____

____

____

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TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

Item 9 - Potential Reuser/Point-Source Combinations

Point Source		Reuser		Identified Present-Worth Costs			
				Wastewater Reuse	Replaceable Water	Total	Non-Monetary Cost Incentive
ID	Q,mgd	ID	Q,mgd				

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Notes on Methodology Logic

Step 1 (Item 1) - The purpose of this step is to determine if there is a reuse potential in the planning area. This might occur where the available water supply limits current or future water demand, where the existing water supply will be subject to environmental degradation due to overuse or the impact of the wastewater discharge, where the wastewater effluent is of a high quality, or where wastewater disposal is unusually expensive. If any of the situations identified in Item 1 exist, then the user of this manual should evaluate the wastewater reuse potential further.

Step 2 (Item 2) - This step will identify the potential reusers in the planning area and also potential new uses for wastewater. A guideline is included in Table 6-13, which identifies some potential reusers. The information for each potential reuser is entered in Item 2. The information developed for reuse of effluent from one point source in the planning area will likely be applicable for the evaluation of wastewater reuse at other point sources.

Step 3a - No discussion.

Step 3b (Item 3) - The purpose of this step is to identify the required level of treatment for utilization of the wastewater to supplement or replace the reuser water supply. The critical water quality parameters for the specific reuser should be identified in the worksheet as completely as possible. One source for this information is reference (11).

The existing wastewater discharge quality can be identified using the method outlined in the TREATMENT FACILITY METHODOLOGY. Any additional treatment required to meet the reuser's needs is identified for each critical parameter. If the required water quality for the reuser is not greater than that for wastewater discharge, then there will not be an increased treatment cost.

Step 3c (Item 3) - In this step, the upgrading technique to meet the reuser's water quality requirements is determined. The user should incorporate information in Appendix H (or comparable sources) regarding the effluent characteristics from alternative treatment processes.

Step 3d (Item 4) - This step identifies the treatment cost schedule for the modification required to meet the reuser's water quality requirements. The

TABLE 6-13

POTENTIAL CUSTOMERS AND APPLICATIONS FOR WASTEWATER REUSE

Customer	Applications
Municipal	Irrigation - Public parks, zoo grounds, government centers, etc. - Public golf courses - School grounds - Publicly-owned farm lands - Right-of-way landscaping - Other
	Groundwater recharge
	Prevention of salt water intrusion
	Recreational lakes
	Public utilities - Cooling water for power plants
Private Industry	Cooling water
	Boiler feed water
	Process purposes
	Irrigation of grounds
Private Irrigation	Crop irrigation
	Salt leaching
	Irrigation of - Golf courses - Duck clubs - Recreational areas, including artificial lakes

*This list is not all inclusive; the individual planning agency should develop a similar listing specific to the planning area.

planner can utilize the TREATMENT FACILITY METHODOLOGY to determine the treatment cost involved in attaining this level of treatment when a general treatment facility upgrading is required. If only a portion of the wastewater flow is utilized for reuse or if only one additional treatment operation is required to upgrade the wastewater quality to meet the reuser's need, then the capital cost can be developed using the TREATMENT FACILITY METHODOLOGY, or an equivalent method, with the required adjustments for construction costs.

Step 3e (Item 5) - In this step, the transportation cost schedule for pumping the required wastewater quantity to the reuser is determined by means of the TRANSPORTATION COST METHODOLOGY.

Step 3f (Item 6) - In this step, the transportation and reuse treatment costs schedules are combined to determine the total project cost schedule. The project present-worth cost can be determined by means of the PRESENT-WORTH METHODOLOGY.

Step 4a (Item 7) - In this step, the cost to the potential wastewater reuser for replaceable water during the planning period is determined. The quantity of replaceable water should represent the portion of the reuser's need that can be satisfied by treated wastewater. Where the reuser's current supply will be inadequate for future needs, the unit cost for developing and treating a new supply should be used.

Step 4b (Item 8) - This step determines if there is an economic incentive for wastewater reuse. Since a potential reuser would not be willing to pay more for the treated wastewater than for the water supply currently used, the maximum revenues from wastewater reuse to the treatment plant control authority will always be less than this current water supply cost. Therefore, the control authority will not find it cost-effective to pursue treatment and reuse where the reuse cost is greater than the current water supply cost, unless other factors (such as a limited water supply) outweigh the monetary cost considerations.

Step 5a - This step is a summary of the potentially cost effective point source/reuser combinations. When the wastewater reuse evaluation has been completed, this list will be useful in ordering priorities for future investigations and for assessing the overall potential for reuse.

6.4.2.9 Impact Area Modification Methodology

Discussion

An impact area modification involves an alteration of the natural characteristics of the receiving water body, such as in-stream reaeration, low-flow augmentation, or a change in the physical location of a discharge point. Reducing the impact of a pollutant discharge is sometimes less expensive than reducing the pollution load in the discharge. This might also be the only feasible alternative remaining if all feasible pollutant load reduction alternatives have already been implemented. In addition, this alternative may also be especially applicable in certain streams where flow is highly regulated. This component methodology will identify the monetary costs associated with several modification schemes: discharge relocation, in-stream reaeration, and flow augmentation.

The initial determination in this evaluation is the desired and critical water quality of the receiving streams. These stream segments are commonly classified as: 1) Effluent Limited (implying that the control level is based on the generally accepted treatment level); or 2) Water Quality Limited (which means that the effluent control level has been based on a waste load allocation calculated as necessary to maintain the desired water quality). The IMPACT AREA MODIFICATION METHODOLOGY is generally more often applicable to Water Quality Limited stream segments, since these tend to have the more stringent treatment requirements.

Discharge relocation is probably the most feasible of the impact area modifications in this evaluation procedure. The required procedure involves identification of alternative discharge sites and evaluation of the points of relocation. The water quality impact analysis techniques described in Chapter 5 must be performed for the modified loading pattern to define the required treatment level at the proposed new location. Then, the costs of treatment and transporation are computed.

In-stream reaeration will be a feasible technique only when the dissolved oxygen in the receiving water is a critical water quality parameter and where aeration is compatible with physical stream characteristics. Chapter 5 techniques are required in this evaluation to redefine the source treatment level. Then, the cost to implement the reaeration system and the cost for the modified treatment control level are evaluated.

Flow augmentation should be considered when water quality standards are violated during low-flow conditions, when means for flow augmentation (such as an upstream reservoir) are available, and when low-flow augmentation will alleviate one or more critical water quality parameters. The methodology guides the user in identifying the flow-augmentation capability in determining the allowable waste loads with the flow augmented, and finally in evaluating all project costs including treatment costs and flow-augmentation costs.

The user should be advised that current EPA policy will not permit use of flow augmentation as a substitute for "adequate treatment". See reference (19). Thus, flow augmentation may only be considered where water quality standards are not met through Best Available Technology (BAT) level of treatment. Even where flow augmentation is warranted, the user should be aware of potential institutional difficulties in securing flow release guarantee agreements with agencies controlling watercourses. The user should also consider potential adverse effects of flow augmentation upon other uses of dams and reservoirs (e.g., water-based recreation, water supply, power generation).

Methodology Logic

A summary of the logic of the IMPACT AREA MODIFICATION METHODOLOGY is presented in Figure 6-23. An expanded flowchart, Figure 6-24, lists the steps to be taken in determining performance and costs. The worksheets for recording the operations are presented as Table 6-14. Notes on specific steps or worksheet items are presented after the worksheets.

FIGURE 6-23

IMPACT AREA MODIFICATION METHODOLOGY
LOGIC SUMMARY

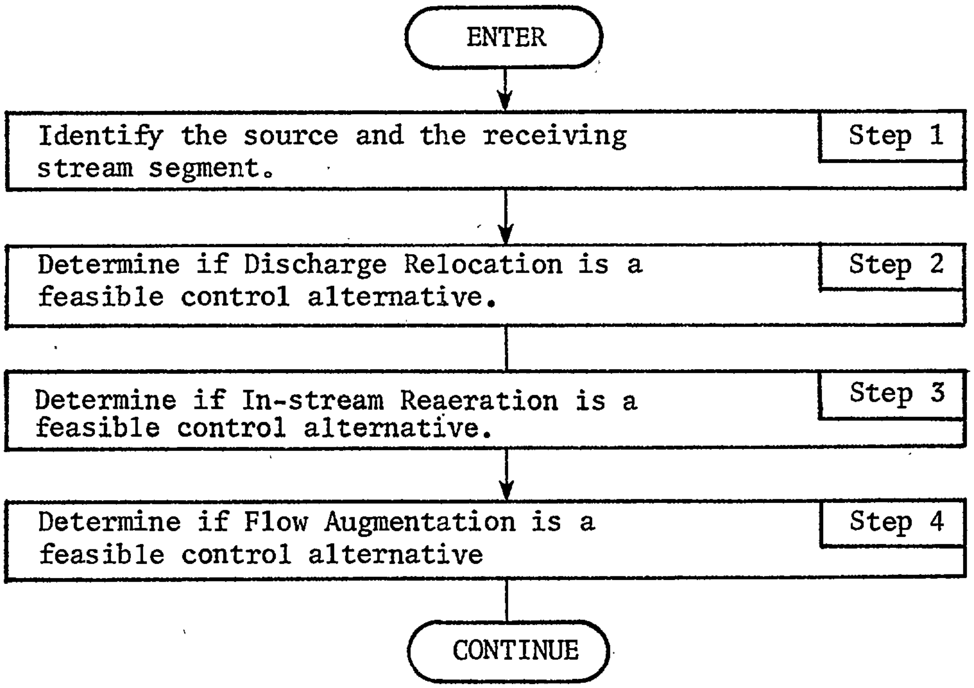


FIGURE 6-24

IMPACT AREA MODIFICATION METHODOLOGY FLOWCHART

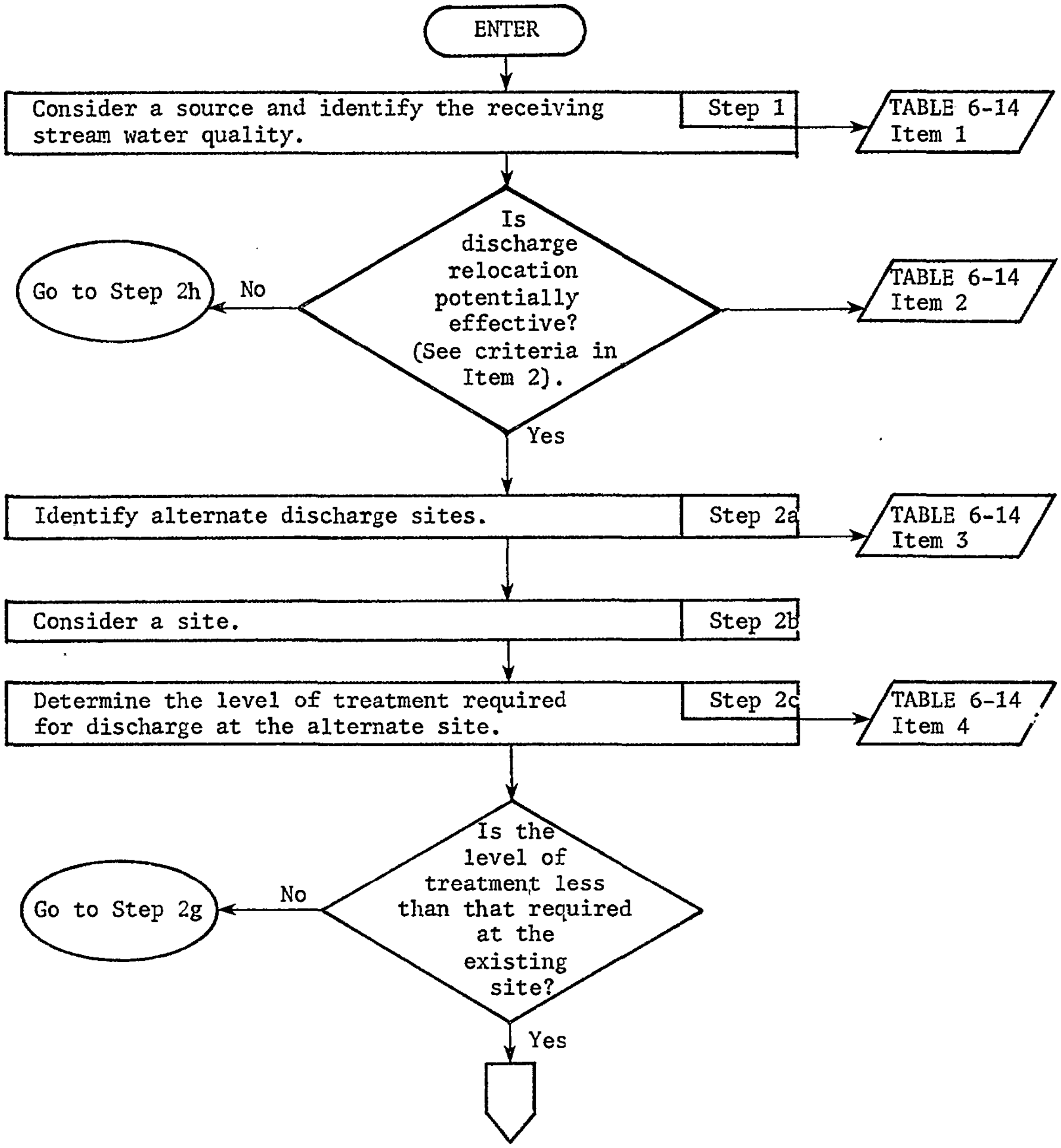


FIGURE 6-24 (CONTINUED)

IMPACT AREA MODIFICATION METHODOLOGY
FLOWCHART

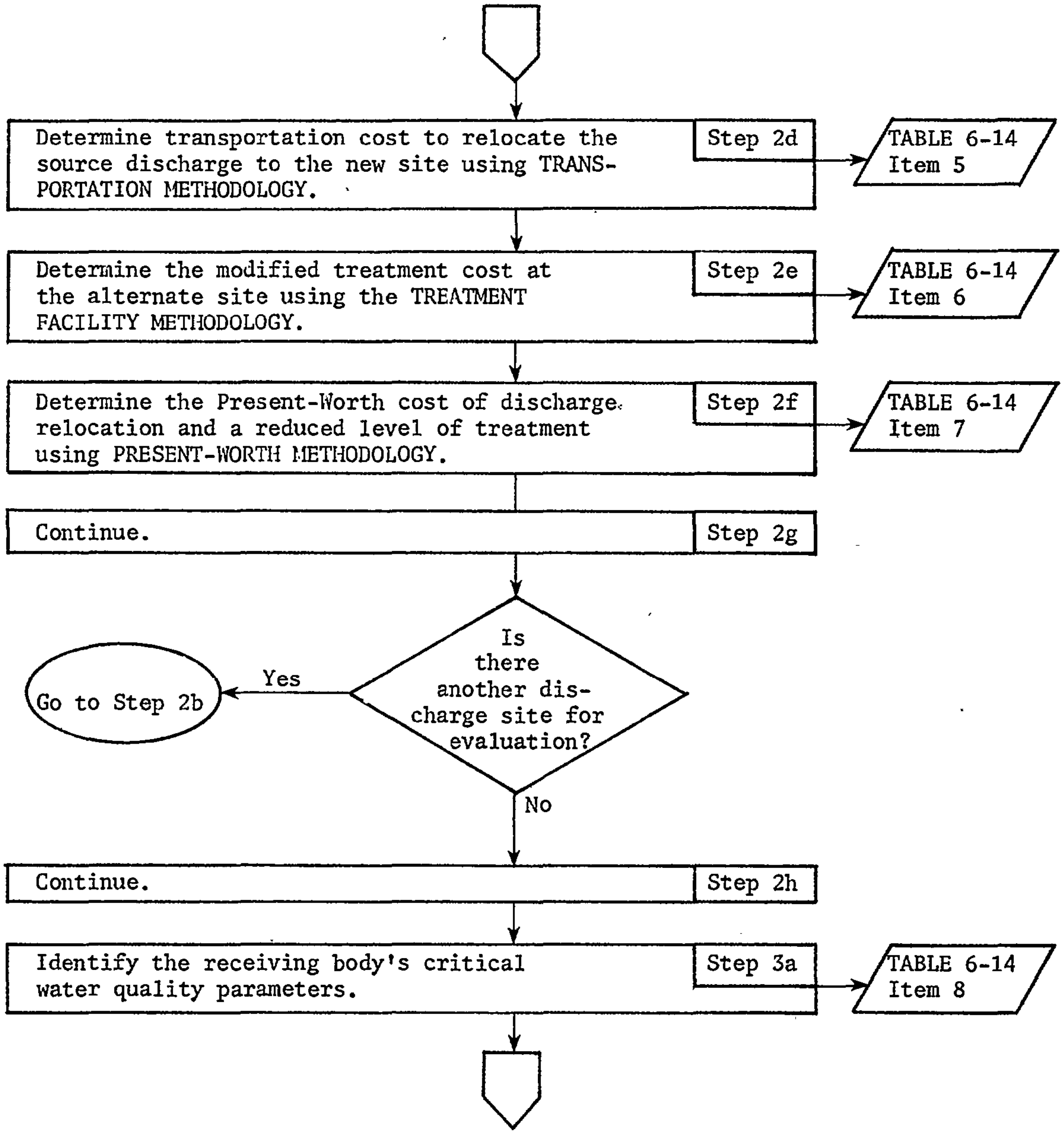


FIGURE 6-24 (CONTINUED)

IMPACT AREA MODIFICATION METHODOLOGY FLOWCHART

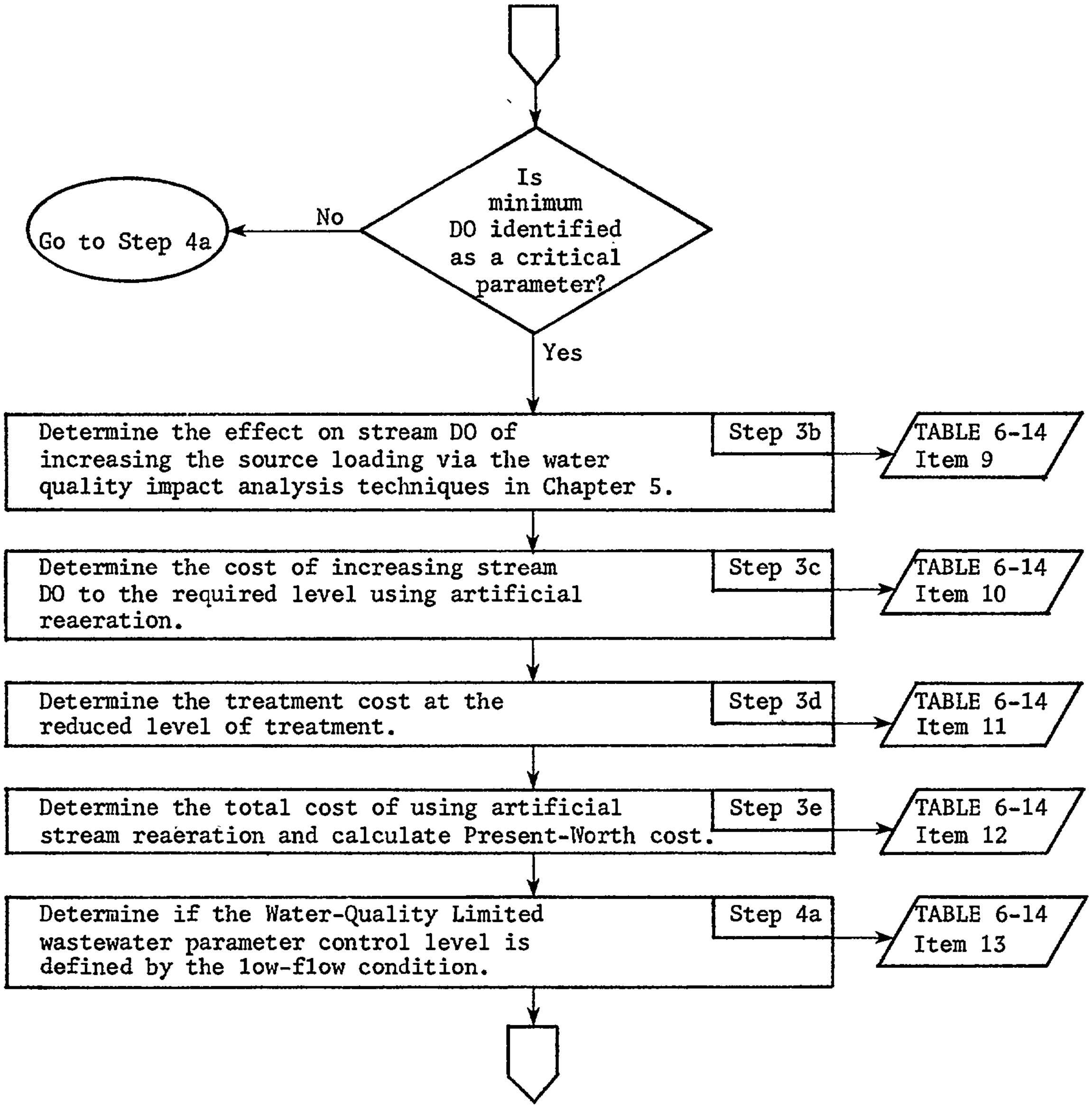
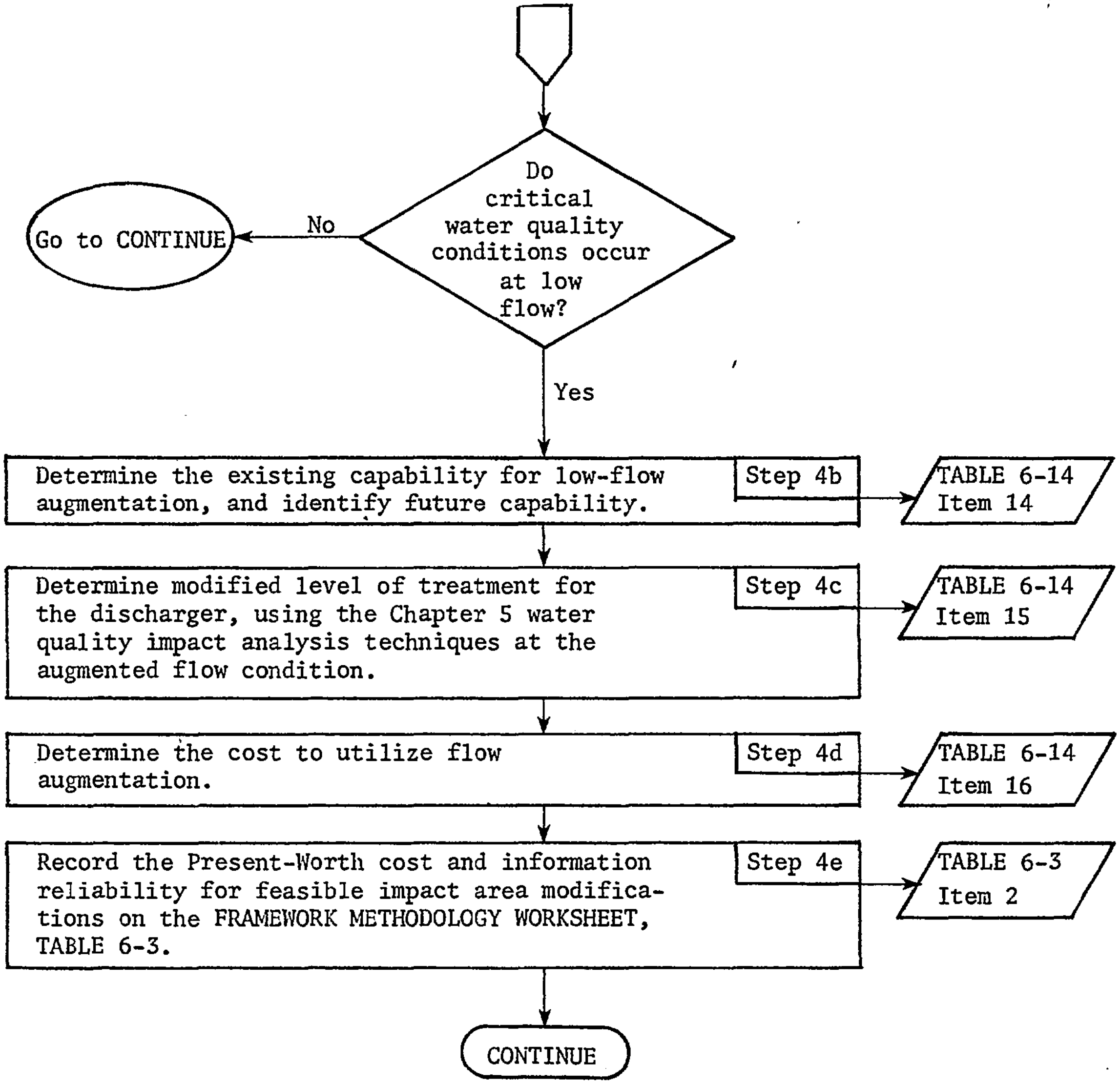


FIGURE 6-24 (CONTINUED)

IMPACT AREA MODIFICATION METHODOLOGY FLOWCHART



Determine the existing capability for low-flow augmentation, and identify future capability.

Determine modified level of treatment for the discharger, using the Chapter 5 water quality impact analysis techniques at the augmented flow condition.

Determine the cost to utilize flow augmentation.

Record the Present-Worth cost and information reliability for feasible impact area modifications on the FRAMEWORK METHODOLOGY WORKSHEET, TABLE 6-3.

TABLE 6-14
IMPACT AREA MODIFICATION METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approach used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - Receiving Water Quality.

Source ____________

Water Quality Conditions: (Summary of existing and future water quality problems by pollutant type for stream segment of interest)

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TABLE 6-14
IMPACT AREA MODIFICATION METHODOLOGY WORKSHEET

Discharge Relocation Evaluation.

Item 2 - General Criteria for Discharge Relocation.

1. Is this source a principal cause of the critical stream condition?	YES	NO
2. Are there several significant sources near this source?	YES	NO
3. Would relocation be relatively inexpensive?	YES	NO
4. Is there a major (or larger) stream nearby that could receive the source?	YES	NO

Item 3 - Alternate Discharge Site Identification.

(Factors: Less stringent water quality criteria; lower net source loading; increased dilution due to tributary flow)

	Site	Location	Distance
a)			
b)			
c)			

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

Site __________ Evaluation

Item 4 - Modified Source Level of Treatment (from Impact Analysis, Chapter 5).

Level	Reference Cost Curve	Parameter Control Levels							
		BOD mg/1	COD mg/1	TSS mg/1	T-P mg/1	NH_3-N mg/1	NO_3-N mg/1	T-N mg/1	T-C #/100ml
Existing Discharge Site									
Alternate Discharge Site A									
Alternate Discharge Site B									

Item 5 - Discharge Relocation Transportation Cost Schedule.
(use the TRANSPORTATION COST METHODOLOGY)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage Value
	Yr.	to Yr.		Start	End	Year	Cost	
Existing Facility								
1								
2								
3								
n								

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

Item 6 - Modified Treatment Level Cost Schedule.
(use TREATMENT FACILITY METHODOLOGY)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

Item 7 - Project Cost.

i. Project Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

ii. Project Present-Worth Cost: $____________________
(use PRESENT-WORTH METHODOLOGY)

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

ARTIFICIAL REAERATION EVALUATION

Item 8 - Receiving-Body Dissolved Oxygen Requirements.

Condition	Dissolved Oxygen, mg/l	@	Location
Water Quality Limit	______		______
Critical Level	______		______

Item 9 - Modified Stream Quality at Revised Source Loads.
(developed from Impact Analysis, Chapter 5)

	Wastewater Load		Resulting		
	Parameter	Discharge Level	Minimum Stream DO	@	Location
A.	BOD	______	______	@	______
	TSS	______			
	(etc.)	______			
B.	BOD	______	______	@	______
	TSS	______			
	(etc.)	______			

Item 10 - Artificial Reaeration Requirement.

i. Oxygen-Transfer Requirements.

Stream flow rate at critical conditions: ______cfs

Critical DO level: ______mg/l

Minimum acceptable DO level: ______mg/l

Oxygen transfer requirement: ______lb/hr for ______hr/yr

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

Item 10 - Artificial Reaeration Requirement (continued).

ii. Reaeration Project Phasing.

Phase	Timing		Reaeration Oxygen Demand, lb/hr		Capital	O&M		Replacement Cost	Salvage Value
	Yr to Yr		Start	End		Start	End		
1									
2									
3									
n									

Item 11 - Treatment Cost Schedule at Modified Treatment Level.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

Item 12 - Artificial Reaeration Project Cost.

i. Project Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

ii. Project Present-Worth Cost: $__________

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

FLOW AUGMENTATION EVALUATION

Item 13 - General Applicability of Flow Augmentation as a feasible control alternative.

Do critical water quality conditions occur at low flow?
(Yes or no) ________ Critical Parameters, if yes; ________________

Comments: (Special conditions, model assumptions, reference sheets)

____

____

____

Item 14 - Flow-Augmentation Capability.

i. Existing Reservoir low-flow augmentation capacity: ____________cfs
Duration: ________days

ii. Proposed Reservoir low-flow augmentation capacity: ____________cfs
Duration: ________days

Item 15 - Modified Source Load Control with Flow Augmentation.

i. Available Flow for Augmentation for the required duration ____________cfs

ii. Revised stream low flow ____________cfs

iii. Revised Level of Treatment

	Critical Parameter	Revised Control Level
a)		
b)		

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

Item 16 - Flow Augmentation Cost

i. Modified Level of Treatment Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage Value
	Yr. to Yr.			Start	End	Year	Cost	
Existing Facility								
1								
2								
3								
n								

ii. Flow Augmentation Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage Value
	Yr. to Yr.			Start	End	Year	Cost	
Existing Facility								
1								
2								
3								
n								

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

Item 16 - Flow Augmentation Cost (continued).

iii. Project Cost Schedule.

Phase	Year to Year	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
1		16i					
		16ii					
	Total Phase 1						
2		16i					
		16ii					
	Total Phase 2						
3		16i					
		16ii					
	Total Phase 3						
n		16i					
		16ii					
	Total Phase n						

Replacement Schedule

Year	Cost

iv. Project Present-Worth cost.

Interest ______________________%

Present-Worth Cost $______________

By ______________ Date ______________ Strategy No. ______________
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Notes on Methodology Logic

Step 1 (Item 1) - This step of the evaluation will indicate if there might be a monetary cost advantage for the use of an environmental modification with the selected control alternatives. This will be indicated by the nature of the control objective as indicated by the desired versus actual water quality condition. This decision can be aided by determining whether the source load-reduction identified for the strategy and for the source under consideration is determined by the water quality standards of the receiving water (Water Quality Limited) or treatment standards (Effluent Limited). If the segment is Water Quality Limited, then modifying the impact of the discharge is more likely to favorably alter the physical characteristic of the receiving water.

Step 2 (Item 2) - The relocation of the source load discharge point might be feasible if another stream or other body of water near the discharge site has less stringent water quality criteria than the proposed receiving water, or if another location on the existing receiving water has a greater assimilative capacity than the existing discharge site, etc., as described by general conditions in this item. The user should note that unless there is a reduced cost associated with treatment, the cost of transporting the treated source effluent to the new discharge site will make discharge relocation an unattractive monetary cost alternative.

Step 2a, b, c (Items 3, 4) - The required level of treatment at the alternate site should be evaluated utilizing the impact analysis in Chapter 5. If this level of treatment is confirmed to be less than that required for discharge at the existing site, then the costs associated with transportation and the lower treatment level should be evaluated. A regionalized discharge also could be considered.

Step 2d, e, f, g, h (Items 5, 6, 7) - The monetary cost associated with discharge relocation for each identified alternative site is evaluated using the appropriate methodologies.

Step 3a (Item 8) - Artificial stream reaeration will be a feasible alternative when the water quality parameter of concern is the minimum D.O. of the receiving water. This information should be available from earlier determinations (Item 1).

Step 3b (Item 9) - Where the receiving water D.O. is the critical water quality parameter, it will be necessary to run the water quality impact analysis in Chapter 5 using an increased wastewater source load to the receiving water to define the effect on stream D.O. (Item 9). The D.O. deficit of the receiving water should therefore be defined using this modified load.

Step 3c (Item 10) - The monetary cost associated with artificial reaeration of the receiving water is determined in Item 10. First, the oxygen deficit at the critical point in the receiving water is calculated on the basis of the stream flow rate at the model conditions, the minimum (critical) D.O. concentration at the critical point, and the desired D.O. level at that point. The required oxygen transfer can therefore be defined. In addition, the estimated time that reaeration will be required should also be estimated. The cost for artificial reaeration can then be computed utilizing site-specific information.

Step 3d, e (Items 11, 12) - The treatment cost for the modified level of treatment associated with the artificial reaeration evaluation should be computed using the TREATMENT FACILITY METHODOLOGY. The Present-Worth cost of the artificial reaeration alternative can then be evaluated using the PRESENT-WORTH METHODOLOGY.

Step 4a (Item 13) - This step is used to determine if augmentation of the receiving water flow might be feasible as an alternative to the proposed level of treatment for the source. That is, if the source load level of treatment is defined by the low-flow condition of the receiving water, then it might be feasible to increase the allowable load of the critical parameters by increasing the flow rate of the receiving water. Therefore, the first condition that must be satisfied is that the critical parameter in the allowable level of discharge is determined by the low-flow condition of the receiving water (Item 4). This information should be available from the water quality impact analysis of Chapter 5.

Step 4b, (Item 14) - Where flow augmentation could reduce the level of treatment, the existing capability for low-flow augmentation must be identified. This evaluation will require information on the available water supply upstream of the source discharge that can be used for low-flow augmentation.

The assigned capacity for low-flow augmentation or the potential capacity for this use should be identified as to the available rate of discharge and also the length of time that this discharge can be maintained.

Step 4c (Item 15) - The low-flow rate in the receiving water should be modified to consider the flow available from the identified water supplies, and the water quality impact analysis should be used to calculate the revised level of treatment identified for the critical parameters.

Step 4d, e, f (Items 15, 16) - A revised treatment cost for this identified level of treatment is developed using the appropriate methodology. The Present-Worth cost of flow-augmentation alternatives can then be evaluated by considering the monetary costs associated with utilizing the flow augmentation capacity of the identified water supply and the treatment cost associated with the reduced level of treatment.

The scope of this evaluation does not include consideration of the alternative of creating a water supply, especially for low-flow augmentation for a specific wastewater source of interest. This alternative, however, could be feasible in a specific case. For example, a situation where the natural topography would lend itself to the creation of a reservoir might be a situation where a low-flow augmentation water supply could be readily developed.

6.4.2.10 Regionalization Methodology

Discussion

The purpose of the REGIONALIZATION METHODOLOGY is to identify situations where the monetary cost associated with a control alternative can be reduced by handling two or more waste discharges at a common site. This methodology will direct the user in evaluating regionalization of several wastewater treatment facilities, as well as the utilization of a common disposal site for several residual generators. However, the residuals disposal portion of this methodology is independent of the treatment portion; therefore, it can and should be used independently.

The REGIONALIZATION METHODOLOGY guides the user in identifying feasible regionalization combinations for wastewater treatment, and then determining the cost associated with these combinations. Although generally applicable to municipal point sources, regionalization may include joint municipal-industrial or cooperative-industrial treatment facilities. In the latter case, the user would not estimate costs, but would be interested in effects upon wasteload allocations and water quality.

Following the analysis of the wastewater source combinations, the user evaluates the development of a regional residual disposal facility. This sequence of steps (wastewater regionalization, residuals disposal combination) is recommended since the residuals generated by a regional plant might differ from those generated by the initially-identified, point-source control alternatives.

Utilization of the REGIONALIZATION METHODOLOGY will be most effective when the user is aware of the factors in the specific planning area that would make a combination of several sources into one facility desirable. In most cases, these will be both positive and negative effects from regionalization which the user should consider. Typically, the greatest advantage of a regional facility versus several smaller facilities is the economy of scale associated with constructing, operating, and managing one large facility rather than several smaller ones. Larger and more efficient operating budgets, and efficient distribution and use of personnel are also benefits of concentrating facilities and responsibilities for wastewater management.

In some cases, however, savings in treatment plant construction may be offset by increased collection system and operations and maintenance costs. In evaluating regionalization alternatives, the user should also consider the following potential effects:

1. Potential higher concentration of pollutants.
2. More pronounced consequences of treatment plant failures.
3. More sophisticated treatment plant which may be beyond local operating capabilities.
4. Inducement of urban growth along interceptor sewers.

Although many of the concerns are not specifically addressed in the REGIONALIZATION METHODOLOGY, the user is encouraged to assess these effects in a broader evaluation framework.

Perhaps the most important step in this REGIONALIZATION METHODOLOGY is the identification of potential regional sites. It is not the intent of this manual to limit the user to identifying existing plants or disposal sites as the only potential regional sites. Any tract of land that would be suitable for a large wastewater treatment facility or disposal site should be evaluated.

The treatment facilities (referred to as satellite facilities) that are combined with the proposed regional site are important to the effective use of the REGIONALIZATION METHODLOGY. Typically, the user would want to identify combinations of regional and satellite facilities with higher individual costs, since these combinations would have the greatest impact on the overall alternative cost. However, the user should not limit his evaluation only to the larger wastewater sources.

A decision step is included in this methodology relative to the identification of additional regional/satellite combinations. This is included after each combination has been evaluated to insure: first, that the user will not have to identify every potential regional/satellite combination before determining the impact of these source combinations; and, secondly, to be certain that the evaluation of regionalization continues until the reasonable combinations have been evaluated. This decision step will require good judgment by the user who should utilize the trends apparent from the use of the previous methodologies to evaluate the potential benefit from further combinations. The user should continue these evaluations until the cost savings cease.

In certain cases, even with demonstrable economies, regionalization may not be locally acceptable. Therefore, as part of the evaluation process, the user should be aware of attitudes concerning regionalization.

Methodology Logic

A summary of the logic of the REGIONALIZATION METHODOLOGY is presented in Figure 6-25. An exapnded flowchart, Figure 6-26, lists the steps to be taken in determining performance and costs. The worksheet for recording the operations is presented as Table 6-15. Notes on specific steps or worksheet items are presented following the worksheets.

FIGURE 6-25

REGIONALIZATION METHODOLOGY
LOGIC SUMMARY

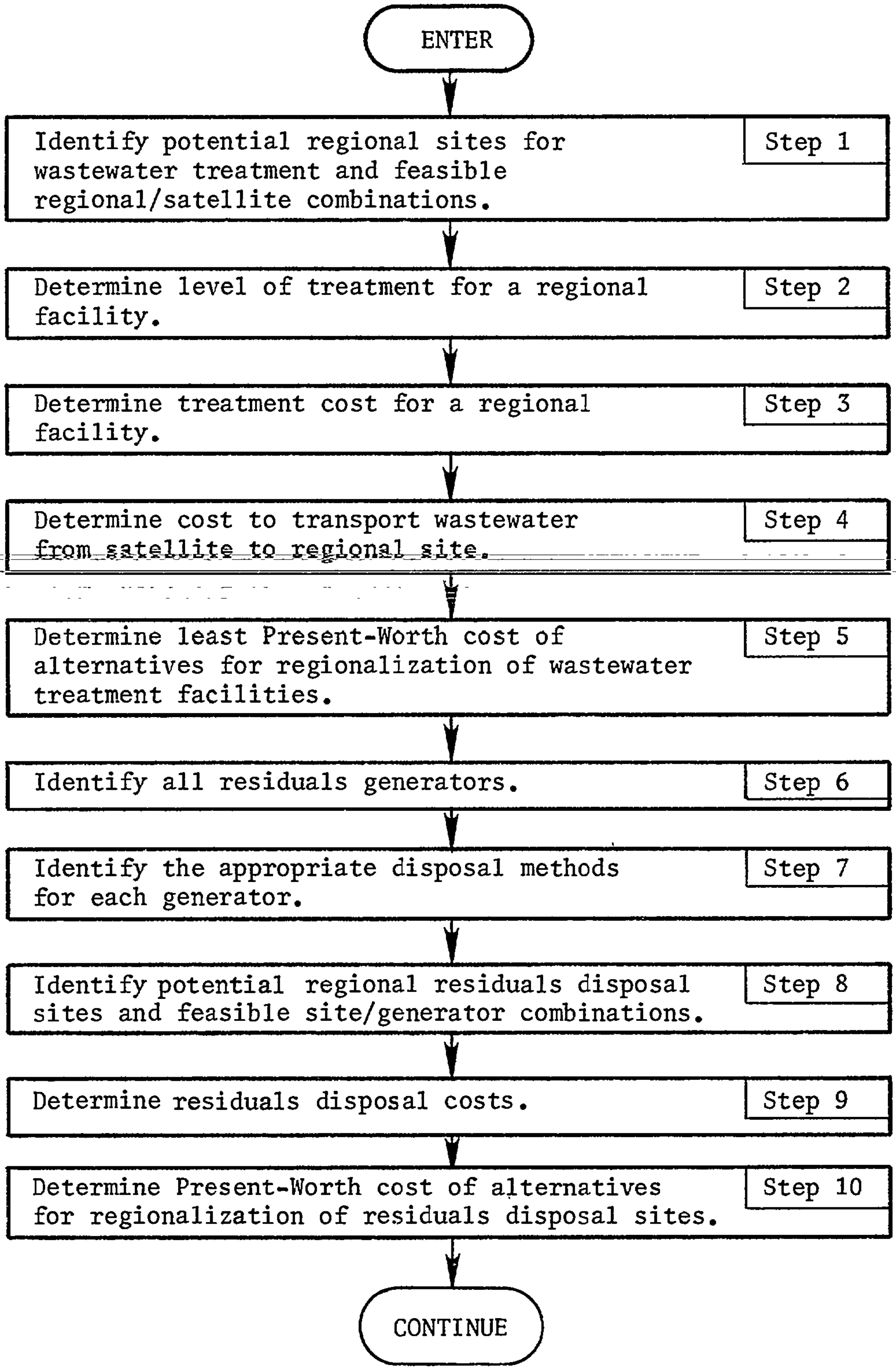

FIGURE 6-26

REGIONALIZATION METHODOLOGY
FLOWCHART

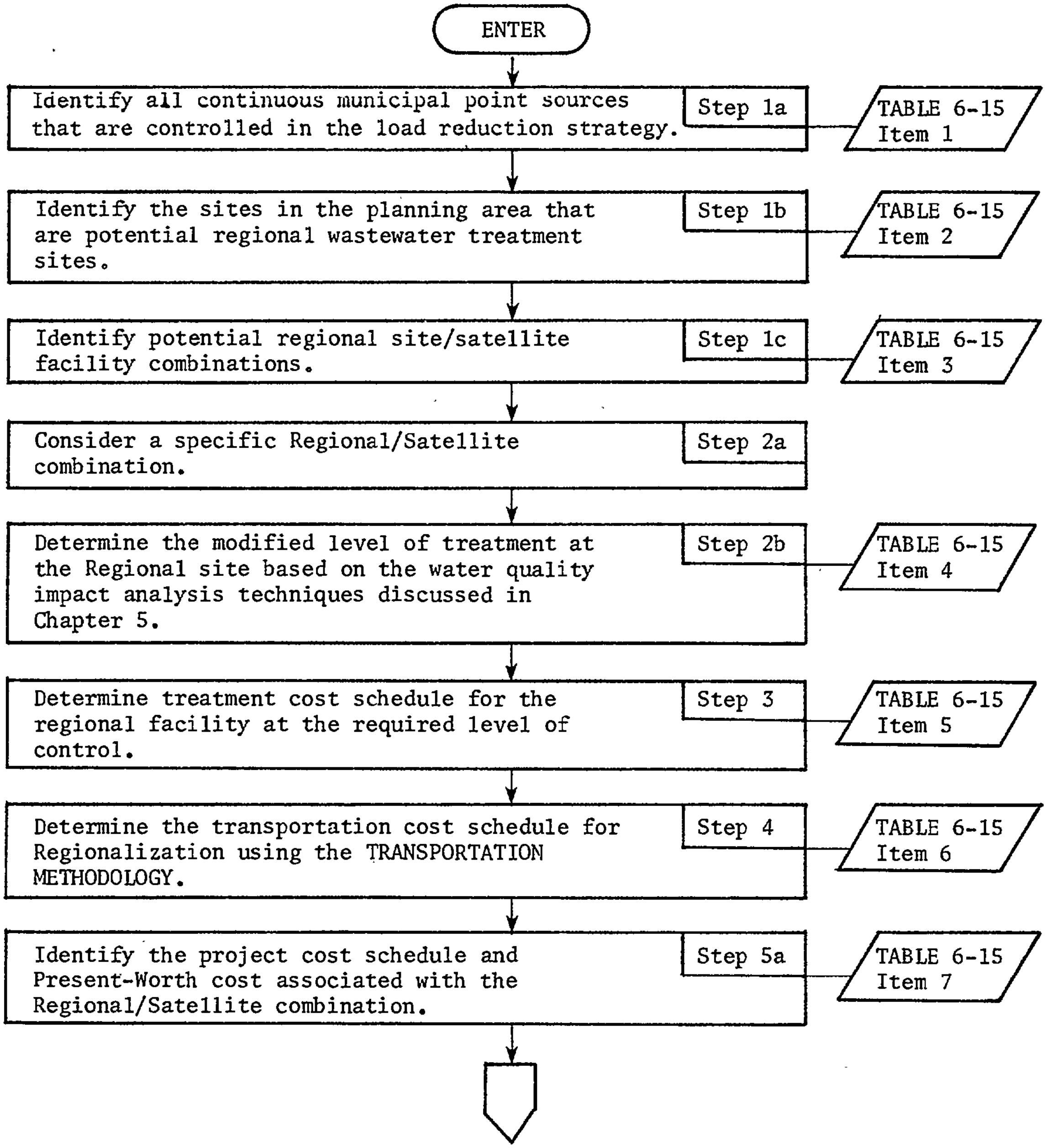

FIGURE 6-26 (CONTINUED)

REGIONALIZATION METHODOLOGY
FLOWCHART

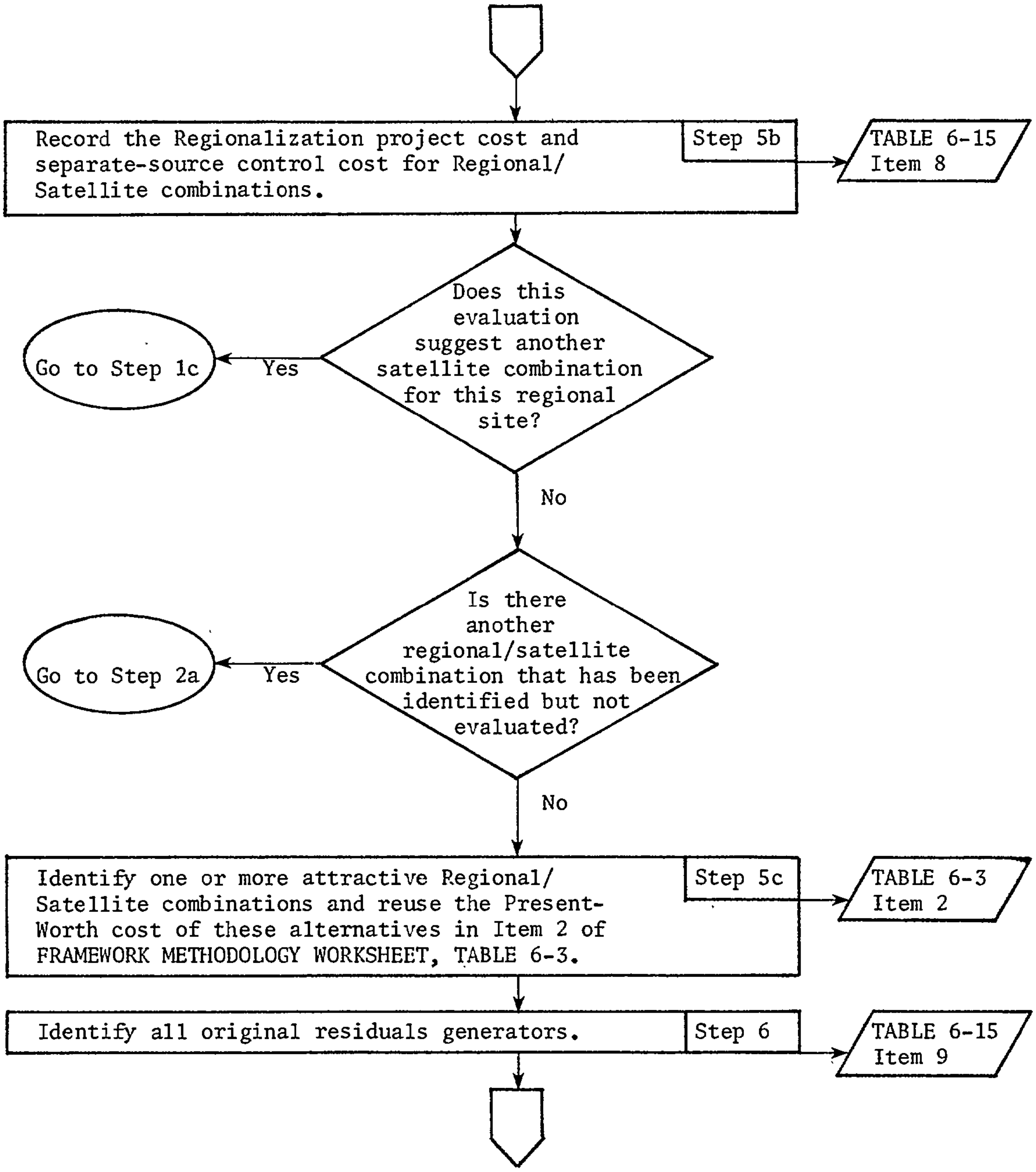

FIGURE 6-26 (CONTINUED)

REGIONALIZATION METHODOLOGY
FLOWCHART

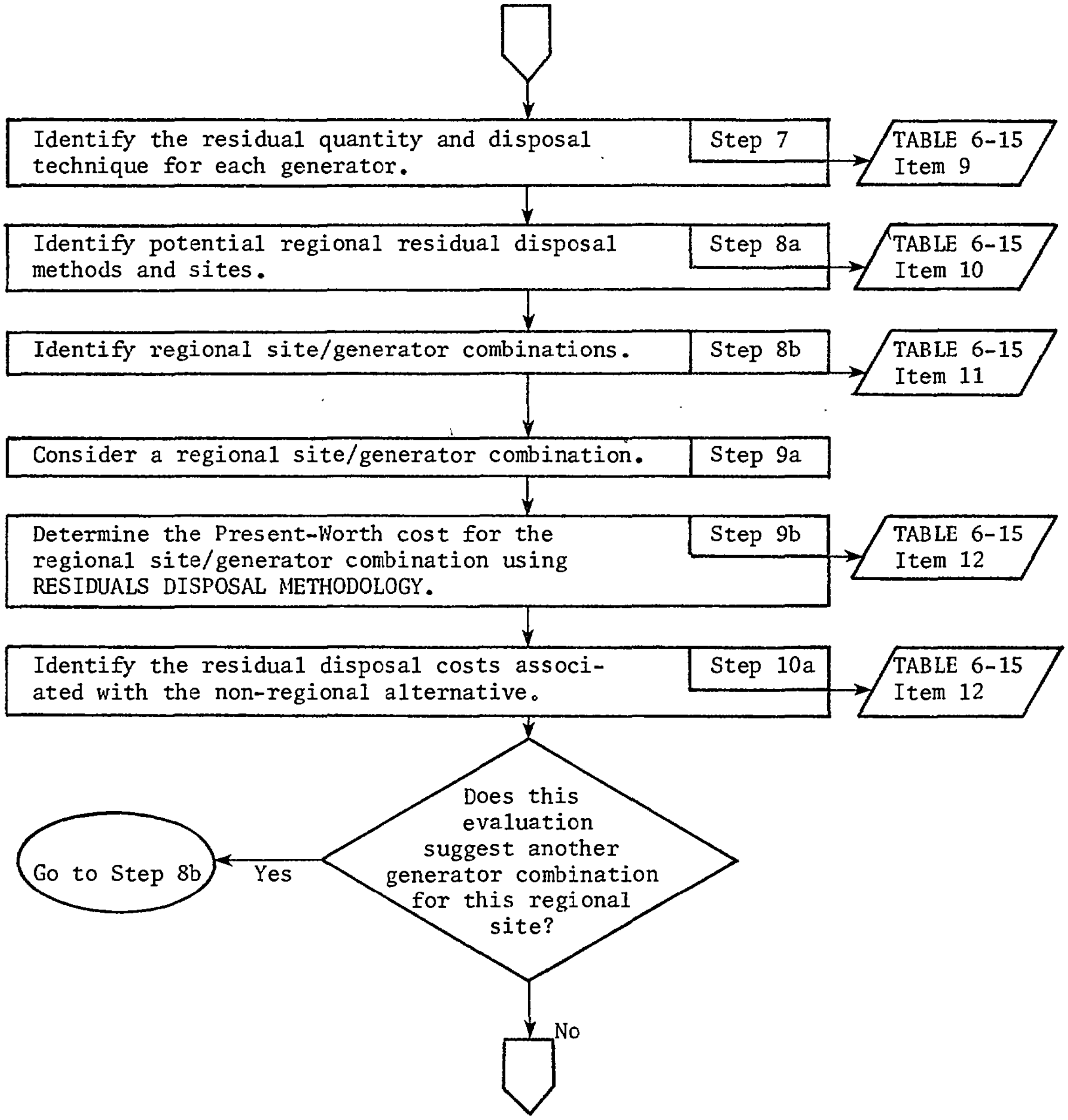

FIGURE 6-26 (CONTINUED)

REGIONALIZATION METHODOLOGY
FLOWCHART

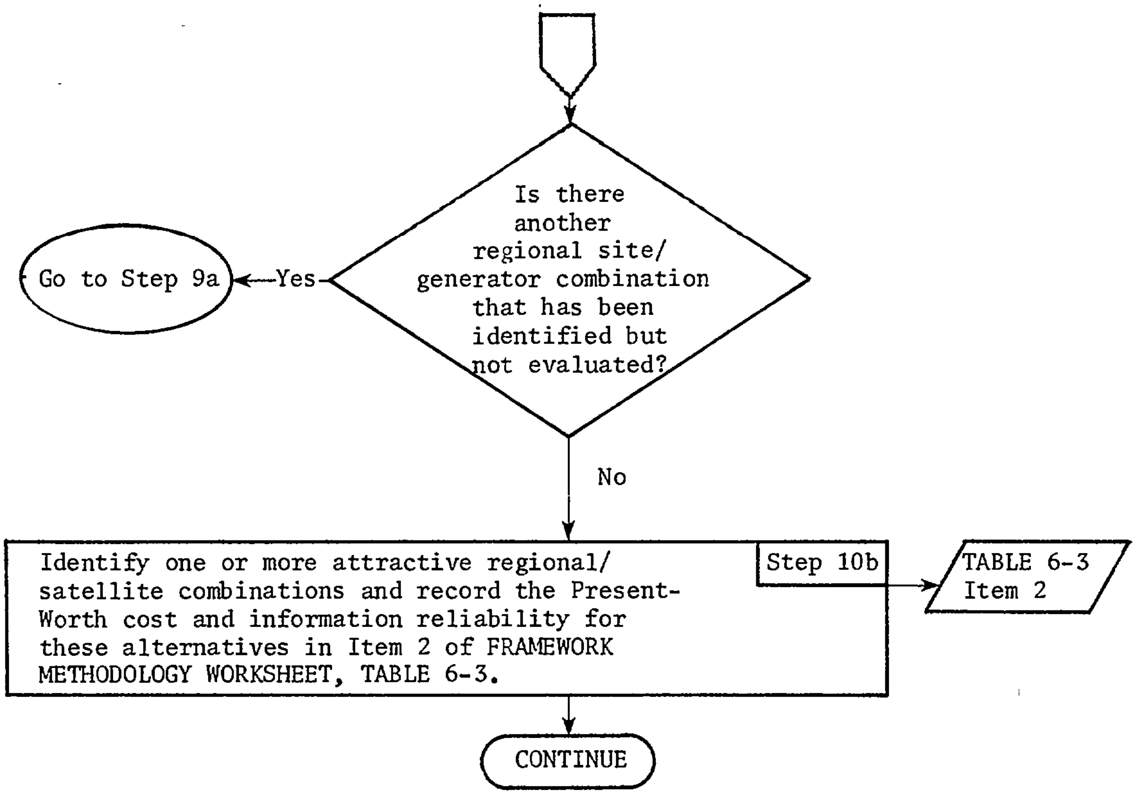

TABLE 6-15
REGIONALIZATION METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approach used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Wastewater Treatment Regional Site Evaluation

Item 1 - Source Identification.
(Attach topographic map with sources located)

	Source/Location	Flow	Level of Control/ Critical Parameters
a)			
b)			
c)			
d)			
e)			
f)			
g)			
h)			
i)			
j)			
k)			
l)			
m)			
n)			

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 2 - Regional Site Identification.
(Attach topographic map with sources located)

	Site/Location	Available Area (acres)	Estimated Max. Flow (mgd)	Existing Condition
a)				
b)				
c)				
d)				
e)				
f)				
g)				
h)				
i)				
j)				
k)				
l)				
m)				
n)				

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 3 - Potential Regional Site/Source Combinations.

Combination I.D. No.	Source 1		Source 2		Regional Site		
	I.D.	Q (mgd)	I.D.	Q (mgd)	I.D.	Max Site Q (mgd)	Max Design Q (mgd)
1							
2							
3							

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

REGIONAL SITE EVALUATION

Regional Combination No. __________

Item 4 - Level of Treatment at Regionalization Site.

	Parameter	Raw Wastewater	Discharge Level	Level of Treatment
a)				
b)				
c)				

Level of Treatment:

Item 5 - Regional Site Wastewater Treatment Cost Schedule.
(Use the TREATMENT FACILITY METHODOLOGY)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

Item 6 - Wastewater Transportation Cost Schedule.
(Use the TRANSPORTATION COST METHODOLOGY)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 7 - Regionalization Cost Evaluation.

i. Cost Schedule (Regional Combination No. __________)

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
1		# 5					
		# 6					
TOTAL PHASE 1							
2		# 5					
		# 6					
TOTAL PHASE 2							
3		# 5					
		# 6					
TOTAL PHASE 3							
4		# 5					
		# 6					
TOTAL PHASE 4							

Replacement Schedule

Item	Year	Cost

ii. Present-Worth Project Cost: $__________

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 8 - Present-Worth Cost.

Regional Site I.D.	Source I.D.		Separate Treatment Costs		Regional Treatment Cost
	No. 1	No. 2	Source 1	Source 2	

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Residuals Disposal Regional Site Evaluation

Item 9 - Residuals Generator Source Identification.

	Source/Location	Flow (mgd)	Residual Disposal Method	Residual Characteristics	
				Quality	Type
a)					
b)					
c)					
d)					
e)					
f)					
g)					
h)					
i)					
j)					
k)					
l)					
m)					
n)					

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 10 - Potential Residual Disposal Sites.

	Site/Location	Available Area (acres)	Estimated Capacity	Existing Condition
a)				
b)				
c)				
d)				
e)				
f)				
g)				
h)				
i)				
j)				
k)				
l)				
m)				
n)				

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 11 - Potential Regional Site/Generator Combinations.

	Regional Site		Generator		
	Site I.D.	Capacity	I.D.	Location	Quantity
a.					
b.					

Item 12 - Regional Site/Generator Combined Present Worth Costs.
(Using the RESIDUALS DISPOSAL METHODOLOGY)

	Combination Generator I.D.	Present Worth Cost at Combination	Present Worth Cost Separate
a.			
		TOTAL	
b.			
		TOTAL	

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Notes on Methodology Logic

Step 1a (Item 1) - The purpose of this step is to identify the wastewater sources that are controlled in the strategy under consideration. The information developed in this step should be useful in identifying the desired source combinations for the separate regional facilitiy evaluations.

Step 1b (Item 2) - The potential regional sites are identified in this step. There are numerous factors that will indicate a potential regional site in the particular planning area. For the general condition, any existing wastewater treatment site is a potential regional site, especially if there is adequate room for expansion. However, it is not the intent to limit potential regional sites to existing facilities; the user should attempt to identify any suitable location in the planning area that satisfies the basic requirements for a wastewater treatment facility. References should be consulted for more information concerning facilities siting. For example, a large undeveloped area lying in the proximity of several small treatment sites might be identified as a potential regional site if it were at a lower elevation than these sites. The information identified for the potential regional sites should be recorded on the worksheet (Item 2) and should include: the location of the site, the available area, the estimated maximum treatment capacity (flow), and the existing condition of the site (e.g., existing structures, utilities, roads, vegetation, etc.).

Step 1c (Item 3) - In identifying the source combinations for further study, the proposed regional site is designated as the regional site, and the source or sources that are to be relocated to the regional site are identified as satellite sources. The worksheet is designed to evaluate two wastewater sources combined at a regional treatment site, where the regional site can be considered as one of the existing sources when desired. The user can evaluate more complex situations by an iterative procedure utilizing these worksheets and by considering a regional site combination as one of the sources. There is an opportunity later in the methodology to identify additional regional/satellite combinations based on the results of the evaluation.

Step 2a - This step selects one of the combinations identified in Step 1c (Item 3) for a detailed monetary cost evaluation.

Step 2b (Item 4) - The water quality impact analysis techniques in Chapter 5 used to define the level of treatment required for each source must be modified to reflect the exclusion of the loads from the sources that have been relocated to the regional site. The level of treatment required at the regional site is then identified to maintain an acceptable water quality for the receiving stream. When regionalization does not cause a significant change in the pollutant loading pattern to the receiving body, the control level for the regional facilities will be the same as that required at the separate sources.

Step 3 (Item 5) - This step identifies the treatment cost schedule for the regional facility. Several alternative treatment methods can be evaluated as desired using the TREATMENT FACILITY and LAND APPLICATION METHODOLOGIES. The worksheet is designed to present a summary of the selected treatment system cost schedule.

Step 4 (Item 6) - This step identifies the wastewater transportation cost schedule for the regionalization project. This can be developed using the TRANSPORTATION COST METHODOLOGY and summarized in the Regionalization Worksheet.

Step 5a (Item 7) - This step presents the total project cost schedule. This will include treatment costs (Item 5), transportation costs (Item 6), and other special costs identified for the project. The PRESENT-WORTH METHODOLOGY is used in this step to develop the Project Present-Worth cost.

Step 5b (Item 8) - This step summarizes the costs developed for the various regionalization evaluations. The user can utilize Item 8 in identifying control alternatives for further (non-monetary cost) evaluation. Following the summarization of a regional cost, the user should determine if another combination is suggested. This might involve the combination of two previously identified regional facilities into another larger regional facility. This can be accomplished in Step 1c by identifying these regional facilities as the satellite sites.

Step 6 (Item 9) - The purpose of this step is to inventory the residual generators in the planning area. This identification should also include the residuals generated in the wastewater regionalization evaluation. The characteristics of each residual should include the nature of the sludge (biological or chemical) and the condition of the sludge (wet, cake, or ash) as identified in the treatment system cost curves, as well as the general area and disposal options available.

Step 7 (Item 9) - The purpose of this step is to identify the proposed disposal methods for each of the residual generators as either land spreading or landfilling. The usefulness of this inventory will be in later identifying potential combinations for a specific regional disposal site. The quantity of residuals from each generator should also be identified in this step.

Step 8a (Item 10) - In this step the potential regional residual disposal sites are identified. This list, of course, should include all sites identified for the disposal of the individual point-source residuals. However, this list should also include future sites that could be developed for a regional residual disposal site; this is particularly important in evaluating the regionalization of residual disposal sites since the capacity at the existing sites might limit regionalization potential. The characteristic of the potential residual disposal sites should specifically address any condition at the site that would enhance or limit its applicability for residual disposal.

Step 8b (Item 11) - In this step the potential regional site/residual generator combinations are identified. In the first analysis of regionalization, the user should identify the combinations that would have the most significant impact on the monetary costs of the control strategy; typically, this would indicate the larger quantities of residuals and those residuals with a significant disposal cost. In addition, the user may wish to consider a combination of a potential regional site with several residual generators. This would allow the user to identify a common disposal site cost and the individual residual transportation cost in one analysis, thereby saving computational effort. The user should rely upon the experience obtained in performing the individual residual disposal evaluations to identify these combinations, as well as any local situation that would impact on a regional residual disposal site.

Step 9a - In this step the user selects a residual disposal combination for the Present-Worth evaluation.

Step 9b (Item 12) - The Present Worth cost for the regional disposal site/ generator combination is evaluated in this step. This cost is developed using the Residual Disposal Methodology and the combined residual characteristics. The costs associated with the regional disposal site include the site cost and the transportation cost for each residual to the site.

Step 10a (Item 12) - This step identifies the residual disposal costs determined when the separate disposal evaluations were performed. The regional disposal cost can then be compared to the separate disposal cost to identify relative monetary costs. This might suggest further residual combinations for evaluation.

Step 10b - In this step the user identifies the least Present-Worth cost residual disposal technique from the information summarized in Item 12.

6.4.2.11 Present-Worth Methodology

Discussion

The purpose of this methodology is to determine the Present-Worth cost of an alternative. The worksheet is designed to evaluate project cost schedules developed by the evaluation procedures for each control alternative; it can handle projects with up to four construction phases, but could readily be modified to include more. The costs identified in the worksheet include construction cost, constant and variable operation and maintenance (O & M) costs, existing facility phase out costs, facility replacement costs, and facility salvage value. This procedure converts these costs over the project life into an equivalent cost that represents the current investment that would be required to satisfy all of the identified project costs for the planning period. For a more detailed discussion, the user may consult any standard engineering economy text, including reference(20).

The construction costs incurred by the project represent single-payment costs that occur at certain times throughout the planning period. The single-payment present-worth factor (sppwf) is used to determine the Present-Worth cost, and is determined by the following formula:

$$\text{sppwf} = \frac{1}{(1 + i)^n} \qquad (6\text{-}1)$$

where:

i is the interest
n is the number of interest periods

The operation and maintenance (O & M) cost includes both constant and variable costs. The constant O & M cost is based on the flow rate at the beginning of the planning period. The variable O & M cost represents the difference between the O & M cost at the flow rate in the final year of the planning period and the constant O & M cost identified by the flow rate at the beginning of the planning period.

The uniform-series present-worth factor (uspwf) is used to convert the constant annual O & M cost to a Present-Worth cost by the formula:

$$\text{uspwf} = \frac{(1 + i)^n - 1}{i\ (1 + i)^n} \qquad (6\text{-}2)$$

where:

i is the interest rate
n is the number of interest periods

For cases where the constant payment is for a period that does not start at the beginning of the planning period (Phase 2 constant O & M costs), the uniform-series factor must be adjusted by multiplying it by the single-payment present-worth factor for the number of years from the beginning of the planning period to the time that the constant payment begins, as in the following:

$$(\text{uspwf}^{t1}) \times (\text{sppwf}^{t2}) \qquad (6\text{-}3)$$

where:

t1 is the number of years that the constant payment will be made

t2 is the number of years from the beginning of the planning period to the time that the constant payment begins

The variable operation and maintenance costs are assumed to vary linearly through the planning period and are multiplied by the gradient-series present-worth factor for the same number of years that the corresponding constant operation and maintenance is paid (gspwf^a-years). This value is computed as:

$$\text{gspwf} = \frac{\left[\dfrac{(1 + i)^n - 1}{i\ (1 + i)^n}\right] - n\left[\dfrac{1}{(1 + i)^n}\right]}{(i)} \qquad (6\text{-}4)$$

where:

i is the interest rate
n is the number of interest periods that the series is in effect

When using this term for computing the Present-Worth of a variable O & M cost, care must be exercised to insure that the gradient O & M is used (i.e., the annual average increase in O & M costs during the phase.

If the gradient series does not start at the beginning of the planning period, it must be adjusted by multiplying it by the single-payment present-worth factor as follows:

$$(gspwf^{t1}) \times (sppwf^{t2}) \qquad (6\text{-}5)$$

where:

t1 is the period in which the gradient series is in effect

t2 is the number of years from the beginning of the planning period to the time the variable payment is started

In practice, the user will find it more convenient to consult appropriate tables in an engineering economy text; if tables are not available, the preceding formulas will provide comparable results.

The facility replacement cost identifies the cost required to extend the useful life of equipment to the end of the planning period. This is computed when a cpaital item has a service life less than the remaining years in the planning period, and is computed by:

$$\text{Replacement Cost} = \frac{\text{Planning Period - Remaining Service Life}}{\text{Service Life}} \times \text{Capital Value} \qquad (6\text{-}6)$$

where Capital Value represents the capital that would be required today to completely replace the facility. This is a single-payment cost, with Present Worth computed using the factor sppwf.

Finally, the salvage valve represents the value remaining for all capital at the end of the planning period, and is computed by:

$$\text{Salvage Value} = \frac{\text{Service Life - Years to Planning End}}{\text{Service Life}} \times \text{Capital} \qquad (6\text{-}7)$$

where Capital (or Capital Value) represents the initial investment (or cost to replace today). This is a negative cost, with the Present-Worth value computed using the factor sppwf.

Methodology Logic

A summary of the logic of the Present-Worth Methodology is presented in Figure 6-27. An expanded flowchart, Figure 6-28, lists the steps to be taken in determining performance and costs. The worksheet for recording the operations is presented as Table 6-16. Notes on specific steps or worksheet items are presented following the worksheets.

FIGURE 6-27

PRESENT-WORTH METHODOLOGY
LOGIC SUMMARY

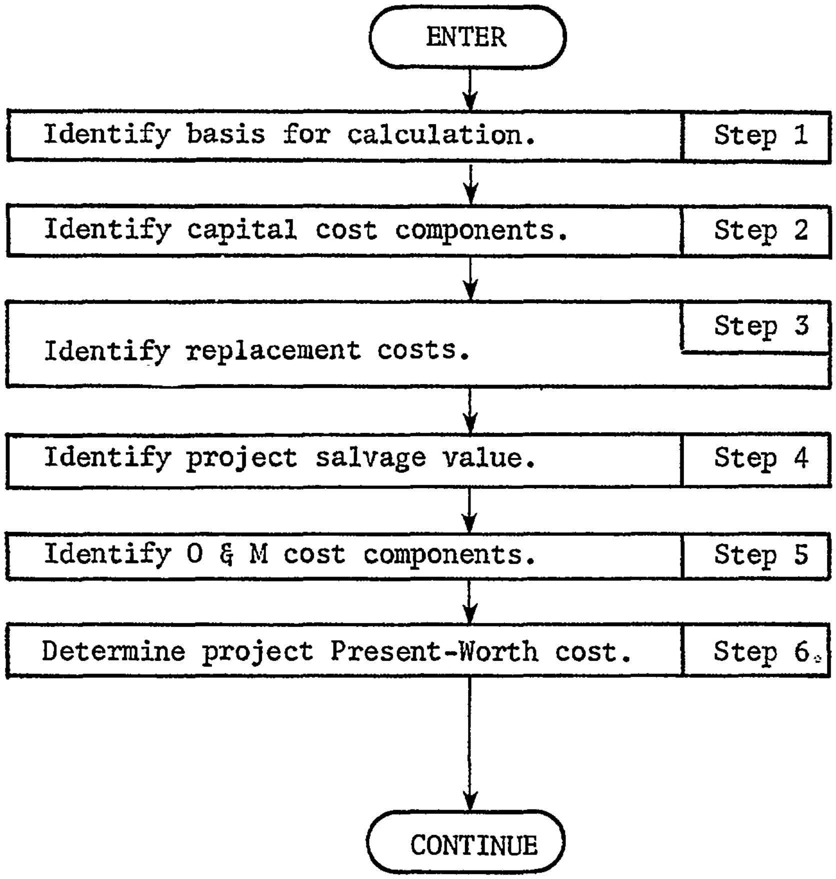

FIGURE 6-28

PRESENT-WORTH METHODOLOGY
FLOWCHART

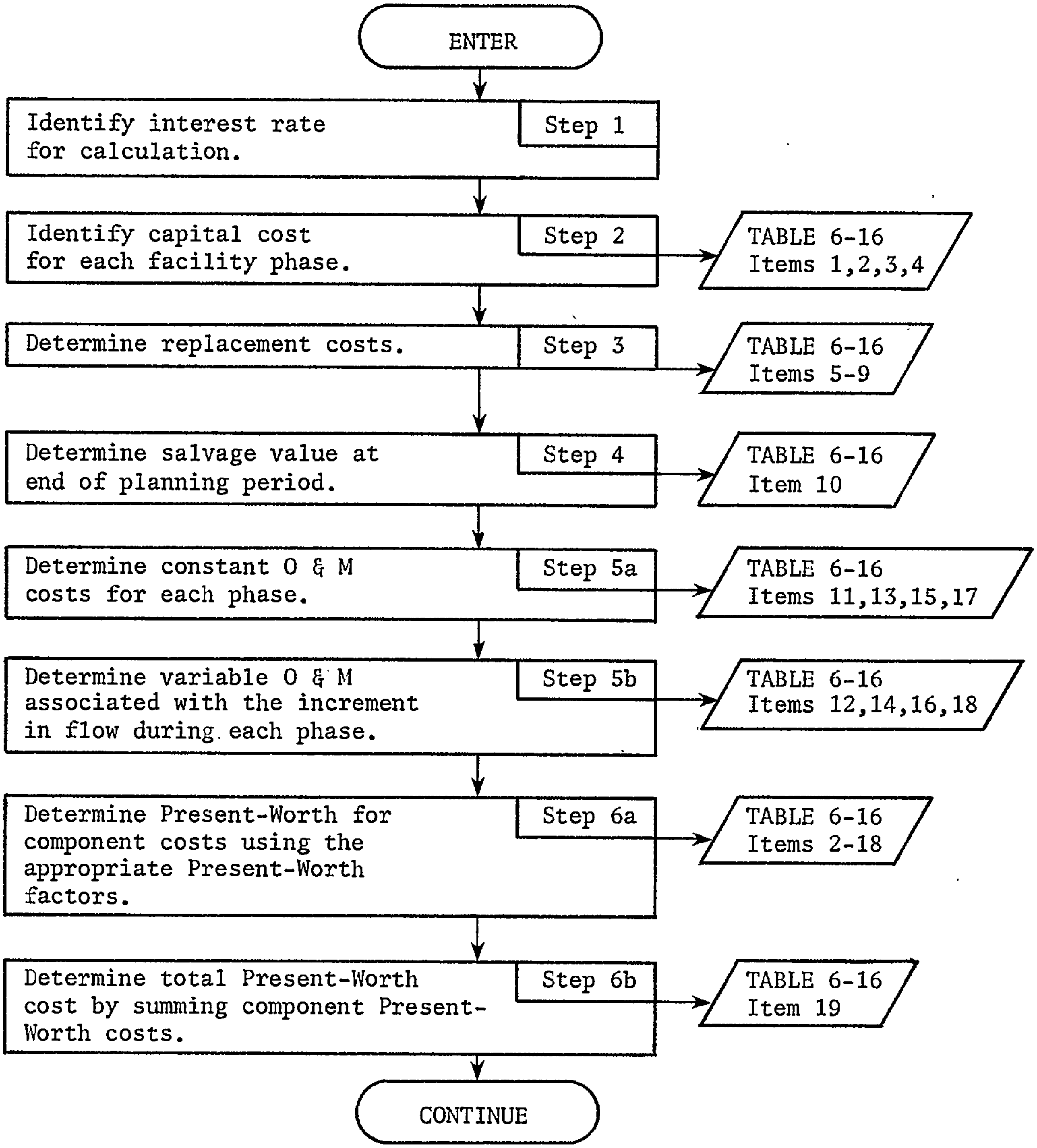

TABLE 6-16
PRESENT-WORTH METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgements presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Items 1-10 Present-Worth Calculation.

Planning Period 20 years   Interest ______ %

Item (Reference Page)	Amount		Present-Worth
1. Phase 1 Capital (pg. ___)	______	x 1.0 (Yr 1)	______
2. Phase 2 Capital (pg. ___)	______	x ______ ($sppwf^a$) (Yr ___)	______
3. Phase 3 Capital (pg. ___)	______	x ______ ($sppwf^b$) (Yr ___)	______
4. Phase n Capital (pg. ___)	______	x ______ ($sppwf^c$) (Yr ___)	______
5. Replacement year (h) (pg. ___)	______	x ______ ($sppwf^h$) (Yr ___)	______
6. Replacement year (i) (pg. ___)	______	x ______ ($sppwf^i$) (Yr ___)	______
7. Replacement year (j) (pg. ___)	______	x ______ ($sppwf^j$) (Yr ___)	______
8. Replacement year (k) (pg. ___)	______	x ______ ($sppwf^k$) (Yr ___)	______
9. Replacement year (1) (pg. ___)	______	x ______ ($sppwf^1$) (Yr ___)	______
10. Salvage Value (Negative Cost) (pg. ___)	______	x ______ ($sppwf^z$) (Yr ___)	______

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TABLE 6-16 (continued)
PRESENT-WORTH METHODOLOGY
WORKSHEET

Item 11 - 19 Present-Worth Calculation.

Planning Period 20 years Interest ________ %

	Item (Reference Page)	Amount		Present-Worth
11.	O&M Phase 1 Constant (pg. ___)	______	x ____(uspwf^{d}) x 1.0 (#Yrs____;Yr 1)	__________
12.	O&M Phase 1 Variable (pg. ___)	______	x ____(gspwf^{d}) x 1.0 (#Yrs____;Yr 1)	__________
13.	O&M Phase 2 Constant (pg. ___)	______	x ____(uspwf^{e} x sppwf^{a}) (#Yrs____;Yr____)	__________
14.	O&M Phase 2 Variable (pg. ___)	______	x ____(gspwf^{e} x sppwf^{a}) (#Yrs____;Yr____)	__________
15.	O&M Phase 3 Constant (pg. ___)	______	x ____(uspwf^{f} x sppwf^{b}) (#Yrs____;Yr____)	__________
16.	O&M Phase 3 Variable (pg. ___)	______	x ____(gspwf^{f} x sppwf^{b}) (#Yrs____;Yr____)	__________
17.	O&M Phase n Constant (pg.___)	______	x ____(uspwf^{g} x sppwf^{c}) (#Yrs____;Yr____)	__________
18.	O&M Phase n Variable (pg.___)	______	x ____(gspwf^{g} x sppwf^{c}) (#Yrs____;Yr____)	__________
19.			TOTAL PRESENT-WORTH	$__________

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Notes on Methodology Logic

Step 1 - The rate of interest should be selected to realistically reflect the prevailing interest rates and inflationary trends. The actual interest rate which must be used is published annually by the U.S. Water Resources Council 18 CFR 704.39, Discount Rate, Federal Register published annually. The interest rate selected should be the same for each alternative.

Step 2 (Items 1, 2, 3, 4) - The capital cost for each phase will be available from the project cost schedule. The timing for each capital investment and all other costs should be recorded on the worksheet in (Yr____), with the page number of the calculation recorded in (pg. ____). The superscripts (a, b, c, etc.) are for reference purposes to aid in identifying the factors that are identical.

Step 3 (Items 5, 6, 7, 8, 9) - The replacement cost schedule will be available from the project cost schedule.

Step 4 (Item 10) - The project salvage value for all capital expenditure will be available from the project cost schedule, and represents a negative cost at the end of the period.

Step 5a (Items 11, 13, 15, 17) - The constant O & M costs will be available from the project cost schedule.

Step 5b (Items 12, 14, 16, 18) - The variable O & M costs will be available from the project cost schedule or can be computed using:

$$\underset{\text{(Phase 1)}}{\text{Variable O \& M}} = \frac{\text{Constant O \& M (Phase 2) - Constant O \& M (Phase I)}}{\text{Years in Phase 1}}$$

Step 6a (Items 2 to 18) - The required Present-Worth factors will generally be available from standard tables. If the required interest rate is not identified in the tables, these factors can be computed using the formulas described in the general discussion section.

Step 6b (Item 19) - The Present-Worth is determined by adding the separate Present-Worth costs and subtracting the Present-Worth of the salvage value.

6.4.2.12 Residuals Disposal Methodology

Discussion

The purpose of this methodology is to identify costs associated with disposing of the residuals generated by a wastewater treatment process. The TREATMENT FACILITY METHODOLOGY identifies residual handling and treatment costs as well as the wastewater treatment costs; therefore, this methodology will identify only the costs associated with transporting the treated residuals to the disposal site and with disposing of the residuals at the site.

This methodology includes two ultimate disposal alternatives for wastewater treatment residues: land spreading or landfilling. These disposal techniques are similar in their transportation requirements and are the most widely used techniques at this time. However, there is no intent to limit the user to evaluation of these alternatives only. For discussion of other residuals handling techniques, the user should consult references(21-24). There may be opportunities for cost-effective disposal of wastes that would apply only to a specific locality, such as an abandoned mine. The user is encouraged to seek a local solution to his disposal problem. Even if the alternatives other than the two suggested above are considered, parts of this methodology can serve as guidance in determining costs. Evaluation of residuals disposal practices must include consideration of local attitudes, potential socio-economic effects, and public health implications.

Methodology Logic

A summary of the logic of the RESIDUAL DISPOSAL METHODOLOGY is presented in Figure 6-29. An expanded flowchart, Figure 6-30, lists the steps to be taken in determining performance and costs. The worksheet for recording the operations is presented in Table 6-17. Notes on specific steps and worksheet items are included after the worksheets.

FIGURE 6-29

RESIDUALS DISPOSAL METHODOLOGY
LOGIC SUMMARY

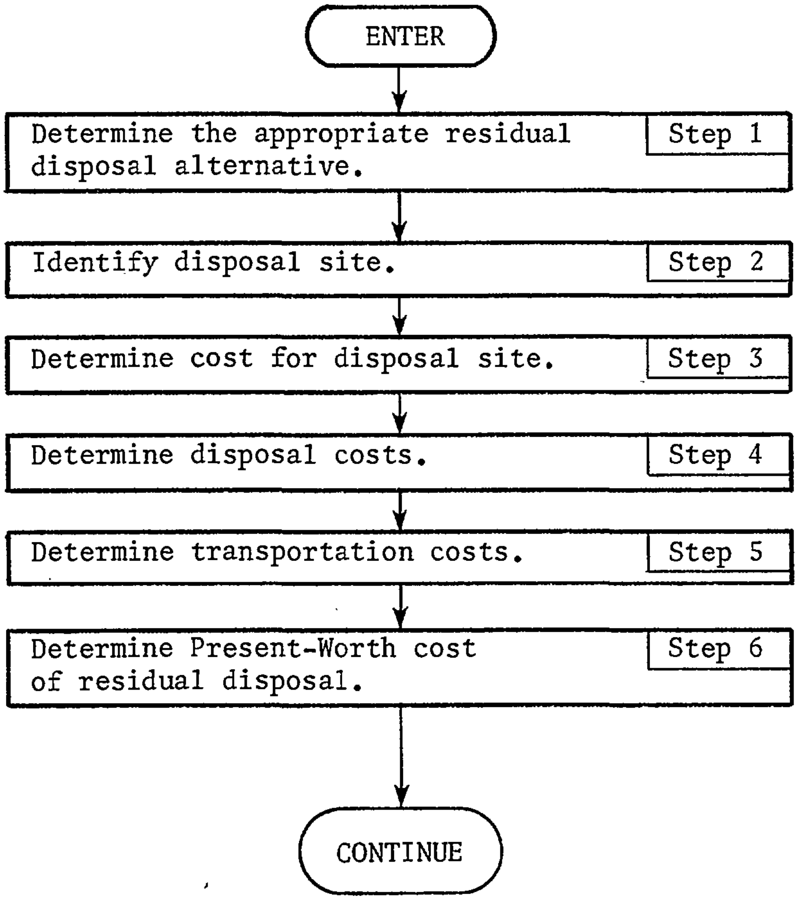


FIGURE 6-30

RESIDUALS DISPOSAL METHODOLOGY FLOWCHART

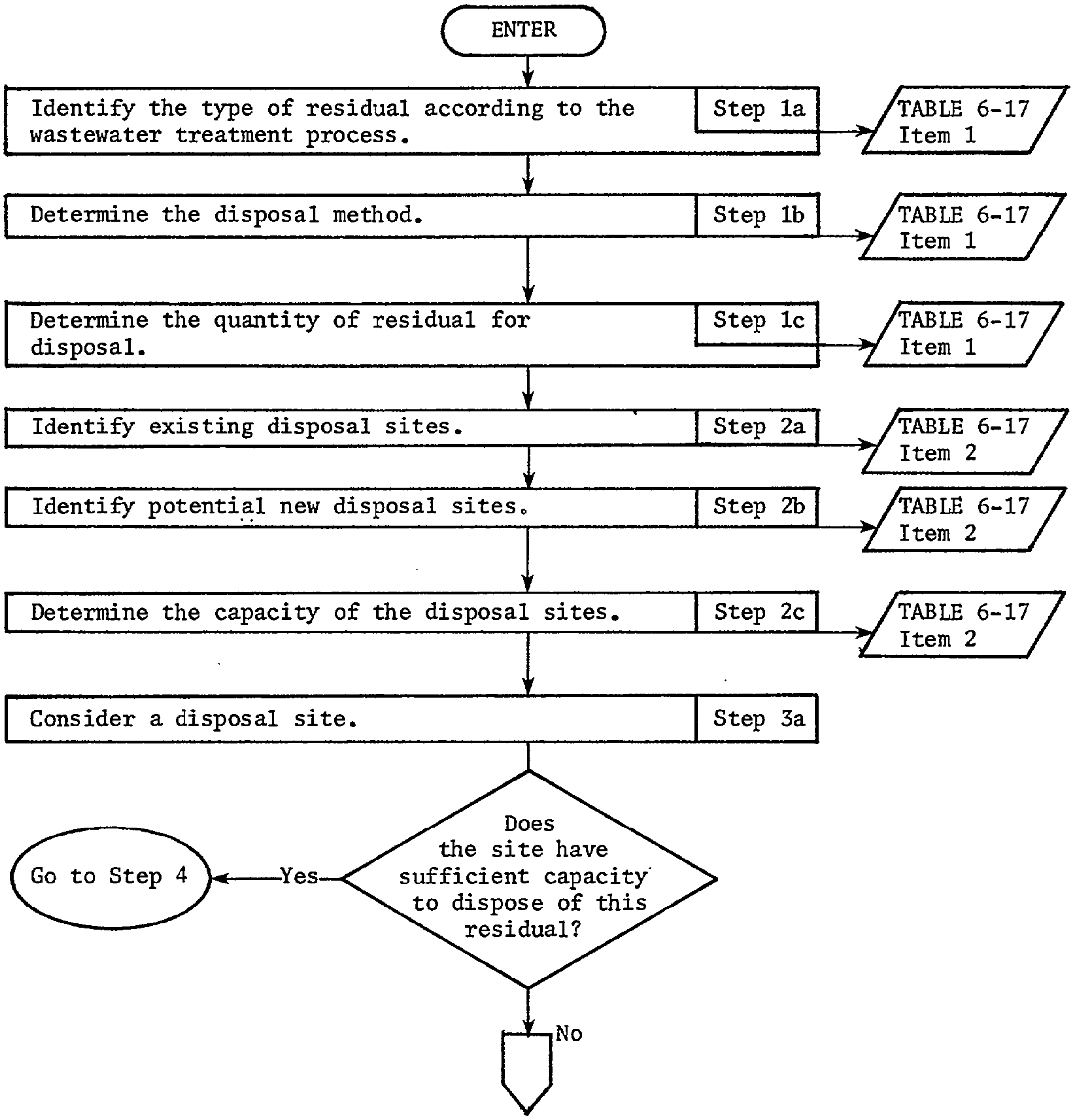


FIGURE 6-30 (continued)

RESIDUALS DISPOSAL METHODOLOGY
FLOWCHART

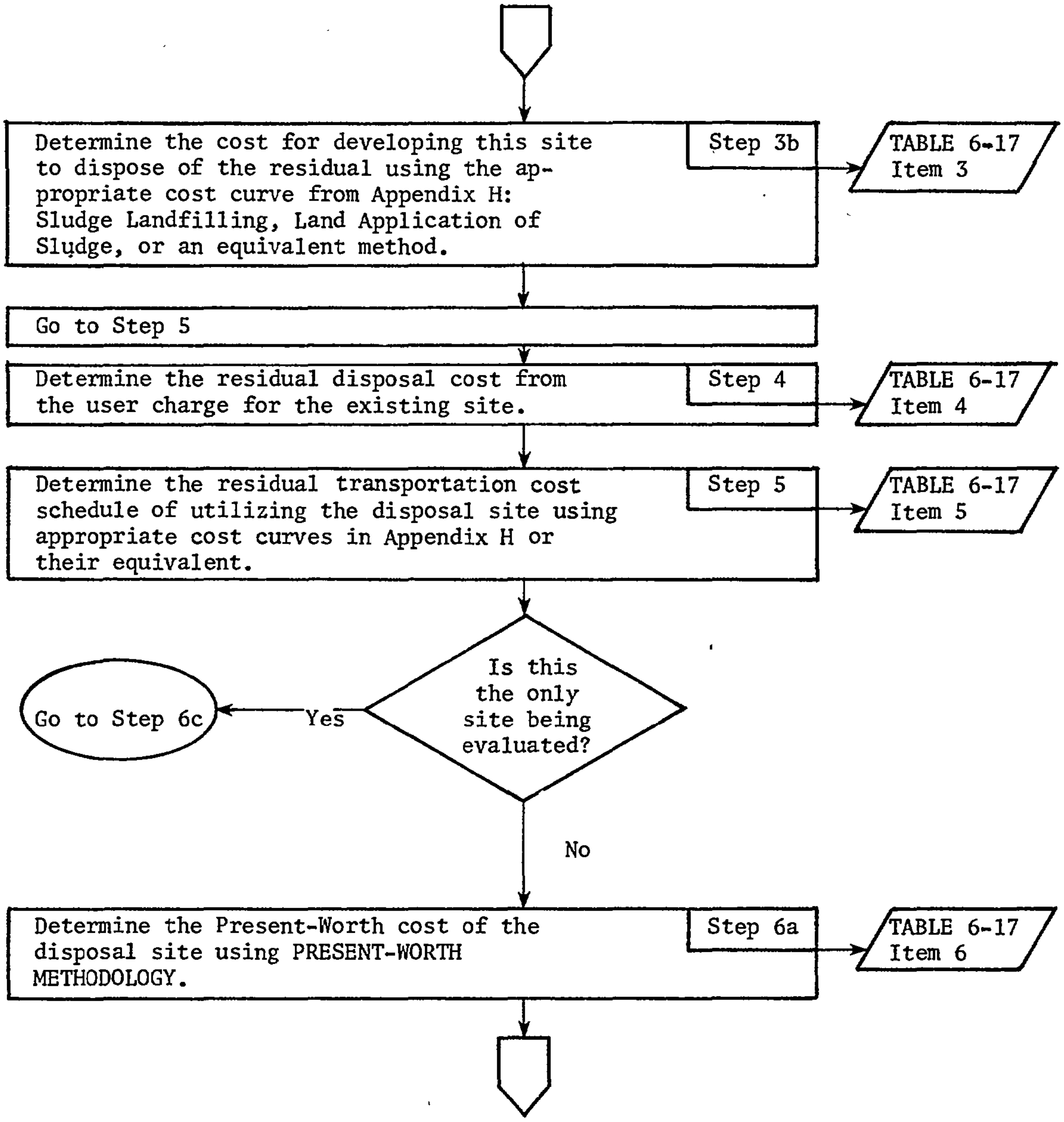
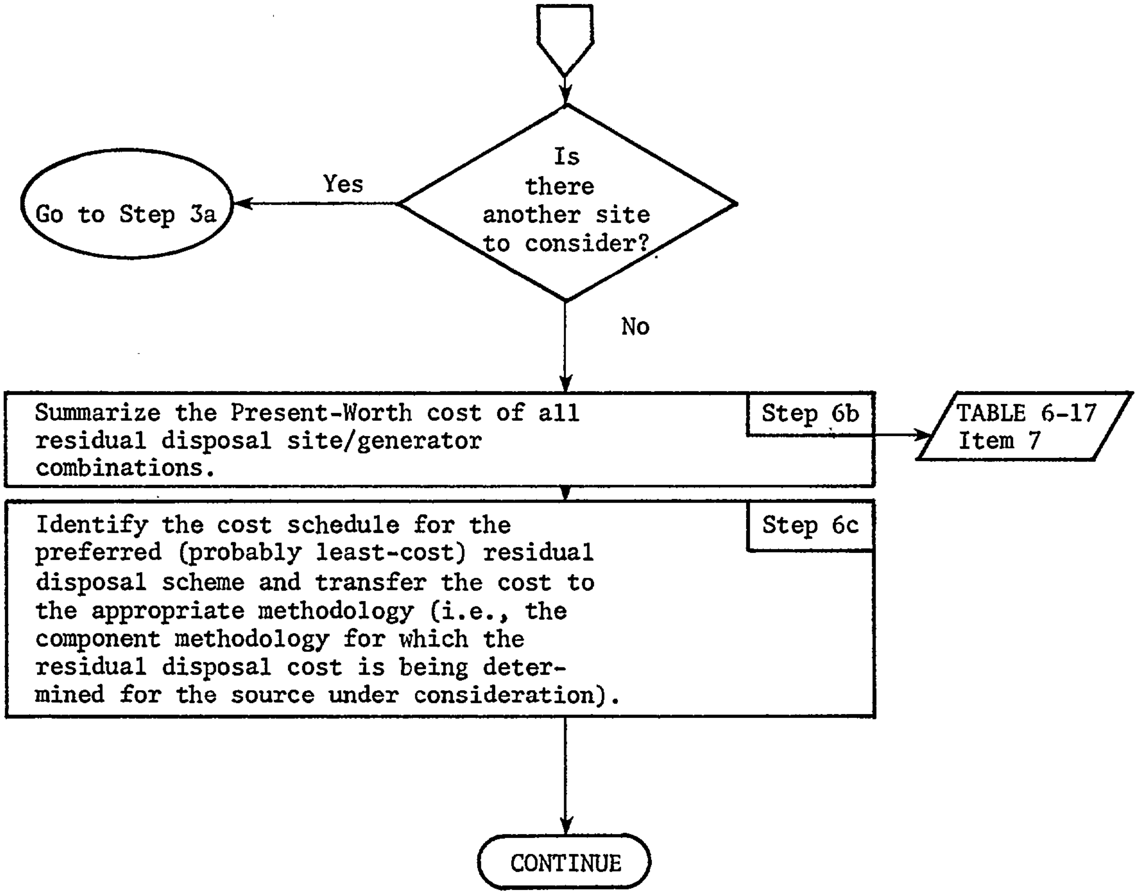


FIGURE 6-30 (continued)

RESIDUALS DISPOSAL METHODOLOGY FLOWCHART



Summarize the Present-Worth cost of all residual disposal site/generator combinations.

Identify the cost schedule for the preferred (probably least-cost) residual disposal scheme and transfer the cost to the appropriate methodology (i.e., the component methodology for which the residual disposal cost is being determined for the source under consideration).

TABLE 6-17
RESIDUALS DISPOSAL METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - Residual Characteristics.

i. Residual generator method of treatment:

____

____

ii. Residual description: ______________________________

iii. Feasible disposal techniques: ______________________________

IV. Residual quantity: (ultimate)

Design Year Flow Rate ______________ mgd

Residual Generation ______________ lb/mgd (dry basis)

Disposal Quantity ______________ lb/day (dry basis)

Item 2 - Residual Disposal Sites.

i. Site characteristics summary.

	Site	Location	Distance from Generator	Estimated Useful Capacity	Comments
a) Existing Land Spreading Sites.	____	____	____	____	____
b) Existing Landfill Sites.	____	____	____	____	____
c) Potential Sites.	____	____	____	____	____
i) Landfill.	____	____	____	____	____
ii) Land Spreading.	____	____	____	____	____

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TABLE 6-17 (continued)
RESIDUALS DISPOSAL METHODOLOGY
WORKSHEET

Item 2 - continued

iii. Site capacity evaluation - existing site ______

a) Useful life at existing disposal rate: __________ years

b) Existing disposal rate: __________ lb/day or ton/day or cu yd/day

c) Proposed disposal rate: __________ (same units as above)
(use average rate for entire planning period)

d) Useful life at proposed disposal rate = (a) x $\frac{(b)}{(c)}$ = __________

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TABLE 6-17 (continued)
RESIDUALS DISPOSAL METHODOLOGY
WORKSHEET

Item 3 - Cost for Development of Residual Disposal Site.

i. Program Schedule.

Phase	Timing Yr to Yr		Design Flows (mgd) Start	End
1	______ to ______		______	______
2	______ to ______		______	______
3	______ to ______		______	______
n	______ to ______		______	______

ii. Site Development Schedule.

Phase	Design Flow (mgd)	Land Required[1] (acre)	Land Cost[2]
1	______	______	______
2	______	______	______
3	______	______	______
n	______	______	______

[1] Land Required = ____________ acre/mgd

[2] Land Cost = $____________/acre

iii. Project Cost Schedule - Disposal Site.

Phase	Timing Yr to Yr		Capital[1]	O&M Start	End	Replacement Cost[2] Year	Cost	Salvage Value[2]
Land Cost								
1								
2								
3								
n								

[1] Capital computed as described for TREATMENT FACILITY METHODOLOGY; includes construction add on's to reflect installed capital. Capital = curve $ x (1.0 + Construction Add on's.)

[2] Cost computed as described for TREATMENT FACILITY METHODOLOGY; attach appropriate computations.

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TABLE 6-17 (continued)
RESIDUALS DISPOSAL METHODOLOGY
WORKSHEET

Item 4 - Residual Disposal Cost for Existing Site.

Program Costs

Phase	Timing Yr to Yr	Design Flow (mgd) Start	Design Flow (mgd) End	Residual Quantity[1] Start	Residual Quantity[1] End	Disposal Cost[2] Start	Disposal Cost[2] End
1							
2							
3							
n							

[1] Determined using __________ lb/mg

[2] Determined using ($________/lb disposal cost) x (Residual Quantity lb/yr)

Item 5 - Residual Transportation Cost (using appropriate cost curves in Appendix H, Figures H-86 to H-90, or equivalent.

i. Transportation Method: ____________________________

Residual Characteristics: Type ________ Solids ________

ii. Transportation Cost Schedule.

Phase	Timing Yr to Yr		Capital[1]	O&M Start	O&M End	Design Flow Start	Design Flow End	Replacement Cost Year	Replacement Cost Cost	Salvage Value
1										
2										
3										
n										

[1] Develop capital estimate based on end design flow for each phase; include construction add on's to reflect installed cost.

[2] Develop O&M estimate (start and end) for each phase.

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TABLE 6-17 (continued)
RESIDUALS DISPOSAL METHODOLOGY
WORKSHEET

Item 6 - Residual Disposal Site Present-Worth Cost

i. Alternative Characteristics:

Residual ______________

Solids ______________

Transportation Method ______________

Disposal Method ______________

ii. Present-Worth Cost $ ______________
(from PRESENT-WORTH METHODOLOGY)

Reference Sheets ______________

Item 7 - Site Alternatives Summary - Residuals Disposal

Site	Location	Present-Worth Cost

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Notes on Methodology Logic

Step 1a (Item 1i, ii) - This step of the evaluation will identify the nature of the residual generated by the wastewater treatment facility, referred to as the generator. The residual will be identified as either biological or chemical, depending on the nature of the treatment at the point source. In addition, the solids content of the residual should be estimated by considering the residual treatment process utilized at the treatment facility. In many instances where more complex treatment processes are required, more than one type of sludge might be identified.

Step 1b (Item 1iii) - The feasible disposal techniques are identified in this step. The table below describes a classification for selecting these techniques that might be useful in identifying the general potential for a particular residual. However, many local, state, and federal regulations exist concerning pathogens, solids content, etc; the user is advised to modify this table to reflect the local condition.

	Residual Type				
	Biological			Chemical	
	Wet	Cake	Ash	Cake	Ash
Land Spreading	X	X	-	-	-
Landfill	-	X	X	X	X

A wet residual represents a thickened or unthickened residual (solids at 1 to 10 percent). A cake represents a dewatered condition (solids at 15 to 50 percent), and an ash represents the product from an incinerator. The user should determine the type of residual generated by the treatment process under consideration. (Although not addressed in this methodology, the user should also consider other residuals handling techniques such as incineration or composting.)

Step 1c (Item 1iv) - In this step, the quantity of residual generated by the source control method is estimated. This information can be developed by the user for special cases. The treatment systems curves in Appendix H indicate an estimate for typical wastewater treatment processes.

Step 2a (Item 2i) - The identification of existing disposal sites is described in this step. The sites identified for a specific residual might include any existing residual disposal sites, but more typically would include the site currently utilized for disposing the existing residual.

Step 2b (Item 2i) - The identification of potential new disposal sites is described in this step, primarily to insure that the user gives some consideration to the site-specific factors associated with development of a new disposal site. In general, the local authorities will favor utilizing an acceptable existing site due to the adverse social and environmental impacts of disposal site development.

Step 2c (Item 2iii) - This step evaluates the remaining disposal capacity at an existing site. This capacity is most easily identified in terms of the useful life of the site while operating at the increased disposal rate.

Step 3a - No discussion.

Step 3b (Item 3) - This step is utilized if a new site is being developed or if the existing site is being expanded because of insufficient capacity. The site development/expansion schedule should be prepared considering the project design flows and anticipated residual generation. The project cost schedule can then be developed using the cost curves in Appendix H or an equivalent method. The user should modify any cost curve capital estimate to reflect total construction cost (e.g., engineering design and construction contingency) as is done in the TREATMENT FACILITY METHODOLOGY (Table 6-4, Item 3) using reasonable construction estimates.

Step 4 (Item 4) - This step is utilized only when the disposal site has sufficient capacity to accept the residual. In this case, the residual disposal cost will consist of the user charge for the particular disposal site of interest.

Step 5 (Item 5) - This step identifies the transportation cost for residual disposal, which can be developed using the cost curves in Appendix H or their equivalent.

Step 6 (Item 6) - This step identifies the Present-Worth cost of the residual disposal alternative under consideration. This step will be utilized to compare the Present-Worth cost of two or more disposal alternatives. When only one alternative is under consideration, this step will not be used.

Step 6b (Item 7) - This step summarizes the Present-Worth cost of alternative residual disposal site evaluations when several sites are under consideration. Note: this step is not required when only one site is being considered because the site's cost schedule will be transferred to the appropriate project cost schedule for the Present-Worth determination.

Step 6c - This step involves the transfer of the cost schedule for the preferred residual disposal site to the project cost schedule for the control alternative for which the residual disposal cost is being determined, e.g., treatment plant construction.

6.4.2.13 Transportation Cost Methodology

Discussion

The TRANSPORTATION COST METHODOLOGY is intended to guide the user in developing a reliable cost estimate for a pipeline conveyance system. This procedure is applicable for wastewater (including land application and reuse) and liquid sludge transportation. It is used in conjunction with other methodologies, such as regionalization and discharge relocation, whenever it is necessary to determine the cost of transporting wastewaters or sludges by pipeline.

The initial determination in this methodology is the identification of a suitable transportation route between the contributing source and the receiving point. The selected route is then divided into segments defined by breaks in the slope of the pipeline. Each of these segments is then analyzed to determine if a gravity-flow condition exists. When it is determined that gravity flow would not be acceptable for the segment of interest, a force main is assumed. The cost for each segment thus consists of the pipeline and the pumping costs required for force main segments. Although not included in this methodology, the user should also consider other factors which will affect costs, such as local soils and other topographic features (e.g., depth to bedrock, stream crossings) and right-of-way acquisition. The cost associated with each segment is then combined to obtain the overall transportation cost for the relocation of the flow from the contributing source to the receiving point.

Methodology Logic

A summary of the logic of the TRANSPORTATION COST METHODOLOGY is presented in Figure 6-31. An expanded flowchart, Figure 6-32, lists the steps to be taken in determining performance and costs. The worksheets for recording the operations are presented in Table 6-18. Notes on specific steps or worksheet items are presented after the worksheets.

FIGURE 6-31

TRANSPORTATION COST METHODOLOGY
LOGIC SUMMARY

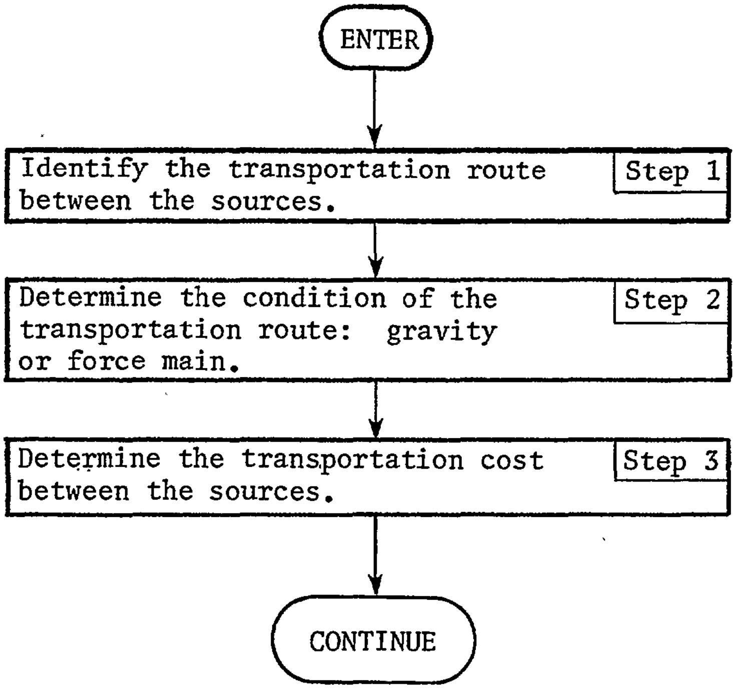
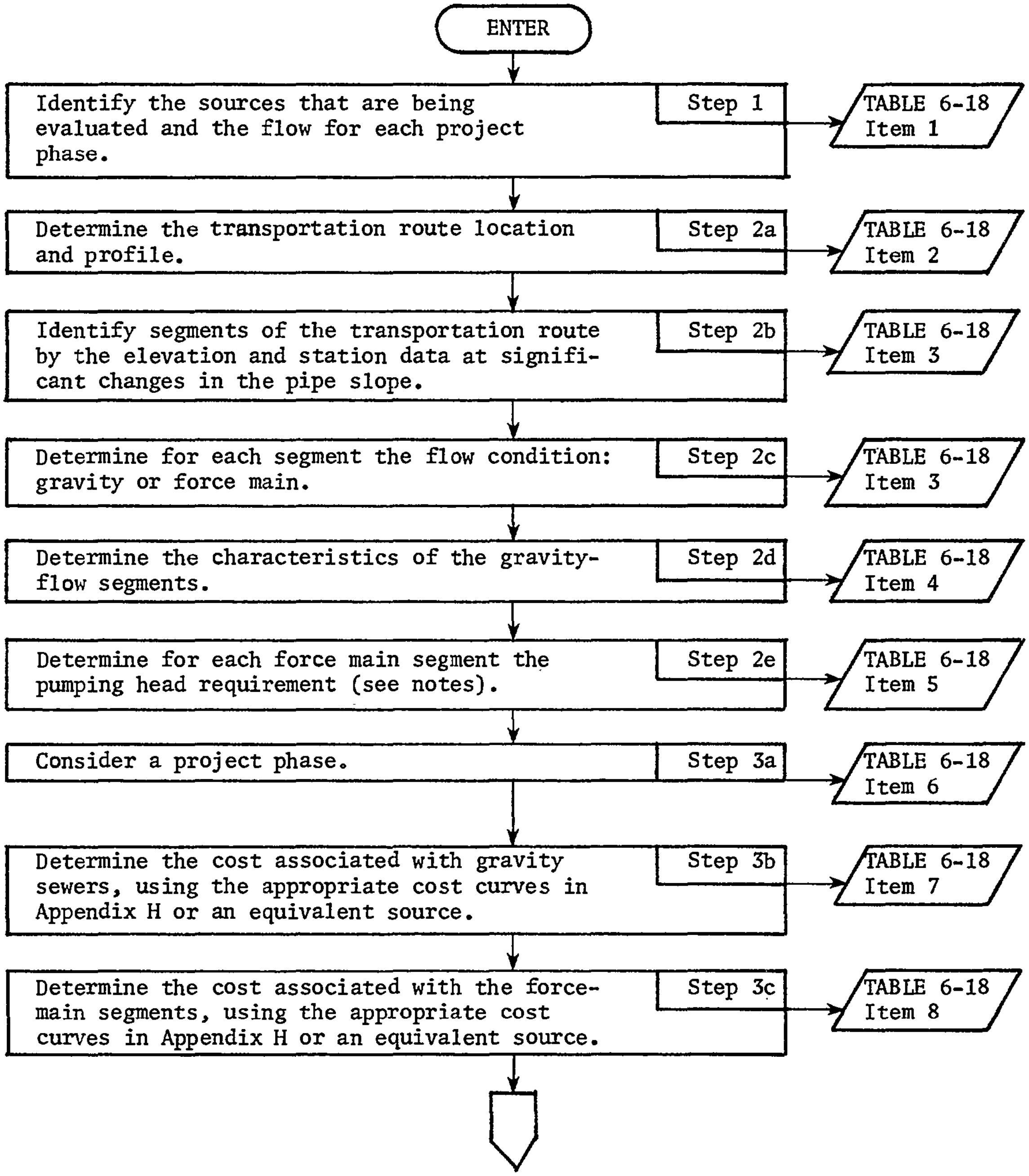


FIGURE 6-32

TRANSPORTATION COST METHODOLOGY
FLOWCHART



Identify the sources that are being evaluated and the flow for each project phase.

Determine the transportation route location and profile.

Identify segments of the transportation route by the elevation and station data at significant changes in the pipe slope.

Determine for each segment the flow condition: gravity or force main.

Determine the characteristics of the gravity-flow segments.

Determine for each force main segment the pumping head requirement (see notes).

Consider a project phase.

Determine the cost associated with gravity sewers, using the appropriate cost curves in Appendix H or an equivalent source.

Determine the cost associated with the force-main segments, using the appropriate cost curves in Appendix H or an equivalent source.

FIGURE 6-32 (continued)

TRANSPORTATION COST METHODOLOGY FLOWCHART

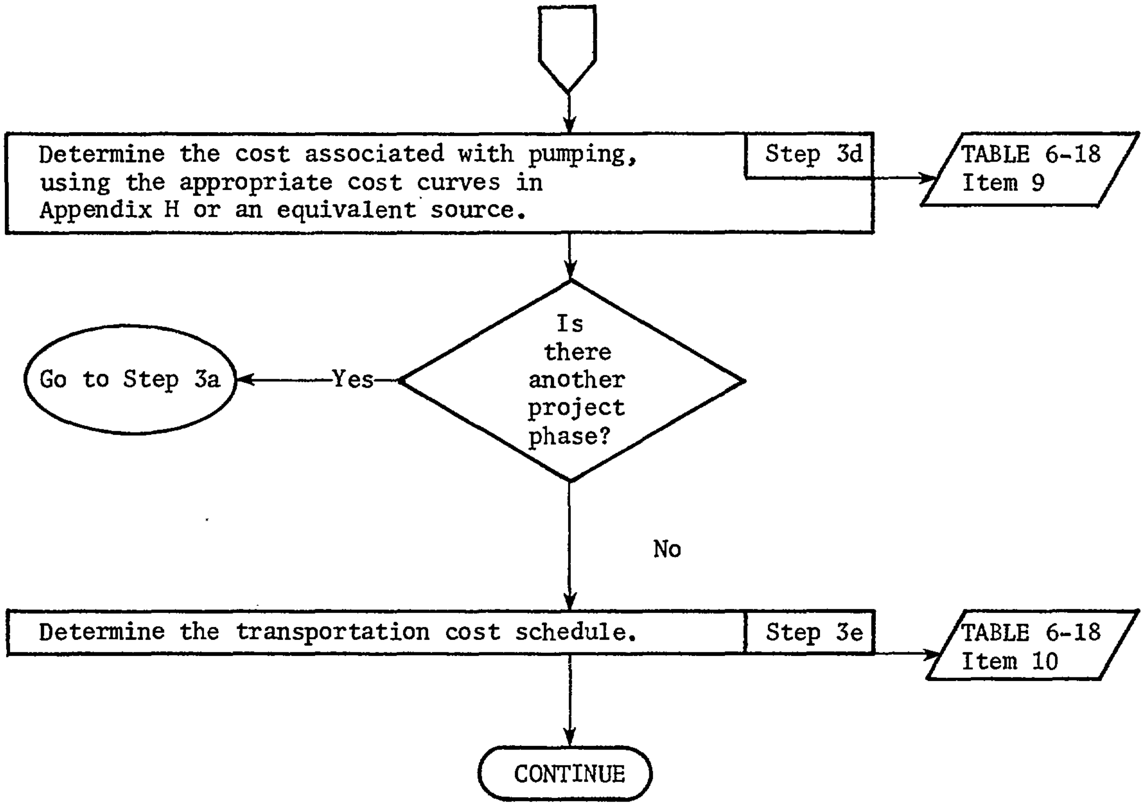


TABLE 6-18
TRANSPORTATION COST METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - Project/Phase/Source Identification.

i. Project: __________

ii. Source Identification

Source	Elevation	Design Flow (mgd) @ Phase No.				
		1	2	3	n	Design Yr

Item 2 - Transportation Route Profile.
(also locate routė on topographic map)

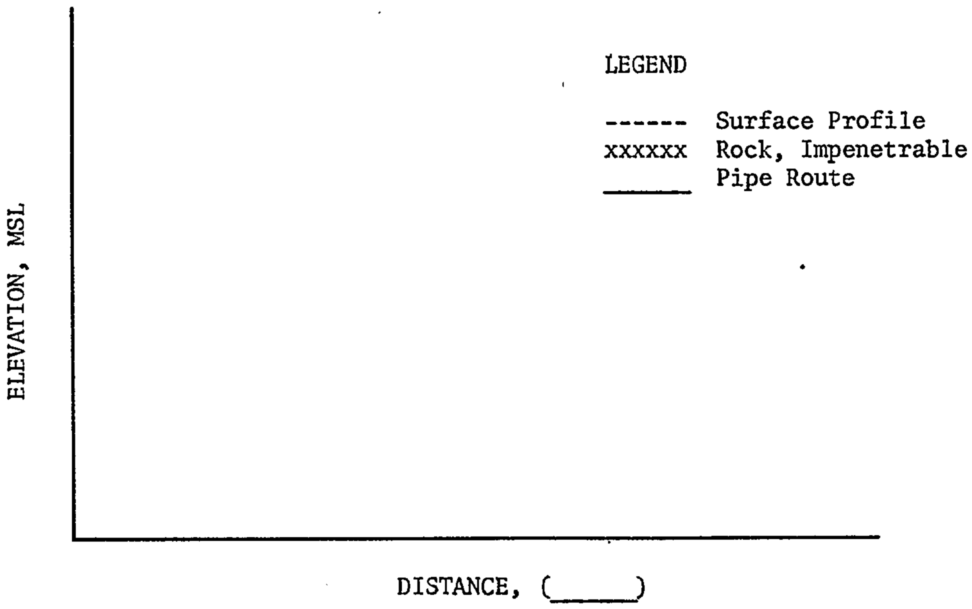


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TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 3 - Critical Segments of the Transportation Route.

Flow Rate: ____________ Assumed n-value: ____________

Segment	Elevation/Station[1] E/S(A)	to E/S(B)	Slope[2]	Velocity[3]	Flow Type[4]
a)					
b)					
c)					
d)					

Notes:

1. Define Elevation Station Data from upstream E/S(A) to downstream E/S(B).
2. Slope = $\frac{E(B) - E(A)}{S(B) - S(A)}$ Units: ft/ft
3. Determine velocity only for positive slope condition; negative slope indicates force main (see discussion); use the attached nomograph (Hydraulic Computations).
4. Flow type: Gravity if positive slope and acceptable velocity (2 fps minimum); force main for other conditions.

Reference calculation sheets: ____________________________

Item 4 - Gravity Segments.

Flow Rate: ____________ mgd

Segment	Flow Rate (mgd)	Length (ft)
a)		
b)		
c)		
d)		
TOTAL		

By ____________ Date ____________ Strategy No. ____________
Checked by ____________ Date ____________ Source No. ____________
Remarks: ______________________________ Page ________

TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 5 - Force Main Segments.

	Segment	Length	Static Head[1]	Dynamic Head[2]	Pumping Head[3]
a)					
b)					
c)					
d)					

Notes: [1] Static head = elevation difference from upstream to downstream.

[2] Dynamic head = See Discussion, Step 2 (Item 5).

[3] Pumping head = Static head + Dynamic head.

Reference calculation sheets: ____________________

By ____________ Date ____________ Strategy No. ____________
Checked by ____________ Date ____________ Source No. ____________
Remarks: ________________________________ Page ____________

TABLE 6-18
TRANSPORTATION COST METHODOLOGY WORKSHEET

COST DETERMINATION

Item 6 - Project/Phase Identification.

Project: ______________________

Phase: ______________________

Item 7 - Gravity Sewer Costs.

i. Reference Cost Curve: Figure H-84 (or equivalent curve)
Service Life: __________ years
Gravity Sewer Length: __________ feet
Design Flow Rate: __________

ii. Cost Determination.

Construction Cost: $__________
(Compute only for Phases that include sewer construction; adjust curve cost to reflect installed cost.)

O&M Cost - Start: $ ______________
O&M Cost - End: $ ______________
Replacement Cost: None
Salvage Value (SV)[1]$ ______________

$$^1\mathrm{SV} = \frac{(\text{Service Life - Years to Project End})}{(\text{Service Life})} \times \text{Capital}$$

By ____________ Date ____________ Strategy No. ____________
Checked by ____________ Date ____________ Source No. ____________
Remarks: ______________________________ Page ________

TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 8 - Force Main Cost.

i. Phase: ____________

Cost Curve: Figure H-85 (or equivalent curve)

Service Life: ______________ years

Length: ______________ ft

a) ____________ ft @ ____________ mgd

b) ____________ ft @ ____________ mgd

c) ____________ ft @ ____________ mgd

ii. Cost Determination.

	Segment		
	a	b	c
Capital Cost[1]			
O&M Cost - Start			
O&M Cost - End			
Replacement Cost	None	None	None
Salvage Value (SV)[2]			

[1] Compute only for Phases that include force main construction; adjust curve cost to reflect installed cost.

[2] $SV = \frac{\text{(Service Life - Years to Project End)}}{\text{(Service Life)}} \times \text{Capital}$

By ____________ Date ____________ Strategy No. ____________
Checked by ____________ Date ____________ Source No. ____________
Remarks: ________________________________ Page ________

TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 9 - Pump Station/Pumping Cost.

i. Phase: ______________

Service Life: ______________ years

Pumping Head/Flow:

Segment	Total Head	Flow Rate	
		Start	End

ii. Cost Determination. (Cost curve Figure H-30, or equivalent)

	(a)	(b)	(c)
Segment:	________	________	________
Capital Cost[1]	________	________	________
O&M Adjustment for head[2]	( )	( )	( )
O&M - Start[3]	________	________	________
O&M - End[3]	________	________	________
Replacement Cost/Year[4]	________	________	________
Salvage Value (SV)[5]	________	________	________

Notes:

[1] Compute only for Phases that include pumping capacity expansion; adjust curve cost to reflect installed cost.

[2] Compute as described by the cost curve.

[3] O&M Cost = Curve Cost + Adjustment

[4] Replacement Cost = $\frac{\text{Years Remaining in Project-Service Life}}{\text{Service Life}}$ x Capital

[5] SV = $\frac{\text{(Service Life - Years Remaining in Project)}}{\text{(Service Life)}}$ x Capital

If Salvage Value is negative, enter as 0 and compute replacement cost.

By ______________ Date ______________ Strategy No. ______________
Checked by ______________ Date ______________ Source No. ______________
Remarks: ______________________________ Page ____________

TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 10 - Transportation Cost Summary.

i. Cost Schedule.

Phase	Item	Capital Cost	Start - O&M	End - O&M	Salvage Value
1	#7				
	#8				
	#9				
	TOTAL PHASE 1				
2	#7				
	#8				
	#9				
	TOTAL PHASE 2				
3	#7				
	#8				
	#9				
	TOTAL PHASE 3				
n	#7				
	#8				
	#9				
	TOTAL PHASE n				

By ________ Date ________ Strategy No. ________
Checked by ________ Date ________ Source No. ________
Remarks: ________ Page ________

TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 10 - Transportation Cost Summary (continued).

Replacement Schedule		
Item	Year	Cost
____	____	____
____	____	____

ii. Present-Worth Cost: $______________________

(Compute only when required.)

By ______________ Date ______________ Strategy No. ______________

Checked by______________ Date ______________ Source No. ______________

Remarks: ____ Page__________

Notes on Methodology Logic

Step 1 (Item 1) - This step identifies the conditions for the transportation cost evaluation. The flow rate for each source at each phase should be available from previous evaluations.

Step 2a (Item 2) - This step identifies the pipeline route for the cost evaluation. The user should identify the route on available topographic maps (e.g., USGS Quadrangle Sheets), with general consideration given to the natural path between the two locations and environmental constraints. Typically, this would indicate that the transportation route follows a stream or other relatively low area. The surface profile between the two sources is then determined and plotted on the transportation route identification graph. It is also desirable that the underlying condition along the path be identified if the information is readily available. The purpose of this determination is to locate any underground conditions that would constrain the type of pipeline that could be installed. For example, a shallow rock deposit would limit the depth to which a gravity sewer could be installed, so this should be identified on the transportation route graph. This information is entered onto the graph in terms of the station and elevation of each critical point (i.e., minimum and maximum elevations for any condition). The pipeline profile can then be developed on this graph, with Source 1 defined as the receiving point and Sources 2, 3, etc., the contributing points.

Step 2b (Item 3) - In this step the user characterizes the pipeline route in terms of critical segments that approximate the anticipated condition. These critical segments should be as short as required to adequately describe the pipeline condition, but the user should try to identify a minimum of these to provide a degree of detail consistent with the other methodologies.

Step 2c (Item 3) - Since a gravity sewer is less expensive than a force main, this step is concerned primarily with identifying the segments that can utilize a gravity sewer. Each segment is defined by its endpoint elevation and station, working from the upstream to the downstream end of the pipeline. The slope for each identified segment should be computed. A positive slope indicates that the upstream end is higher than the downstream end of the segment. The flow velocity can be computed using the nomograph (Figure 6-33), or by any other accepted method. The flow type for each segment is gravity flow if the slope is positive and the resultant flow velocity is sufficient (minimum 2 feet per second). Force main segments are those that cannot be identified as gravity.

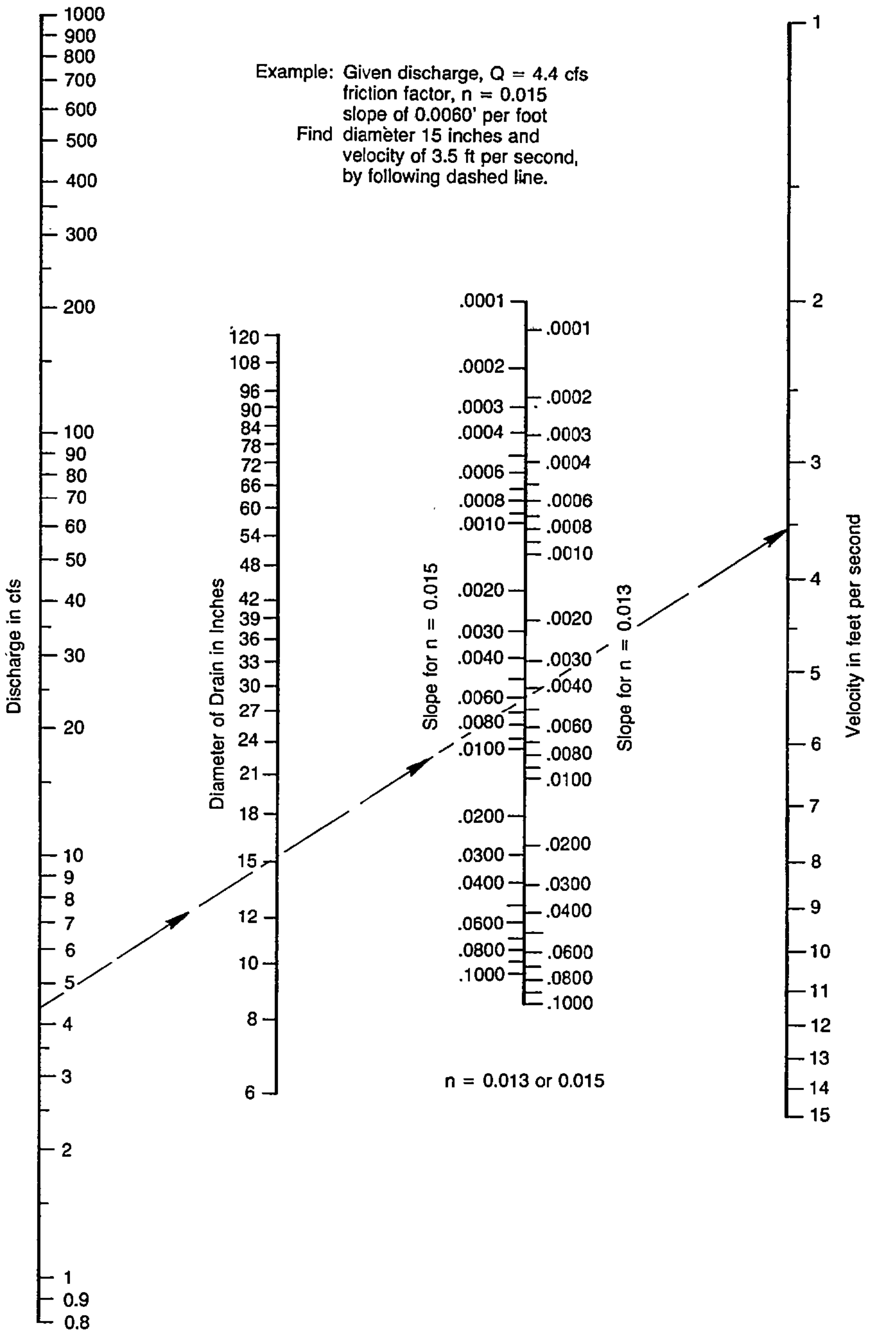
Step 2d (Item 4) - This step identifies the peak flow rate (typically 2.5 to 3.0 times the average flow rate at the design-year condition) and the length of the critical gravity flow segments.

Step 2e (Item 5) - In this step, each force main segment is characterized as to the energy (pumping head) required to maintain flow through that segment. The static head refers to the energy required to lift the water from the downstream point to the upstream point, and can be considered the elevation difference between these points. The dynamic head represents the energy losses (dynamic losses) in the pipeline associated with flow through the pipe. These dynamic losses are a function of the velocity in the pipe and the pipe characteristics. As a first approximation, the user can define the dynamic losses to be equal to 20 percent of the static head since, under the average conditions, the design engineer would limit total dynamic losses to a number in this range. For the situation where a more accurate estimate is required, or when there is a long force main/low lifting head situation, the user should determine this friction loss using standard engineering methods.

Step 3a (Item 6) - This step identifies the project and phase to which the calculations in Items 7 to 10 apply.

Step 3b (Item 7) - In this step, the gravity sewer capital cost is determined for phases that include pipeline construction. The operation and maintenance cost associated with the gravity sewer should be determined for every project phase.

Step 3c (Item 8) - In this step, the force main capital cost is determined for phases that include pipeline construction. The operation and maintenance cost associated with the force main should be determined for every project phase.



Example: Given discharge, Q = 4.4 cfs
friction factor, n = 0.015
slope of 0.0060' per foot
Find diameter 15 inches and
velocity of 3.5 ft per second,
by following dashed line.

Adapted from Engineering Manual, Department of Defense, Corps of Engineers, Part XIII, Chapter 1, June 1955.

FIGURE 6-33 SIZE OF CIRCULAR DRAIN FLOWING FULL

Step 3d (Item 9) - In this step, the various cost items associated with wastewater pumping are determined. The project schedule should identify the desired pumping capacity during the various phases of the planning period. After this step, the user is directed to evaluate the next project phase for the cost items.

Step 3e (Item 10) - This step summarizes the costs determined in Items 6, 7, 8, and 9 in a Transportation Cost Schedule. This schedule can be transferred to the appropriate methodology for inclusion in the project cost schedule or can be used to develop the Present-Worth cost.

6.5 Illustrative Example

This illustrative example is presented to demonstrate how the approach presented in Chapter 6 is employed to develop and evaluate control alternatives for a 208 planning area. The setting for the example is the hypothetical Jefferson City area on the South River, which has been used throughout this manual.

The methodology is employed just as it would be in an actual problem setting. Water quality objectives are defined, load-reduction strategies are developed, and the FRAMEWORK METHODOLOGY guides the user in utilizing the component methodologies to evaluate control alternatives for each wastewater source of interest.

Notes are recorded directly on the worksheets and on separate sheets entitled "Illustrative Example Supplemental Notes" to give the user additional information about the source of a numerical value, the reason for a particular operation, or other explanations which clarify the procedure. In order to reduce the volume of the example, the various methodology worksheets are filled out completely only the first time they are utilized. If a set of worksheets is needed more than once, a note is included to indicate how the user would proceed. An assumption is made about the result if it is needed for further determinations.

6.5.1 Water Quality Objectives

Dissolved Oxygen criteria and suggested objectives for other water quality parameters for the South River are presented in Table 6-1. As explained earlier in this chapter, the water quality standards that are in force for a particular situation do not always address all important water quality parameters. A standard often must be considered a minimum objective for an area. Separate objectives may be set to address parameters other than the most commonly addressed one, i.e., Dissolved Oxygen. Therefore, a range of objectives is available for an area, for example, from simply meeting the water quality standard to controlling pollution from storm water runoff also. Control alternatives to meet these objectives will probably require a wide range of costs.

For the South River area, various objectives for control of individual parameters and combinations of parameters are proposed for further evaluation. These objectives are recorded in Table 6-19.

6.5.2 Load-Reduction Strategies

As explained earlier in Chapter 6, a load-reduction strategy is a combination of percentage load reductions at various sources which will meet a given water quality objective. There may be a number of load-reduction strategies which will meet any one objective. A combination of a water quality objective and a strategy is referred to simply as a load-reduction strategy.

For any area, there may be a number of water quality objectives, several strategies to achieve each objective, and a wide range of costs to implement the strategies. This suggests that a decision-maker will have to make decisions on two levels. First, he must decide on the water quality objective which will be sought; then he must decide on the most cost-effective control alternative to achieve that objective. The first decision generally will identify the order of magnitude of dollars to be spent to achieve water quality (however it might be defined). Control of more parameters with fewer allowable violations usually requires more money, and selection of a particular control alternative identifies the approach which supposedly will have the least total cost to society. Since the decision-maker cannot perform his function without input, the planner or engineer must develop appropriate cost information for all feasible and potentially desirable strategies.

The Load-Reduction Strategy Matrix, Table 6-19, represents a number of different water quality objectives for the South River area, and several strategies for achieving these objectives. The strategies were formulated by using the water quality impact analysis techniques in Chapter 5. Obviously, in an area which has a number of sources, there is a very large number of possible combinations of load reductions which will achieve the desired objective. The strategies formulated for a planning area should include those combinations which present a choice, by including significantly different technological solutions to achieve the desired water quality objectives.

The objectives and strategies presented in Table 6-19 range from control of one water quality parameter and two sources, to control of five parameters

TABLE 6-19

LOAD REDUCTION STRATEGY MATRIX

Allocation Designation	Objective	STP I (% Removal)	STP II (% Removal)	Separate Storm Sewer Runoff (% Removal)	Combined Sewer Overflow (% Removal).
1a	DO(1)-1	55% CBOD 55% NBOD	55% CBOD 55% NBOD	-	-
1b	DO-1	24% CBOD 24% NBOD	75% CBOD 75% NBOD	-	-
1c	DO-1	83% CBOD 42% NBOD	83% CBOD 42% NBOD	-	-
1d	DO-1	16% CBOD 75% NBOD	16% CBOD 75% NBOD	-	-
2a	DO-1,DO-2	55% CBOD 55% NBOD	55% CBOD 55% NBOD	-	39% UOD
2b	DO-1,DO-2	55% CBOD 55% NBOD	55% CBOD 55% NBOD	25% UOD	25% UOD
2c	DO-1,DO-2	75% CBOD 75% CBOD	75% CBOD 75% NBOD	-	24% UOD
3a	TC(2)-1	-	-	95% TC	99.83% TC
3b	TC-1	-	-	99.62% TC	99.62% TC
4	TSS(3)-1	-	-	65% TSS	-
5a	N(4)-1	58% N	58% N	-	-
5b	P(5)-1	80% P	80% P	-	-
6 = 2a	DO-1,DO-2 TC-1	55% CBOD 55% NBOD	55% CBOD 55% NBOD	- 95% TC	39% UOD 99.83% TC
7 = 2a, 3a 3a, 4	DO-1,DO-2 TC-1,TSS-1	55% CBOD 55% NBOD	55% NBOD 55% NBOD	65% TSS 95% TC	39% UOD 99.83% TC
8 = 2a, 3a, 5a, 5b	DO-1,DO-2, TC-1,N-1,P-1	55% CBOD 55% NBOD 58% N 80% P	55% CBOD 55% NBOD 58% N 80% P	95% TC	39% UOD 99.83% TC
9 = 2a, 3a, 4, 5a, 5b	DO-1,DO-2, TC-1,TSS-1, N-1,P-1	55% CBOD 55% NBOD 58% N 80% P	55% CBOD 55% NBOD 58% N 80% P	65% TSS 95% TC	39% UOD 99.83% TC

NOTES:

(1)DO = dissolved oxygen

(2)TC = total coliform organisms

(3)TSS = total suspended solids

(4)N = total nitrogen

(5)P = total phosphorus

and four sources. In an actual situation, a user would probably elect to start his analysis with the simplest case and proceed to the most complex. Determinations made for the sources under the simplest case very often will be relevant in evaluating more complex strategies.

6.5.3 Development and Evaluation of Control Alternatives

When the Load-Reduction Strategy Matrix has been developed, the user is ready to begin the development and evaluation of control alternatives for his 208 area. He does this by utilizing the FRAMEWORK METHODOLOGY, and the component methodologies determined to be appropriate for the area involved. Worksheets from both the framework and the component methodologies are filled out and filed as illustrated in the pages that follow.

To further aid the user, a list is presented (in Table 6-20) of the methodologies employed in this illustrative example; each component methodology is used at least once. The illustrative example worksheets are numbered in the lower right corner just as they would be in actual use. The report page number appears at the bottom center of the page and is prefixed by a "6-". Page references within the illustrative example mean the illustrative-example page numbers in the lower right corner.

TABLE 6-20
INDEX TO COMPONENT METHODOLOGIES USED IN ILLUSTRATIVE EXAMPLE

Strategy	Source	Methodology[1]	Illustrative Example Page Number
N/A	N/A	Framework	1
9	1	Treatment Facility	9
		Residuals Disposal	19
		Present-Worth	27
		Land Application	31
		Transportation	38
		Present-Worth	50
		Wastewater Reuse	55
		Treatment Facility	58
		Impact Area Modification	67
		Regionalization	77
9	3	Land Management	87
		Collection System Control	91
		Storage/Treatment	100

---

[1] Component methodologies utilized by other component methodologies are included.

TABLE 6-3
FRAMEWORK METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1

i. Identification of water quality objectives, load reduction strategies, and sources.

a. Define water quality objectives by number and parameters to be controlled.

Water Quality Objective #	Receiving Water Constituents to be Controlled: D.O. (Dry Weather)	D.O. (Wet Weather)	Total Suspended Solids (Wet Weather)	Total Nitrogen	Total Phosphorus	Total Coliforms (Wet Weather)
1	x					
2	x	x				
3						x
4			x			
5				x	x	
6	x	x				x
7	x	x	x			
8	x	x		x	x	x
9	x	x	x	x	x	x

b. Load reduction strategies represent differing percentage reductions in load at the various sources of interest for a particular water quality objective. These strategies are identified by a letter (a, b, c, etc.) where more than one strategy is proposed for a particular water quality objective.

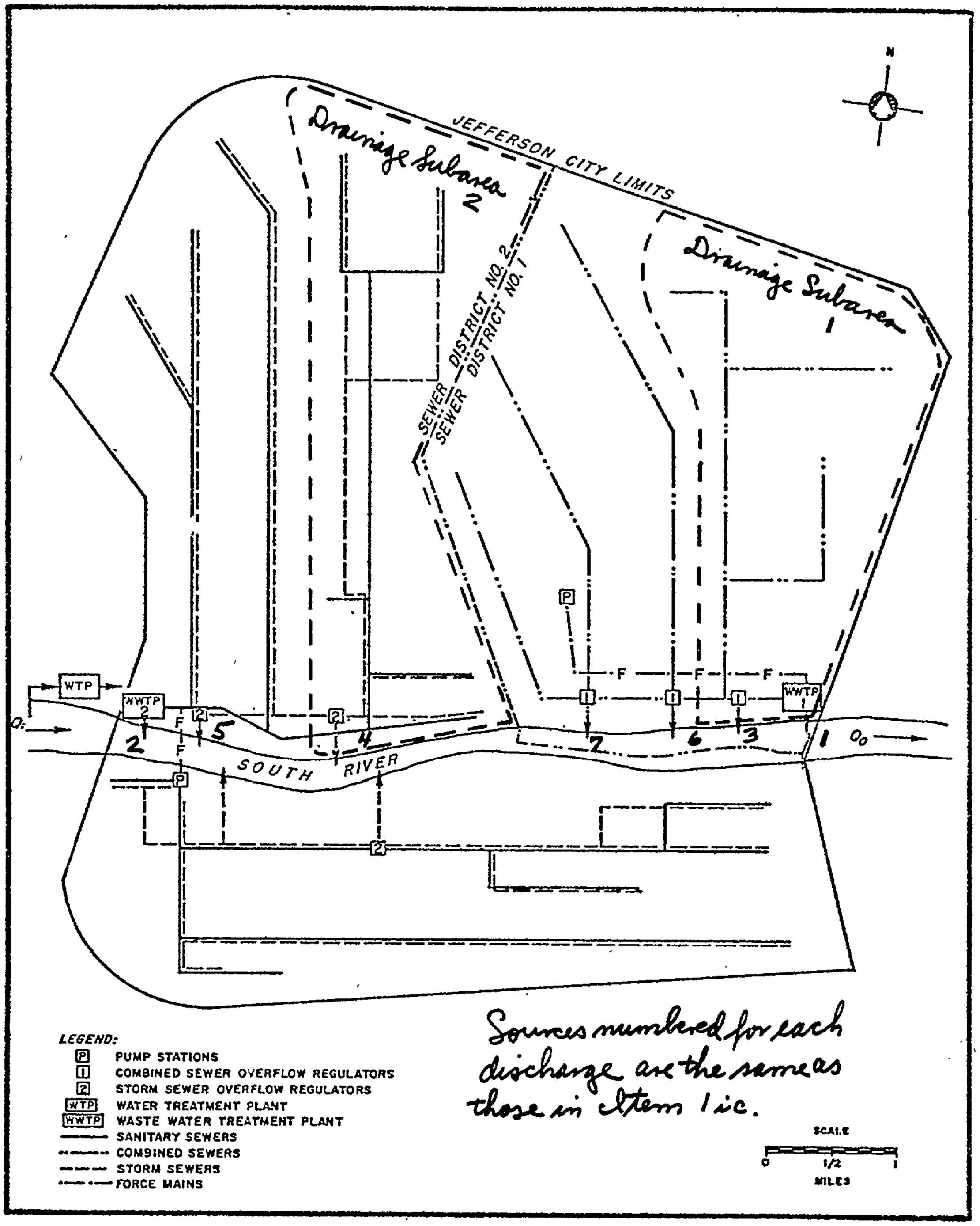
c. Identify sources by number: Attached Figure 5-7 (page 2) defines the location of each source.

Source #	Source Type (Wet or Dry)	Source Description
1	Dry	Sewer District (SD 1), Wastewater Treatment Plant 1
2	Dry	Sewer District (SD 2), Wastewater Treatment Plant 2
3	Wet	Combined Sewer Overflow, SD 1
4	Wet	Separate Storm Sewer Runoff, SD 2
5	Wet	Separate Storm Sewer Runoff, SD 2
6	Wet	Combined Sewer Overflow, SD 1
7	Wet	Combined Sewer Overflow, SD 1

By ______ Date ______ Strategy No. N/A

Checked by ______ Date ______ Source No. N/A

Remarks: ______ Page 1



Sources numbered for each discharge are the same as those in Item 1ic.

FIGURE 5-7
JEFFERSON CITY STUDY AREA
COLLECTION SYSTEM

By ____________ Date ____________ Strategy No. ____________
Checked by ____________ Date ____________ Source No. ____________
Remarks: ________________________________ Page 2

Illustrative Example Supplemental Notes

The water quality objectives defined in Item 1i are those indicated by Table 6-1. These numbers will be referenced throughout this example by "Number-Letter", where the letter refers to the individual strategy that meets the water quality objective. An objective/strategy combination will be referred to as a load-reduction strategy or simply as the strategy.

The impact analysis in Chapter 5 deals with the combined sewer overflows (identified as Sources 3, 6, 7) and separate storm sewer runoff (Sources 4, 5) as aggregated loads.

For this example, the assumption has been made, for illustrative purposes, that these aggregated loads can be controlled to the desired level by implementing a control program for Sources 1, 2, 3, and 4.

By ____________ Date ____________ Strategy No. N/A
Checked by ____________ Date ____________ Source No. N/A
Remarks: ____________ Page 3

Illustrative Example Supplemental Notes

The strategies have been formulated and numbered, and the sources numbered. At this point, the user begins the iterative evaluation of initial alternatives for the various strategies and sources defined above, using: Figure 6-6 as a guide, Item 1ii (page 5) for keeping track of strategies and sources considered, and Item 2 (page 7) to record Present-Worth costs and information reliability for the various control alternatives considered.

By ______ Date ______ Strategy No. N/A
Checked by ______ Date ______ Source No. N/A
Remarks: ______ Page 4

TABLE 6-3 (continued)
FRAMEWORK METHODOLOGY
WORKSHEET

Item 1 (continued)

ii. Record of load reduction strategies and sources considered.

- Check (x) the sources to be considered under each load-reduction strategy.
- Circle the checks in the matrix after all appropriate control alternatives have been considered for a source.
- Go to next load reduction strategy when all sources have been considered for that strategy.
- End when all strategies have been considered.

Load Reduction Strategy	Source Number					
	1	2	3	4	5	6
1a	x	x				
1b	x	x				
1c	x	x				
1d	x	x				
2a	x	x		x		
2b	x	x	x	x		
2c	x	x		x		
3a			x	x		
3b			x	x		
4			x			
5a	x	x				
5b	x	x				
6	x	x	x	x		
7	x	x	x	x		
8	x	x	x	x		
9	ⓧ	ⓧ	ⓧ	ⓧ		

By ____________ Date ____________ Strategy No. N/A
Checked by ____________ Date ____________ Source No. N/A
Remarks: ____________ Page 5

Illustrative Example Supplemental Notes

For the illustrative example, Strategy 9 will be evaluated. This strategy has been selected because it requires use of all component methodologies. Actually Strategy 9 is a combination of a number of less comprehensive strategies (See Table 6-1). Since the example considers only Strategy 9, only sources for that strategy are circled in Item 1ii as a record of evaluation. In an actual case, all x's would be circled since all sources for all strategies would be considered.

By ____________ Date ____________ Strategy No. N/A

Checked by ____________ Date ____________ Source No. N/A

Remarks: ____________ Page 6

TABLE 6-3 (continued)
FRAMEWORK METHODOLOGY
WORKSHEET

Item 2 - Feasible Control Alternatives.

i. Record the Present-Worth cost of control alternatives determined using the component methodologies.

ii. Record the worksheet page number (from lower right corner) where the present worth is recorded in the appropriate component methodology.

iii. Record the relative reliability of the performance and cost information for the control alternative as identified in Appendixes G and H or at the discretion of the user.

Load Reduction Strategy	Source	Control Alternative	Present-Worth $	Page	Information Reliability	
					Performance	Cost
1a		Would be evaluated by user in an actual case. Illustrative example considers only Strategy 9.				
8						
9	1	Treatment Facility	$61,700,000	26	B	B
9	1	Land Application	$42,120,000	53	B	B
9	1	Wastewater Reuse	Cost not generated; see note, page 54			
9	1	Impact Area Modif.	Cost not generated; see note, page 66			
9	2	Treatment Facility	Cost not generated for these control alternatives since the operations are the same as for Source 1.			
9	2	Land Application				
9	2	Wastewater Reuse				
9	2	Impact Area Modif.				
9	1,2	Regionalization				
9	3	Land Management	Cost estimates not computed for this example.			
9	3	Collection Sys. Control				
9	3	Storage/Treatment	$2,734,000	119	C	C
9	3	Wastewater Reuse	Costs not generated since operations similar to those already presented.			
9	3	Impact Area Modif.				
9	4	All Control Alternatives				
9	3,4	Regionalization				

By ______ Date ______ Strategy No. N/A
Checked by ______ Date ______ Source No. N/A
Remarks: ______ Page 7

Illustrative Example Supplemental Notes

The treatment facility evaluation included in the next few worksheets considers a single-phase project utilizing the existing treatment facility. Alternatives (not evaluated) would be abandonment of the existing facility or a multi-phased project. A complete residuals disposal evaluation is included in this treatment facility upgrading, involving consideration of a new landfill site. Transportation of the treatment plant residual (sludge) is by truck. The Present-Worth cost of the upgrading project is determined.

By ______________ Date ______________ Strategy No. N/A

Checked by ______________ Date ______________ Source No. N/A

Remarks: ______________ Page 8

TABLE 6-4

TREATMENT FACILITY METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - Program Implementation Schedule.

i. Planning Period: 20 years

ii. Construction phases: 1

Phase	Timing			Flow Projection (mgd)		Design Flow (mgd)
	Year	to	Year	Start	End	
1*	Present	to	20	9	17	9
2		to				
3		to				
4		to				
n		to				

*Existing facility not utilized at full capacity.

By ____________ Date ____________ Strategy No. 9
Checked by ____________ Date ____________ Source No. 1
Remarks: ____________ Page 9

TABLE 6-4 (continued)
TREATMENT FACILITY METHODOLOGY WORKSHEET

iii. Treatment Objectives.

Note: Dissolved oxygen deficits use ultimate oxygen demand inputs (Table 6-3). These must be reconverted back to CBOD and NBOD (NH_3) concentrations to determine discharge limitations (See Appendix H discussion of Treatment Systems Performance Matrix).

		Effluent Quality							
Phase	Reference Cost Curve**	BOD mg/l	COD mg/l	TSS mg/l	T-P mg/l	NH_3-N mg/l	NO_3-N mg/l	T-N mg/l	T-C #/100ml
Existing Facility	H-2	130	250	100	9	20	0	20	—
1	H-12/S-1				2[a]			8[a]	
2	(See note, page 11)								
3									
n									

**Treatment System curve number (Appendix H, Figures H-2 to H-15) or reference number for synthesized system cost curve developed from unit process curves (Appendix H).

iv. Existing Facility Characteristics.

Design Capacity: 9 mgd

Service Life: 34 years

Years in Service: 20 years

Remaining Service: 14 years

Note:

[a] These parameters are the most critical and, therefore, define the required treatment level

By ____________ Date ____________ Strategy No. 9

Checked by ____________ Date ____________ Source No. 1

Remarks: Strategy to expand/upgrade existing facility

Page 10

Illustrative Example Supplemental Notes

The required treatment level suggests that Treatment System H-12 could be used. However, for purposes of this example, it is assumed that local conditions require that an alternate system be used. Therefore a treatment system is synthesized using some of the unit process curves in Appendix H and some hypothetical "local cost estimates". This synthesized system, referred to as S-1, will provide the required treatment level. System units and construction and O & M costs are summarized, in the table below:

Synthesized System Cost Estimate

Treatment System Units	App H Curve No.	Construction Cost (10^6 $)	O&M Cost	Service Life
Wastewater Treatment:				
Lift Pumps	H-30	1.10	32,000	15
Preliminary Treatment	H-31	0.27	38,000	30
Primary Clarifier	Local Cost Est.	2.4	73,000	50
Act. Sludge	H-34	2.30	160,000	40
Second. Clar.	Local Cost Est.	2.3	120,000	30
Nitrification	Local Cost Est.	3.6	200,000	40
Denitrification	Local Cost Est.	2.5	600,000	30
Two Stage Lime	H-53	2.10	400,000	40
Filtration	H-56	2.00	170,000	30
Disinfection	H-58	0.28	87,000	15
Biological Sludge:				
Gravity Thick.	H-64	0.15	6,000	50
Anaerobic Digester	H-73	0.90	100,000	50
Vacuum Filt.	Local Cost Est.	1.15	200,000	20
Chemical Sludge:				
Gravity Thick.	H-64	0.15	6,000	50
Vacuum Filt.	Local Cost Est.	1.45	400,000	20
Misc. Structures	H-29	0.35	8,000	50
Support Personnel	H-28	-	100,000	-
TOTAL		23.00	2,700,000	

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 2 - Existing Facility Cost.
Note: For the first phase of new facility construction, Items 2i, 2ii, and 2iii will equal zero since there is no existing facility.

i. Capital Value (i.e., construction cost plus add-ons).

a. Design Q = 9 mgd

b. Level of Treatment: Reference Cost Curve H-2
Service Life 34 years

c.	Construction Cost (Curve $)	= $2,600,000
d.	plus Piping - Curve $ x 15%	= 390,000
	Electrical - Curve $ x 12%	= 312,000
	Instrumentation - Curve $ x 8%	= 208,000
	Site Preparation - Curve $ x 5%	= 130,000
	Miscellaneous Structures (None)	= 0
e.	Sub-Total 1, Construction Cost (c+d)	$3,640,000
f.	plus Sub-Total 1 x Engineering and Construction 15%	= 546,000
	Sub-Total 1 x Contingencies 15%	= 546,000
g.	Sub-Total 2: Capital Cost (e+f)	$4,732,000

h. CAPITAL VALUE OF EXISTING FACILITY = Sub-Total 2 x $\frac{\text{ENR (Current)}}{2475^*}$ 2499 = $4,780,000
* ENR = 2475, September, 1976.

ii. Replacement Cost.
(compute only if planning period is greater than remaining service life)

$$\text{Replacement Cost} = \frac{\text{Planning Period - Remaining Service Life}}{\text{Planning Period}} \times \text{Capital Value}$$

$\frac{20-14}{20}$ x 4,780,000 = $1,430,000 at year 14

iii. Salvage Value. Service life @ 14 years is less than 20 years to planning period end.
(compute only if remaining service life is greater than planning period)

$$\text{Salvage Value} = \frac{\text{Remaining Service - Years to Planning End}}{\text{Remaining Service}} \times \text{Capital Value}$$

= None at end of planning period.

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 3 - Expansion Program or New Facility Construction.

Phase Number 1

i. Existing Capacity = 9 mgd (previous phase or existing facility; zero if new facility)

ii. Expanded or New Facility Capacity = 17 mgd (design capacity of next phase)

iii. Level of Treatment: Reference Cost Curve H-2
Service Life 34 years
(This is the treatment level of existing facility)

iv. Construction cost of expanded or new facility - enter cost curve at expanded or new facility at capacity (ii) 17 mgd — $4,000,000

v. Construction cost of existing facility - enter cost curve at existing facility at capacity (i) 9 mgd — 2,600,000

vi. Sub-Total 1: Expanded or New Facility Construction Cost (iv-v) = $1,400,000

vii. plus Sub-Total 1 x Piping 15% = 210,000
Sub-Total 1 x Electrical 12% = 168,000
Sub-Total 1 x Instrumentation 8% = 112,000
Sub-Total 1 x Site Preparation 5% = 70,000

viii. Sub-Total 2: Construction Cost (vi + vii) — $1,960,000

ix. plus Sub-Total 2 x Expansion/Upgrading Factor 5% (1) = 98,000
Sub-Total 2 x Engineering and Construction 15% = 294,000
Sub-Total 2 x Contingencies 15% = 294,000

x. Sub-Total 3: Capital Cost (viii + ix) — $2,646,000

xi. CAPITAL COST OF EXPANSION OR OF NEW FACILITY = Sub-Total 3 x $\frac{\overset{2499}{\text{ENR (Current)}}}{2475^*}$ = $2,670,000

* ENR (Engineering News Record) = 2475, September, 1976.

(1) Expansion/upgrading factor taken from Figure H-1 by entering the curve at 17 mgd.

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 4 - Upgrading Program.

Phase Number 1

i. Existing Level of Treatment: Reference Cost Curve H-2
(previous phase or existing facility)

ii. Required Level of Treatment: Reference Cost Curve S-1
(for the identified phase) Service Life 35 years (1)

iii. Q = 17 mgd (design capacity)

iv. Construction Cost at required level of treatment - curve from ii $23,000,000

v. Construction Cost at existing level of treatment - curve from i 4,000,000

vi. Sub-Total 1: Construction Cost of Upgrading (iv-v) = $19,000,000

vii. plus Sub-Total 1 x Piping 15% = 2,850,000
Sub-Total 1 x Electrical 12% = 2,280,000
Sub-Total 1 x Instrumentation 8% = 1,520,000
Sub-Total 1 x Site Preparation 5% = 950,000

viii. Sub-Total 2: Construction Cost (vi + vii) $26,600,000

ix. plus Sub-Total 2 x Expansion/Upgrading Factor 5% (2) = 1,330,000
Sub-Total 2 x Engineering and Construction 15% = 3,990,000
Sub-Total 2 x Contingencies 15% = 3,990,000

x. Sub-Total 3: Capital Cost (viii + ix) $35,910,000

xi. CAPITAL COST OF UPGRADING = Sub-Total 3 x $\frac{\overset{2449}{\text{ENR (Current)}}}{2475^*}$ = $36,300,000

* ENR = 2475, September, 1976.

(1) Composite service life. Average of service lives of individual units weighed by construction costs.

(2) Expansion/upgrading factor is from Figure H-1 by entering at the flow rate of 17 mgd.

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 5 - O&M Constant and Variable Cost.

Note: this level is used to identify O+M cost since it represents the actual treatment level.

Phase 1

Level of Treatment: Reference Cost Curve S-1

Timing		Design Flow		O&M Cost	
Start (yr.)	End (yr.)	Start (mgd)	End (mgd)	Start	End
1	20	9	17	$1,600,000*	$2,700,000

*Developed similarly to S-1 cost estimate at 17 mgd on page 11.

Item 6 - Phase 1 Replacement Costs (Upgraded and/or Expanded Portion)
(Compute if planning period is greater than phase service life)
Replacement Cost Schedule.

There are no replacement costs for this phase.

Expansion		Upgrading		Total	
Year	Cost	Year	Cost	Year	Cost
None		None		None	

Replacement Cost for Phase 1 =

$$\frac{\text{Years from Time of Replacement to end of Planning Period}}{\text{Service Life}} \times \text{Capital}$$

Note: the replacement cost for the existing facility has been estimated in Item 2.

Item 7 - Phase 1 Salvage Value at End of Planning Period.
(Compute if phase service life is greater than years to planning period end)

$$\text{Salvage Value} = \frac{(\text{Service Life - Years to Planning End})}{\text{Service Life}} \times \text{Capital}$$

Expansion S.V. = $\frac{34-20}{34} \times 2,670,000$ $1,100,000

Upgrading S.V. = $\frac{35-20}{35} \times 36,300,000$ $15,600,000

Total Phase S.V. $16,700,000

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Illustrative Example Supplemental Notes

For a more complex problem, there would probably be more than one phase for evaluation. If so, the sheets with Items 3, 4, 5, 6, and 7 would be repeated at this point in the evaluation, once for each project phase.

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 8 - Residual Disposal Cost, using RESIDUALS DISPOSAL METHODOLOGY.

i. Residual Disposal Technique.

Solids Nature dewatered solids (20%, 30%)

Residual Type biological, chemical

Disposal Method landfill

Transportation Truck

ii. Residual Disposal Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Land Cost[1]	1	20	184,000					184,000
1 Bio[2]	1	20	248,000	130,000	140,000	15	43,000	0
1 chem[2]	1	20	652,000	240,000	290,000	15	117,000	0
3								
n								
Total			1,084,000	370,000	430,000	15	160,000	184,000

Notes:

1. The cost for land is taken from Residuals Disposal, Item 3 iii (page 21).
2. The cost for Phase 1 (Bio + chem) is the sum of the landfill site and truck costs taken from Item 3 iii (page 21) and Item 5 iii (page 23).

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Illustrative Example Supplemental Notes

Pages 19-26 are worksheets for the RESIDUALS DISPOSAL METHODOLOGY, which is another component methodology utilized by the TREATMENT FACILITY METHODOLOGY to make a specific determination, the cost of residuals disposal. The use of component methodologies in this fashion will occur throughout the example. See Table 6-20 for an overview of which component methodologies are utilized in evaluating control alternatives for the various sources.

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TABLE 6-17 (continued)
RESIDUALS DISPOSAL METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - Residual Characteristics.

i. Residual generator method of treatment:

Primary clarification, activated sludge, nitrification, denitrification, two-stage lime

ii. Residual description: Biological and chemical (lime)

iii. Feasible disposal techniques: Landfill

IV. Residual quantity: (ultimate)

Design Year Flow Rate 17 mgd

Residual Generation 900(bio)+6,300(chem) lb/mgd (dry basis) (Note 3)

Disposal Quantity 122,400 lb/day (dry basis)
(7200 X 17 mgd)

Item 2 - Residual Disposal Sites.

i. Site characteristics summary.

	Site	Location	Distance from Generator	Estimated Useful Capacity	Comments
a) Existing Land Spreading Sites.		Not Applicable			
b) Existing Landfill Sites.	1	SD1 Facility	0 miles	0.12yr[a]	Site inadequate for future use.
c) Potential Sites.					
i) Landfill.	2	off Hwy 3 east of SD1	10 miles	Uncertain (200acres)	Site recently cleared.
ii) Land Spreading.		Not Applicable			

(a) Determined on page 20', Item 2 ii.

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Remarks: These calculation sheets (pages 18-25) develop cost schedule for page 17 of TREATMENT FACILITY METHODOLOGY. Page 19

TABLE 6-17 (continued)
RESIDUALS DISPOSAL METHODOLOGY
WORKSHEET

Item 2 - continued

iii. Site capacity evaluation - existing site #1

a) Useful life at existing disposal rate: 2 years

b) Existing disposal rate: *(615 lb/mgd)(9 mgd) = 5,500 [✓ lb/day] or ton/day or cu yd/day

c) Proposed disposal rate: 93,600' lbs/day (same units as above)

(use average rate for entire planning period)

d) Useful life at proposed disposal rate = (a) x $\frac{(b)}{(c)}$ = 0.1 yr ²

* Primary treatment (assumed for illustrative purposes)

Note:

' Average proposed disposal rate =

$$= \left(\frac{9\,mgd + 17\,mgd}{2}\right)\left(\frac{122{,}400\,lb/day}{17\,mgd}\right) =$$

$$= 93{,}600$$

² Useful Life = $2\,yr \times \frac{5{,}500\,lb/day}{93{,}600\,lb/day} = 0.12\,yr$

The user should refer to sludge handling curves (H-64—H-72) appropriate to treatment process. In this example, biological sludge is processed by an aerobic digester (H-73), and chemical sludge is processed by a vacuum filter (local cost estimate; residuals generation rate comparable to other chemical sludge processes.

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TABLE 6-17
RESIDUALS DISPOSAL METHODOLOGY
WORKSHEET

Item 3 - Cost for Development of Residual Disposal Site.

i. Program Schedule.

Phase	Timing			Design Flows (mgd)	
	Yr	to	Yr	Start	End
1	1	to	20	9	17
2		to			
3		to			
n		to			

ii. Site Development Schedule.

Phase	Design Flow (mgd)	Land Required[1] (acre)	Land Cost[2]
1	17	92 (19 bio/73 chem)	184,000
2			
3			
n			

[1] Land Required = 1.1 (bio) + 4.3 (chem) acre/mgd

[2] Land Cost = $ 2,000 /acre

Note: from cost curve information in Appendix H (Figures H-81 and H-82)

iii. Project Cost Schedule - Disposal Site.

Phase	Timing		Capital[1]	O&M		Replacement Cost[2]		Salvage Value[2]
	Yr	to Yr		Start	End	Year	Cost	
Land Cost	1	20	184,000	N/A	N/A	N/A	N/A	184,000
1 Bio	1	20	118,000	30,000	40,000	None		0
1 chem	1	20	302,000	70,000	120,000	None		0
3								
n								

[1] Capital computed as described for TREATMENT FACILITY METHODOLOGY; includes construction add on's to reflect installed capital. Capital = curve $ x (1.0 + Construction Add on's.)

[2] Cost computed as described for TREATMENT FACILITY METHODOLOGY; attach appropriate computations.

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Remarks: Reference page 22 for capital (Item iii) estimate. Page 21

Supplemental Calculations: Residuals Disposal

Item 3 iii

Biological Landfill: (Service Life = 20 years)

Flow = 17 mgd

Curve $ = 90,000

Add On's:

Engineering/Const. (15%) = 13,500

Contingency (15%) = 13,500

Sub 2 $117,000

$ENR = \frac{2499}{2475} \times \text{Sub 2}$

Construction = $118,000

No phase out cost
No salvage value
No replacement cost
} Both landfills

Chemical Landfill: (Service Life = 20 years)

Flow = 17 mgd

Curve $ = 230,000

Add On's:

Engineering/Const (15%) = 34,500

Contingency (15%) = 34,500

Sub 2 $299,000

$ENR = \frac{2499}{2475} \times \text{Sub 2}$

Capital = $ 302,000

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TABLE 6-17 (continued)
RESIDUALS DISPOSAL METHODOLOGY
WORKSHEET

Item 4 - Residual Disposal Cost for Existing Site.

Program Costs

Phase	Timing Yr to Yr	Design Flow (mgd) Start	 End	Residual Quantity[1] Start	 End	Disposal Cost[2] Start	 End
1	This is not applicable for this evaluation because an existing						
2	site is not being considered.						
3							
n							

[1]Determined using ________lb/mg

[2]Determined using ($_______/lb disposal cost) x (Residual Quantity lb/yr)

Item 5 - Residual Transportation Cost (using appropriate cost curves in Appendix H, Figures H-86 to H-90, or equivalent.

i. Transportation Method: truck

Residual Characteristics: Type Bio Solids 20%
Chem 30%

ii. Transportation Cost Schedule. Bio sludge is 900/7200 = 13% total; chem = 87%

Phase	Timing Yr to Yr		Capital[1]	O&M Start	 End	Design Flow Start	 End	Replacement Cost Year	 Cost	Salvage Value
Bio 1	1	20	130,000	100,000	100,000	*	*	15	43,000	0
Chem 1	1	20	350,000	170,000	170,000	*	*	15	117,000	0
3										
n										

* Average condition was used for simplicity. See page 24.

[1]Develop capital estimate based on end design flow for each phase; include construction add on's to reflect installed cost.

[2]Develop O&M estimate (start and end) for each phase.

[a] O+M costs are adjusted for haul distance as described in supplemental calculations.

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Supplemental Calculations : Residual Disposal

Item 5 i i

1) It is assumed that biological sludge at 20% solids has 340 lb solids/cu yd, and a chemical sludge has 500 lb solids/cu yd at 30%.

2) The biological sludge represents $\frac{900}{900+6300}$ = 13% of the residual, with the chemical sludge at 87%.

3) Capital cost will be developed for an average residual quantity, defined on calculation page 20 (note 1) to be 93,600 lb/day.

4) O&M cost (for simplicity) will also be calculated for this average quantity.

Biological Residual

Quantity/day = (93,600 lb/day)(0.13) $\left(\frac{1}{340 \text{ lb/cu yd}}\right)$ = 36 cu yd/day

Capital cost = $130,000 Year 1, service life 15 years.

~~Therefore~~, Replacement Cost = $\left(\frac{20-15}{15}\right)$(130,000) = $43,000 at year 15.

O&M (start) = $100,000 = end O&M

Chemical Residual

Quantity/day = (93,600)(0.87)$\left(\frac{1}{500}\right)$ = 163 cu yd/day

Capital cost = $350,000 Year 1, service life 15 years.

Replacement Cost = $\left(\frac{20-15}{15}\right)$(350,000) = $117,000 at year 15.

O&M (start) = $170,000 = O&M (end)

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TABLE 6-17 (continued)
RESIDUALS DISPOSAL METHODOLOGY
WORKSHEET

Item 6 - Residual Disposal Site Present-Worth Cost

i. Alternative Characteristics:

Residual	This section is not completed
Solids	for this evaluation because
Transportation Method	only one residual disposal
Disposal Method	scheme is under consideration.

ii. Present-Worth Cost (from PRESENT-WORTH METHODOLOGY) $ ______

Reference Sheets ______

Item 7 - Site Alternatives Summary - Residuals Disposal

Site	Location	Present-Worth Cost

This schedule is not completed for the illustrative example because only one residual site is under consideration.

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 9 - Project Cost Schedule (Summary of costs developed in TREATMENT FACILITY METHODOLOGY).

Phase	Year to Year	Item No.	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
1	1 to 20	3 (Expand/New)	2,670,000	-	-	-	-
		4 (Upgrade	36,300,000	-	-	-	-
		5/6/7	-	1,600,000	2,700,000	-	16,700,000
		8 (Residual)	1,084,000	370,000	430,000	-	184,000
	Total Phase 1		40,050,000	1,970,000	3,130,000	58,000	16,900,000
2		3 (Expand/New)					
		4 (Upgrade)					
		5/6/7					
		8 (Residuals)					
	Total Phase 2						
3		3 (Expand/New)					
		4 Upgrade)					
		5/6/7					
		8 (Residuals)					
	Total Phase 3						
n		3 (Expand/New)					
		4 (Upgrade)					
		5/6/7					
		8 (Residuals)					
	Total Phase n						

Phase 1 Variable O+M =

$$\frac{3,130,000 - 1,970,000}{20}$$

Replacement Schedule

Year	Cost
14	1,430,000
15	160,000

The year 14 cost is from page 12 of these calculations; Year 15 cost is from page 23.

Item 10 - Present-Worth Cost, using PRESENT-WORTH METHODOLOGY

Interest 7 % (from Water Resources Council 18 CFR 704.39, Discount Rate, published annually)

Present-Worth Cost $ 61,700,000

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Remarks: This sheet returns to TREATMENT FACILITY METHODOLOGY Page 26

TABLE 6-16
PRESENT-WORTH METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgements presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Items 1-10 Present-Worth Calculation.

Planning Period 20 years Interest 7 %

	Item (Reference Page)	Amount		Present-Worth
1.	Phase 1 Capital (pg. 26)	40,050,000	x 1.0 (Yr 1)	40,050,000
2.	Phase 2 Capital (pg. ___)	___	x ___ $(sppwf^a)$ (Yr ___)	___
3.	Phase 3 Capital (pg. ___)	___	x ___ $(sppwf^b)$ (Yr ___)	___
4.	Phase n Capital (pg. ___)	___	x ___ $(sppwf^c)$ (Yr ___)	___
5.	Replacement year (h) (pg. 26)	1,430,000	x 0.39 $(sppwf^h)$ (Yr 14)	560,000
6.	Replacement year (i) (pg. 26)	160,000	x 0.36 $(sppwf^i)$ (Yr 15)	58,000
7.	Replacement year (j) (pg. ___)	___	x ___ $(sppwf^j)$ (Yr ___)	___
8.	Replacement year (k) (pg. ___)	___	x ___ $(sppwf^k)$ (Yr ___)	___
9.	Replacement year (1) (pg. ___)	___	x ___ $(sppwf^1)$ (Yr ___)	___
10.	Salvage Value (Negative Cost) (pg. 26)	(-16,900,000)	x 0.26 $(sppwf^z)$ (Yr 20)	(-4,390,000

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Remarks: These sheets develop Present-Worth Costs for TREATMENT FACILITY METHODOLOGY Page 27

TABLE 6-16 (continued)
PRESENT-WORTH METHODOLOGY WORKSHEET

Item 11 - 19 Present-Worth Calculation.

Planning Period 20 years Interest 7 %

	Item (Reference Page)	Amount		Present-Worth
11.	O&M Phase 1 Constant (pg. 26)	1,970,000	x 10.6 (uspwf^{d}) x 1.0 (#Yrs 20; Yr 1)	20,880,000
12.	O&M Phase 1 Variable (pg. 26)	58,000	x 77.5 (gspwf^{d}) x 1.0 (#Yrs 20; Yr 1)	4,495,000
13.	O&M Phase 2 Constant (pg. ___)	______	x ____(uspwf^{e} x sppwf^{a}) (#Yrs____; Yr____)	
14.	O&M Phase 2 Variable (pg. ___)	______	x ____(gspwf^{e} x sppwf^{a}) (#Yrs____; Yr____)	
15.	O&M Phase 3 Constant (pg. ___)	______	x ____(uspwf^{f} x sppwf^{b}) (#Yrs____; Yr____)	
16.	O&M Phase 3 Variable (pg. ___)	______	x ____(gspwf^{f} x sppwf^{b}) (#Yrs____; Yr____)	
17.	O&M Phase n Constant (pg. ___)	______	x ____(uspwf^{g} x sppwf^{c}) (#Yrs____; Yr____)	
18.	O&M Phase n Variable (pg. ___)	______	x ____(gspwf^{g} x sppwf^{c}) (#Yrs____; Yr____)	
19.			TOTAL PRESENT-WORTH	$ 61,700,000

Reference

a) <u>Principles of Engineering Economy</u>, Grant and Ireson, 5th Edition, 1970. sppwf^{b}, sppwf^{c}, sppwf^{a}, uspwf^{d}

b) <u>Principles and Problems Engineering Economics</u>, U.S. Army Engineering School, Fort Belvoir, VA, 15 march 1969. gspwf^{d}

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Illustrative Example Supplemental Notes

The next evaluation for Source 1, as indicated by the flowchart Figure 6-6, is land application of the wastewater. A complete evaluation of an undrained application site is included in the attached worksheets. Also included is a complete cost determination for the wastewater transportation pipeline from the existing treatment facility site to the application site. The Present-Worth calculations are also included.

An underdrained system could also be evaluated using these worksheets. In this case there would still be a discharge to the receiving water.

The pipeline route evaluated for this example includes force main and gravity segments and the associated pumping costs. Only one route is evaluated, but if another route were to be considered, the user could determine the Present-Worth cost for each.

The Present-Worth analysis included in this example includes all expected construction expenditures and operation and maintenance items. Salvage values at the end of the project life and replacement costs for equipment are included. Finally, a negative cost is identified to reflect assumed revenues from the land application site for marketable crops.

The land application site evaluated for this example was an undrained site; thus, there would be no direct discharge of wastewater from Source 1 to the receiving stream. Therefore, the original load-reduction strategy, which was based on a discharge from Source 1 and Source 2, could likely be modified to allow less treatment at Source 2.

In an actual case similar to this situation, the user would utilize the water quality impact analysis techniques presented in Chapter 5 to reevaluate the waste load allocations for use in evaluating control alternatives for the

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Illustrative Example Supplemental Notes (continued)

other sources if undrained land application were to be the selected alternative for the source under consideration. Of course, for control alternatives other than undrained land application for Source 1, the original load allocations for all sources would again be used.

For this example, the original load allocations will be used throughout since the techniques are the same and, thus, serve for illustration.

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approach used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

PROJECT SCHEDULE

Item 1 - Program Implementation Schedule.

i. Planning Period: 20 years

ii. Existing Facility Conditions. Treatment Facility

Design Capacity: 9 mgd
Years of Service: 20 years
Remaining Service: 14 years

iii. Treatment Levels.

Note: Dissolved oxygen deficits use ultimate oxygen demand inputs (Table 6-3). These must be reconverted back to CBOD and NBOD (NH_3) concentrations in order to determine discharge limitations (See Appendix H discussion of Treatment Systems Performance Ratios).

Level	Reference Cost Curve	Parameter Control Levels BOD mg/l	COD mg/l	TSS mg/l	T-P mg/l	NH_3-N mg/l	NO_3-N mg/l	T-N mg/l	T-C #/100ml
Existing Facility	H-2	130	250	100	9	20	0	20	-
Pretreatment	H-6(1)	20	-	20	7	17			
Discharge Limitations	H-12/S-1		(See Note page 11)		2			8	

iv. Construction Phases: 1

(1) This is considered to be the minimum acceptable treatment level (only for this example).

Phase	Timing Year to Year	Design Flow (mgd) Flow Projection Start	Flow Projection End	Pretreatment Start	Pretreatment End	Transportation	Site Start	Site End
1	1 - 20	9	17	9	17	17	9	17
2								
3								
n								

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

GENERAL SITE EVALUATION

Item 2 - Land Application Ultimate Area Requirement.

i. Maximum Annual Flow Rate at end of Planning Period = 17 mgd

ii. Application Rate[1] = 2.5 in./week (Based on general application rates in the area)

iii. Non-operating time = 13 weeks/year

iv. Area Required = 2,800 acres, without buffer zone (includes area for roads, buildings, etc.)

Gross Area Required (with 200 ft buffer zone)[2]= 3,100 acres

(Use Nomograph "Total Land Requirement", Figure 6-13, or equivalent)

---

[1] Items 2ii and 2iii can be used for determining maximum capacity for potential LA sites in Item 5.

[2] Use a more stringent buffer zone limitation if indicated by applicable Federal, State, or local regulations or site conditions.

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

Item 3 - Potential LA Sites - Location.

(Attach USGS Quad Sheet or equivalent with potential sites outlined and identified.)

Attached is Figure 2-5 upon which is outlined the location of three potential LA sites. This sheet can be used to approximate distances to the three sites from the treatment systems.

A USGS Quad Sheet or other suitable map would be preferred since it would give more reliable information concerning the existing land use at these specific sites.

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Remarks: Referenced figures are in Chapter 2 of this manual. Page 33

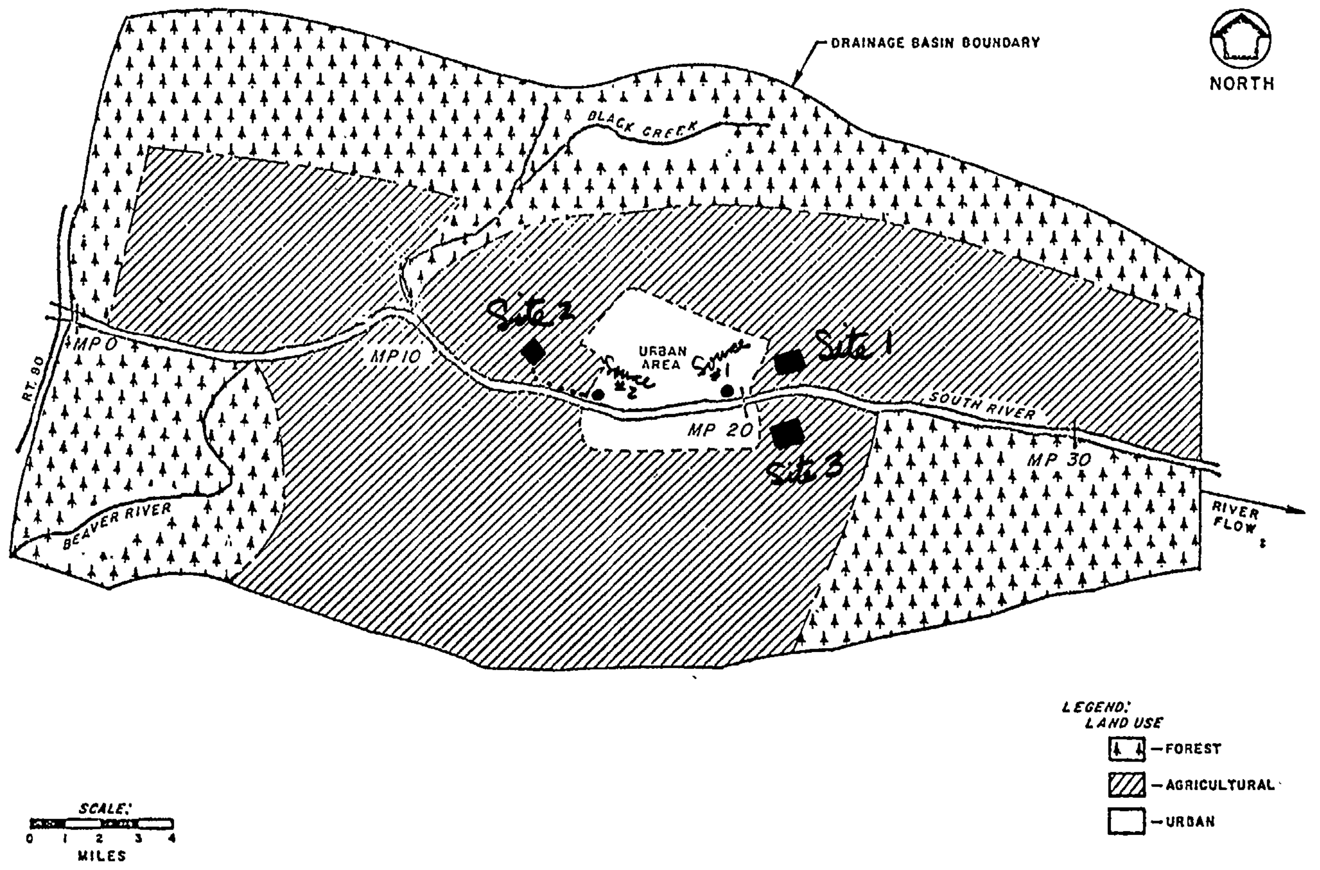

FIGURE 2-5
SIMPLIFIED LAND USE PATTERNS
(SOUTH RIVER, U.S.A.)
HYPOTHETICAL EXAMPLE

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TABLE 6-5

LAND APPLICATION METHODOLOGY WORKSHEET

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Item 4 - Potential LA Sites Data Sheet. (Sample of factors to be considered. This is not an inclusive list; see LA references for additional considerations.)

Site	Approximate Available area (acres)	Estimated treatment capacity (mgd)	Area of site presently irrigated (percent)	Distance from plant (feet)	Elevation difference from plant (feet)	Homes onsite (No.)	Other buildings onsite (No.)	Roads onsite (miles)	Comments-Major problems or advantages
1	3,500	17.5	0	6,900	185	4	5	3.7	1) close to river 2) test for contamination from leachate
2	7,000	37	0	28,000	320	5	2	4.9	1) pumping required 2) close to river; leachate 3) upstream of water treatment intake
3	12,000	73	0	7,950	100	8	13	8.3	1) close to river; potential leachate 2) across river requires pumping & river cross 3) potential regional site for SD 1&2

TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

SPECIFIC SITE EVALUATION
SITE 1

Item 5 - Site Implementation Schedule.

i. Planning Period: 20 years

ii. Construction Phases: 1
(If different from Project Schedule, then describe.)

iii. Pretreatment Requirements: Reference Cost Curve H-6
(If different from Project Schedule, then describe.)

iv. Performance Characteristics - existing facility: Reference Cost Curve H-2 (From Item 1iii)

Item 6 - Pre-application Treatment Cost.

i. Use Treatment Facility Methodology.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Existing Facility	Note: This information would be entered from the worksheets							
1	for TREATMENT FACILITY METHODOLOGY, TABLE 6-4; that							
2	methodology has been used in a previous portion of this							
3	Illustrative Example.							
n								

Item 7 - Land Cost. Note: This schedule will not be used because the cost curve in Figure H-16 (Land Application of Wastewater) includes land cost at $1,000/acre which is adequate for this example.

Application Rate = ________ in./week
Curve Rate = ________ in./week
Factor = Application Rate/Curve Rate = ________

Phase	Design[1] Flow	Adjusted[2] Flow	Land Cost	Salvage Value
1				
2				
3				
n				

[1] Design Flow is the desired application site daily capacity addition for the project Phase.
[2] Adjusted Flow = Design Flow x Factor

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

Item 8 - Transportation Cost.

i. Use Table 6-18, TRANSPORTATION COST METHODOLOGY to complete following schedule:

Phase	Capital	O&M		Salvage Value
		Start	End	
1	3,564,000	120,000	199,000	402,000
2				
3				
n				

Replacement Schedule

Year	Cost
15	1,000,000

Item 9 - Application Site Costs.

i. Use cost curve in Appendix H, Figure H-16, or equivalent method:

Curve No. H-16
Service Life 30 years

Phase	Timing	Flow Rate, mgd		Design, mgd	Capital Cost	O&M		Salvage[2] Value	Revenue[3]	
	Yr to Yr	Start	End			Start	End		Start	End
1	1 - 20	9	17	17	17,000,000	490,000	930,000	5,670,000	60,000	140,000
2										
3										
n										

Replacement Schedule

Year	Cost
None	

$\frac{30-20}{30}$ x Capital

---

[1] Adjust curve cost to reflect installed cost.

[2] Develop Salvage Value = $\frac{\text{Service Life - Years to Planning End}}{\text{Service Life}}$ x Capital which reflects the remaining Phase value at the planning period end.

[3] Include crop revenues, etc.

Net Revenue Estimate: Start = \$40/acre/yr
End = \$50/acre/yr

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TABLE 6-18
TRANSPORTATION COST METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

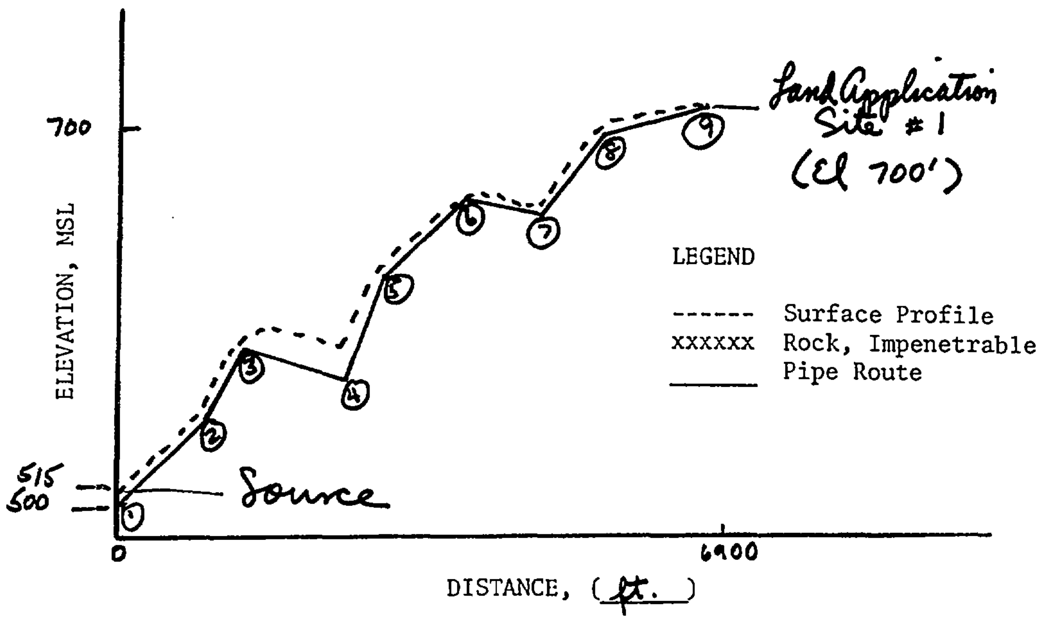
Item 1 - Project/Phase/Source Identification.

i. Project: Land Application

ii. Source Identification

Source	Elevation	Design Flow (mgd) @ Phase No.				
		1	2	3	n	Design Yr
Land Application Site #1	700' MSL					
Source	515' MSL	-	-	-	-	-

Item 2 - Transportation Route Profile.
(also locate route on topographic map)



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Remarks: These sheets (pages 38 to 47) develop transportation cost for LA page 37. Page 38

TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 3 - Critical Segments of the Transportation Route.

Flow Rate: 17 mgd (11.0 cfs)   Assumed n-value: 0.015

	Segment	Elevation/Station[1] E/S(A)	to	E/S(B)	Slope[2]	Velocity[3]	Flow Type[4]
a)	①-③	584		515	-0.05	-	FM
b)	③-④	572		584	+0.01	5.2 fps	G
c)	④-⑥	652		572	-0.06	-	FM
d)	⑥-⑦	660		652	+ 0.01	5.2 fps	G
	⑦-⑨	700		660	- 0.02	-	FM

Notes:

1. Define Elevation Station Data from upstream E/S(A) to downstream E/S(B).

2. Slope = $\frac{E(B) - E(A)}{S(B) - S(A)}$ Units: ft/ft

3. Determine velocity only for positive slope condition; negative slope indicates force main (see discussion); use the attached nomograph (Hydraulic Computations).

4. Flow type: Gravity if positive slope and acceptable velocity (2 fps minimum); force main for other conditions.

Reference calculation sheets: None

Item 4 - Gravity Segments.

Flow Rate: 17 mgd

	Segment	Flow Rate (mgd)	Length (ft)
a)	3 - 4	17	1,100
b)	6 - 7	17	800
c)			
d)			
TOTAL			

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TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 5 - Force Main Segments.

	Segment	Length	Static Head[1]	Dynamic Head[2]	Pumping Head[3]
a)	①-③	1,400'	69'	14'	83'
b)	④-⑥	1,400'	80'	16'	76'
c)	⑦-⑨	2,200'	40'	8'	48'
d)					

Notes: [1] Static head = elevation difference from upstream to downstream.

[2] Dynamic head = See Discussion, Step 2 (Item 5).

[3] Pumping head = Static head + Dynamic head.

Reference calculation sheets: None

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TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

COST DETERMINATION

Item 6 - Project/Phase Identification.

Project: Land Application

Phase: 1

Item 7 - Gravity Sewer Costs.

i. Reference Cost Curve: Figure II-84 (or equivalent curve)
Service Life: 50 years.
Gravity Sewer Length: 0.36 mile ~~feet~~
Design Flow Rate: 17 mgd

ii. Cost Determination.

Construction Cost: $ 154,000 [a]
(Compute only for Phases that include sewer construction; adjust curve cost to reflect installed cost.)

O&M Cost - Start: $ 500 [b]
O&M Cost - End: $ 500
Replacement Cost: None
Salvage Value (SV)[1] $ 92,000 [c]

$$^1\text{SV} = \frac{(\text{Service Life - Years to Project End})}{(\text{Service Life})} \times \text{Capital}$$

[a] Construction cost includes 30% of curve cost for Engineering, Construction Supervision, and Contingency

$$\text{Cost} = \left(\frac{330,000}{\text{mile}}\right)(0.36\,\text{mile})(1.0+0.3) = \$154,000$$

[b] $$\text{O+M} = \left(\frac{1300}{\text{mile}}\right)(0.36) = 470 \approx 500$$

[c] $$\text{SV} = \frac{50-20}{50} \times 154,000 = 92,000$$

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TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 8 - Force Main Cost.

i. Phase: 1

Cost Curve: Figure H-85 (or equivalent curve)

Service Life: 50 years

Length: 5,000 ft

a) 0.95 miles ~~ft~~ @ 17 mgd

b) ____ ft @ ____ mgd

c) ____ ft @ ____ mgd

ii. Cost Determination.

	Segment		
	a	b	c
Capital Cost[1]	410,000[a]		
O&M Cost - Start	500[b]		
O&M Cost - End	500		
Replacement Cost	None	None	None
Salvage Value (SV)[2]	310,000[c]		

[1] Compute only for Phases that include force main construction; adjust curve cost to reflect installed cost.

[2] $SV = \frac{\text{(Service Life - Years to Project End)}}{\text{(Service Life)}} \times \text{Capital}$

a $\text{Cost} = \left(\frac{330{,}000}{\text{mile}}\right)(0.95)(1+0.3) = 410{,}000$

b $\text{O\&M} = \left(\frac{\$500}{\text{mile}}\right)(0.95) = 480 \approx 500$

c $SV = \frac{50-20}{50} \times 410{,}000 = 310{,}000$

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TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 9 - Pump Station/Pumping Cost.

i. Phase: 1

Service Life: 15 years

Pumping Head/Flow: from page 40

	Segment	Total Head	Flow Rate	
			Start	End
(a)	1 - 3	83'	9 mgd	17 mgd
(b)	4 - 6	76'	9 mgd	17 mgd
(c)	7 - 9	48'	9 mgd	17 mgd

ii. Cost Determination. (Cost curve Figure H-30, or equivalent)

	(a)	(b)	(c)
Segment:			
Capital Cost[1]	1,000,000	1,000,000	1,000,000
O&M Adjustment for head[2]	( )	( )	( )
O&M - Start[3]	44,000	42,000	33,000
O&M - End[3]	74,000	70,000	54,000
Replacement Cost/Year[4]	330,000/15	330,000/15	330,000/15
Salvage Value (SV)[5]	None	None	None

(Service life < years to project end)

Notes: [1] Compute only for Phases that include pumping capacity expansion; adjust curve cost to reflect installed cost.

see page 44

[2] Compute as described by the cost curve.

[3] O&M Cost = Curve Cost + Adjustment $\frac{20-15}{15} \times 1{,}000{,}000 = 330{,}000$

④Replacement Cost = $\frac{\text{Years Remaining in Project-Service Life}}{\text{Service Life}}$ x Capital

[5] $SV = \frac{\text{(Service Life - Years Remaining in Project)}}{\text{(Service Life)}}$ x Capital

If Salvage Value is negative, enter as 0 and compute replacement cost.

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Supplemental Calculation: Transportation

Item 9ii

O&M adjustment for TDH other than 10'

	Start O&M			End O&M		
Segment =	a	b	c	a	b	c
Actual TDH	83'	76'	48'	83'	76'	48'
Q, actual, mgd	9	9	9	17	17	17
Q_e*, mgd	75	68	43	141	129	82
Power Cost @ Q_e	25,000	23,000	14,000	47,000	43,000	27,000
Labor @ Q	12,000	12,000	12,000	15,000	15,000	15,000
Material @ Q	7,000	7,000	7,000	12,000	12,000	12,000
TOTAL	44,000	42,000	33,000	74,000	70,000	54,000

Note:

1. Power cost for $Q_e > 100$ mgd made by extending the curve.
2. Q_e estimated by: $Q_e = Q_{design} \times \frac{TDH}{10}$

a) $Q_e = 17 \times 83/10 = 141$ mgd (end), 75 mgd (start)

b) $Q_e = 17 \times 76/10 = 129$ mgd (end), 68 mgd (start)

c) $Q_e = 17 \times 48/10 = 82$ mgd (end), 43 mgd (start)

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Illustrative Example Supplemental Notes

For certain projects, additional phases might be identified to consider the staging of pump stations, etc. The appropriate sheets (Items 6, 7, 8, and 9) would be included at this point.

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TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 10 - Transportation Cost Summary.

i. Cost Schedule.

Phase	Item	Capital Cost	Start - O&M	End - O&M	Salvage Value	
1	#7	154,000	500	500	92,000	Pg.41
	#8	410,000	500	500	310,000	Pg.42
	#9	3,000,000	119,000	198,000	None	Pg.43
TOTAL PHASE 1		3,564,000	120,000	199,000	402,000	
2	#7					
	#8					
	#9					
TOTAL PHASE 2						
3	#7					
	#8					
	#9					
TOTAL PHASE 3						
n	#7					
	#8					
	#9					
TOTAL PHASE n						

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TABLE 6-18 (continued)
TRANSPORTATION COST METHODOLOGY
WORKSHEET

Item 10 - Transportation Cost Summary (continued).

Replacement Schedule		
Item	Year	Cost
9	15	1,000,000

ii. Present-Worth Cost: $ Not required. Cost schedule will be transfered to page 37 of the Land Application evaluation for Source 1.

(Compute only when required.)

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

Item 10 - Monetary Cost Evaluation

i. Cost Schedule. (All costs: $ x 10^6)

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value	Revenues[2]	
								Start	End
1	1 - 20	#6*	10.66	0.44	0.67	-	3.76	0	0
		#7		Cost for land included in Item 8					
		#8	3.56	0.12	0.20	-	0.40	0	0
		#9	17.00	0.49	0.93	-	5.67	0.06	0.14
	TOTAL PHASE 1		31.22	1.05	1.80	0.038[1]	9.83	0.06	0.14
2		#6							
		#7							
		#8							
		#9							
	TOTAL PHASE 2								
3		#6							
		#7							
		#8							
		#9							
	TOTAL PHASE 3								

*Assumed Cost to allow completion of example.

[1] Variable O+M (Phase 1) = $\frac{1.80 - 1.05}{20} = 0.038$

[2] Variable Revenue (Phase 1) = $\frac{0.14 - 0.06}{20} = 0.004$

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TABLE 6-5 (CONTINUED)
LAND APPLICATION METHODOLOGY
WORKSHEET

Item 10 - Monetary Cost Evaluation (Continued).

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value	Revenues Start End
4		#6						
		#7						
		#8						
		#9						
	TOTAL PHASE 4							

Replacement Schedule ($ x 10^6)

Item	Year	Cost
8	15	1.00

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TABLE 6-16
PRESENT-WORTH METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgements presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Items 1-10 Present-Worth Calculation.

Planning Period 20 years Interest 7 %

	Item (Reference Page)	Amount			Present-Worth
1.	Phase 1 Capital (pg. 48)	31,220,000	x	1.0 (Yr 1)	31,220,000
2.	Phase 2 Capital (pg. ___)	______	x	______ (sppwf^a) (Yr ___)	______
3.	Phase 3 Capital (pg. ___)	______	x	______ (sppwf^b) (Yr ___)	______
4.	Phase n Capital (pg. ___)	______	x	______ (sppwf^c) (Yr ___)	______
5.	Replacement year (h) (pg. ___)	1,000,000	x	0.36 (sppwf^h) (Yr 15)	360,000
6.	Replacement year (i) (pg. ___)	______	x	______ (sppwf^i) (Yr ___)	______
7.	Replacement year (j) (pg. ___)	______	x	______ (sppwf^j) (Yr ___)	______
8.	Replacement year (k) (pg. ___)	______	x	______ (sppwf^k) (Yr ___)	______
9.	Replacement year (1) (pg. ___)	______	x	______ (sppwf^1) (Yr ___)	______
10.	Salvage Value (Negative Cost) (pg. 48)	-9,830,000	x	0.26 (sppwf^z) (Yr 20)	-2,560,000

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TABLE 6-16 (continued)
PRESENT-WORTH METHODOLOGY
WORKSHEET

Item 11 - 19 Present-Worth Calculation.

Planning Period 20 years

Interest 7 %

	Item (Reference Page)	Amount		Present-Worth
11.	O&M Phase 1 Constant (pg. ___)	1,050,000	x 10.6 (uspwf^d) x 1.0 (#Yrs 20 ;Yr 1)	11,100,000
12.	O&M Phase 1 Variable (pg. ___)	38,000	x 77.5 (gspwf^d) x 1.0 (#Yrs 20 ;Yr 1)	2,950,000
13.	O&M Phase 2 Constant (pg. ___)		x ____(uspwf^e x sppwf^a) (#Yrs____;Yr____)	
14.	O&M Phase 2 Variable (pg. ___)		x ____(gspwf^e x sppwf^a) (#Yrs____;Yr____)	
15.	Revenue* ~~O&M~~ Phase ~~A~~ 1 Constant (pg. ___)	-60,000	x (10.6)(1.0) (uspwf^f x sppwf^b) (#Yrs 20 ;Yr 1)	-640,000
16.	Revenue* ~~O&M~~ Phase ~~A~~ 1 Variable (pg. ___)	-4,000	x (77.5)(1.0) (gspwf^f x sppwf^b) (#Yrs 20 ;Yr 1)	- 310,000
17.	O&M Phase n Constant (pg.___)		x ____(uspwf^g x sppwf^c) (#Yrs____;Yr____)	
18.	O&M Phase n Variable (pg.___)		x ____(gspwf^g x sppwf^c) (#Yrs____;Yr____)	
19.			TOTAL PRESENT-WORTH	$ 42,120,000

*Negative cost

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Illustrative Example Supplemental Notes

In many situations, the user would wish to evaluate another potential land application site. This would be done by repeating the calculations for Items 5, 6, 7, 8, 9, and 10 and inserting them at this location. After evaluating all alternatives, the user would continue to Item 11.

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TABLE 6-5 (CONTINUED)

LAND APPLICATION METHODOLOGY WORKSHEET

PRESENT-WORTH COST EVALUATION

Item 11 - Present-Worth Cost.

Site	Present-Worth Cost	Reference Sheet
1	$42,120,000	50,51

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Illustrative Example Supplemental Notes

The reuse of the treated wastewater from Source 1 is evaluated in the following worksheets. This example is not complete in that Present-Worth costs are not computed. However, the cost schedules specific to this evaluation are developed with appropriate commentary.

The potential reusers identified in this example currently use irrigation water from groundwater wells. For this example, additional treatment of the treated wastewater is assumed necessary for the reuser's use to demonstrate the development of the additional treatment cost. Also developed is a cost schedule for the reuser's alternative water cost for the replaceable portion of their water supply (i.e., the portion that could be replaced by treated wastewater).

These wastewater reuse cost evaluations develop two Present-Worth costs. The first is the estimated cost incurred for transportation and treatment of the wastewater for reuse. The second is the alternative cost of the reuser utilizing the projected water supply. Comparison of these two costs indicates which represents the least monetary cost, and thus indicates if an economic incentive might exist for reuse (i.e., reuse cost less than projected supply cost).

If reuse appears economically attractive, the user can develop total project costs for reuse by using the appropriate component methodologies. The Present-Worth cost of the total project would be carried to the FRAMEWORK METHODOLOGY WORKSHEET, page 8, to compare with other control alternatives.

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TABLE 6-12
WASTEWATER REUSE METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approach used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - General Reuse Criteria.

i. Total municipal water demand approaching existing water supply? YES [✓ NO]

ii. Existing water supplies unavailable or insufficient for new uses (e.g., irrigation, industry)? YES [✓ NO]

iii. Existing water supply subject to environmental degradation (e.g., salt-water intrusion, aquifer drawdown)? YES [✓ NO]

iv. Point source wastewater effluent available in sufficient quantity to satisfy potential new needs? [✓ YES] NO

v. Point source disposal technique involves unusually high costs? [✓ YES] NO

vi. Planning area includes any large uses of non-potable water? [✓ YES] NO

vii. Absence of restrictions (riparian rights) or related water law? [✓ YES] NO

viii. Expressed interest on the part of potential wastewater reusers? [✓ YES] NO

Note: The user should determine from the answers to these general questions the relative demand for reuse. This is a judgment to be made by the planner or engineer.

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TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

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Item 2 - Potential Reuse Source Identification

	Potential Water Reuser Identification	Water Source/ Treatment	Water Use	Present Water Cost	Projected Water Cost	Quantity		Quality		Reliability	Distance
						Current	Design Yr	Desired	Minimum		
	EXAMPLE:										
1.	Georgia Power Plant, Mitchell. Mr. John Phillips Plant Engineer	Flint River/ screened, settled.	Cooling Water	$0.12/1000 gal	$0.24	1.4 mgd (0.6 mgd firm minimum)	1.8 mgd (1985)	76°F pH, 7.2	84°F pH 7.0-7.8	complete	3 miles, across Flint River
2.	Blue Valley Pecan Farm. Mr. Tom Young Owner	Private well, no treatment	Irrigation	$.08/1000 gal pumping cost	$0.22	400,000 gpd~ (May-Nov)		No toxic metals, low dissolving solids, normal pH	same	low	2 miles
	Albany County Club	Private well, no treatment	Irrigation	$.14/1000 gal pumping cost	$0.38	500,000 gpd (May-Nov)	700,000 gpd (1980)	same		low	8 miles

Note: the above example data will be utilized in the Illustrative Example.

TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

POTENTIAL REUSER EVALUATION
Reuser: #2, Blue Valley Pecan Farm, Albany Country Club

Item 3 - Point Source Treatment to Meet Reuse Criteria.

	Reference Cost Curve	Reuser/Point Source Critical Parameters							
		Q	BOD	TSS	N	P	T	TDS	Cl_2
Reuser Water Quality Requirements *	N/A	mg/L 0.7	mg/L 19	mg/L 26	-	-	-	mg/L 658	mg/L -
Existing Wastewater Effluent Quality	H-3	9.0	100	50	-	-	-	-	-
Existing Wastewater Effluent Quality Control Criteria	H-12	9.0	5	5	-	-	-	-	-
Upgrade Technique[1]	H-59	0.7	None	None				None	H-59

[1] Identify the cost curve for upgrading each identified parameter to the reuser criterion; identify the required system under "Reference Cost Curve" from individual curves.

Item 4 - Treatment Requirement Cost Schedule to Meet Reuser Criteria.

(Use TREATMENT FACILITY METHODOLOGY to define costs for treatment above that required for discharge)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage Value
	Yr.	to Yr.		Start	End	Year	Cost	
Existing Facility	Existing facility must be upgraded to H-12 level for discharge							
1	1	20	37,000	6,000	9,000	15	12,000	None
2								
3								
n								

* Reuser water quality requirements from Table 6-14. The requirements for zero chlorine residual is for illustrative purposes only, and does not represent a real situation.

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Remarks: Reference pages 58-61 describe costs for Item 4. Page 57

TABLE 6-4
TREATMENT FACILITY METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - Program Implementation Schedule.

i. Planning Period: 20 years

ii. Construction phases: 1

Phase	Timing			Flow Projection (mgd)		Design Flow (mgd)
	Year	to	Year	Start	End	
1*	Present	to	N/A			
~~2~~ 1	1	to	20	0.7	1.5	1.5
3		to		assumes	Blue	
4		to		Valley	growth	
n		to		to 800,000	gpd	
				demand		

*Existing facility not utilized at full capacity.

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Remarks: ______ Page 58

TABLE 6-4 (continued)
TREATMENT FACILITY METHODOLOGY WORKSHEET

iii. Treatment Objectives.

Note: Dissolved oxygen deficits use ultimate oxygen demand inputs (Table 6-3). These must be reconverted back to CBOD and NBOD (NH_3) concentrations to determine discharge limitations (See Appendix H discussion of Treatment Systems Performance Matrix).

	Effluent Quality								
Phase	Reference Cost Curve**	BOD mg/1	COD mg/1	TSS mg/1	T-P mg/1	NH_3-N mg/1	NO_3-N mg/1	T-N mg/1	T-C #/100ml
Existing Facility	Not applicable			- looking		at	treatment	alone	discharge limit
1	H-59								≈0
2									
3									
n									

**Treatment System curve number (Appendix H, Figures H-2 to H-15) or reference number for synthesized system cost curve developed from unit process curves (Appendix H).

iv. Existing Facility Characteristics.

Design Capacity: None mgd

Service Life: ______ years

Years in Service: ______ years

Remaining Service: ______ years

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Remarks: new facility for treatment + reuse of treated effluent Page 59

TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 3 - Expansion Program or [✓ New Facility Construction.]

Phase Number 1

i. Existing Capacity = None mgd (previous phase or existing facility; zero if new facility)

ii. Expanded or New Facility Capacity = 1.5 mgd (design capacity of next phase)

iii. Level of Treatment: Reference Cost Curve H-59
Service Life 15 yr

iv. Construction cost of expanded or new facility - enter cost curve at expanded or new facility at capacity (ii) $20,000

v. Construction cost of existing facility - enter cost curve at existing facility at capacity (i) —

vi. Sub-Total 1: Expanded or New Facility Construction Cost (iv-v) = $20,000

vii. plus Sub-Total 1 x Piping 15% = 3,000
Sub-Total 1 x Electrical 12% = 2,400
Sub-Total 1 x Instrumentation 8% = 1,600
Sub-Total 1 x Site Preparation 5% = 1,000

viii. Sub-Total 2: Construction Cost (vi + vii) $28,000

ix. plus Sub-Total 2 x Expansion/Upgrading Factor N/A = —
Sub-Total 2 x Engineering and Construction 15% = 4,200
Sub-Total 2 x Contingencies 15% = 4,200

x. Sub-Total 3: Capital Cost (viii + ix) $36,400

xi. CAPITAL COST OF EXPANSION OR OF NEW FACILITY = Sub-Total 3 x $\frac{\text{ENR (Current)}}{2475^*}$ $\frac{2499}{2475}$ = $37,000

* ENR = 2475, September, 1976.

Note: Items 2 and 4 not included because these calculations are not required.

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TABLE 6-4 (CONTINUED)
TREATMENT FACILITY METHODOLOGY
WORKSHEET

Item 5 - O&M Constant and Variable Cost.

Phase 1

Level of Treatment: Reference Cost Curve H-59

Timing		Design Flow		O&M Cost	
Start (yr.)	End (yr.)	Start (mgd)	End (mgd)	Start	End
1	20	0.7	1.5	$ 6,000	$ 9,000

Item 6 - Phase ____ Replacement Costs (Upgraded and/or Expanded Portion) (Compute if planning period is greater than phase service life) Replacement Cost Schedule.

~~Expansion~~ New		Upgrading		Total	
Year	Cost	Year	Cost	Year	Cost
15	12,000			15	12,000

Replacement Cost for Phase 1 =

$$\frac{\text{Years from Time of Replacement to end of Planning Period}}{\text{Service Life}} \text{ x Capital}$$

$$RC = \frac{20-15}{15} \times 37{,}000 = 12{,}000$$

Item 7 - Phase 1 Salvage Value at End of Planning Period. (Compute if phase service life is greater than years to planning period end)

$$\text{Salvage Value} = \frac{\text{(Service Life - Years to Planning End)}}{\text{Service Life}} \text{ x Capital}$$

~~Expansion~~ New S.V. = None $ None

Upgrading S.V. = $ ____

Total Phase S.V. $ None

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TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

Item 5 - Reuse Transportation Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Existing Facility			Not applicable					
1*	1	20	2,200,000	20,000	30,000	15	80,000	1,200,000
2								
3	*Costs are assumed for illustrative purposes.							
n								

Item 6 - Wastewater Reuse Project Costs.

i. Project Cost Schedule.

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
1		#4	37,000	6,000	9,000	–	0
		#5	2,200,000	20,000	30,000	–	1,200,000
	TOTAL PHASE 1		2,240,000	26,000	39,000	700	1,200,000
2		#4					
		#5					
	TOTAL PHASE 2						
3		#4					
		#5					
	TOTAL PHASE 3						

Variable O+M = $\frac{39,000 - 26,000}{20}$

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Remarks: Return to WASTEWATER REUSE METHODOLOGY Page 62

TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

Item 6 - Wastewater Reuse Project Costs (continued).

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
n		#4					
		#5					

TOTAL PHASE n

Replacement Schedule

Year	Cost
15	12,800
15	80,000

ii. Present-Worth Cost (using PRESENT WORTH METHODOLOGY).

Interest 7 %

Present-Worth Cost $

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TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

Item 7 - Reuser Replaceable Water Costs.
(Cost that would be incurred for replaceable water supply without wastewater recycle)

i. Cost Schedule.

Phase	Timing	Recycle Water Use[1], gpd		Unit Water Cost[2]	Annual Water Cost[3], $/year	
	Yr to Yr	Start	End	$/1000 gal	Start	End
1	1 20 Blue Valley	400,000	800,000	Start End 0.08 0.22	6,800	37,000
~~2~~ 1	1 20 Albany CC	300,000	700,000	0.14 0.38	8,900	57,000
3						
n						

[1] Recycle Water Use represents the portion of total reuser water requirement that could be satisfied by treated wastewater.

[2] Unit Water Cost represents projected cost for existing supply if adequate, or the unit cost for development and treatment of a new supply.

[3] Annual Water Cost = (gpd) ($/gal) (365 days/yr) (required May - Nov. from Item 2)

ii. Reuser Replaceable Water Present-Worth Cost.
(using PRESENT-WORTH METHODOLOGY): $__________

Item 8 - Wastewater Reuser Relative Costs.

i. Replaceable Water Cost: $__________
(Alternative Present-Worth cost for reuser water needs that can be satisfied during the planning period by treated wastewater; from Item 7 ii)

ii. Reused Wastewater Cost: $__________
(Present-Worth cost to utilize treated wastewater; from Item 6 ii)

iii. Other Factors: (describe factors other than cost that will affect further evaluation)

Further evaluation depends upon the location of additional reusers in the vicinity of these reusers to share the cost of a force main system. Also, if a substantial reuser were identified & reliable, the required treatment facility cost could be reduced.

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TABLE 6-12 (continued)
WASTEWATER REUSE METHODOLOGY
WORKSHEET

Item 9 - Potential Reuser/Point-Source Combinations

Point Source		Reuser		Identified Present-Worth Costs			
ID	Q,mgd	ID	Q,mgd	Wastewater Reuse	Replaceable Water	Total	Non-Monetary Cost Incentive

None identified

Note: The user would normally continue evaluation of reuse candidates at this point, and would then transfer pertinent information on the favorable candidates to Item 9.

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Illustrative Example Supplemental Notes

The following worksheets outline the evaluation of Impact Area Modifications as a control alternative for Source 1. This portion of the example is not worked to completion because the required cost computations have been previously demonstrated in the treatment facility and land application control alternatives evaluations. However, the determinations specific to this methodology have been utilized.

The general receiving water condition in this example has been defined using the water quality impact analysis techniques in Chapter 5 of this manual, which consider all contributing sources and their relative impact. This information identifies the potential usefulness of the modifications presented in this example.

If the discharge relocation example had been carried further, the Chapter 5 impact analysis would be utilized to determine the level of treatment at the proposed discharge site. If a reduced treatment level were indicated, then a cost reduction for treatment might be realized, depending on the magnitude of the transportation cost that would also be incurred.

Artificial reaeration is not a viable control alternative for this example because the critical water quality parameters are nitrogen and phosphorus, which would not be substantially affected by reaeration. The information and steps that would be considered for a potential situation are indicated on the worksheets.

Flow augmentation also is not a viable control alternative for this example because the critical water quality condition is defined by the long-term concentration of nitrogen and phosphorus, but this modification is appropriate primarily to relieve a short-term stiuation. The worksheets have been marked to indicate the appropriate entries to be made when this evaluation is appropriate.

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TABLE 6-14
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approach used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 - Receiving Water Quality.

Source # 1

Water Quality Conditions:

The user is referred to Table 6-2 which describes the desired water quality in the South River for this hypothetical example. The information in this table is used to identify which impact area modifications might alleviate the undesireable water quality conditions.

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TABLE 6-14
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

Discharge Relocation Evaluation.

Item 2 - General Criteria for Discharge Relocation.*

1. Is this source a principal cause of the critical stream condition? [✓ YES] NO
2. Are there several significant sources near this source? [✓ YES] NO
3. Would relocation be relatively inexpensive? one other could be [✓ YES] NO
4. Is there a major (or larger) stream nearby that could receive the source? YES [✓ NO]

Item 3 - Alternate Discharge Site Identification.

(Factors: Less stringent water quality criteria; lower net source loading; increased dilution due to tributary flow)

	Site	Location	Distance
a)	Mile 30	Downstream from Source 1 on S. River	appr. 10 miles
b)			
c)			

Reference Figure 2-4, Map of Hypothetical Planning Area, South River, USA.

* These evaluations are qualitative in nature and will be useful in defining priorities for further cost evaluation.

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

Site ____________ Evaluation

Item 4 - Modified Source Level of Treatment (from Impact Analysis, Chapter 5).

Level	Reference Cost Curve	Parameter Control Levels							
		BOD mg/l	COD mg/l	TSS mg/l	T-P mg/l	NH_3-N mg/l	NO_3-N mg/l	T-N mg/l	T-C #/100ml
Existing Discharge Site									
Alternate Discharge Site A									
Alternate Discharge Site B									

This information will be defined by the results of the Chapter 5 Impact Analysis. The required level of treatment (indicated by the Reference Cost Curve) should be lower for any alternative Discharge Site that is investigated further.

Item 5 - Discharge Relocation Transportation Cost Schedule.
(use the TRANSPORTATION COST METHODOLOGY)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Existing Facility								
1								
2								
3								
n								

This cost schedule is developed as previously demonstrated.

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

Item 6 - Modified Treatment Level Cost Schedule.
(use TREATMENT FACILITY METHODOLOGY)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Existing Facility								
1			This cost schedule is developed					
2			as previously demonstrated.					
3								
n								

Item 7 - Project Cost.

i. Project Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr. to Yr.			Start	End	Year	Cost	Value
Existing Facility								
1			This schedule combines the costs from					
2			items 4, 5, 6 and any other					
3			identified cost.					
n								

ii. Project Present-Worth Cost: $____________
(use PRESENT-WORTH METHODOLOGY)

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TABLE 6-14
IMPACT AREA MODIFICATION METHODOLOGY WORKSHEET

ARTIFICIAL REAERATION EVALUATION

Item 8 - Receiving-Body Dissolved Oxygen Requirements.

Condition	Dissolved Oxygen, mg/l	@	Location
Water Quality Limit (1)	5.0 mg/L / 4.0 mg/L		M.P. 0→20/20→35
Critical Level (2)	(Deficit) None / 3.10 mg/L		M.P. 19.5/23

References: (1) - Table 6-1; (2) - Table 6-2.

Item 9 - Modified Stream Quality at Revised Source Loads. (developed from Impact Analysis, Chapter 5)

	Wastewater Load		Resulting		
	Parameter	Discharge Level	Minimum Stream DO	@	Location
A.	BOD		Not affected	@	
	TSS	Not affected			
	T-N/P	Not affected by aeration			
B.	BOD			@	
	TSS				

Note: the user would determine the resulting stream D.O. at a modified load condition in the case where total nitrogen and phosphorus were not critical.

Item 10 - Artificial Reaeration Requirement.

i. Oxygen-Transfer Requirements.

Stream flow rate at critical conditions: (from stream data) cfs

Critical DO level: 3.10 mg/l (deficit)

Minimum acceptable DO level: 4.0 mg/l

Oxygen transfer requirement: ‹calculate› lb/hr for (from stream data) hr/yr

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY WORKSHEET

Item 10 - Artificial Reaeration Requirement (continued).

ii. Reaeration Project Phasing.

Phase	Timing		Reaeration Oxygen Demand, lb/hr		Capital	O&M		Replacement Cost	Salvage Value
	Yr	to Yr	Start	End		Start	End		
1			This cost	schedule	would	be developed			
2			using site-specific		information			and	
3			the TREATMENT	FACILITY		worksheets.			
n									

Item 11 - Treatment Cost Schedule at Modified Treatment Level.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage Value
	Yr.	to Yr.		Start	End	Year	Cost	
Existing Facility								
1			This cost	schedule	is	developed		
2			as previously		demonstrated.			
3								
n								

Item 12 - Artificial Reaeration Project Cost.

i. Project Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage Value
	Yr.	to Yr.		Start	End	Year	Cost	
Existing Facility								
1			This cost	schedule	can	be developed		
2			using the	TREATMENT		FACILITY		
3			worksheet.					
n								

ii. Project Present-Worth Cost: $__________

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

FLOW AUGMENTATION EVALUATION

Item 13 - General Applicability of Flow Augmentation as a feasible control alternative.

Do critical water quality conditions occur at low flow?
(Yes or no) Yes Critical Parameters, if yes; D.O. (no for N & P)

Comments: (Special conditions, model assumptions, reference sheets)

See Tables 6-1 and 6-2.

Item 14 - Flow-Augmentation Capability.

i. Existing Reservoir low-flow augmentation capacity: from water supply data cfs
Duration: ______ days

ii. Proposed Reservoir low-flow augmentation capacity: this would be a site specific determination cfs
Duration: ______ days

Item 15 - Modified Source Load Control with Flow Augmentation.

i. Available Flow for Augmentation for the required duration (Item 14) cfs

ii. Revised stream low flow Item 14 and Item 10 source ~~cfs~~

iii. Revised Level of Treatment
from Impact Analysis (Chapter 5) performed with the original pollutant loads and the revised stream flow in (ii) above.

	Critical Parameter	Revised Control Level
a)		
b)		

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY
WORKSHEET

Item 16 - Flow Augmentation Cost

i. Modified Level of Treatment Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Existing Facility								
1			This cost	schedule	is	developed		
2			using the	TREATMENT		FACILITY		
3			worksheets.					
n								

ii. Flow Augmentation Cost Schedule.

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Existing Facility			This cost	schedule	can	be developed	using	
1			the TREATMENT		FACILITY	worksheets	in	
2			certain	situations.	However,	in many	cases,	
3			this	information	would	be available	from	
n			existing	water	supply	surveys.		

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TABLE 6-14 (CONTINUED)
IMPACT AREA MODIFICATION METHODOLOGY WORKSHEET

Item 16 - Flow Augmentation Cost (continued).

iii. Project Cost Schedule.

Phase	Year to Year	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
1		16i					
		16ii					
	Total Phase 1						
2		16i					
		16ii					
	Total Phase 2						
3		16i					
		16ii					
	Total Phase 3						
n		16i					
		16ii					
	Total Phase n						

This cost schedule is developed as demonstrated in other similar cost schedule tables.

Replacement Schedule

Year	Cost

iv. Project Present-Worth cost.

Interest ____________%

Present-Worth Cost $____________

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Remarks: ____________ Page 75

Illustrative Example Supplemental Notes

The evaluation process of Strategy 9 would normally continue at this point with an evaluation of the control alternatives for the next source, as indicated by the flow diagram (Figure 6-6). The evaluation of Source 2 control alternatives is identical to that shown for Source 1; therefore, these calculations have not been included in this example.

Following the evaluation of all sources for each strategy, the evaluation of regionalized control facilities is investigated. The following worksheets demonstrate one approach for regionalization.

Typical information concerning potential site identification and evaluation has been included in the appropriate worksheet items. However, other factors may be more significant for the planner's area of interest.

This example demonstrates regionalization of residuals disposal as well as treatment facilities. Again, only the information concerning site identification has been indicated since all cost determinations are similar to those presented in previous methodologies.

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TABLE 6-15
REGIONALIZATION METHODOLOGY
WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approach used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Wastewater Treatment Regional Site Evaluation

Item 1 - Source Identification.
(Attach topographic map with sources located)

	Source/Location	Flow	Level of Control/ Critical Parameters
a)	1 / SD#1	9 → 17	Curve H-12/S-1*
b)	2 / SD#2	11 → 30	Curve H-12
c)			
d)			
e)			
f)			
g)			
h)			
i)			
j)			
k)			
l)			
m)			
n)			

* See page 11.

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 2 - Regional Site Identification.
(Attach topographic map with sources located)

	Site/Location	Available Area (acres)	Estimated Max. Flow (mgd)	Existing Condition
a)	SD #1 STP	30	60	9 mgd primary STP
b)	SD #2 STP	25	70	12 mgd secondary STP
c)	S. River, 2 miles	110	400	Farmland and
d)	downstream of SD #1 STP			light forest,
e)				undeveloped.
f)				
g)				
h)				
i)				
j)				
k)				
l)				
m)				
n)				

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 3 - Potential Regional Site/Source Combinations.

Combination I.D. No.	Source 1		Source 2		Regional Site		
	I.D.	Q (mgd)	I.D.	Q (mgd)	I.D.	Max Site Q (mgd)	Max Design Q (mgd)
1	a) SD#1	17	b) SD#2	30	a) SD#1 STP	60	47
2	a) SD#1	17	b) SD#2	30	b) SD#2 STP	70	47
3	a) SD#1	17	b) SD#2	30	c) S. River downstream	400	47

Note: This list presents a running account of the alternatives under consideration for wastewater reuse. More than two sources can be combined if desired.

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Remarks: ________ Page 79

TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

REGIONAL SITE EVALUATION

(S.D #1 + S.D. #2 @ S. River,
2 miles downstream of S.D. #1 STP)

Regional Combination No. 3

Item 4 - Level of Treatment at Regionalization Site.

	Parameter	Raw Wastewater	Discharge Level	Level of Treatment
a)	BOD			
b)	T-P		2 mg/L	H-12
c)	T-N		8 mg/L	H-12

Level of Treatment: H-12 Note: in the general case, the Chapter 5 Impact Analysis would be performed to define the level of treatment.

Item 5 - Regional Site Wastewater Treatment Cost Schedule.
(Use the TREATMENT FACILITY METHODOLOGY)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Existing Facility			This cost is computed exactly as before with a new					
1			facility because this is a new site. In this example,					
2			a phase-out cost might exist if the facilities at					
3			S.D. #1 and S.D. #2 are not utilized.					
n								

Item 6 - Wastewater Transportation Cost Schedule.
(Use the TRANSPORTATION COST METHODOLOGY)

Phase	Timing		Capital	O&M		Replacement Cost		Salvage
	Yr.	to Yr.		Start	End	Year	Cost	Value
Existing Facility			This cost is computed exactly as before for two flows					
1			in the pipeline: one for the flow between the					
2			two existing discharges, and for both flows combined.					
3								
n								

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 7 - Regionalization Cost Evaluation.

i. Cost Schedule (Regional Combination No. ________)

Phase	Timing Yr to Yr	Item	Capital Cost	Start O&M	End O&M	Variable O&M	Salvage Value
1		# 5	This schedule would be filled out as other similar schedules.				
		# 6					
TOTAL PHASE 1							
2		# 5					
		# 6					
TOTAL PHASE 2							
3		# 5					
		# 6					
TOTAL PHASE 3							
4		# 5					
		# 6					
TOTAL PHASE 4							

Replacement Schedule

Item	Year	Cost

ii. Present-Worth Project Cost: $________

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 8 - Present-Worth Cost.

Regional Site I.D.	Source I.D.		Separate Treatment Costs		Regional Treatment Cost
	No. 1	No. 2	Source 1	Source 2	

The costs determined for each alternative are recorded here. The comparison of the individual treatment cost to the regionalized cost indicates if a monetary cost incentive exists. The results on this table should indicate the types of combinations that are most effective which will assist in the identification of additional combinations.

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Residuals Disposal Regional Site Evaluation

Item 9 - Residuals Generator Source Identification.

	Source/Location	Flow (mgd)	Residual Disposal Method	Residual Characteristics	
				Quality	Type
a)	S.D. #1	17	landfill	dewatered (20%)	biological
b)	S.D. #2	30	landfill	dewatered (30%)	chem. (lime)
c)				(same)	
d)					
e)					
f)					
g)					
h)					
i)					
j)					
k)					
l)					
m)					
n)					

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TABLE 6-15 (continued)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 10 - Potential Residual Disposal Sites.

	Site/Location	Available Area (acres)	Estimated Capacity	Existing Condition
a)	off Hwy 3, East of S.D. # 1	200	*	Site cleared (proposed site for S.D. # 1 landfill)
~~b)~~				
b)	S.D. # 2	20	*	Existing site for S.D. # 2
d)				
e)				
f)				
g)				
h)				
i)				
j)				
k)				
l)				
m)				
n)				

* This should be developed in terms of useful life for a design flow rate.
E.g. 50 years as landfill @ 30 mgd
(curve H-12 equivalent sludge)
or S-1.

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TABLE 6-15 (CONTINUED)
REGIONALIZATION METHODOLOGY
WORKSHEET

Item 11 - Potential Regional Site/Generator Combinations.

	Regional Site		Generator			
	Site I.D.	Capacity	I.D.	Location	Quantity	
a.	off Hwy 3	adequate for 20 years	(a)	S.D. #1	17 mgd	S-1
			(b)	S.D. #2	30 mgd	H-12
b.						

Item 12 - Regional Site/Generator Combined Present Worth Costs. (Using the RESIDUALS DISPOSAL METHODOLOGY)

	Combination Generator I.D.	Present Worth Cost at Combination	Present Worth Cost Separate
a.	off Hwy 3	*	**
	S.D. #1		**
	S.D. #2		
		TOTAL	
b.			
		TOTAL	

* This number would be computed using the information in Item 11 and the RESIDUALS DISPOSAL METHODOLOGY.

** This information would be available from previous computations.

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Illustrative Example Supplemental Notes

The monetary cost evaluation for achievement of Strategy 9 load reductions for Sources 1 and 2 is complete at this point. All costs for this evaluation have been entered in Table 6-3, Item 2, as they were generated.

The illustrative example continues at this point with an evaluation of wet weather, Sources 3 and 4.

The first of the control alternatives to be considered for wet weather sources is the LAND MANAGEMENT METHODOLOGY. For Source 3, the feasibility of controlling the TSS and TC loadings through land management is considered.

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TABLE 6-6
LAND MANAGEMENT METHODOLOGY
WORKSHEET

> The procedures, calculations, assumptions, and judgements presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 Land uses and land use activities of concern.

Land Uses and Land Use Activities[1]	Wasteload BOD_5	N	P	TSS	TC[2]	Applicable Land Management Alternatives
Residential (Low-density)	NOTE: Wasteloads from these land uses can be generated using techniques presented in Chapters 2 and 3, or by equivalent methods					Sediment and erosion control program
Residential (Medium density)						Street sweeping
Residential (High density)						Street sweeping
Light Industry						Site runoff controls
Commercial						Street sweeping

Item 2

i) Load reduction requirements:

	Percent Reduction BOD_5	N	P	TSS	TC
Load reduction requirements:				65%	95%

ii) Land management alternative performance capability.

Land Uses and Land Use Activities	Applicable Land Management Alternative	Performance Range-Percent Reduction BOD_5	N	P	TSS	TC
Residential (Low density)	Sediment and erosion control program	(Would be determined by applying unit estimates (e.g. 5 lb BOD_5/acre/yr) of pollutant removals to the areas under consideration and figuring percent removals based on estimated overall loadings)				
Residential (medium density)	Street Sweeping					
Residential (High density)	Street Sweeping					
Light Industry	Site Runoff Controls					
Commercial	Street Sweeping					

[1] See Table 6-7.
[2] Most probable number/100 ml

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TABLE 6-6 (continued)
LAND MANAGEMENT METHODOLOGY
WORKSHEET

Item 3 Identification of alternatives which will achieve required reduction.

Land Uses and Land Use Activities	Land Management Alternatives	Parameter	[Wasteload] x	[Performance Capa.] =	[Wasteload Red.]
Residential (high and medium density)	Street Sweeping	TSS			
Commercial	Street Sweeping	TSS			

Note: The implementation of a street sweeping program in the high- and medium-density residential and commercial areas was chosen for evaluation for illustrative purposes. This choice was based on the relative acreage of the land uses in Subarea 1, on the relative magnitude of contribution of the pollutants of concern, and on the potential effectiveness of a street sweeping program in achieving the required load reductions of total suspended solids (TSS). It is assumed that none of the land management alternatives applicable to Subarea 1 would achieve near the required reductions of the total coliform (TC); therefore, other control alternatives would have to be applied for this pollutant

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TABLE 6-6 (continued)
LAND MANAGEMENT METHODOLOGY
WORKSHEET

Item 4 Capital and O&M Cost (From Appendix G).

Land Management Alternative	Affected Land Use/Activity (Acres)	Capital Cost	O&M Cost
Street Sweeping	∼ 5,000 acres	(Would be determined by	
		applying unit estimates	
		[e.g. $ /curb mile/year]	
		to areas of concern)	

Item 5 Present worth cost of land management alternatives.

Land Management Alternatives	Present Worth Cost	Information Reliability[1] Performance	Cost
Street Sweeping	(Using PRESENT-	(Based on judgment	
	WORTH METHODOLOGY)	of the user)	

[1] From Appendix G

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Illustrative Example Supplemental Notes

The degree of pollutant reduction achievable through land management control alternatives may or may not be what is required by the load reduction strategy. Therefore, collection system controls, the next wet-weather control alternative in the framework (Figure 6-6), may be evaluated as an additional control alternative or as an alternate approach. In this example, collection system controls will be evaluated as a separate control alternative.

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TABLE 6-8
COLLECTION SYSTEM CONTROL METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State and local guidelines and regulations.

Item 1 Sewer System Characteristics.

Outfall No.	Subarea Location[1]	Sewer Segment Type[2]	Type of Overflow Control Device[3]
1			
2			
3	Subarea1 *	Combined	Regulator (new)
4	Subarea 2*	Storm	None
5			

* See map, page 2.

---

[1] Locations should be referenced to a map using outfall and subarea numbers.

[2] Combined sewer, storm sewer or unsewered.

[3] Swirl separator or conventional regulator.

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 2 Pollutants (BOD_5, TSS, TC, P, N) to be controlled (from Load Reduction Strategy Matrix, Table 6-19).

Segment No.	Pollutant Parameter	Required % Reduction
3	TSS TC	65% 95%
4	UOD TC	39% 99.8%

Item 3 In-line storage volume.

Segment No.	Hydraulic Capacity	Dry Weather Flow	Internal Storage Capacity
3 (First segment considered)	See note.	See note	0.8 million gallons (assumed for example)

NOTE: Here the user would prepare a schematic of the sewer segment which indicates the routing, interconnections, interceptor capacities, and other features similar to Figure 3-19 on page 3-76 of this manual. The hydraulic capacity of the interceptors can be determined from standard pipe flow equations. Dry weather flow to the combined sewer can be estimated using contributing population times a gallons per capita factor. Internal storage capacity can be determined using a simulator, as described in Section 3.4.4, by considering existing excess hydraulic capacity over the dry weather flow, as in Table 3-8, and using slopes to determine available volumes, or by other methods. For this example, it is assumed that 10 weirs placed at various locations in sewer segment #1 will result in an internal storage of 0.8 million gallons.

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 4 Cost of utilizing available in-line storage from cost information information in Appendix G, or an equivalent method.

Segment No.	Type of Control[1]	Construction Costs	O&M Cost	Total Present Worth Costs[2]
3	10 weirs	(Determine costs	using	EPA or
		other appropriate	guidance)	

Item 5 Sewer segments with deposition problems.

Segment No.	Extent of Problem	Source of Problem[3]
3	① 1500' between manholes	Slope allows solids
	7 and 11	to settle.
	② 1000' between manholes	Slope allows solids
	22 and 25	to settle.

[1] Weir, gate, etc.

[2] Using PRESENT-WORTH METHODOLOGY

[3] Obstructions, slack velocity, etc.

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 6 Impact of sewer flushing on pollutant concentrations in overflows.

Outfall No.	Segment No.	Pollutant Parameter	Concentration Before	Concentration After
3	3	TSS TC	372 mg/L no change	340 mg/l *

*NOTE: Concentration after flushing assumed for example purposes. The impact of sewer flushing will have to be estimated depending on the performance data found in Appendix G. The impact will be highly site-specific and will depend on the design storm, the magnitude of runoff, the extent of flushing, and other factors.

Item 7 Cost of sewer flushing using cost information, in Appendix G, or an equivalent method.

Segment No.	O&M	Total Present-Worth Costs
3	(See Appendix G or other appropriate guidance for estimating sewer flushing costs)	

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 8 Cost of measuring conveyance capability using cost curves in Appendix G or an equivalent method.

i. Polymer injection costs.

Segment No.	Construction Costs	O&M Costs	Total Present-Worth Costs[1]
3	(Determine costs using EPA or other		
	appropriate guidance)		

ii. Increased Pumping capacity at the wastewater treatment plant where influent interceptor is surcharged.

Existing Capacity	Increased Capacity	Construction Cost	O&M Costs	Total Present-Worth Costs[1]
		None		

[1] Using PRESENT-WORTH METHODOLOGY

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 9 Cost of other collection system controls as appropriate, using Appendix G or an equivalent method.

Segment No.	Type of Control	Construction Costs	O&M Costs	Total Present-Worth Costs[1]
		None		

[1] Using PRESENT-WORTH METHODOLOGY

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 10 Collective effect on runoff volumes and loads of all collection system controls found to be feasible.

Segment No.	Control Alternative	% Runoff Controlled	Pollutant Parameter	Load Reduction Achieved
3	In-line storage	0.8 mg		
3	Sewer flushing of two sections		TSS	70 mg/L*
3	Polymer injection	*		*

(this table would be filled out similarly for each of the critical sewer segments and the collective effect of the controls would be added up.)

*NOTE: It will be assumed for this example that enough TSS are to be removed by sewer flushing to reduce the required load reduction from the remaining overflow from 65% to 60%. Polymer injection results in improved conveyance capability which is very difficult to represent quantitatively as a runoff control or load reduction.

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TABLE 6-8 (continued)
COLLECTION SYSTEM CONTROL METHODOLOGY
WORKSHEET

Item 11 Summary of feasible collection system control alternatives.

Collection System Control	Present-Worth Costs
In-line storage	(Costs from previous
Sewer flushing	calculations)
Polymer injection	

Total ______

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Illustrative Example Supplemental Notes

The investigation of storage/treatment options in this example takes into account the internal storage achieved in the collection system. A storage/treatment option is selected for evaluation which involves storage and treatment of the storm overflow at the overflow site. Other options are covered in the discussion.

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TABLE 6-9
STORAGE/TREATMENT METHODOLOGY WORKSHEET

The procedures, calculations, assumptions, and judgments presented in the flowcharts and worksheets are for guidance only, and should not be interpreted as the only approach available (or even as the preferred approach). However, any approaches used should be consistent with EPA Cost Effectiveness Analysis Guidelines and all other EPA, State, and local guidelines and regulations.

Item 1 Sewer System Characteristics.

Outfall No.	Subarea Location[1]	Sewer Segment Type[2]	Type of Control Device[3]
3	Subarea #1 (see map, page 2)	Combined	Regulator (weir)
4	Subarea #2 (see map, page 2)	Storm	None

Note: This information is the same as that in Item 1 of COLLECTION SYSTEM CONTROL METHODOLOGY Illustrative Example worksheet, page 91.

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[1] Locations should be referenced to a map using Outfall and Subarea numbers.
[2] Combined sewer, storm sewer, or unsewered.
[3] Swirl separator or conventional regulator.

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 2 Pollutant Parameters (BOD_5, TSS, TC, P, N).

Segment No.	Pollutant Parameter	Required % Reduction
3	TSS TC	65% 95%
4	TSS TC	39% 99.8%

Note: This information is the same as that in Item 2 of COLLECTION SYSTEM CONTROL METHODOLOGY Illustrative Example Worksheet, page 92.

Item 3 Results of collection system controls.

i. Quantity of design storm runoff volume stored in internal storage of collection system 0.8 mg. (from page 92)

ii. Remaining runoff volumes and load:

Volume: 6 mg.

Flow: 63 mgd. (peak)

Load: ______ mg/l BOD_5

340 mg/l TSS

______ mg/l P

______ mg/l N

______ #/100 ml

Note: Volume and flow can be determined from the overflow hydrograph. Loads are determined using the load assessment techniques in Chapters 2 and 3, modified by control alternatives previously evaluated, e.g., land management or collection system controls

Note: Collection system controls do not satisfy the load reduction requirements; therefore, other feasible control options were investigated. Table 6-10 was used to identify other options. Control option 5 will be evaluated for illustration purposes.

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 4 i. Cost of Regulator, from information in Appendix G, or equivalent method.

Construction Cost $ ____________

O&M Cost $ ____________

Total Present-Worth Cost $ ________________
(using PRESENT-WORTH METHODOLOGY)

Note: Not applicable since swirl separator is to be used.

ii. Cost of swirl separator, using Table 6-11, or equivalent method.

Design flow 50 mgd (assumed for illustrative purposes

Construction Cost $ 30,500 (amortized - see Table 6-11)

O&M Cost $ 9,000

Total Present-Worth Cost $ 419,000
(using PRESENT-WORTH METHODOLOGY)

Item 5 Cost of storage tanks, from Table 6-11, or equivalent method.

Type of storage: Settling ____________ Complete mix ____________

Construction Cost $ ____________

O&M Cost $ ____________

Total Present-Worth Cost $ ________________
(using PRESENT-WORTH METHODOLOGY)

Note: not applicable since treatment is to be provided. Item 6 will be used to determine the least-cost combination of storage and treatment.

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6

i. Design Storm Characteristics.

Intensity ____________ in/hour

Duration ____________ hour

Frequency ____________ /year

Note: The user can select the design storm based on an analysis of local rainfall/runoff characteristics, pollution reduction requirements and other factors, and can develop the inlet hydrograph using an applicable method of his choice. For this illustration, the inlet hydrograph below is assumed.

ii. Inlet hydrograph(s) for design storm (storm runoff entering the sewer system).
Method (Assumed for illustrative purposes.)

Sub-area 1		Sub-area ____		Sub-area ____	
TIME (hr)	FLOW (mgd)	TIME	FLOW	TIME	FLOW
0	0				
0.5	5				
1.0	15				
1.5	40				
2.0	62.5				
2.5	60				
3.0	40				
3.5	21				
4.0	13				
4.5	8				
5.0	4.5				
5.5	2				
6.0	0				

NOTE: Plot hydrographs for each subarea on separate sheets of graph paper.

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

iii. Inflow hydrograph to overflow control structure.

Routing Procedure ____________________

TIME		FLOW
Inlet	Inflow	
1.5	0	40
2	0.5	62.5
2.5	1.0	60
3.0	1.5	40
3.5	2.0	21
4.0	2.5	13
4.5	3.0	8
5.0	3.5	4.5
5.5	4.0	2
6.0	4.5	0

iv. Overflow hydrograph from the control structure.

(Attach rating curve for specific structure)

TIME	FLOW
Same co	ordinates
as shown	above

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Illustrative Example Supplemental Notes

The user may use any applicable routing procedure to develop the inflow hydrograph to the overflow control structure. The routing procedure should take into account the use of in-line storage.

For this illustrative example, the volume retained by in-line storage (0.8 mg) is deleted from the hydrograph (shown by the shaded area on Plot A), and the actual inflow hydrograph begins at a time equal to 1.5 hours. The ordinates shown were taken from the inlet hydrograph plot.

The user can determine the overflow hydrograph using the inflow hydrograph and a rating curve for the particular overflow control device. For this illustrative example the overflow hydrograph is assumed to be the same as the inflow hydrograph to the control structure.

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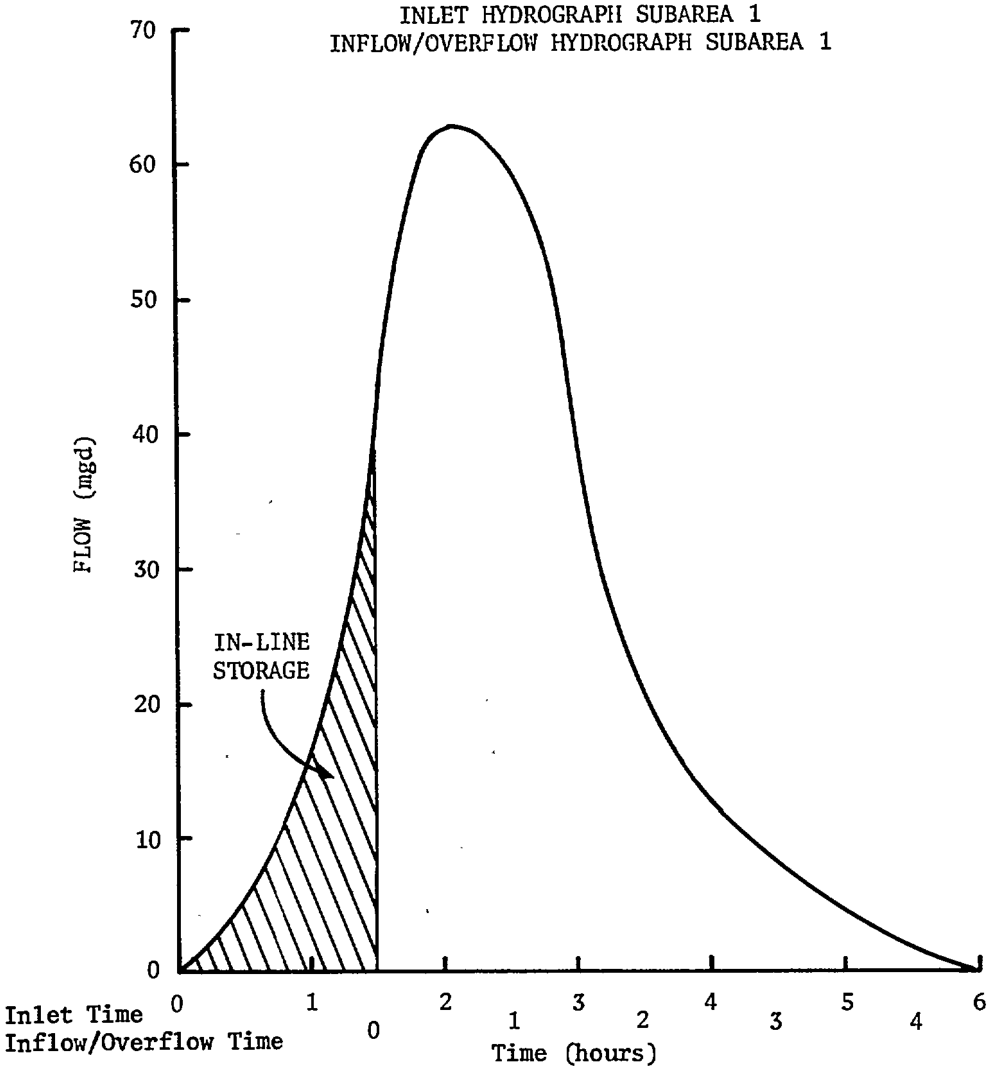
PLOT A

INLET HYDROGRAPH SUBAREA 1
INFLOW/OVERFLOW HYDROGRAPH SUBAREA 1



Inflow/Overflow TIME 0 1 2 3 4

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

Volume = area under hydrograph. For the illustrative example, trapezoidal approximations of areas were used.

v. Mass Curve from Overflow hydrograph.

TIME	INCREMENTAL VOLUME (mg)	CUMULATIVE VOLUME (mg)
0	0	0
0.1	0.18	0.18
0.2	0.22	0.4
0.3	0.24	0.64
0.4	0.26	0.9
0.5	0.28	1.18
0.75	0.69	1.87
1.00	0.67	2.54
1.5	1.17	3.71
2.0	0.61	4.32
2.5	0.35	4.67
3.0	0.22	4.89
4.0	0.2	5.09

NOTE: Plot Mass Curve on separate sheet of standard graph paper.

See Plot B

vi. Storage/Treatment requirements.

Using mass curve shown in Plot B

% MAXIMUM TREATMENT RATE	TREATMENT RATE mgd	STORAGE VOLUME mg
100	62.5	0
75	47	0.8
50	31	1.8
25	16	3.1
0	0	5.1

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PLOT B

MASS CURVE

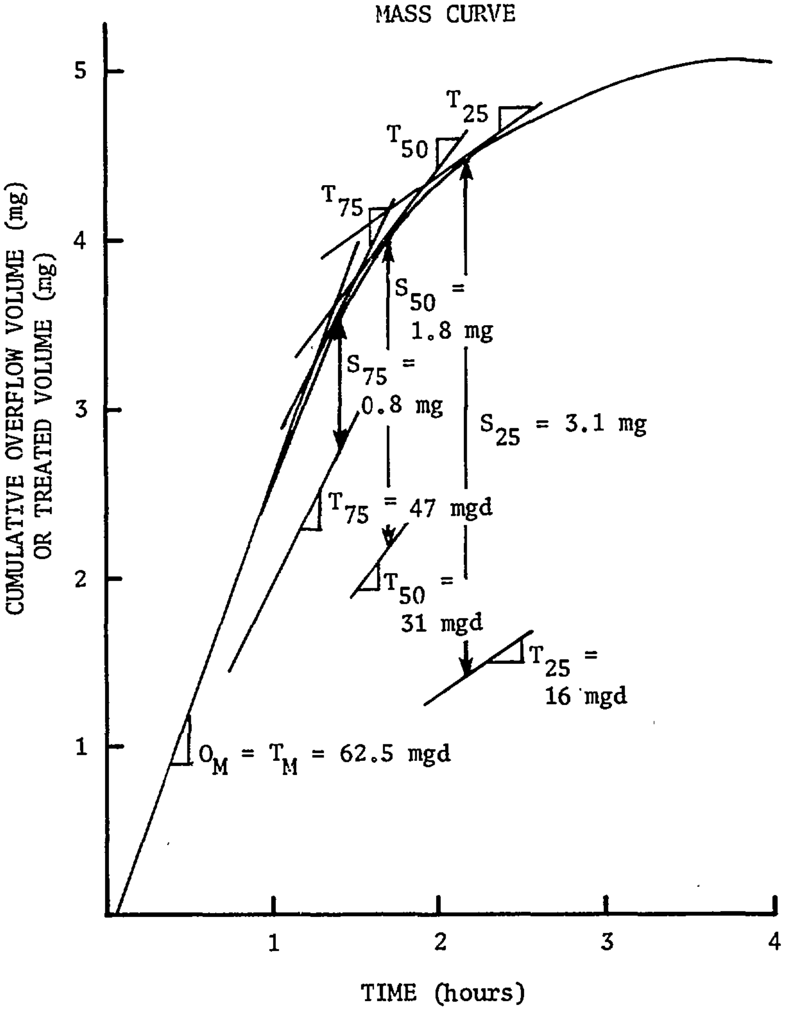

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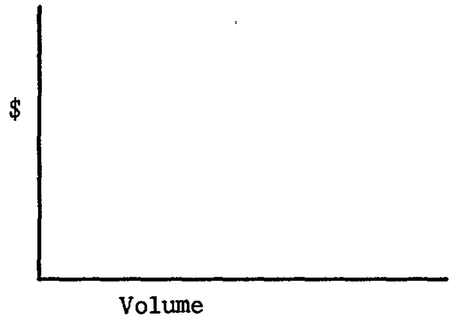
TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

vii. Cost of storage, using cost functions from Table 6-11, or equivalent method, PRESENT-WORTH METHODOLOGY.
Type of storage High Density.

Volume to be Stored	Total Annual Cost ~~Construction~~ Cost	O&M Cost	Present Worth Cost
0	0		0
0.8	40,800		432,000
1.8	91,800		973,000
3.1	158,000		1,676,000
5.1	260,000		2,757,000

NOTE: Plot Volume to be stored and Present Worth Cost.



Present worth costs obtained above are shown in Plot C.

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

viii. Cost of on-site treatment, using cost functions from Table 6-11, or equivalent method, and PRESENT-WORTH METHODOLOGY.

Treatment Unit Microscreens

Treatment Rate	amortized Construction Cost	O&M Cost	Present Worth Cost
62.5	170,000	43,000	2,257,000
47	137,000	34,000	1,812,000
31	100,000	25,000	1,324,000
16	60,000	15,000	795,000

Treatment Unit Dissolved Air Flotation

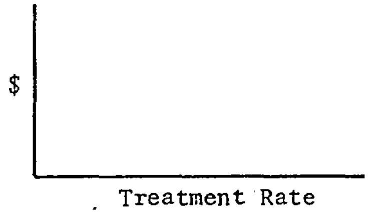
Treatment Rate	Construction Cost	O&M Cost	Present-Worth Cost
62.5	263,000	66,000	3,485,000
47	207,000	52,000	2,744,000
31	146,000	36,000	1,928,000
16	84,000	21,000	1,112,000

Treatment Unit

Treatment Rate	Construction Cost	O&M Cost	Present-Worth Cost

NOTE: Plot a cost curve for each type of treatment unit on the same set of axes.

See Plot C.



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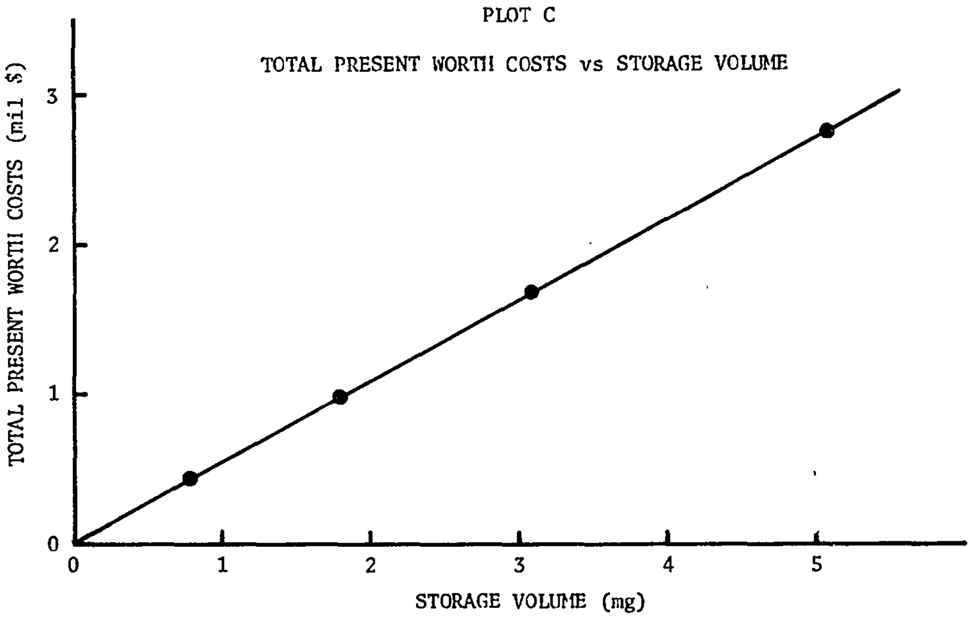
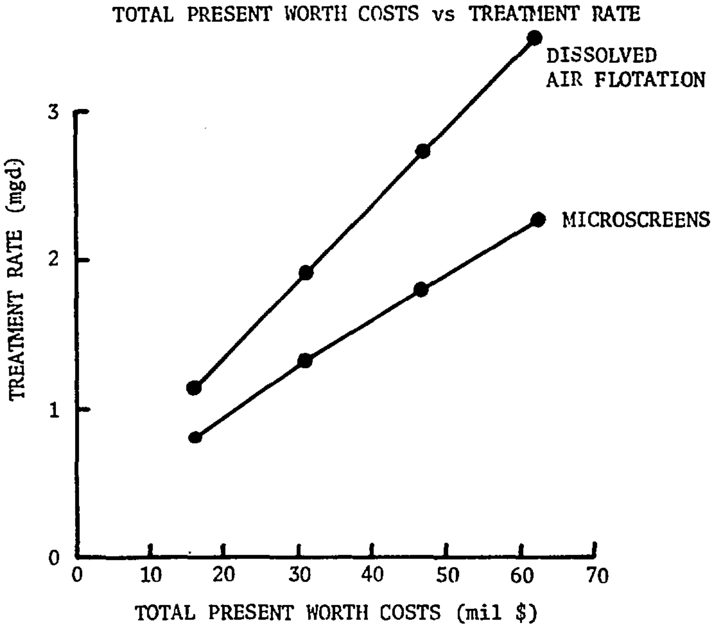


PLOT C

TOTAL PRESENT WORTH COSTS vs STORAGE VOLUME



TOTAL PRESENT WORTH COSTS vs TREATMENT RATE

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

Note: not applicable for this option.

ix. Cost of treatment at the existing plant.

a) Cost of discharging effluent to the interceptor and treating at existing plant.

i. If interceptor is adjacent to overflow control device (as in combined sewer), no cost is associated with discharge to the interceptor.

ii. If interceptor is not adjacent to the overflow control device, the cost to construct a sewer to transport the effluent can be determined using the TRANSPORTATION COST METHODOLOGY. PRESENT WORTH METHODOLOGY:

Flow to be Transported and Treated	Sewer Construction Construction Cost	 O&M Cost	Present Worth Cost

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

Note: not applicable for this option.

b) Costs of upgrading/expanding existing treatment facility using TREATMENT FACILITY METHODOLOGY and PRESENT-WORTH METHODOLOGY.

Treatment Rate	Construction Cost	O&M Cost	Present-Worth Cost

c) Total Present Worth cost of treatment at existing facility.

Treatment Rate	Total Present-Worth Cost of Transportation and Treatment

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 6 - Continued

x. Least-cost combination of storage and treatment.

Treatment Rate mgd	Least-cost Treatment Unit	Present-Worth Cost of Treatment	Storage Volume	Present-Worth Cost of Storage[1]	Total Present-Worth Cost
62.5	Microscreen	2,257,000	0	0	2,257,000
47	Microscreen	1,812,000	0.8	432,000	2,244,000
31	Microscreen	1,324,000	1.8	973,000	2,297,000
16	Microscreen	795,000	3.1	1,680,000	2,475,000
0	—	0	5.1	2,780,000	2,780,000

[1] Table 6-11 was used to generate storage costs. A storage for high density areas (15 people/acre) was selected.

NOTE: Plot total Present-Worth cost of storage and treatment versus treatment rate. See Plot D.

Least-cost combination of storage and treatment.

$ 2,200,000

Note: For an actual case, the user would utilize the above table to locate the approximate area of the minimum point, and then generate more treatment rate/present-worth cost combinations in order to more exactly locate the minimum cost combination. For this example, the minimum point is estimated to occur approximately at 47 mgd.

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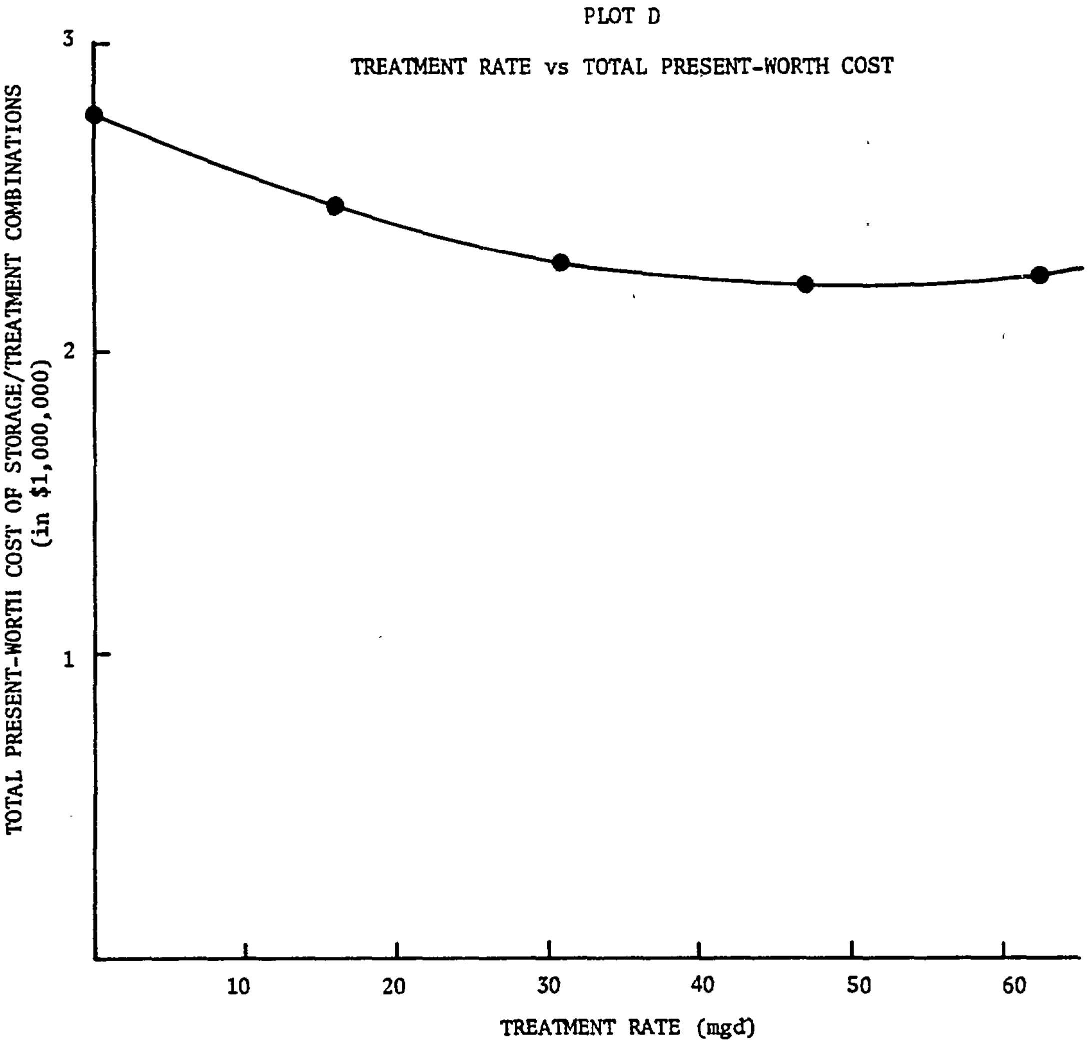

PLOT D

TREATMENT RATE vs TOTAL PRESENT-WORTH COST

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 7

i. Cost of laying pipe to connect new regulator to existing outfall pipe (if significant), or cost of laying a new or larger outfall pipe, from cost curve in Appendix H, Figure H-84.

Construction Cost $ ________

O&M Cost $ ________

not applicable for Option 5, Table 6-10.

Total Present Worth Cost $ ________
(using PRESENT WORTH METHODOLOGY)

ii. Cost of Disinfection (where required) from curve in Appendix H, Figure H-26.

Construction Cost $ 60,000

O&M Cost $ 1,100

Total Present Worth Cost $ 72,000
(using PRESENT WORTH METHODOLOGY)

Design Average Flow = 30 mgd (assumed for this example).

O+M cost is based on 60 days/year operation.

O+M Present worth Cost = (O+M)(Present Worth Factor)
(1,100)(10.594)
= 11,600

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 8 Cost of sludge handling.

not applicable for Option 5, Table 6-10.

i. On-site sludge handling.
  a. Sludge treatment (using cost curves in Appendix H).
    1) Organic sludges

      Lime stabilization (Figure H-79)
        Construction Cost $ ____________
        O&M Cost $ ____________

      Vacuum Filtration (Figure H-68)
        Construction Cost $ ____________
        O&M Cost $ ____________

    2) Inorganic sludges

      Vacuum Filtration (Figure H-69)
        Construction Cost $ ____________
        O&M Cost $ ____________

    3) Subtotal $ ____________

  b. Sludge transport (using cost curves in Appendix H, Figures H-86 through H-90).

    Method: ____________________
        Construction Cost $ ____________
        O&M Cost $ ____________

  c. Sludge disposal (using cost curves in Appendix H, Figures H-81 through H-83).

    Method: ____________________
        Construction Cost $ ____________
        O&M Cost $ ____________

  d. Total cost for on-site sludge handling.

        Construction Cost $ ____________
        O&M Cost $ ____________

  e. Total Present Worth Cost $ ____________________
    (using PRESENT WORTH METHODOLOGY)

ii. Sludge handling at existing wastewater treatment facility.
  a. Sludge transport to existing facility

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 8 - Continued

1) If sewer is storm sewer, determine cost to construct sewer to connect with sanitary sewer interceptor to treatment plant, using TRANSPORTATION COST METHODOLOGY.

Construction Cost $ ____________
O&M Cost $ ____________

2) If sewer is combined, existing interceptor capacity should be sufficient to transport sludges to the existing wastewater treatment plant.

b. Sludge treatment.

Determine if there is capacity available in existing sludge handling facilities to accept additional sludge volumes from the treatment of storm overflows. If not, determine cost to provide additional sludge handling capacity at the existing facility, using TREATMENT FACILITY METHODOLOGY.

Construction Cost $ ____________
O&M Cost $ ____________

c. Sludge disposal (using cost curves in Appendix H, Figures H-81, 82, or 83).

Method: ____________________
Construction Cost $ ____________
O&M Cost $ ____________

d. Total cost for sludge handling at existing wastewater treatment facilities.

Construction Cost $ ____________
O&M Cost $ ____________

e. Total Present Worth Cost $ ____________________
(using PRESENT WORTH METHODOLOGY)

Note: It is assumed for this example that there is existing capacity in the sludge handling facilities at the existing plant, and that no additional costs are incurred in transporting, treating, or disposing of the sludge from the sewer overflow treatment units (swirl separator and microscreen). In an actual case, the user might want to assign a cost for handling of this sludge, even though there is existing capacity at the plant, based on the relative proportion of total sludge volume which it represents.

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 9 Total Present Worth Cost.

Sewer Segment #1 Sewer Type Combined

i. For each storage/treatment option, record the Present-Worth costs of each component determined using Items 4-8.

ii. Determine the total Present Worth cost of each option.

Present Worth Cost (in $1,000)

Option #	Regulator	Swirl	Storage w/ Settling	Storage w/ mixing	On-site WW Treatment	Treatment at Existing Plant	Interceptor Construction	Disinfection	On-site Sludge Handling	Sludge Handling at Existing Plant	Total Present-Worth Cost
5	-	419	-	432	1,812	-	-	71	-	No cost	2,734

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TABLE 6-9 (continued)
STORAGE/TREATMENT METHODOLOGY
WORKSHEET

Item 10 Regionalization of storage/treatment components, using REGIONALIZATION METHODOLOGY.

Components regionalized:

Regional Facility:

Construction Cost $ ____________

O&M Cost $ ____________

Present Worth Cost $ ____________
(using PRESENT WORTH METHODOLOGY)

Not investigated for this example.

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Illustrative Example Supplemental Notes

The evaluation of control alternatives for wet weather sources also includes consideration of reuse. This utilizes the same component methodology as for dry weather sources; therefore, the worksheets are not included in this example. The differences in the evaluation procedure will typically include consideration of storage requirements and reuser reliability requirements.

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Illustrative Example Supplemental Notes

The potential for Impact Area Modifications to reduce the undesirable effects of wet weather sources is considered at this point in the load reduction strategy evaluation. The use of In-stream Reaeration and Low-Flow Augmentation are not relevant to the control of a wet weather source. However, Discharge Relocation is potentially viable. This evaluation would be identical to that for a dry weather source; therefore, the worksheets are not included here.

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Illustrative Example Supplemental Notes

The planner would generally continue the analysis of Load Reduction Strategy 9 at this point by considering for Source 4 the various control alternatives just described for Source 3. The user is referred to the previous worksheets for illustration of the appropriate methodologies.

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Illustrative Example Supplemental Notes

Following the evaluation of control alternatives for all sources identified in Load Reduction Strategy 9, the user would consider the effect of regionalized facilities on total project cost. The worksheets for this evaluation are not included for this example because all of the component methodologies have been previously utilized.

The evaluation of regional facilities for this example would include wet and dry weather sources. In addition, the residual disposal portion of the REGIONALIZATION METHODOLOGY would be useful in identifying effective alternatives.

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Illustrative Example Supplemental Notes

At this point, the user will have completed the development and evaluation of feasible control alternatives for the load reduction strategies of interest. The total monetary cost of each control alternative will have been developed and expressed as a Present-Worth cost. Also, a relative information reliability will have been determined for each control alternative.

The user is now ready to evaluate criteria other than monetary cost, such as implementability and environmental, social, and economic costs, in order to determine which load reduction strategy has the least total cost to society. This part of the evaluation leading to final selection is not covered by this manual. However, the monetary costs calculated using this manual are a major input to that determination.

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6.6 References

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