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Research and Development

Urban Stormwater Management Workshop Proceedings

Edison NJ
December 1, 1977

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URBAN STORMWATER MANAGEMENT WORKSHOP PROCEEDINGS

Edison, New Jersey
December 1, 1977

Edited By

Richard Field
Storm and Combined Sewer Section
Wastewater Research Division
Municipal Environmental Research Laboratory (Cincinnati)
Edison, New Jersey 08817

Based on Project Nos. 68-03-2617, R802411, R805238, R804578, S804432

Project Officers

Richard Field
Anthony N. Tafuri   Hugh E. Masters
Chi-Yuan Fan   Richard P. Traver
Storm and Combined Sewer Section
Wastewater Research Division
Municipal Environmental Research Laboratory (Cincinnati)
Edison, New Jersey 08817

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

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic efforts of pollution. This publication is one of the products of that research; a most vital communication link between the researcher and the user community.

The workshop proceedings contained herein include discussions of the urban stormwater management technology manual update; new version of the Storm Water Management Model (SWMM); characterization of pollutant accumulation rates in urban areas; and application of nonstructural control of urban runoff pollution.

Francis T. Mayo
Director
Municipal Environmental Research
Laboratory

ABSTRACT

The workshop on urban stormwater management technology was held on December 1, 1977 at the offices of the USEPA in Edison, New Jersey. The purpose of the workshop was to exchange and disseminate the most up-to-date research results and technical information from projects sponsored under the USEPA Urban Runoff Control Research Development and Demonstration Program. The proceedings contained herein represent the contributions from participating lecturers and include the following topics:

a. Urban stormwater management and technology manual (update),

b. Comprehensive planning for control of urban storm runoff and combined sewer overflows,

c. Low cost-effective alternative and comparative analysis from 208 areawide assessment study on combined sewer overflow and urban stormwater pollution control,

d. Statistical characterization of runoff loading rates and cost functions of control measures,

e. Dry weather pollutant deposition in sewerage systems and associated first flush combined sewer overflow pollution control by dry weather sewer flushing,

f. Nonpoint pollution abatement through improved street cleaning practices.

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LIST OF WORKSHOP PARTICIPANTS

LECTURING PARTICIPANTS

Robert Berwick, Ph.D., Meta Systems, Inc., 10 Holworthy Street, Cambridge, Massachusetts 02138

Richard Field, Chief, Storm and Combined Sewer Section, Wastewater Research Division, Municipal Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Edison, New Jersey 08817

E. John Finnemore, Ph.D., Metcalf & Eddy, Inc., 1029 Corporation Way, Palo Alto, California 94303

James P. Heaney, Ph.D., Associate Professor, University of Florida, Department of Environmental Engineering, A.P. Black Hall, Gainesville, Florida 32611

Wayne C. Huber, Ph.D., Associate Professor, University of Florida, Department of Environmental Engineering, A.P. Black Hall, Gainesville, Florida 32611

William G. Lynard, Metcalf & Eddy, Inc., 1029 Corporation Way, Palo Alto, California 94303

Daniel F. Luecke, Ph.D., Meta Systems, Inc., 10 Holworthy Street, Cambridge, Massachusetts 02138

William C. Pisano, Ph.D., Technical Director, Energy & Environmental Analysis, Inc., 257 Vassar Street, Cambridge, Massachusetts 02139

Robert E. Pitt, Environmental Engineer, Woodward-Clyde Consultants, 3 Embarcadero Center, Suite 700, San Francisco, California 94111

PARTICIPANTS

D. Dean Adrian, Ph.D.
Director of Environmental Engineering Program
Department of Civil Engineering
University of Massachusetts
Amherst, Massachusetts 01002

Ralph W. Christensen
Office of Great Lakes Coordinator
Region V
U.S. Environmental Protection Agency
230 South Dearborn Street
Chicago, Illinois 60604

Francis Condon
Program Manager
Waste Management Division
Office of Air, Land and Water Use
Office of Research & Development
U.S. Environmental Protection Agency
Washington, D.C. 20460

Frank Drehwing
Vice President
O'Brien & Gere Engineers, Inc.
1304 Buckley Road
Syracuse, New York 13201

Chi-Yuan Fan
Staff Engineer
Storm & Combined Sewer Section
Wastewater Research Division
Municipal Environmental Research Laboratory
U.S. Environmental Protection Agency
Edison, New Jersey 08817

Shinichi Hiromoto
Sewage Works Bureau of Kyoto City
Planning Section
12 Higashi Sanno
Higashi-Kujo Minami-Ku
Kyoto, Japan

George James
Clinton Bogart Associates
2083 Central Avenue
Fort Lee, New Jersey 07024

Thomas K. Jewell
Research Assistant
Department of Civil Engineering
University of Massachusetts
Amherst, Massachusetts 01002

Dennis Lai, Ph.D.
Clinton Bogart Associates
2083 Central Avenue
Fort Lee, New Jersey 07024

Hugh E. Masters
Staff Engineer
Storm & Combined Sewer Section
Wastewater Research Division
Municipal Environmental Research Laboratory
U.S. Environmental Protection Agency
Edison, New Jersey 08817

Cornelius B. Murphy, Jr., Ph.D.
O'Brien & Gere Engineers, Inc.
1304 Buckley Road
Syracuse, New York 13201

Walter Su
Clinton Bogart Associates
2083 Central Avenue
Fort Lee, New Jersey 07024

Anthony N. Tafuri
Staff Engineer
Storm & Combined Sewer Section
Wastewater Research Division
Municipal Environmental Research Laboratory
U.S. Environmental Protection Agency
Edison, New Jersey 08817

Richard P. Traver
Staff Engineer
Storm & Combined Sewer Section
Wastewater Research Division
Municipal Environmental Research Laboratory
U.S. Environmental Protection Agency
Edison, New Jersey 08817

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I would like to thank the lecturing participants and the other workshop participants. Special appreciation is to be given to Chi-Yuan Fan of the Storm and Combined Sewer Section for his devoted effort in planning and executing the workshop. Finally, I acknowledge the help of our secretaries, Elizabeth H. Mohary and Marily A. Nelson, for their valuable assistance in organizing the workshop and related material.

Richard Field

URBAN STORMWATER MANAGEMENT PROGRAM WORKSHOP

Introduction

by

Richard Field*

Welcome to the Second Technical Interproject Workshop in the Urban Stormwater Management Program.

The principal purpose of these workshops is for an exchange of project information and data, and to establish liaisons between principal investigators on USEPA sponsored research projects, with similar objectives, for the mutual benefit of their respective investigations and USEPA personnel. The workshop serves as an effective means for technology transfer and provides impetus in surfacing the most up-to-date information in the field.

There is a crying need for (1) cost-performance data for full-scale nonstructural urban stormwater pollution control measures or "best management practices" (BMP's), and (2) for the appropriate planning methodology to optimumly integrate nonstructural with structural control on an area-wide basis. A third very important need is for collection and better evaluation of pollutant emission/water quality data. We have heard the cry for this information from 208 agencies, USEPA and many others. Today's session will be aimed at broadening the technology for these needs.

The project titles and names of the investigating agencies involved in this workshop are indicated in Table 1-1. Guest speakers will represent these projects as will be described a little later. The common project objectives and their interrelationships are presented in Table 1-2. This is a simple matrix to follow. As you can see, and may have anticipated, all the projects relate to the basic objectives of this workshop.

The first speakers will be Dr. John Finnemore and Bill Lynard, Project Managers from Metcalf & Eddy, Inc., Palo Alto, California. They will discuss results from the completed "Urban Stormwater Management and Technology Update" project, the plan of the new project on cost-effectiveness and impacts assessment, and the operational program of the San Francisco stormwater treatment pilot plant. Their presentation should be of great interest. We have just received the published update SOTA report which is appropriate to hand out at this workshop.

*Chief, Storm & Combined Sewer Section, Municipal Environmental Research Laboratory (Cincinnati), U.S. Environmental Protection Agency, Edison, New Jersey 08817

Next, Drs. James Heaney and Wayne Huber will present the new features of the SWMM program. These include a revised storage/treatment block to predict dry and wet-weather flow treatment performance and sludge handling facility requirements and costs. As you all know, the University of Florida was involved with the original SWMM development and has been retained for its improvement since 1974. In addition, they also developed "desk-top" analysis methods for preliminary screening procedures and comparative evaluation of storage-treatment and other management practices. The desk-top procedures are in two USEPA reports that have been used by 208 planning agencies.

The third speaker will be Dr. William Pisano who will discuss his involvement in area-wide wastewater management planning projects in Massachusetts.

The first lecture after lunch is on the Meta Systems' project. Drs. Dan Luecke and Bob Berwick will present the preliminary results of their study on urban pollutant loadings, runoff control options and related costs, and storm event analysis. The basic objective of this project is to develop and evaluate pollutant production, flowrate and costs for stormwater pollution control in new residential developments. The project began July 1977.

Afterward, Dr. William Pisano will present his second lecture on his study on dry-weather pollutant deposition in sewerage systems and upstream source control by sewer flushing. The project involved a vast field sampling and analysis program for sewer deposition and flushing, and resulted in published methods for predicting the total daily mass of pollutant deposition accumulations.

The last speaker will be Mr. Robert Pitt of Woodward-Clyde Consultants' San Francisco office. He will present interesting results of his work in the City of San Jose on a demonstration project which includes full-scale street cleaning equipment tests for runoff pollution control, analysis of particulate routing and mass balances in storm drainage, and economic evaluation of control alternatives.

Each lecture will be one hour long, which includes 45 minutes for presentation and 15 minutes for questions and answers. We will collect the lunch sandwich selections now before we proceed with the lectures. Lunch will be served from 12:15 pm to 1:00 pm in this room.

Thank you and have a fruitful meeting.

Table 1-1. Project Title and Investigating Agency

Investigating Agency	Project Title and Number
1. Metcalf & Eddy	"Assessment of the Cost-Effectiveness and Impacts of Completed and Operating Stormwater and Combined Sewer Overflow Remedial Systems for Future Guidance" (68-03-2617)
2. University of Florida	"Comprehensive Planning for Control of Urban Storm Runoff and Combined Sewer Overflow" (R-802411)
3. Meta Systems	"New Residential Developments and the Quantity and Quality of Runoff" (R-805238)
4. Northeastern University/EEA	"Characterization of Solids Behavior in, and Variability Testing of Selected Flushing Techniques for Combined Sewer Systems" (R-804578)
5. City of San Jose/ Woodward-Clyde	"Demonstration of Non-Point Pollution Abatement Through Improved Street Cleaning Practices" (S-804432)
6. Monroe County/ O'Brien & Gere	"Combined Sewer Overflow Abatement Program - Rochester, New York" (Y-005141)

Table 1-2. Common Project Objectives

Project	Meth for Integrated Struct/Nonstruct Control Optimization	Nonstructural Control/BMP Performance	Collect/ Evaluate Field Data
1. (M&E)	X	X	X
2. (UF)	X	X	X
3. (MS)	X	X	X
4. (EEA)	X	X	X
5. (W-C)	X	X	X
6. (O&G)	X	X	X

STATE-OF-THE-ART OF URBAN STORMWATER MANAGEMENT

By

William G. Lynard, P.E.* and E.John Finnemore, P.E.*

Within the last decade, a concentrated effort has been made by the EPA, local, and private agencies to investigate the effects and impacts of stormwater and combined sewer overflows on the receiving water environment. Over this period, a greater awareness of the adverse stormwater contributions to the aggregate quality of the surface waters of the nation have come to the forefront; and as the goals of clean water and restoration rise, and as increasingly effective coutermeasure implementation is achieved, the role of noncontinuous stormwater discharges has become increasingly important.

A series of state-of-the-art documents, developed by Metcalf & Eddy, to provide current assessments, problem identification, characterization, and control/countermeasure implementation practices have been published by the EPA. In 1974 "Urban Stormwater Management and Technology: An Assessment," EPA-670/2-74-040, presented a comprehensive investigation and assessment of promising, completed, and ongoing stormwater projects, representative of the state-of-the-art in abatement theory and technology. The second of the series, "Urban Stormwater Management and Technology: Update and Users' Guide," EPA-600/8-77-014 (just released), is the subject of part of this paper, and is a continuation and reexamination of the state-of-the-art of storm and combined sewer overflow technology. Recommendations from this study have identified the need for more detailed investigation of the most promising structural control facilities and of the state of technology for source control mitigation practices, termed "Best Management Practices" (BMPs)--the subject of the current study and third in the series of technology assessments.

To complement and put into practice the results of these studies, a pilot plant testing program has been initiated as a part of a wet-weather Step 1 facilities plan for the City and County of San Francisco. This project features the most current structural mitigation and management concepts developed for the control of combined sewer overflows.

URBAN STORMWATER MANAGEMENT AND TECHNOLOGY: UPDATE AND USERS' GUIDE

The unquestioned need for solutions to urban runoff pollution have spawned a major research and development effort resulting in the evolution and

*Project Engineer and Project Manager, respectively, Metcalf & Eddy, Inc. Western Regional Office, Palo Alto, California.

application of a new technology which emphasizes time and spatial effects and total system consciousness. Solutions are found not only in improved hardware and process operations, but even more so in the stressing of management practices that limit the spread of the problem and attack it at its source. Assessment of the urban runoff problem is referenced to the developing national data base, then localized through site selective monitoring and analysis, and is quantified as to potential source and magnitude using modeling techniques or simplified desktop procedures.

Control of urban runoff pollution is separable into two categories of countermeasure implementation: nonstructural or low-structural source controls/BMPs, and structural alternatives. BMPs focus on source abatement, whereas, structural alternatives roughly parallel conventional wastewater treatment practices of end-of-pipe correction.

Approach to Stormwater Management

The basic approach concept may be viewed as a four step process: (1) quantifying the need, (2) selective field monitoring, (3) cost-effectiveness assessment, and (4) impact simulations. However, surface runoff generated problems and appropriate mitigation measures are difficult to assess because:

- The events are irregular and unpredictable
- The impacts are likely to be highly time and location variable
- Other discharges or conditions tend to mask actual results
- Relatively little usable local data are available and new data are extremely time consuming and costly to obtain
- Mitigation measures are largely conceptual and effectiveness is ill defined

User assistance tools and planning guides are available where gross assessments of relative loads, sources, and their impact on water quality are required. User assistance tools range from desktop procedures to highly complex digital computer simulations. These tools have been characterized into four categories and are described in Table 2-1.

Among the user planning guides published since January 1974, which are designed to aid managers, three have been noted for their fuller treatment of the five components summarized in Table 2-2.

Characteristics of storm flow pollutants are also of particular interest to the designer-manager implementing urban runoff mitigation measures. These include: (1) source of pollutants, (2) discharge loadings, (3) process residuals, and (4) receiving water impacts. A summary of typical values of stormwater quality parameters is shown in Table 2-3.

Table 2-1. Levels of Stormwater Management Tools

Analysis level	Model type	Model complexity	Purpose of model	Model characteristics
I	Desktop	Low to medium	Problem assessment, preliminary planning, alternative screening.	No computers. Equations, nomographs based on statistical analyses of many years of records.
II	Continuous simulation	Low to medium	Problem assessment, planning, preliminary sizing of facilities (particularly storage), alternative screening. Assess long-term impacts of designs.	Program of few hundred to few thousand statements. Uses many years of rainfall records with daily time steps, or worst 2 years with hourly time steps. May include flow routing and continuous receiving water analysis.
III	Single event simulation	Medium to high	Analysis for design, detailed planning	Program to over 10,000 statements. Higher modeling precision, from rainfall through sewers, possibly to receiving waters. Short-time steps and simulation times. Fewer alternatives to be evaluated.
IV	Operational	Medium	Real-time coverage of sewerage systems	Uses telemetered rainfall data and feedback from sewer system sensors to continually make short-term predictions of system responses, and so produce control decisions during storms.

Table 2-2. Summary of Principal Planning Guides

Title	SWMM: Level I Preliminary Screening Procedures	Water Quality Management Planning for Urban Runoff	Areawide Assessment Procedures Manual
Prepared by	University of Florida	URS Research Company	EPA Municipal Environmental Research Laboratory
Release date	October 1976	December 1974	July 1976
Reference No.	1	2	3
Complexity level(s)	I-low	II-medium	III-low to high
Coverages			
• Discharge quality and quantity	Yes	Yes	Yes
• Control alternatives	Yes	General discussions, only	Yes
• Receiving water impacts	No	No	Yes
• Control costs and benefits	Yes	No	Not clear
• Example applications	Partial	Yes	Yes

Table 2-3. Comparison of Typical Values for Urban Storm Flow Discharges[a]

	TSS	VSS	BOD	COD	Kjeldahl nitrogen	Total nitrogen	PO_4-P	OPO_4-P	Lead	Fecal coliforms
Background levels	5-100	...	0.5-3	20	....	0.05-0.5[b]	0.01-0.2[c]	....	<0.1	....
Stormwater runoff	415	90	20	115	1.4	3.1	0.6	0.4	0.35	14,500
Combined sewer overflow	370	140	115	375	3.8	9.1	1.9	1.0	0.37	670,000
Sanitary sewage	200	150	375	500	40	40	10	7	0.17	2,500,000-- 50,000,000

a. All values mg/L except fecal coliforms which are organisms/100 mL.

b. NO_3 as N.

c. Total phosphorus as P.

Often overlooked in countermeasure planning is the impact of residual sludge/solids from stormwater treatment processes. The relatively high loadings of highly inorganic solids may cause major treatment and disposal problems, especially if solids are returned to dry-weather treatment facilities for processing.

Stormwater and Combined Sewer Overflow Control Measures

Best Management Practices. Nonstructural and low-structural source controls offer considerable promise as the first line of defense to control urban runoff pollution. For developing areas, BMPs are implemented through planning, legislation, and enforcement with goals of maximizing detention-percolation, avoidance of over-development or land misuse, and minimizing impacts of construction activities. For developed areas, sound maintenance and operation practices are required for (1) litter and chemical use control, (2) street cleaning and repair, (3) catchbasin and collection system maintenance, (4) runoff flow controls, and (5) public support and involvement. BMPs have decided benefits over structural alternatives including lower cost, earlier results, and an improved and cleaner neighborhood environment. The greatest difficulty, however, is that the action-impact relationships are almost totally unquantified.

In planning, the concept of preventing and reducing the source of stormwater pollution best applies to developing urban areas, for these are areas where man's encroachment is yet minimal, or at least controllable, and drainage essentially conforms to natural patterns and levels. Such lands, in consequence, offer the greatest flexibility of approach in preventing pollution. What is required, therefore, is to manage development in such a way that a runoff regime may be retained close to natural levels, thereby minimizing the need for expensive downstream structural controls. Effective land use planning can be used to control the type and mix of land activities to meet water quality standards. Land use planning elements include:

- Utilization of greenways and detention ponds
- Utilization of pervious areas for recharge

- Avoidance of steep slopes for development
- Maintenance of maximum land area in a natural undisturbed state
- Prohibiting development on floodplains
- Utilization of porous pavements where applicable
- Utilization of natural drainage features

Construction controls such as minimizing the area and duration of exposure, protecting the soil with mulch and vegetative cover, increasing infiltration rates, and construction of temporary storage basins or protective dikes to limit storm runoff can significantly reduce receiving water impacts caused by erosion.

Proper maintenance and cleanliness of the entire urban area can have a significant impact on the quality of pollutants washed from an area by stormwater. Cleanliness of an urban area starts with control of litter, debris, and agricultural chemicals, such as pesticides and fertilizers. Regular street repair and sweeping can further minimize the pollutants picked up in stormwater runoff. Proper use and maintenance of both catchbasins and the collection system can improve control of pollutants by directing them to treatment or disposal.

Program success is dependent on legislation or ordinances, to force or encourage conformance with the intended BMP, and a concerted effort to monitor compliance and educate not only those who will bear the responsibility of regulation, but the public as well. Legislation has been implemented successfully in several communities for surface runoff control as summarized in Table 2-4.

Structural Alternatives. Structural alternatives involve storage (volume sensitive) and treatment (rate sensitive) options and balances. A substantial arsenal of devices has been developed through research and demonstration projects and much is being learned with respect to their feasibility, efficiencies, costs, and problems. Cost-effective applications of structural alternatives may be implemented through multiprocess or integrated treatment installations, or both. Multiprocess operations are characterized by the optimization of storage and treatment to control both quantity and quality. Optimization can also be accomplished by integration of a wet-weather treatment process with a dry-weather treatment facility where either excess dry-weather capacity is utilized for stormwater treatment or where the wet-weather facility provides an added measure of control for the dry-weather treatment operation.

Storage facilities are frequently used to attenuate peak flows, thereby reducing the size of facilities required for further treatment. Storage, however, with the resulting sedimentation that occurs due to increased detention times, can also be considered a treatment process. Storage can be implemented through (1) inline storage, where the unused volume in interceptor and trunk sewers is used to store runoff; and (2) offline storage, where structures independent of the interceptor/trunk sewer system are used to store runoff.

Table 2-4. Summary of Legislative Programs

Location	Description of legislation
Denver Urban Renewal Authority	Requires private developers to pond rainfall on rooftops and in plazas of all new and renovated construction. The design criteria for plazas require a runoff rate of 1 in./hr and a water depth of 0.75 in. during the 10 year rain. The values for rooftops are 0.5 in./hr and a depth of 1 in. for the 10 year storm or 3 in. during a 100 year rain.
Naperville, Illinois	Plumbing, sewer, and water ordinance requiring that runoff release rate be regulated by the safe capacity of the receiving water, but no more than 0.15 in./hr. Storage must be designed for the 100 year storm. The ordinance is applicable to all new subdivisions and compliance is required for approval of development permits.
Joliet, Illinois	Ordinance similar to that of Naperville. Requires runoff to meet a variety of criteria: (1) runoff rate shall not exceed historic values, (2) allowable runoff rates are prorated on the basis of stream capacity, and (3) runoff rate shall not exceed that of 2 year storm with a runoff coefficient of 0.3 unless facilities can handle the flow. The ordinance is enforced for 10 acre residential areas and 5 acre nonresidential developments through the issuance of building permits.
Albuquerque Metropolitan Arroyo Flood Control Authority	Requires stormwater detention for all new developments such that downstream drainage facility capacity is not exceeded or the rate of runoff does not exceed the natural rate of flow. Compliance is required for building permits and subdivision plat approval. In addition, a developer not in compliance can be sued as creating a public nuisance.
Arvada, Colorado	Requires detention for runoff greater than predevelopment rates for new construction. If a developer chooses not to provide the detention he is assessed a one time fee that reflects the cost the city will pay to develop a drainage system. If detention is provided, no fee is assessed.
Boulder, Colorado	Monthly drainage fee that is assessed against all property in the city on the basis of surface area and runoff coefficient. Efforts to retain runoff onsite will result in lower monthly charges.
Metropolitan Sanitary District of Greater Chicago	Requires provision for stormwater retention before granting sewer connection permits to new developments. The maximum release rate is computed by the Rational Formula with a 3 year rain and a coefficient of 0.15. Storage must be designed for the 100 year storm.
Montgomery County, Maryland	The State of Maryland has classified sediment as a pollutant under its Water Pollution Control Act and Montgomery County's program is an example of the result. The recommendations of the SCS on erosion control must be met to obtain clearing and grading permits in the county. Detention ponds are part of the requirements for approval.
Fairfax County, Virginia	The county has a history of runoff control similar to that of Montgomery County. Erosion and sediment control has been mandated during construction since the late 1960s. Temporary detention ponds were used at most sites and permanent detention must be evaluated for all new developers.
Springfield, Illinois	Sewer ordinance for combined sewer areas that has decreased runoff by a successful campaign to disconnect sewer downspots from the sewer system.

in. x 2.54 = cm
acre x 0.405 = ha

An inline, computer controlled storage system in Seattle [4] has effectively demonstrated the use of such a system, under varying degrees of control, for reducing overflow volumes. The increased storage effectiveness as a result of increased system control is shown in Figure 2-1.

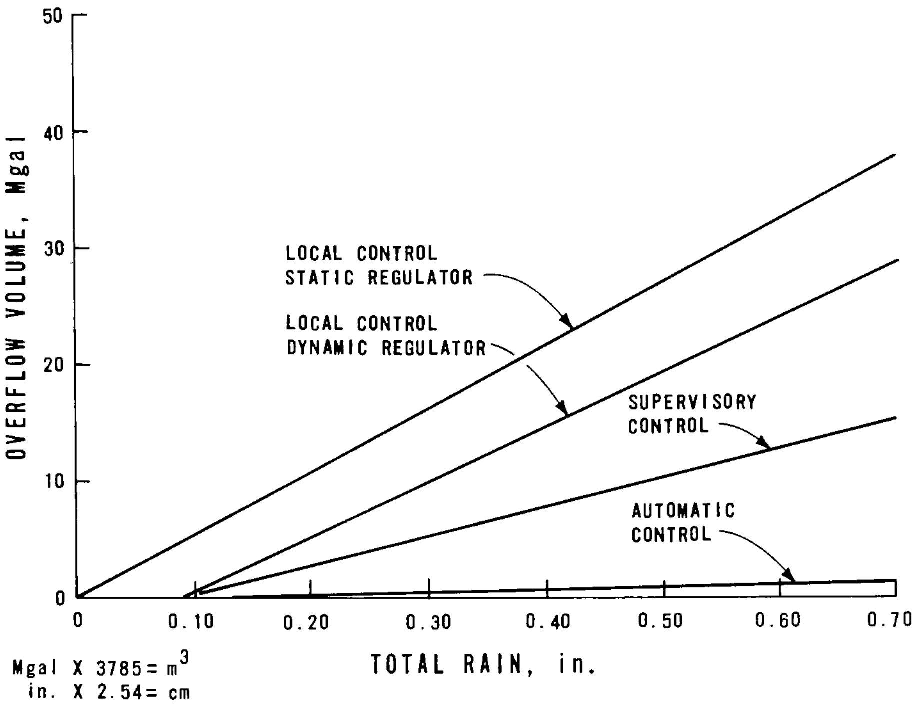


Figure 2-1. Inline storage effectiveness regression lines for each mode of control, Seattle, WA.

Both physical and biological treatment systems have been demonstrated throughout the United States either with pilot scale or full-scale prototype facilities. Of these processes, physical treatment systems have demonstrated best the capability of controlling high and variable influent concentrations and flowrates, and operate independently of other treatment facilities. A comparison of typical ranges of pollutant removal and average construction costs for several physical treatment alternatives is shown in Table 2-5.

Table 2-5. Comparison of Physical Treatment Systems

	Percent reduction						
Physical unit process	Suspended solids	BOD_5	COD	Settleable solids	Total phosphorus	Total Kjeldahl nitrogen	Average cost, \$/Mgal d[a]
Sedimentation							
Without chemicals	20-60	30	34	90-95	20	38	23,000
Chemically assisted	68	68	45	--	--	--	23,000
Swirl concentrator	40-60	25-60	--	50-90	--	--	4,500
Screening	10-90	10-50	22	10-95	13	16	19,000
Dissolved air flotation[b]	45-85	30-80	55	93[c]	55	35	34,000
High rate filtration[d]	50-80	20-55	40	55-95	50	21	58,000

a. ENR 2000.

b. Process efficiencies include both prescreening and dissolved air flotation with chemical addition.

c. From pilot plant analysis.

d. Includes chemical addition.

The range of removals by physical processes is influenced by the quality of the influent storm flow, the specific type of process equipment used, and type and amount of chemical additives used (polyelectrolytes and chemical coagulants).

Portions of the work on which this presentation is based were performed pursuant to Contract No. 68-03-2228 with the Environmental Protection Agency.

CURRENT STORMWATER PROJECTS

Stormwater and Combined Sewer Overflow Assessments

Experience in stormwater management to date has shown that there are no absolutes with respect to design and performance criteria. Generalizations to provide national coverage on applicability greatly overextend a very limited data and experience base. Given the broad, comparative information on control measures in two EPA state-of-the-art reports, planners and designers next will need more in-depth evaluations (performance, cost effectiveness, operating problems, environmental and social impacts) of the most promising alternatives, as demonstrated in real, problem solving applications.

A project has been initiated with the EPA which will be focused to provide summaries of essential data, professional judgments as to applicability, and recommendations as to the most promising approach methodologies. Recognizing that the technology base and demands are continuously changing, the primary goal of this project will be to improve and accelerate the transfer of the most promising technology to the potential user. The project will therefore be selective in its content.

Objectives. The objectives of this project are to evaluate, for future guidance, the successes of the most promising completed and operating urban stormwater pollution control measures in fulfilling their problem solving objectives, including cost effectiveness and environmental and social impacts, and to give priority to advancing our presently lagging knowledge of nonstructural and low-structural measures (BMPs).

Needs. Estimated nationwide costs for treatment and control of combined sewage and urban stormwater have been estimated in the $10 billions [5, 6, 7]. However, these costs and studies were based on simplified assumptions. To establish better figures and estimates, it is necessary to conduct a cost-effective analysis based on constructed and operating stormwater control/treatment facilities. Of particular interest is the evaluation of the effect of the stormwater control/treatment facilities on pollution abatement and water quality improvement in the receiving system. Furthermore, the effect of such facilities on local planning and implementation of other storm runoff correction measures within the local area should also be evaluated and could incorporate a more sensitive construction and operating cost analysis.

Method of Approach. A selected number of combined sewer overflow/stormwater and flood/erosion control projects will be reviewed in the following three categories:

1. The effectiveness of completed and operating nonstructural/low-structural source control programs (BMPs) will be evaluated for selected projects or studies determined to be most promising in stormwater pollution abatement. In-depth action-impact analysis will be emphasized. These projects will be selected from the following source control areas with emphasis on multipurpose/benefit application: (a) natural drainage, (b) porous pavements, (c) onsite/source storage, (d) maintenance practices (street sweeping), and (e) erosion/flood control. Assessment of actual or apparent benefits with respect to effectiveness, costs, local and environmental impacts will be made.

2. The most promising, new and ongoing BMP projects will also be selected for in-depth case studies. Projects of this type resulting from Section 201 and 208 plans currently being implemented will be considered in this category. The basis and reasons for their preferred and successful selection at the respective sites will be investigated, and their apparent benefits (single and multipurpose) will be assessed.

3. Selected completed and operating combined sewer overflow treatment facilities will be evaluated from a treatment systems approach. The projects will be selected for in-depth re-evaluation from the following structural treatment control alternatives:

    - Inline storage
    - Offline storage
    - Storage/sedimentation
    - Swirl concentrator
    - Screens
    - Dissolved air flotation
    - Treatment lagoons

Essential elements of the treatment systems approach include definition and detailed evaluation of system effectiveness; design basis and background; annual costs, including operation and maintenance; energy consumption; achievement of stormwater quality goals and return benefits of the system (single or multipurpose); operation and maintenance difficulties including expected usable life; and receiving water impacts. The anlaysis will also identify socio-economic impacts such as (a) effects on land values and taxes, (b) housing and relocation, (c) general public acceptance and aesthetics, and (d) local community facilities. Applications of value engineering techniques to optimize system costs will also be evaluated. To the extent possible, this approach and analysis will be applied to the case studies in all three categories.

To identify as many potential candidate projects as possible, including those local projects that may not be known nationally or have been uncovered through past studies, over 100 survey questionnaires have been sent to federal, local, and private agencies requesting nomination of any known project for inclusion in this study. The main emphasis of this study will be centered around regionally encompassing projects or full-scale prototype treatment facilities.

The work upon which this portion of the paper is based was performed pursuant to Contract No. 68-03-2617 with the Environmental Protection Agency.

San Francisco Pilot Plant Project

As an integral element of San Francisco's 201 facilities planning effort to control and treat combined sewer overflows from the entire city area and treat dry-weather flows from the west side of the city, a pilot plant was constructed to study and test the applicability, effectiveness, and costs of various unit processes. The pilot facilities consist of three process systems: (1) dry-weather treatment, (2) wet-weather treatment, and (3) sludge processing.

The pilot plant program was developed before the enactment of the amendments of PL 92-500, with the presumption of secondary levels of treatment for dry-weather flows.

Scope and Objectives of the Program. The proposed prototype secondary facilities would treat dry-weather sewage to effluent quality levels required for ocean discharge. The secondary facilities would also be integrated with the wet-weather facilities to treat wet-weather flows up to its peak hydraulic capacity. A wet-weather treatment plant would be operated in parallel with the secondary plant on an intermittent basis, as required, to accommodate wet-weather flows in excess of the peak hydraulic capacity of the secondary plant.

The primary sites under consideration for these facilities are limited in area with possible restrictions on surface use, and the site's proximity to high use public recreational areas create complications for an already very difficult waste treatment problem. In addition, wet-weather discharge requirements have not been finalized pending the results of a cost-effectiveness study of wet-weather treatment alternatives. Other uncertanties of the program include:

- During wet weather, the composition of the feed to the secondary plant will be continuously changing. The response of secondary plant effluent quality to expected variations in pollutant load and chemical characteristics during storms is unknown.
- An attractive alternative for wet-weather operation is to pass as much wet-weather sewage through the secondary plant as is possible without serious degradation of effluent quality. This minimizes the volume of wet-weather treatment necessary, minimizes the number of times per year the wet-weather treatment plant must be operated, and provides a better effluent quality for small storms. The

extent to which the secondary plant can be hydraulically forced before treatment breakdown is unknown.

- It is possible to provide a short-term upgrading of the secondary plant capacity by providing chemical addition to primary clarifiers and flexible use of secondary clarifiers. The technical feasibility, cost effectiveness, and long-term effect of this on secondary plant operations are unknown.

- Secondary clarifier performance could be the limiting factor on hydraulically forcing the secondary plant. Both the effect of the increased flow and the sudden reduction of biosludge age on the secondary clarification process cannot be forecast for the transient conditions expected during wet-weather operations.

The pilot plant program will seek to answer these important questions to ensure the cost effectiveness of the proposed facility. The program will also include study of sewage sludge processing and disposal alternatives. Emphasis will be on wet-weather sludges because of their unknown characteristics and potential impacts on existing sludge disposal operations.

The pilot plant program will consist of evaluation and comparison of individual unit processes under controlled conditions. This will be done primarily with synthetic wet-weather wastewater. One or more treatment process trains will also be demonstrated with emphasis on reliability, effluent quality stability, and development of operating procedures for wet-weather plant startup, shutdown, and interstorm conditions.

Process Systems. The secondary treatment alternatives selected for pilot plant evaluation include conventional activated sludge, oxygen activated sludge, and rotating biological contactors; they are shown schematically in Figure 2-2.

The wet-weather treatment units selected for evaluation are shown in Figure 2-3. The candidate processes include:

- Chemically assisted primary sedimentation using a variety of chemical additives
- Primary sedimentation without chemical addition
- High-rate gravity sedimentation using a lamella separator to reduce equipment volume
- Induced air floation which may offer capital and operating costs comparable to chemically assisted primary sedimentation with much lower space requirements than conventional dissolved air flotation processes. However, their performance in stormwater service is unknown.
- Microscreens (23 micron mesh size) can provide high rate solids separation in a small space.

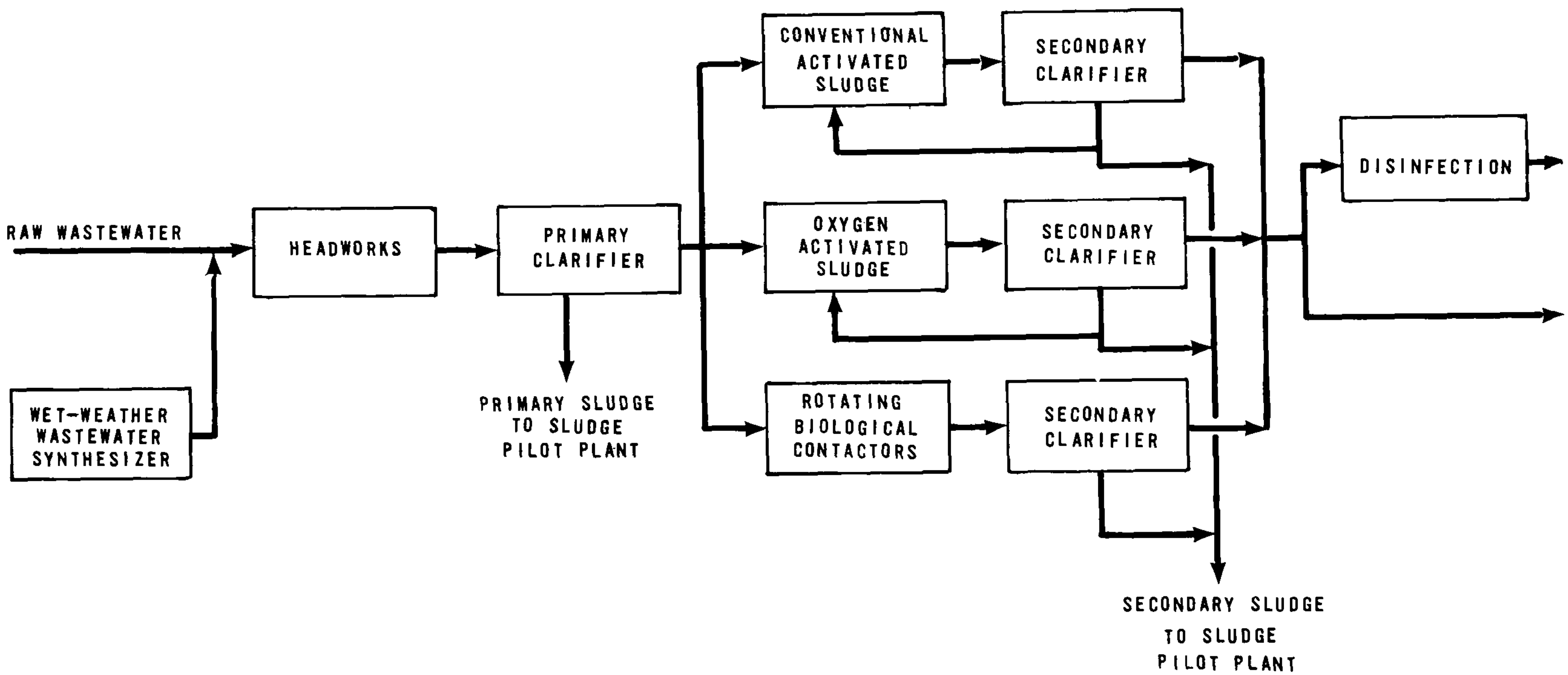


Figure 2-2. Preliminary Process Flow Diagram-
Secondary Pilot Plant.

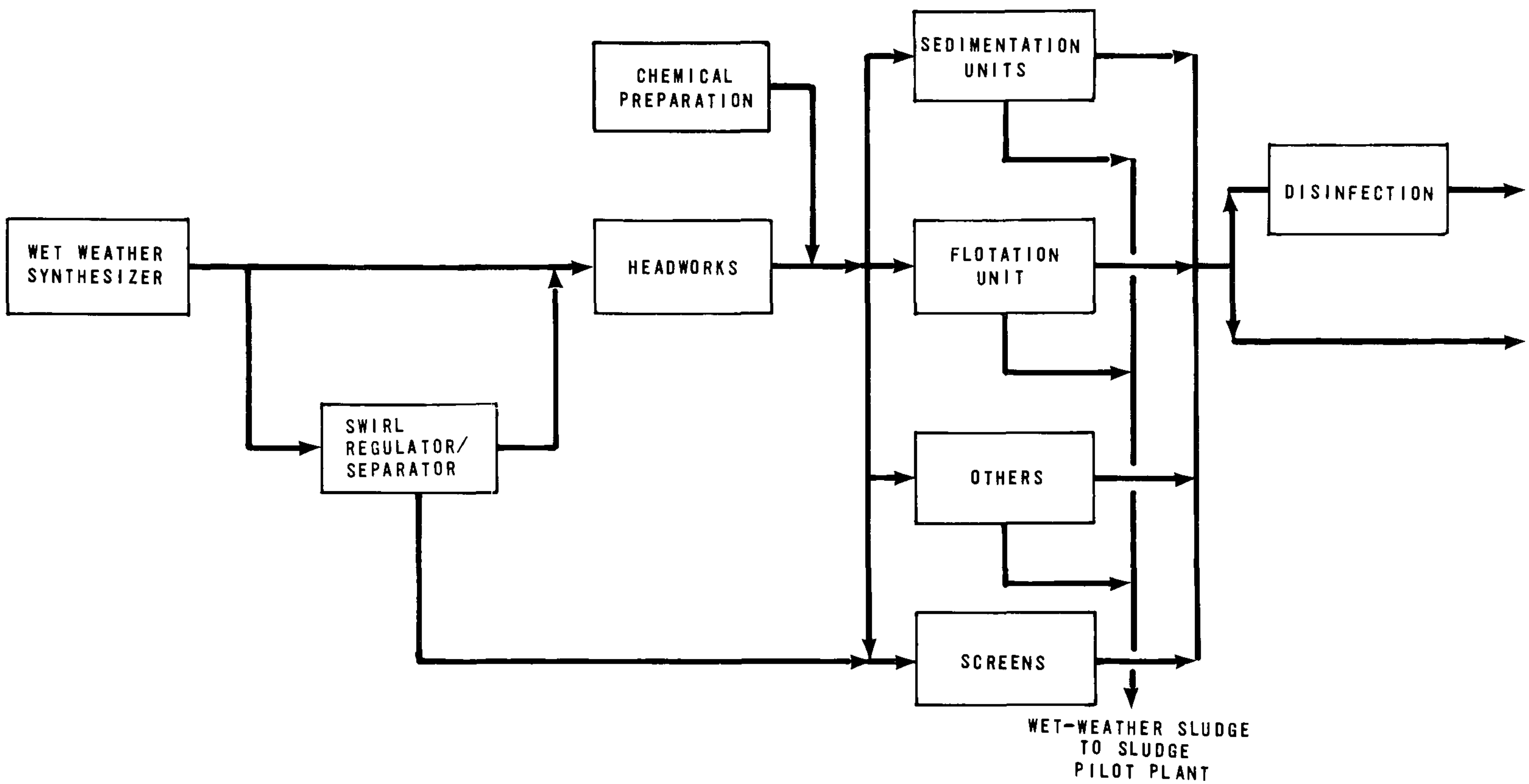


Figure 2-3. Preliminary Process Flow Diagram-
Wet-Weather Pilot Plant.

- Swirl regulator/separators have the potential to concentrate the majority of the suspended solid material in a minority of the flow volume. Their utility in this application will depend on whether the quality of the overflow is suitable for ocean discharge under wet-weather conditions.

- Disinfection will be evaluated for ability to control biological problems in an effluent of variable quality and flow.

A very preliminary process flow diagram for the sludge processing pilot plant is presented in Figure 2-4. The unit operations and processes actually tested during the program will depend on the types of sludges produced by the processes which are selected in the dry- and wet-weather evaluations. Included in the sludge testing program will be evaluations of sludge storage and transport. The very restricted site conditions may preclude any sludge processing at the site itself.

A unique feature of the pilot plant will be the utilization of a wet-weather synthesizer which will be capable of providing the various process trains with simulated storm flow hydrographs and transient pollutant loads. This facility is being provided to enable pilot plant operation and evaluation during non-rain periods. The synthesizer will be operated by mixing various quantities of raw sewage, potable water, solids, brine, and oil-simulating mass emission curves resulting from monitoring at the headworks of the existing treatment plants and from citywide computer simulation runs.

Photographs of the pilot plant facility are shown in Figure 2-5.

The dry weather portion of the pilot plant, first operational in November 1977, is currently under evaluation. The wet-weather facilities are nearing completion, and testing is expected to commence in January 1978.

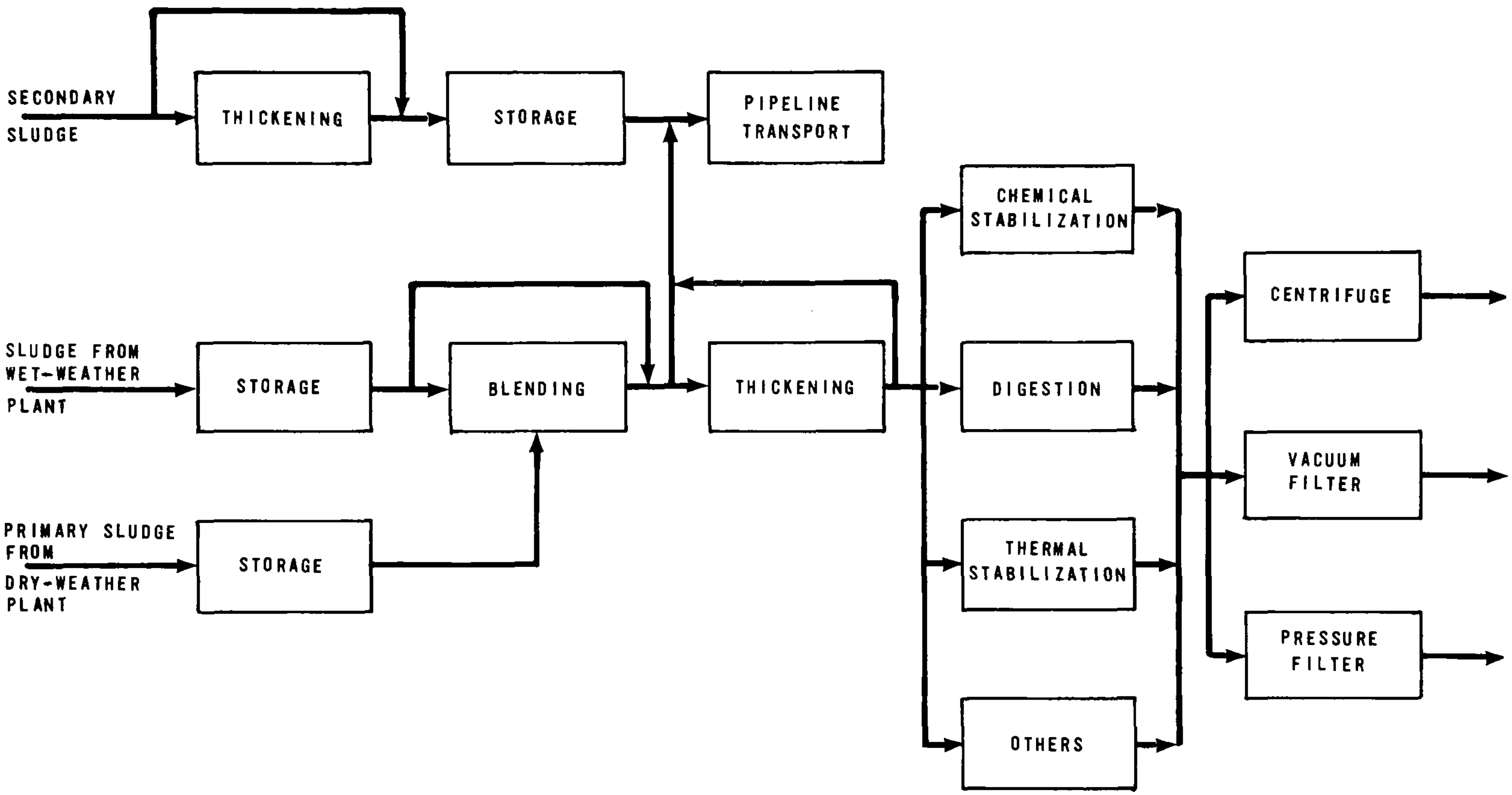
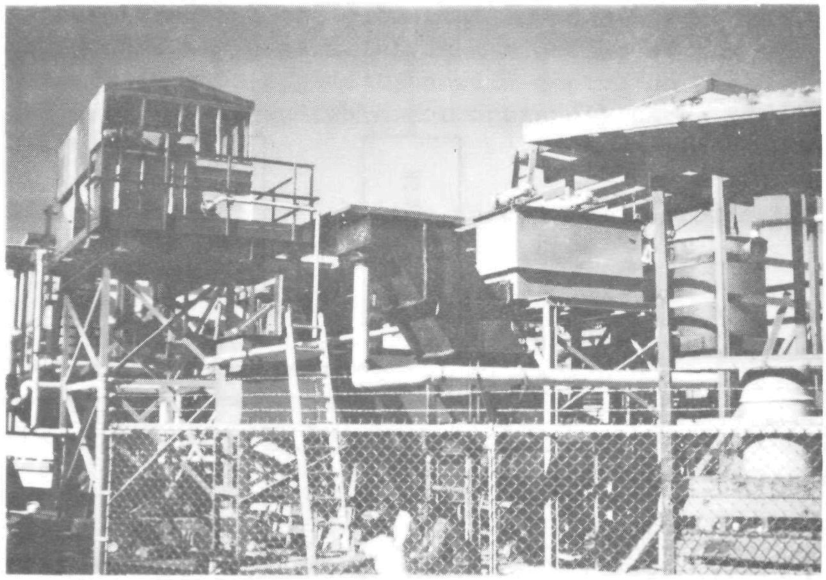
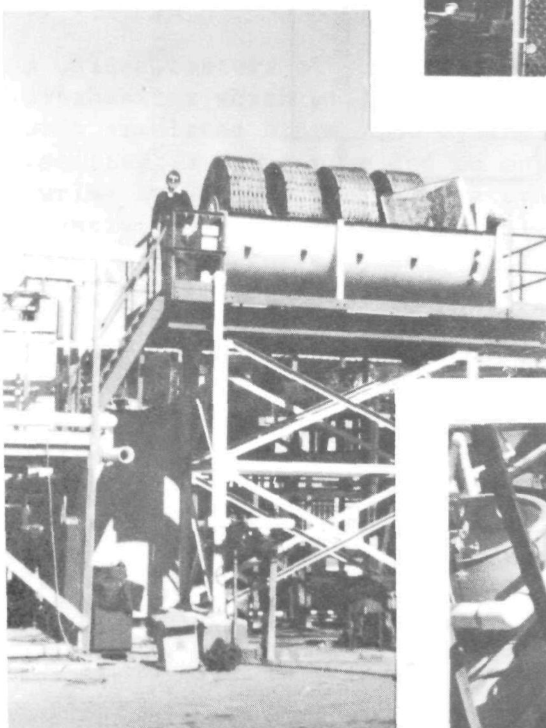


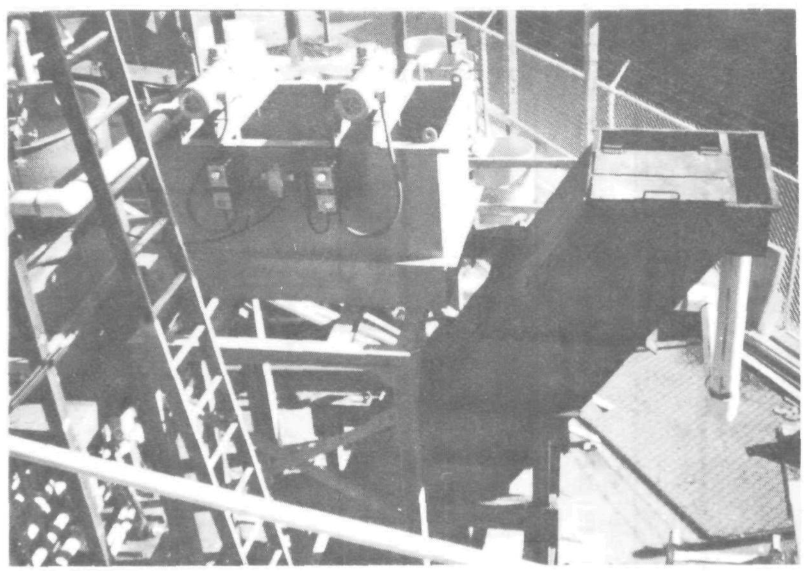
Figure 2-4. Preliminary Process Flow Diagram-
Sludge Pilot Plant.



(a)



(b)



(c)

Figure 2-5. San Francisco pilot plant facilities. (a) Overall view of pilot plant-RBC enclosure and high-rate lamella settler. (b) RBC unit under construction. (c) Overhead view of lamella settler.

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SUMMARY CHARACTERIZATION OF URBAN WASTEWATER MANAGEMENT OPTIONS

By

Stephan J. Nix, James P. Heaney, and Kevin Smolenyak*

Evaluation of control alternatives has become much more complex with the advent of 208 planning in urban areas throughout the United States. The traditional problem of locating and sizing wastewater treatment facilities for control of sewage has been expanded to include wet-weather pollution control. Unlike uniform wastewater flows, the wet-weather flows are intermittent. Furthermore, control options to be considered consist of, not only the traditional structural approaches, e.g., storage-treatment devices, but also non-structural alternatives, e.g., street sweeping.

The problem facing the 208 analyst is to find the combination of all of the above control options which achieves a specified pollutant control level at minimum cost. As part of earlier EPA sponsored studies, we have developed procedures for performing these analyses [1, 2, 3]. The method can be expressed as a problem in production economics, i.e.,

$$\begin{aligned} &\text{minimize } Z = f(\bar{X}) \\ &\text{subject to } g(\bar{Y},\bar{X}) = 0 \quad \ldots\ldots\ldots\ldots \quad (1) \\ &\qquad \bar{Y} = \hat{Y} \end{aligned}$$

where Z = cost of attaining a specified level of pollutant control $\bar{Y}$;
$f(\bar{X})$ = cost function for a vector of inputs, $\bar{X}$;
$\bar{X}$ = vector of inputs, $\bar{X} = (x_1, x_2, \ldots, x_i, \ldots, x_m)$;
$\bar{Y}$ = vector of outputs, $\bar{Y} = (y_1, y_2, \ldots, y_j, \ldots, y_n)$;
$\hat{Y}$ = specified output level, $\hat{Y} = (\hat{y}_1, \hat{y}_2, \ldots, \hat{y}_j, \ldots, \hat{y}_n)$;

and

$g(\bar{Y},\bar{X})$ = 0 is a production function expressing the <u>maximum</u> attainable level of output which can be achieved with a given vector of inputs, $\bar{X}$.

This rather esoteric formulation can be translated into a relevant engineering tool if appropriate cost data and knowledge of the production

*Graduate Research Assistant, Associate Professor, and Graduate Assistant, Dept. of Environmental Engineering Sciences, University of Florida, Gainesville, FL, 32611.

function are available. While engineers are accustomed to deriving cost functions, they are usually unfamiliar with notions of production functions. Production functions define the locus of technically efficient combinations of inputs and outputs. In this presentation, a single output is used, i.e., the proportion of pollutant discharge which is controlled. Inputs are two control technologies, storage and treatment. Linear objective functions are used. The same problem is solved graphically and analytically to show the relative merits of each approach. Data from earlier studies of Minneapolis are used for the test application.

PRODUCTION FUNCTION FOR STORAGE-TREATMENT

The STORM model was used in an earlier study to generate isoquants of the percent pollutant control, y, as a function of treatment rate, x_1, and storage volume, x_2 [1,4]. Results for Minneapolis are shown in Figure 3-1. The isoquants are interpolations between the data points from the simulation runs. Each data point is the result of a one year simulation with a specified storage-treatment configuration. These isoquants exhibit several relevant properties:

1) They slope downward and to the right because as one input increases, it takes less of the other input to achieve the same level of output.
2) They are convex to the origin because of the decreasing ability of one input to be substituted for another to obtain a given level of output. This is known as the principle of diminishing marginal rate of substitution.
3) The isoquants intersect the x_1 axis indicating that it is possible to use this input exclusively.
4) The isoquants become asymptotic to the x_2 axis at the point where x_1 is being used continuously. Further increases in x_2 are nonproductive because x_1 is limitational.

Exponential Production Function

In the earlier studies, the following functional form was used to obtain the equation for a fixed value of y [1, 2]:

$$x_1 = \underline{x}_1 + (\bar{x}_1 - \underline{x}_1)e^{-kx_2} \quad . . . . \quad (2)$$

where $\underline{x}_1$ = treatment rate at which the isoquant becomes asymptotic to the ordinate, inches/hour,

$\bar{x}_1$ = treatment rate at which isoquant intersects the abscissa, inches/hour, and

k = constant, inch^{-1}.

The lower limit on treatment, $\underline{x}_1$, can be found directly, as follows, for 8760 hours per year:

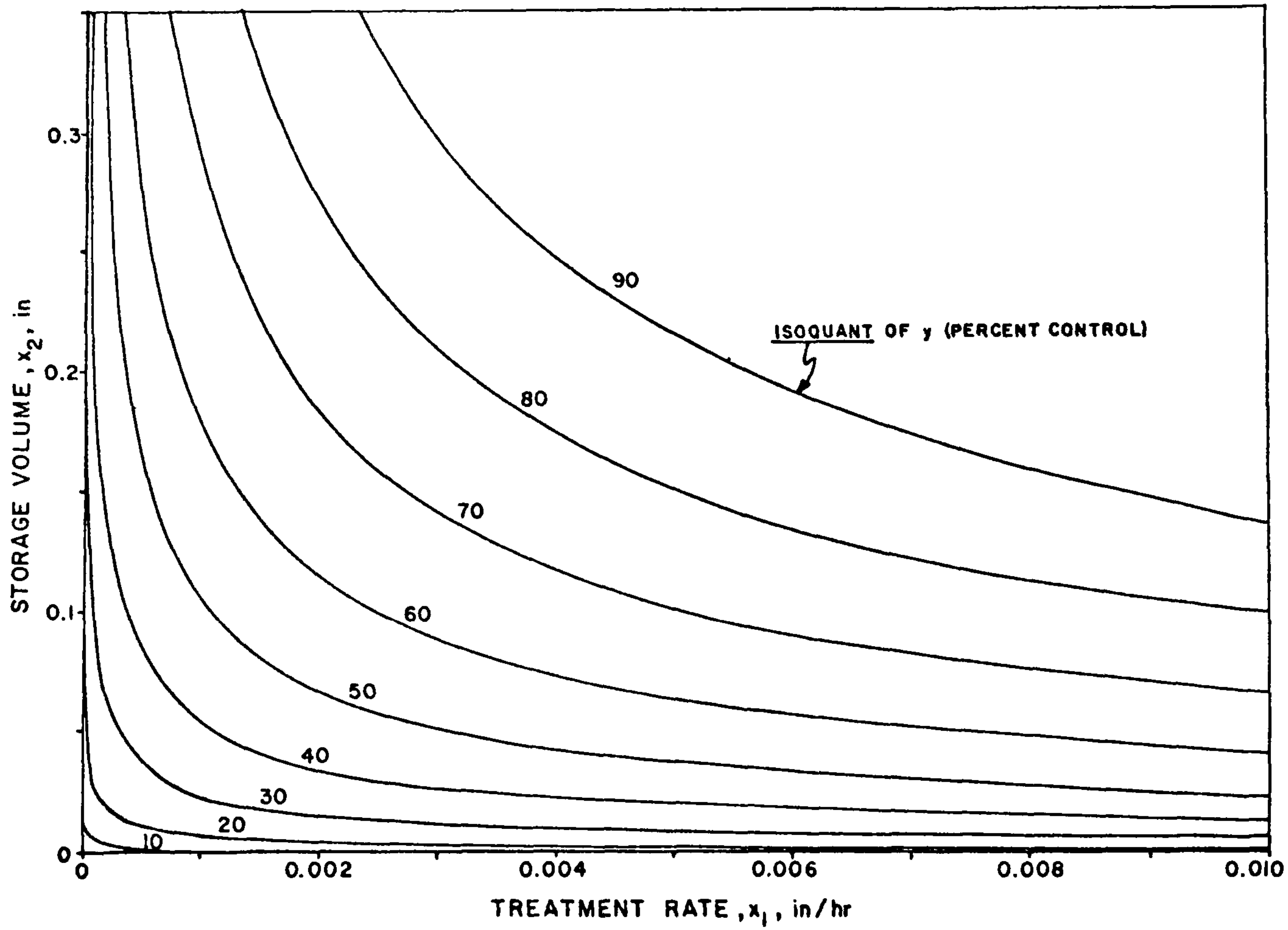

Figure 3-1. Storage-Treatment Isoquants for Minneapolis, Minnesota

$$\underline{x}_1 = \frac{AR}{8760}(y) = ay \quad (3)$$

where AR = annual runoff, inches/year, and
a = coefficient.

By relating the parameters $\underline{x}_1$, $\bar{x}_1 - \underline{x}_1$, and k to the level of control, y, one equation was developed for the entire response surface. The $\bar{x}_1 - \underline{x}_1$ and k terms were found to be of the following general form:

$$\bar{x}_1 - \underline{x}_1 = be^{hy}, \text{ and} \quad (4)$$

$$k = de^{-fy} \quad (5)$$

where b, d, f, h = coefficients.

Substituting equations 3, 4, and 5 into equation 2 yields the production function:

$$x_1 = ay + be^{hy} - (de^{-fy})x_2 \quad (6)$$

This function, while statistically the most accurate, proved clumsy to manipulate, e.g., finding $\partial y/\partial x_i$, the marginal productivity of each input, in the subsequent optimization. Thus, another functional form was evaluated.

Cobb-Douglas Production Function

While a wide variety of production functions are used by economists [5, 6], this study examines only the so-called Cobb-Douglas production function of one output and two inputs. It has the following form:

$$y = Ax_1^{\alpha_1} x_2^{\alpha_2} \quad (7)$$

where A = coefficient, and
α_1, α_2 = input intensity parameters, $\alpha_1 < 1$, $\alpha_2 < 1$.

This function has several properties which make it convenient to use [5], e.g.,

1) it is a homogeneous equation;

2) if $\alpha_1 + \alpha_2 \begin{cases} <1 \rightarrow \text{decreasing returns to scale} \\ =1 \rightarrow \text{constant returns to scale} \\ >1 \rightarrow \text{economies of scale;} \end{cases}$

3) marginal products, $\partial y/\partial x_i$, are positive but decreasing as x increases; and

4) the optimal solution for any value of y can be expressed directly as a function of the inputs.

A potentially major limitation of this functional form is that it is asymptotic to both input axes. Thus, it does not permit using treatment, x_1, alone. This problem can be avoided by defining another single-input production function,

$$y = A_1 x_1^{\alpha} \quad \text{. . . . .} \quad (8)$$

where A_1 = coefficient, and

α = input intensity parameter,

which is used in the optimization procedure as an alternative to equation 7. Comparison of the results will tell the range over which each production function is superior. Another limitation of the Cobb-Douglas functional form is that it loses accuracy as one approaches the x_2 asymptote. However, experience to date indicates that the "elbow" of the function is the area of primary concern.

In the earlier work, isoquants (using the exponential function) were generated for San Francisco, Denver, Minneapolis, Atlanta, and Washington, D.C. [1]. Table 3-1 shows the results of fitting two-input and single-input production functions to this same data. The sum of the α's is less than one, indicating diseconomies of scale. Lastly, a single predictive equation for the entire Untied States was determined by using the data for the five cities with annual runoff as the third input. The result for the two-input model is:

$$y = [967\ (AR)^{-0.239}]\ x_1^{0.233}\ x_2^{0.390} \quad \text{. . . . .} \quad (9)$$

SYSTEM OPTIMIZATION

Graphical Method

The question of what functional form is suited to a particular optimization problem is avoided by a graphical solution procedure. The general optimization problem for storage-treatment is

$$\text{minimize } Z = c_1 x_1^{\beta_1} + c_2 x_2^{\beta_2} \quad \text{. . . . .} \quad (10)$$

$$\text{subject to } g(y,\ x_1,\ x_2) = 0 \quad \text{. . . . .} \quad (11)$$

$$x_1,\ x_2 \geq 0$$

where Z = total annual control costs, \$/acre,

β_1, β_2 = cost exponents,

c_1 = unit cost of x_1, annual \$/inch/hour,

c_2 = unit cost of x_2, annual \$/inch, and

$g(y,\ x_1,\ x_2)$ = 0 is the production function derived by interpolation between data points.

Table 3-1. Parameters for Cobb-Douglas Production Function for Five Cities

City	j=	Annual Runoff,[a] Inches/Year	Developed Population Density,[a] Persons/Acre	Cobb-Douglas Production Function Coefficients				
				$y^b = A\, x_1^{\alpha_1}\, x_2^{\alpha_2}$			$y^c = A_1\, x_1^{\alpha}$	
				A	α_1	α_2	A_1	α
San Francisco	1	9.37	9.96	489	0.198	0.410	566	0.65
Denver	2	5.59	9.11	584	0.252	0.334	449	0.56
Minneapolis	3	10.50	7.92	464	0.213	0.333	410	0.56
Atlanta	4	16.18	8.24	660	0.246	0.490	481	0.69
Washington, D.C.	5	17.22	20.02	570	0.255	0.394	440	0.63

[a]Source: Heaney, J.P., et al., Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges: Volume II: Cost Assessment and Impacts, EPA-600/2-77-064, March, 1977.

[b]Multiple correlation coefficients $\geq .95$

[c]Correlation coefficients ≥ 0.92

The optimal combination of x_1 and x_2 and the associated cost, Z*, for any value of y is found by locating the point of tangency of the isoquant (equation 11) and isocost (equation 10) lines giving the lowest value of Z.

Exponential Production Function

The formulation of the optimization model using the exponential production function, i.e., equation 2, and assuming linear control costs, is [1, 2]:

$$\text{minimize } Z = c_1x_1 + c_2x_2 \qquad (12)$$

$$\text{subject to } x_1 = \underline{x}_1 + (\bar{x}_1 - \underline{x}_1)e^{-kx_2} \qquad (13)$$

$$x_1, x_2 \geq 0$$

Solving this constrained optimization problem yields

$$x_2^* = \max \left[\frac{1}{k} \ln \frac{c_1}{c_2} [k(\bar{x}_1 - \underline{x}_2), 0 \right] \qquad (14)$$

where x_2^* = optimal amount of storage, inches,

$$\text{and } x_1^* = \underline{x}_1 + (\bar{x}_1 - \underline{x}_1)e^{-kx_2^*} \qquad (15)$$

where x_1^* = optimal amount of treatment, inches/hour.

Note that x_1^* is a function of x_2^* so it is necessary to find x_2^* first. Knowing x_1^* and x_2^*, the optimal solution is

$$Z^* = c_1x_1^* + c_2x_2^* \qquad (16)$$

where Z* = total annual cost for optimal solution, \$/acre.

The procedure used to find Z* for any level of control was to solve the optimization problem for several assumed values of y and then fit a function to the result. The equation of best fit was:

$$Z^* = me^{\gamma} \qquad (17)$$

where m, γ = coefficients.

This equation is unsuitable at low levels of y since $Z^* \to m$ as $y \to 0$.

Cobb-Douglas Production Function

The above problem is formulated below using a Cobb-Douglas production function and a nonlinear cost function with economies of scale in storage and treatment.

$$\text{Minimize } Z = c_1x_1^{\beta_1} + c_2x_2^{\beta_2} \qquad (18)$$

$$\text{subject to } y = A_1x_1^{\alpha_1}x_2^{\alpha_2}$$

$$x_1, x_2 \geq 0$$

where β_1, β_2 = cost exponents and $\beta_i < 1$ implies economies of scale in the cost.

Solution of this problem yields:

$$x_1^* = \left[\frac{\alpha_1}{\alpha_2} \; \frac{\beta_2}{\beta_1} \; \frac{c_2}{c_1}\right]^{\frac{1}{\beta_1}} x_2^{\frac{\beta_2}{\beta_1}} \quad \ldots\ldots\ldots \quad (19)$$

Note that the optimal combination of x_1 and x_2 is independent of y. The final solution is shown below:

$$Z^* = Ky^{\frac{\beta_1 \beta_2}{\alpha_1 \beta_2 + \alpha_2 \beta_1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (20)$$

$$\text{where } K = c_2 \left[\frac{\beta_2\alpha_1 + \beta_1\alpha_2}{\beta_1\alpha_2}\right] \left(\frac{\left[\frac{\beta_1 c_1 \alpha_2}{\beta_2 c_2 \alpha_1}\right]^{\alpha_1}}{A^{\beta_1}}\right)^{\frac{\beta_2}{\alpha_2\beta_1 + \alpha_1\beta_2}}$$

Equations 19 and 20 are simplified if linear costs are assumed, i.e., $\beta_1 = \beta_2 = 1$.

The solution shown in equation 20 provides an excellent summary characterization of whether overall economies of scale exist. Most environmental control facilities exhibit economies of scale in the cost of construction and operation and maintenance, i.e., $\beta_i < 1$. However, these savings are offset by the diminishing marginal productivity of the inputs, i.e., $\alpha_i < 1$. Thus, the general test for the effect of scale is whether

$$\frac{\beta_1 \beta_2}{\alpha_1\beta_2 + \alpha_2\beta_1} \begin{cases} > 1 \rightarrow \text{diseconomies of scale} \\ = 1 \rightarrow \text{constant returns to scale} \\ < 1 \rightarrow \text{economies of scale} \end{cases}$$

Furthermore, it is relatively straightforward to do sensitivity analysis using equation 20. Of course, the Cobb-Douglas production function is not as accurate as equation 6. Nevertheless, it may be preferable to use it in certain cases.

Example Comparison

With data for the developed portion of Minneapolis and linear costs reported earlier, an optimal strategy will be derived using each of the three methods [4, 2]. The developed area of Minneapolis encompasses 215,000 acres and has a population density of 7.92 persons per acre. The runoff for the year used to generate the data points with STORM was 10.50 inches. The unit cost of secondary treatment, c_1, is \$9810/ac-in/hr, whereas the unit cost of storage, c_2, is \$219/ac-in. Secondary treatment is assumed to have a constant removal efficiency of 85 percent. In all the above equations, y is assumed to be the pollutant control at a removal efficiency of 100 percent. If another efficiency is desired, the following transformation is made:

$$y = \frac{100y'}{\eta} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

where y' = actual percent pollutant control, $0 \leq y' \leq \eta$, and
η = removal efficiency, $0 \leq \eta \leq 100$.

Application of the graphical method and the expansion path produced by each method is shown in Figure 3-2. The cost equation and optimal strategy for various levels of pollutant (BOD) control are shown in Table 3-2. The results reveal discrepancies among the methods, but all are reasonably close to the answer provided by the graphical solution. It is assumed that the graphical solution is the "best" answer since it is unaffected by the error introduced by fitting functions. However, for preliminary estimates, each method seems to provide results well within the accuracy required for such an analysis.

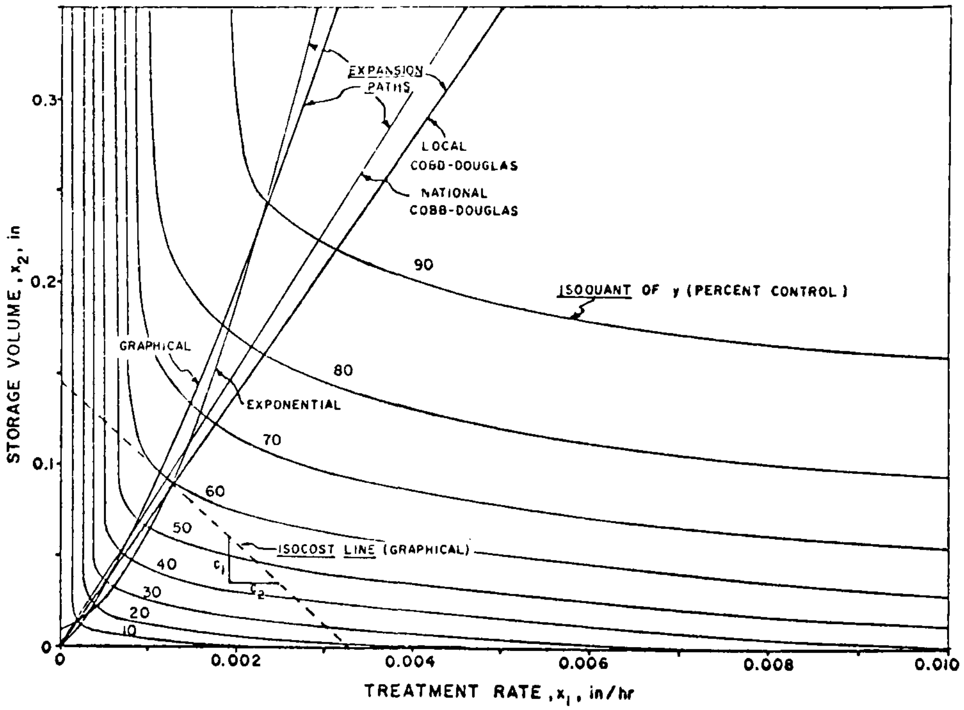

Figure 3-2. Application of Graphical Method and Expansion Paths

Table 3-2. Results of System Optimization, Minneapolis, Minnesota (see text for units of variables)

Percent Pollutant (BOD) Control, y', $0 \le y' \le 85$	Graphical Method			Exponential $Z^* = 4.65R^{0.0382y'}$			Local Cobb-Douglas $Z^* = 0.0331(y')^{1.832}$			National Cobb-Douglas $Z^* = 0.0907(y')^{1.605}$		
	x_1^*	x_2^*	Z^*	x_1^*	x_2^*	Z^*	x_1^*	x_2^*	Z^*	x_1^*	x_2^*	Z^*
10	.00028	.014	6	.00027	.017	7	.00009	.006	2	.00014	.011	4
25	.00052	.036	13	.00058	.032	13	.00048	.034	12	.00061	.046	16
50	.0011	.092	31	.00125	.092	32	.00170	.119	43	.00104	.138	48
75	.0023	.235	74	.00238	.254	79	.00358	.251	90	.00354	.266	93

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OVERVIEW OF NOVEMBER 1977 RELEASE OF SWMM

by

Wayne C. Huber, Stephan J. Nix, W. Alan Peltz and James P. Heaney*

CHRONOLOGY AND STATUS

The most recent previous SWMM release prepared by the University of Florida (UF) was in September 1976. This version differed in minor ways (error cleanup, I/O improvements) from the release made in the spring of 1976. Since September 1976, a list of errors has been prepared and distributed, but the program on the tapes given to users has remained the September 1976 version.

A November 1977 (NOV77) release has been prepared. As usual, it is available from both UF and EPA (through Harry Torno). The package includes a tape with the Fortran source listing and sample input data. The Version II SWMM User's Manual (1) will still apply as far as general input data preparation and formats for some blocks. However, as for the 1976 releases, there is also included a documentation packet containing revised input formats (mainly for Runoff and Storage/Treatment), documentation for WRE Transport (EXTRAN) and, as much as possible, documentation of new procedures included in the NOV77 release. This documentation includes descriptions of some procedures placed in the 1976 releases as well (e.g., infiltration routine in Runoff). Hopefully, it is organized in a better manner than in the past such that users will be able to locate relevant material more easily, (e.g., it has a table of contents).

A new SWMM User's Manual is in preparation and should be ready by summer 1978. Most pending modifications will have been completed and documented at that time.

The most significant differences between the NOV77 and earlier releases are as follows. More details are presented subsequently.

1. Snowmelt is included in the Runoff Block, for both single event and continuous simulation.

2. Continuous simulation capability has been greatly enhanced in terms of I/O, documentation, algorithms and useable output. Continuous simulation uses only the Runoff and Storage/Treatment (S/T) Blocks.

*Dept. of Environmental Engineering Sciences, University of Florida, Gainesville, Florida 32611.

3. Sludge generation and simple solids handling routines are included in the S/T Block. All S/T routines are working.

4. The latest cost data are in the S/T Block. These will have some actual usefulness when running continuously, since O and M costs are computed on the basis of actual hours of operation.

5. Known bugs have been corrected.

Different levels of urban stormwater analysis continue to be evaluated. Two "Level I" reports were issued during 1977 (2,3). A Level III Receiving Model (simplified, continuous) is nearing completion with a report scheduled for winter 1978. The Level IV SWMM report consists of the forthcoming Users Manual, as discussed earlier.

RUNOFF

Continuous Simulation

Although 1976 SWMM release had the capability for continuous simulation, it was poorly documented and there were bugs. The NOV77 release hopefully has few bugs and is understandable. Using US Weather Service "Hourly Precipitation Deck 488" or user-supplied precipitation input, the model will run for an unlimited number of time steps. Infiltration capacity, depression storage and pollutant loads are regenerated during dry time steps. Output is available on a time step, daily, monthly, annual and grand total basis. The fifty highest hourly precipitation, runoff and BOD loads are tabulated also.

Preliminary comparisons have been made between continuous SWMM and STORM (4). For similar schematizations, STORM runs are cheaper and produce comparable output. SWMM might be chosen over STORM if 1) gutter/pipe routing is desired, 2) SWMM algorithms are preferred, or 3) the more flexible and realistic SWMM S/T procedures are preferred. STORM may be advantageous for rural areas since it allows the use of the SCS curve number technique for these areas.

Snowmelt

Following the earlier work of the Canadian SWMM study by Proctor and Redfern and James F. MacLaren (5,6) snowmelt simulation has been added for both single event and continuous simulation. Most techniques are drawn from Anderson's (7) work for the US Weather Service (USWS). For single event simulation, temperature data are input for each time step. For continuous simulation, daily max-min temperatures from the USWS "WBAN Summary of the Day, Deck 345" are converted to hourly values by sinusoidal interpolation.

Each subcatchment is divided into the areas sketched in Figure 4-1. Urban snow removal practices may be simulated through the "redistribution fractions" input for each subcatchment, through alteration of the melt coefficients and base temperatures for the three regions of each subcatchment, and through the areal depletion curves used for continuous simulation.

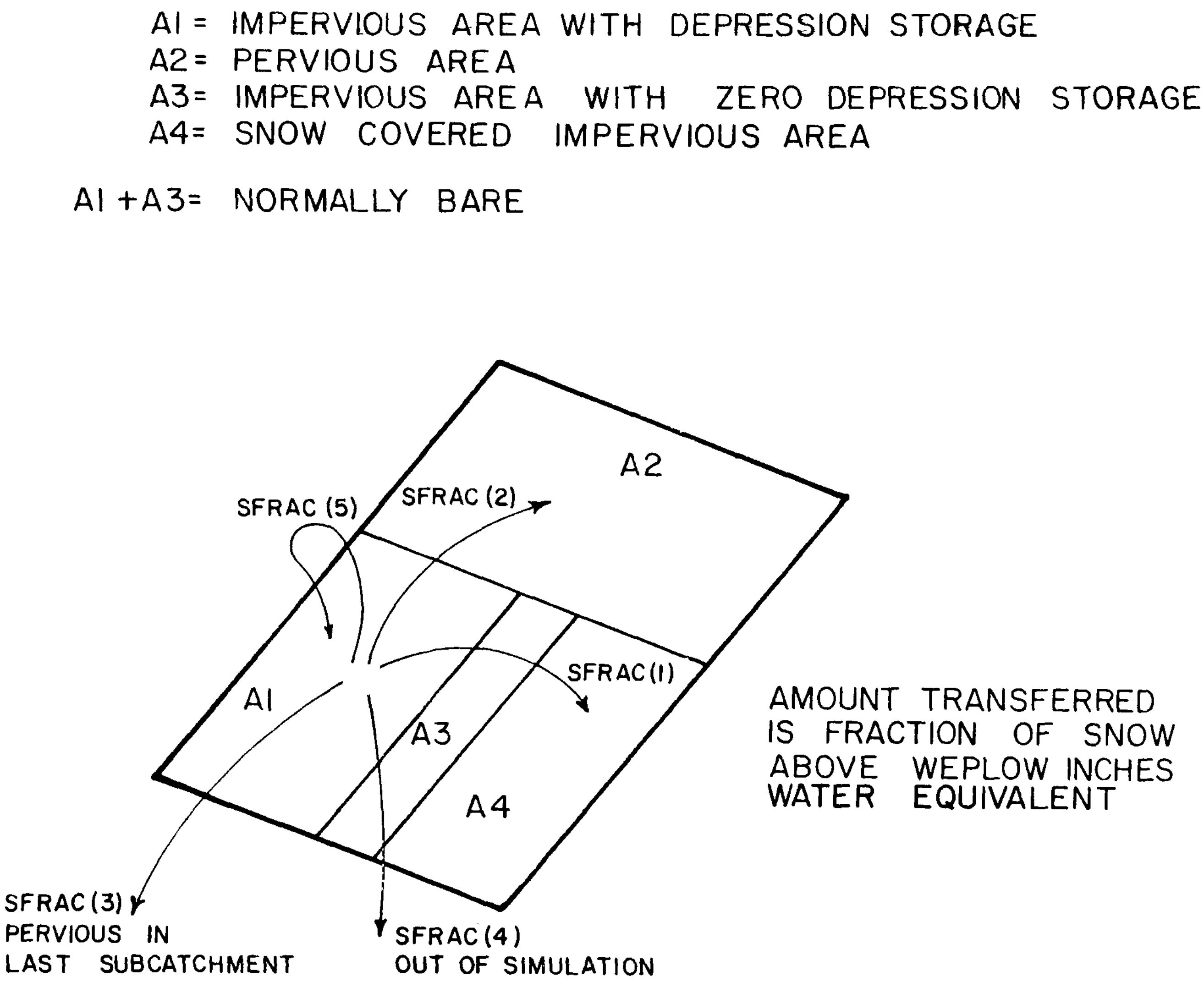

Figure 4-1. Subcatchment schematization during simulation of snowmelt. "Normally bare" areas may represent streets, sidewalks, etc. that are kept clean of snow. Snow on these areas is redistributed (e.g., plowed) according to the indicated fractions (i.e, values of SFRAC).

Anderson's (7) temperature-index and heat balance melt equations are used for melt computations during dry and rainy periods, respectively. For continuous simulation, the "cold content" of the pack is maintained in order to "ripen" the snow before melting. Routing of melt water through the snow pack is performed as a simple reservoir routing procedure, as in the Canadian study (5,6).

The presence of a snow pack is assumed to have no effect on overland flow processes beneath it. Melt is treated in the same manner as rainfall.

Quality Routines

As in the 1976 releases, the use of the dust and dirt etc. factors may be avoided if desired by simply inputing loadings for each pollutant, in lb/ac, for each subcatchment. Only one option will be available for calculation of SS washoff (that used for all other pollutants); thus, the "ISS" parameter has been removed. This follows a careful analysis of the two procedures by the University of Massachusetts (8).

An availability factor has been included in the street sweeping equations to reflect the fraction of total impervious area actually available to be swept (e.g., only the roadways). This follows work of Heaney and Nix (3). In general, the presence of snow is assumed to have no effect on pollutant load regeneration or washoff. However, there is no street sweeping if snow is present on a subcatchment. In addition, one user-supplied pollutant may be simulated, to model chlorides, for example. Although it is treated identically to all other pollutants, as a user option, it may be regenerated only when snow is present, thus avoiding high predicted chloride concentrations during summer months.

Other

Output from the NOV77 release should be more useful in terms of printed explanations and user flexibility. For instance, inflows as well as outflows from gutter/pipes may be printed out, and error messages contain somewhat improved syntax. The present size of the Runoff Block used in conjunction with the Executive Block is about 85K words. Some programming changes have been made to enhance compatability with CDC compilers, although the NOV77 release has only been compiled on the UF Amdahl 470 (similar to the IBM 370/165).

TRANSPORT AND WRE TRANSPORT

No significant changes have been made to either of these routines. Thus, some problems remain as to the documentation and sample data for the latter. UF has received a somewhat improved version of the WRE Transport Model (9) and has incorporated it into the model. However, it is strongly recommended that users encountering problems with the WRE Transport Model apply directly to Water Resources Engineers for assistance. Although they are unable to provide free consulting, they are better equipped at the moment to solve the problems.

STORAGE/TREATMENT

Modifications to the Storage/Treatment Block have been carried out in two steps. The first step was to correct the many bugs found throughout the block. Most corrections were concentrated in the subroutines TREAT and STRAGE, the majority of them dealing with flow routing and pollutant removal in storage units (sedimentation and external storage). Other significant changes involved the swirl concentrator (flow routing and negative pollutant removals) and the bleed-off from the sedimentation unit in the wet-weather treatment string to the dry-weather plant during dry periods. Another correction concerned the proper accounting of evaporation from storage units operating under the assumption of plug flow. Earlier versions, although accounting for evaporation from the total storage volume, did not evaporate water from the individual plugs in the unit. Other errors, although numerous, were not individually significant. Most were unit conversion and minor I/O errors. As part of this phase, the Storage/Treatment Block was tested in the continuous simulation mode and has worked successfully.

The second step centered around modifying pollutant removal mechanisms and adding a simplified sludge accounting procedure. In earlier versions, most of the treatment units employed a very simple (in some cases, a "black box") removal mechanism. In this version, the simplicity is retained but the user is given more latitude with equation parameters in order to account for the nature of the incoming sewage and local conditions. Sludge generation is performed by assuming that the suspended solids removed equals the sludge generated. The dry weight equivalent is then retained for a user specified detention time and released for disposal. Although the sludge procedures are greatly oversimplified, it is believed that the real value of this routine is in showing the variations in a long term simulation.

The cost functions in the original single event simulation were very simple. The major changes incorporated in this version include updated cost functions for a wide variety of dry-weather treatment facilities. More importantly, the costs of wet-weather operation are determined based on the operating hours as tabulated in the continuous simulation.

RECEIVING

There have been no major changes made to this block.

USER ACCESS AND FEEDBACK

As in the past, the program and documentation may be obtained in two ways. First, (and usually fastest), they will be provided free from Harry Torno of EPA (address below) upon receipt of a magnetic tape. Second, they will be provided by UF (contacts below) upon receipt of an unused 9-track magnetic tape or monetary equivalent ($15). In addition, a nominal additional charge (yet to be determined) is made to non-public agencies to cover the cost of tape preparation and documentation duplication at UF. Turn-around time following a request is usually about two weeks.

The primary EPA contact is thus,

Mr. Harry C. Torno
Staff Engineer
Office of Research & Development (RD682)
US Environmental Protection Agency
Washington, D.C. 20460 Phone (202)426-0810

The primary UF contacts are

Dr. Wayne C. Huber or
Mr. W. Alan Peltz
Dept. of Environmental Engineering Sciences
A.P. Black Hall
University of Florida
Gainesville, Florida 32611 Phone (904)392-0846

Feedback of users of program problems is heartily encouraged and should be addressed to either of the UF contacts above. In addition, UF will provide modest trouble-shooting and user assistance via phone or letter.

All users are urged to "join" the SWMM Users Group. This is accomplished simply by contacting Harry Torno with a name and address. He publishes aperiodic newsletters with current information on SWMM and many other current hydrologic analysis models and techniques. All program corrections and updates will be issued via his newsletter. Also, Users Group Meetings are held at approximately a semi-annual frequency at which presentations and group discussions are held on all facets of hydrologic modeling, including SWMM and many other models. These are currently being held in cooperation with the Ontario Ministry for the Environment for service to users of models supported by their office. Their contact is

Mr. Donald Weatherbe
Ontario Ministry for the Environment
135 St. Clair Avenue, W. - 2nd Floor
Toronto, Ontario M4V 1P5 Phone (416)965-6194

SWMM is just one of many available analysis techniques. Users are urged to stay in contact with the above groups for current information.

A CAVEAT TO USERS

The University of Florida has maintained and updated SWMM as only a part of contract and grant research sponsored by EPA. As such, UF is not in the "modeling business" and gladly performs these responsibilities in conjunction with many other urban runoff analysis and evaluation efforts.

As a result, the NOV77 SWMM release has not been thoroughly tested. Almost all aspects of the program may be expected to execute properly, but the many combinations of input parameters preclude adequate testing of all features. Hence, some bugs are to be expected.

As a practical matter, UF depends upon users to perform much of the actual testing of the model. As much as possible, user feedback is then incorporated into future SWMM releases.

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CASE STUDY: BEST MANAGEMENT PRACTICE (BMP) SOLUTION FOR A COMBINED SEWER PROBLEM

by

DR. WILLIAM C. PISANO*

FOREWORD

This paper summarizes the findings of a recent section 208 combined sewer management study for portions of the sewerage system in the City of Fitchburg, Massachusetts. The results showed that sewerage system remedial repairs and slight piping modifications were an order of magnitude less expensive than the nominal BMP practices of sewer flushing, street sweeping and catchbasin cleaning, and several orders of magnitude less than alternative structural options. An infiltration/inflow study is presently being conducted in the remaining sewered areas within the City. The general methodology and problem solving orientation of the aforementioned 208 study was used as guidance in the preparation of the scope of services for this new work.

BACKGROUND

The Nashua River is located in the northwestern portion of the State of Massachusetts and ultimately discharges into the Merrimack River. There are several old communities discharging industrial and domestic wastes into the upper headwaters of the Nashua River. The City of Fitchburg with a population of 40,000 is one of these communities. In recent years significant water quality improvement has been achieved by treatment of pulp and paper plant discharges. Advanced forms of treatment are currently envisioned for the two treatment plants in the City. The financial base of the City will be considerably taxed by these two treatment works.

Sewers within the City are a complicated mix of separated, partially separated and combined subsystems. Older portions in the system are combined but have been partially separated in some areas. Sewers within more recent portions of the City are separated but frequently were inter-connected into the older combined sewered areas. Complete intact and up-to-date sewerage system maps together with accurate inventories of service type per subarea and number and location of overflow points did not exist at the onset of this work.

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*Technical Director, Energy & Environmental Analysis, Inc. (EEA), 257 Vassar Street, Cambridge, Massachusetts 02139.

The State of Massachusetts, Division of Water Pollution Control has been monitoring the river water quality in the Fitchburg urban area over the last several years and desires that the combined sewer overflows in this area be eliminated. The Montachusett Regional Planning Commission, Fitchburg, Massachusetts was given a section 208 areawide wastewater management grant to prepare as part of their overall mandate, a combined sewer management plan that would be responsive to the limited funds the City has available for abating this problem. Energy and Environmental Analysis, Inc. (EEA), was contracted to prepare the combined sewer subplan for a representative area entailing 10-12 miles of sewer. This area was to be selected after completion of a general inventory of the sewers in the City.

PURPOSE OF PROJECT

The aim of the prototype combined sewer management study was to study in fine detail, the impact of potential structural and non-structural control options for a representative portion of the Fitchburg system and then select the least cost program for the area. This study was designed to investigate in detail, various mixes of structural and principally, non-structural control options and to quantify the attendant pollutant reduction and program flushing and maintenance programs, street cleaning, inflow control by downspout elimination and off-line storage tanks. The principal motivation and emphasis of the study from a methodological standpoint was to demonstrate the feasibility and practicality of keying almost the entire effort and resources on developing low cost, non-structural solutions for mitigating the number and magnitude of combined sewer overflows from the prototype study area.

SUMMARY OF PROJECT

Inventory and Mapping. A considerable level of effort was expended to develop the informational base needed to select the analysis area. The inventory phase indicated that the sewerage system in the City is mostly separated with numerous small "slivers" of combined sewer streets. There is however, one 500 acre section in the northeasternly portion of the City served by 10-12 miles of combined sewers with four well-defined overflow points. This area was deemed extremely suitable for the purposes of the study and satisfied the contractual level of effort constraint. Land use in the study area includes a heavy commercial downtown area within a mixture of dense single family and multi-family dwellings. Topography in the area is mostly hilly with a number of fairly flat cross-streets.

The sewerage system in the study area consisted of three collection subsystems. Sewer plan maps of the area and three special maps depicting each complete collection subsystem in profile view were prepared. These maps are called "flow-line" profile maps and visually show the entire collection system slope characteristics. These maps were used in the development of upstream collection system controls. They were particularly useful during the detailed physical surveys of the system and were used to rapidly establish potential sites for upstream off-line storage tanks.

Sewerage System Physical Surveys. The next phase of the work involved intensive physical surveys of the collection system manholes, regulators and other pertinent control points in the combined sewer study area. Roughly 150 manholes were inspected and approximately 100 spot flow measurements in the collection system were performed. The regulators, overflow conduits and other major control points within the system were repeatedly observed during both dry and wet weather conditions over a nine month period.

It was determined from the inspection program that overflow weir levels at three of the four regulators could easily be raised a foot without any adverse backwater effects. Minor structural and piping modifications would be required at one of these locations to permit the desired rise in weir level elevation. The overflow chamber at the fourth regulator merges separate storm and sanitary wastes from a 45 acre area with two combined sewer trunk sewers and two wet weather overflow conduits from an upstream relief point. The 45 acre area was recently separated and is served by a 30" trunk storm drain and an 18" trunk sanitary sewer. The chamber acts as an effective mixing tank for all of the influent waste streams prior to discharging through an 18" dry weather outlet and a 51" wet weather relief outlet. The net result of the junction chamber as it presently exists, is to re-combine the separate sanitary and storm drainage from the 45 acre catchment area with the other combined sewer inputs. The envisioned alteration to the junction chamber is to install steel I-beams across the chamber and extend the 30" drain directly across the chamber and install a side weir for discharge into the 51" wet weather overflow conduit. This recommendation was determined only after repeated observations and careful scrutiny of outdated construction drawings because most of the inlet pipes in the chamber are submerged.

It was also determined during the intensive field surveys that there are two storm drainage subsystems within the combined sewer study area that could be rerouted to minimize the amount of stormwater flow entering the sewerage system. The first area is a 16 acre parcel served by separate storm and sanitary systems. The total flow for this area is connected into the downstream combined sewer system. All of the stormwater currently flowing from this area could be rerouted to tie into an existing trunk storm drain discharging directly to the Nashua River. The required piping would be approximately 200' in length and the area through which the pipe would be constructed is a vacant lot lying between the adjacent streets. The second area is a two acre parking lot whose drainage is then piped into a nearby combined sewer. If the storm drain connection was extended 20' beyond the combined sewer connection, natural drainage would convey the parking lot runoff down an enbankment to the river. There are no buildings at that point due to the proximity of the river and the steepness of the embankment, hence it would require only the crudest of chutes and/or rip-rap to provide relief from the system.

Several corrections for illegal dry weather sewage discharges into overflow conduits were also developed. Although not part of the study effort a number of other illegal dry weather sewage overflows were identified in other areas of the City. The on-going infiltration/inflow study will focus on the correction of these problems.

Sewer Flushing Considerations. Deposition of sewage solids during dry weather in combined sewer systems has long been recognized as a major contributor to "first-flush" phenomena occurring during wet weather runoff periods. The magnitude of these loadings during runoff periods has been estimated to range up to 30 percent of the total daily dry weather sewage loadings.

Dry weather deposition is a potential problem in combined sewers in that the pipes are oversized to convey both dry and wet weather flows. During dry weather sewage flow is low, resulting in the deposition of sewage solids. These deposited pollutant loads are scoured during wet weather periods and may be carried to the Nashua River if overflows occur. Flushing of these materials during dry weather periods would help to mitigate these "first flush" shock pollutant loadings.

The 12 miles of combined sewer pipe in the prototype study area were represented by 322 manhole-to-manhole segments. Physical characteristics of each of the collection pipe segments such as segment length, diameter and slope were tabulated, data processed and utilized in a computerized network procedure for estimating daily dry weather pollutant deposition loadings per segment.

It is estimated that the total daily deposition of sewage solids within this system is 71.6 lb/day. Roughly half of this loading can be attributed to 51 out of the 322 segments, or 16 percent of the total. All of these segments are small upstream segments with diameters ranging from 8" to 15" pipe. The relative fraction of the daily overall deposition fraction of the total system input is low, 4.1%, in comparison to results from other studies where deposition rates were estimated to be as high as 30% with 10-15% being average. This low number is obviously accounted for by the severe topographic relief in the City.

The 51 segments identified as moderate/heavy deposition segments were physically surveyed for feasibility and ease in flushing using manual means and external flush water sources. All segments were inspected for physical suitability, that is, whether the manholes were excessively deep and/or whether the manholes and channel bottoms could sustain flushing injection. A segment was not eliminated if no channel bottom existed. These costs are nominal and were included in the flushing program costs. The segments were also reviewed with respect to traffic and congestion. Finally, the segments were evaluated for proximity to fire hydrants, as a source of flush waters. An arbitrary criteria of 2000 feet was established as a cut-off distance for either fire hose-connected flushing or filling a tanker and then moving to the injection site.

A total of 46 segments were deemed suitable for flushing by manual means. Roughly 33 lbs/day would be removed by a flushing program for the 46 segments. Overview details are given in Table 5-1. Roughly half of the daily dry weather deposition loading could be feasibly removed by a low level labor-intensive flushing program. It is interesting to note that the City of Fitchburg Sewer Department routinely flushed troublesome sewer segments on the average of once a week nearly a decade ago. Limited public works department funds have since curtailed this activity.

Subsystem	A	B	C	D	E	F
1	176	42.95	28	18.32	26	17.36
2	53	12.56	11	7.15	11	7.15
3	93	16.12	12	9.66	9	8.53
Total	322	71.63	51	35.13	46	33.04

Legend: A - Number of manhole-to-manhole segments.
B - Estimated daily solids deposition load (lb/day).
C - Number of moderate/heavy deposition segments.
D - Total load attributable to moderate/heavy deposition segments (lb/day).
E - Number of segments noted in C suitable for sewer flushing after field inspections.
F - Total deposition load removable by sewer flushing.

Table 5-1. SUMMARY OF SEWER FLUSHING POTENTIAL

The manual method of flushing was recommended on the basis of conclusions drawn from an operational study of sewer flushing practices. EEA and Northeastern University (NU), Boston, have completed a massive field-oriented sewer flushing study (EPA Research Grant No. R-804578) aimed at assessing the pollutant removal effectiveness of various methods of flushing small combined sewer collection system laterals. The field results indicate that extremely high pollutant removal efficiencies are possible over a single segment (85-95%). Removal efficiencies over several segments using a single flush point (600-800 feet) result in favorable removals (75-85%) depending on the segment and the pollutant. The average flush water volumes for all experiments is 350 gallons. All manual methods appear favorable. The easiest and most feasible method at this point in time appears to be injection by tanker and/or from a near-by hydrant. The state of the art with respect to operational automated flushing methods, equipment, sensing interfaces, etc. has not been demonstrated at this point in time. It is for this reason that only manual flushing methods are recommended. Their effectiveness and ease in operation has been adequately demonstrated by the EEA/NU research program.

Street Sweeping Considerations. Street sweeping is an extremely public works oriented activity aimed at keeping streets visually clear, preventing the filling and clogging of catchbasins and drainage lines, and removing to some degree, the accumulation of pollutants depositing on street surfaces during dry weather between storm events. The quantification of street sweeping cleaning effectiveness depends upon a host of factors such as sweeping frequency, the number of passes per sweeping, the type of equipment, the condition of road surfaces, the existance of curbs, street surface density and material, and adequate and enforced off-street parking restriction ordinances.

The City of Fitchburg streets, within the 208 combined sewer study area were carefully surveyed for street sweeping feasibility. Roughly 64% of all streets within the study area were inspected. Four criteria were used by the field crews: 1) existance of curbs; 2) condition of street surface; 3) street grade; and 4) degree of congestion and traffic. If the first three criteria were favorable for a given street (block to block) and the traffic was reasonable, the street segment was deemed suitable. If a street was scored favorable for the first three criteria but the traffic and/or congestion was heavy or parking was poor, the street was considered unfavorable. If any one of the three criteria were scored poorly for a given street, it was considered unsuitable.

The fraction of streets considered suitable for street sweeping within the study area is 36.3 percent. The percentage of streets termed unfavorably is considerable and represents roughly a quarter of all streets. It is very likely that a portion of these streets would be suitable for flushing during off-hours and/or during weekends. The fraction of streets considered unsuitable is 44.8 percent and represents a sizeable area where surface and street accumulations are in some sense not controllable.

Inflow Correction Considerations. Illegal downspout connections from households and commercial/industrial buildings into sanitary sewerage systems is a frequently occurring problem in the older communities throughout the country. Most of the original sewerage systems were combined and connection of downspouts was acceptable. Sewer separation brought about tighter restrictions in building and plumbing codes forbidding the connection of downspouts into the sewers. It is not uncommon, however, to find residential areas in separated areas with a blend of legal and illegal connections. Assuming that half of the residences have illegal downspout connections in separated areas results in a considerable stormwater inflow problem. A separate system would therefore act as a combined sewer during rainfall events.

The degree and extent of illegal downspout connections in the study area was investigated in this study since the City of Fitchburg has an on-going program to separate small pockets or "slivers" of combined sewered streets in the future. This information was also of use to the City in providing baseline information for the on-going city-wide infiltration/inflow study.

In this study the field crews selected four representative areas having different land use characteristics within the study area for "wind shield" inspections of illegal downspout connections. The purpose of the surveys was to primarily identify potential problems relegating the actual determination of critical illegal downspout connections to the on-going infiltration/inflow study. The survey results indicate that the estimated number of illegal connections per acre for the residential areas range from .3 to 1 depending on the relative land use density. The survey also showed that there are a number of commercial buildings near the river with flat roofs with central roof drainage systems suspected of direct connection into the sewer systems. Revamping these drains in the commercial district could significantly reduce the stormwater inflow.

Off-Line Storage Potential Considerations. A preliminary feasibility study of storage potential at a number of sites throughout the combined sewer study area was prepared for considering structural alternatives. It was assumed that storage could be developed in two ways: small upstream modules located along collection branches meant to capture highly concentrated upstream loads during storm periods and large downstream facilities for capturing overflows from entire collections networks.

Initially the special collection system "flow-line" profile maps were used to determining potential upstream storage locations. These drawings were prepared by cataloging the entire network of collection systems, pipe by pipe, from detailed analyses of City of Fitchburg sewer maps. These drawings present a pictorial view of the slopes for an entire collection system. The relative elevation of all pipes within any collection subsystem can be easily ascertained. The three collection systems in the study area were visually scanned for locating subareas for potential upstream storage module locations. Such sites are ideally located at the end of collection subareas near a trunk sewer. A site near the intersection of the trunk sewer having a large elevation drop, say, 10 feet, was then established. Such a location is ideal for an upstream storage module. The inlet is located at a high point and the discharge outlet at a low point such that gravity control devices can be used and piping kept to a minimum. Downstream locations near regulators were identified in the same manner. A list of potential sites was generated.

The field inspection program determined the practical feasibility of constructing storage modules. Parking lots, vacant lots, adjacent parks, wide median strips and other open areas were considered as prime choices. There were other considerations investigated in the review of these areas such as the disruptive effects on traffic/business during construction and the mechanical practicality of gravity feed to a storage tank and drain by gravity with minimal amount of piping.

There are 13 possible locations for considering storage to capture wet weather combined sewer flows. The amount of storage that could be considered in each of the three subsystems is ample and well beyond the requirements for capturing portions and/or the entire overflow volumes for slight to moderate level storm events.

Combined Sewer Overflow Measurements. During the spring of 1977 a measurement program was initiated to monitor the three combined sewer overflow points located within the first collection subsystem of the study area. Discharges were monitored for three storm events for all three sites. Pollutant characteristics were monitored at one location for the three storm events. The fourth control point is the complex junction chamber mentioned earlier. This location is the effective control point for the remaining two subsystems. This location was not monitored because of the complexity of the chamber. Two separate storm sewer catchment areas tributary to this location were monitored for flow during this period. This information was used to establish reasonable parameter values of a combined sewer runoff simulation model of the three subsystems within the study area.

One important observation noted from the analysis of the overflow pollutographs was that the response times for "first-flush" to occur and pass the monitoring sites were extremely rapid, on the order of 10 to 20 minutes for the storms that were observed. Analysis of the hydraulic profiles at the three regulators indicated that raising the overflow weir levels would permit much of the "first-flush" loadings to be transported to the treatment plant headworks. This observation somewhat reduced the technical attractiveness of sewer flushing and street sweeping since the treatment plant receiving these wastes is presently underutilized. The City was nevertheless concerned over transporting and treating additional wet weather flows since advanced forms of treatment would soon be required. This position tempered the final selection of recommended management alternatives in that a concerted effort was made to maintain the present wet weather flows regime to the treatment plant. This balance was accomplished by approximating the additional combined sewer overflow volumes captured by raising overflow weir elevations at three regulators in one portion of the study area with the elimination of several direct stormwater drainage inputs tributary to the fourth regulator.

Combined Sewer Runoff Simulation Model. A combined sewer runoff/overflow simulation model was developed for each of the three subsystems within the study area. This model was used to determine the pollutant emission reduction effectiveness of various alternative combined sewer control mixes. Estimates of Total Suspended Solids, BOD, TKN and TP overflow loadings per storm were computed by the model. A synthetic unit hydrograph approach was used to compute wet weather hydrographs. These hydrographs were defined at fine time intervals, smaller than the lag time of the peak flow of the particular catchment area. The stage discharge relationships developed for the overflow conduits and downstream sewer conduits were approximated by third order polynomials. A Newton-Raphson procedure was used to balance head and discharge levels in the computation of the overflow hydrographs at the regulators. Surface pollutant accumulations were computed in a similar manner as in model STORM. Algorithms were incorporated to keep track of the succession of storm washoff, street sweeping and sewer flushing events along time. Control options included modification of the regulator hydraulic characteristics, elimination of storm drainage inputs, inflow reduction, street sweeping, sewer flushing and storage. Levels of control used in the model for each of these options were determined from the feasibility analyses previously described.

Control Option Costs. The four regulator modifications and the two storm drain re-piping alterations totalled $26,500. The downspout inflow correction program including legal costs was estimated at $312,000. This cost assumes 40 connections costing $5000 each for commercial buildings and 70 household connections costing $600 each.

Sewer flushing program costs for a daily operation are shown in Table 5-2. Two alternative methods are considered for the 46 segments deemed suitable for flushing. The first alternative consists of utilizing automatic flushing modules programmed to flush daily. EEA has successfully fabricated, installed and operated an inexpensive air-cylinder driven back-up and release gated device in Dorchester as part of the R&D sewer flushing research study cited earlier. Further testing of such devices are necessary before recommendation

Number of Segments: 46

<u>Daily Flushing Program</u>

Alternative 1 - Automatic Flushing Module Operation

<u>First Costs:</u>	
- Site preparation (grout manhole, fix base, clean segment)	$1,500/segment
- Fabricate & install air-operated module	7,500
	$9,000/segment
<u>Annual Operational Costs (Total Program):</u>	
- 3 men @ $15,000/yr.	45,000
- Truck rental, gas, insurance	8,000
- Equipment component replacement $300/yr/module	14,700
- Sewer cleaning contingency	10,000
- Water	2,300
	$80,000
Cost/module/yr = $1,630	
Present value/segment = $18,745	
<u>Total Present Value Cost (Segment)</u>	
- First costs	9,000
- O&M costs	18,745
	$27,745

Alternative 2: Manual Flushing Mode

<u>First Costs:</u>	
- 3 outfitted water tankers @ 18,000 or $1100/segment	54,000
<u>Annual Operational Costs (Total Program)</u>	
- 6 men @ 15,000/yr.	90,000
- Insurance, gas, maintenance	3,000
- Water	2,000
	$95,000
Cost/segment/yr. = $1938	
Present Value/segment = $22,287	
<u>Total Present Value Cost/Segment</u>	
- Site Preparation	1,500
- First Costs	1,100
- O&M	22,290
	$24,880
Use $25,000/segment	

Total Present Value Costs: $1,150,000

TABLE 5-2. SEWER FLUSHING PROGRAM COSTS
(Daily Operation)

for general application can be made because no full scale network of automated sewer flushing modules has ever been tried in practice. Costs for this type of operation are nevertheless presented for comparison with a manual operation using water tankers.

Initial costs including $1500 per segment for site preparation work entailing repairing and in some cases, installing manhole tables, grouting the flush manhole walls, mechanical cleaning and jetting of the flush segment and other miscellaneous contingency items. Fabrication and installation of the devices is estimated at $7,500 each. A three man maintenance crew is assumed to ensure proper operation of the configuration of the 46 modules. Equipment component replacement costs of $300 per module per year are also assumed. In addition, a contingency factor of $10,000 per year is included to cover possible blockages, malfunctions etc. The estimated annual budget for this operation is $80,000. The O&M cost per module per year is $1630. The present value of O&M cost per module assuming a 20 year discount period at 8 percent interest is $18,745. Total present value cost per module is $27,745.

Alternative program costs consisting of manually flushing 46 segments using three water-tankers and a six man labor force are shown in the second half of Table 2. Total present value costs for this alternative are $24,880 per segment. Present value sewer flushing program costs for the study area is $1,226,000.

Sewer flushing program costs for a manual operation with a weekly frequency for flushing was also estimated. It is assumed that a single crew can complete the flushing circuit within a one week period. The total present value costs over a twenty year discount period for this option is $863,000.

Street sweeping program costs were computed as follows. Three sweepers are assumed for the entire study area. Single pass operation on a weekly schedule was assumed. An average cost of $35,000 per sweeper is used to estimate the capital outlay. This cost represents reasonable average of current available technology. O&M costs are estimated at $80,000 per year. Total present value street sweeping costs for the area is $1,024,500.

Off-line storage costs were computed using an average value of $3 per gallon for capital costs. Annual O&M costs were assumed to be 10 percent of the first cost total.

Assessment of Alternative Control Programs. A total of 24 alternative management plans for the study area were analyzed using the computer simulation model for estimating the relative reduction of overflow volumes and pollutant emissions. These plans consisted of various control option mixes in the areas tributory to the four regulators in the study area. The plans considered were developed by additive combinations of the various control options. Each option was not tested for its individual effect nor were all possible combinations considered.

The simulation analysis used as input 73 rainfall events occurring over one calendar year that were measured at a primary gage in the study area. The period chosen for the analysis represented an average year in terms of storm duration, intensity and antecedent dry periods. The simulation results for each storm were then summed and average results were reported Under present conditions, the annual combined sewer overflow volumes from the four control points in the study area is 166.9 x 10^5 cubic feet. The estimated number of overflows for three of the four regulators range from 23 to 55 per year depending on the location. Overflows occur 95 percent of the time at the major junction chamber on the opposite side of the study area. Roughly 43 x 10^5 cubic feet of overflow at this chamber is attributable to the direct storm drainage inputs. For low intensity storms in which no overflows occur, this storm drainage is conveyed to the treatment plant. Annual combined sewer emissions for all four overflow points is estimated at 10,530 lb-BOD/year.

Summary overflow and pollutant reduction results and present value costs of four major alternative plans are shown in Table 5-3. These results capsulate the analysis of the 24 alternative program mixes considered. Plan A consists of the regulator modification and storm drainage re-piping programs. Plan B adds the sewer flushing program (daily operation) and the street sweeping program (weekly schedule) to the elements cited for plan A. In plan C the inflow control program is added to plan B. Finally, in plan D 43,000 cubic feet of storage at 4 sites is added to the plan C control elements. The following information is provided for each plan: the reduction of combined sewer overflows per year, the percentage reduction of overflows from base-line conditions, the annual emission of BOD loadings in the combined sewer overflows, the percent reduction of BOD emissions from base-line conditions and the total present value cost using a 20 year discount period and an 8% interest rate.

	Plan			
	A	B	C	D
Annual Combined Sewer Overflow Volume Reduced (10^5 cf)	68.24	68.24	78.24	92.97
Percentage Overflow Reduction from Present Conditions	40.9	40.9	46.9	55.7
Annual Combined Sewer BOD Loading Reduction (lb/year)	4624	5642	6135	7031
Percentage BOD Loading Reduction from Present Conditions	43.9	53.6	58.3	66.8
Present Value Costs (20 yr, 8%)	$26,500	$2,201,000	$2,513,000	$4,678,000

Plan A: Regulator Modification & Storm Drainage Revisions
Plan B: Plan A + Sewer Flushing + Street Sweeping
Plan C: Plan B + Inflow Correction   Plan D: Plan C + 43,000 cf Storage

TABLE 5-3. OVERVIEW OF OVERFLOW/POLLUTANT REDUCTIONS AND COSTS OF FOUR ALTERNATIVE COMBINED SEWER MANAGEMENT PLANS

The results in Table 3 show that plan A is the superior cost-effective alternative. Reductions of 41 percent of the annual combined sewer overflow volumes and 44 percent of the annual combined sewer BOD emissions are expected. Similar reductions for the other pollutants were noted for the four plans considered. Further overflow and pollutant loading reductions are possible but are disproportionate to the estimated costs. Approximately 70% reduction of combined sewer pollutant emissions can be achieved under Plan D, but the present value costs are nearly 4.7 million dollars.

Under plan A the number of overflows for three of the four regulators is expected to reduce from 23 to 55 per year down to 5 to 14 per year. Roughly 18 x 10^5 cubic feet of combined sewer overflow would be diverted from these three overflow points to the downstream waste treatment plant instead of discharge to the Nashua River. The number of overflows would not be significantly reduced at the fourth regulator, the major junction chamber. The volumes of combined sewer overflows would, however, be considerably reduced. Approximately 50 x 10^5 cubic feet per year of storm drainage would directly discharge into the overflow conduit instead of being mixed with sewage. Furthermore 7 x 10^5 cubic feet per year of storm drainage occurring during low level storms would not be discharged to the treatment plant. The net additional discharge to the treatment plant would be 11 x 10^5 cubic feet per year.

The alternative combined sewer management plans were then assessed in view of wet weather water quality impact analyses incorporating point sources and major non-point sources in the area including storm drainage from the balance of the City. The storm drainage emissions from the rest of the City are an order of magnitude greater than the combined sewered loadings. A "rough-cut" feasibility study for controlling storm drainage pollutant loadings throughout the City showed that less than 10 percent of the loadings could be realistically reduced and only at exhorbatant costs. Control of storm drainage pollutant loadings in the balance of the City was considered infeasible.

Water Quality impact analyses were performed using the steady-state model of the Nashua River supplied by the Division of Water Pollution Control, State of Massachusetts, (DWPC). The physical parameters of the model originally calibrated by DWPC for dry weather conditions required modification to define time of travel, depth of flow and reaeration under wet weather mean flow conditions. The uncontrolled future (1995) non-point source wet weather loads for the entire 208 area were combined with the loadings from the alternative dry weather control programs and were used in the DWPC Nashua River model for assessing wet weather water quality impacts. The impact analysis indicated that on the average, there would be no dissolved oxygen water quality problems during wet weather runoff.

These results collectively defined the basis used to establish the level of combined sewer overflow abatement in the study area. It was therefore mutually agreed by DWPC and the 208 planning agency that the combined sewer management plan for the study area should reduce in a cost-effective manner, the combined sewer overflow levels and pollutant emissions only to a reasonable extent. High levels of combined sewer abatement for the study area are not warrented since the separated storm sewer loadings are extremely large in comparison to the uncontrolled (present condition) combined sewer loadings. The combined sewer management control plan A, consisting of the regulator modification and storm drainage re-piping programs, was therefore deemed the preferred plan and was recommended in the final 208 report.

CONCLUSION

Overall costs of various alternative control programs which were carefully developed from field feasibility studies together with their attendant pollutant reductions were analyzed to determine the most favorable cost-effective approach. The preferred combined sewer management plan for the study area is estimated to cost roughly $27,000 and will eliminate approximately half of the combined sewer overflows and pollutant loadings while only slightly increasing the hydraulic load to the Fitchburg East Waste Water Treatment Plant.

The preferred plan consisted of minor repairs to four overflow structures and several small alterations of storm sewer piping. This plan is far more economical than the nominal "Best Management Practices" of street sweeping, sewer flushing, catchbasin cleaning, etc. and several orders of magnitude cheaper than structurally oriented programs.

This plan was developed on the a *priori* premise that significant control of combined sewer overflows can in fact be accomplished by a purposeful effort in restoring the condition of the existing system and jointly maximizing any system control. The work program for the combined sewer study included atypical detailed field inspection surveys that are not performed in most 208 studies. The final results, however, speak clearly to the point that even before new and/or old technologies for combined sewer control can be employed, a rational "first-principles" intensive effort at carefully understanding the complex details of collection systems must be accomplished. This task is by no means simply, particularly for old complicated combined sewerage systems like Fitchburg, but the results are both responsive to the needs of the community and to our nation's resources, particularly the federal commitment to spend large sums of money in the near future for combined sewer overflow control.

ACKNOWLEDGMENTS

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Mr. Celso Queiroz, environmental systems analysis (EEA), developed and utilized the combined sewer simulation model in the assessment of alternatives.
Mr. Gerald Aronson, Senior Project Engineer (EEA) supervised the mapping and field measurement program. Dr. William Pisano was the project manager for EEA and prepared the final report.

STATISTICAL CHARACTERIZATION OF RUNOFF LOADING RATES AND COST FUNCTIONS OF CONTROL MEASURES

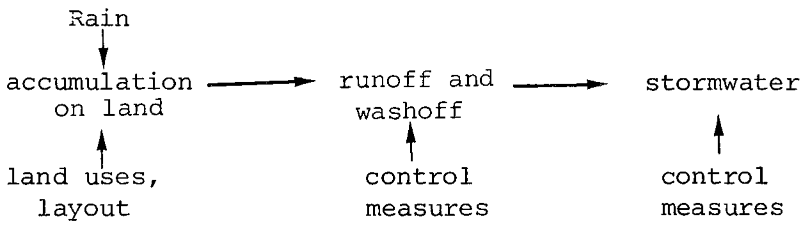
by

R. Berwick, J. Kühner, D. Luecke, M. Shapiro*

INTRODUCTION

An on-going Environmental Protection Agency (EPA) study (EPA Research Grant No. R-805238) has been investigating the impact of new residential development on stormwater quantity and quality. The primary concern of the study is the feasibility of estimating useful relationships between precipitation and runoff given various on-site control measures and their associated capital and operating costs. If we look at a typical picture of the stormwater runoff process (Figure 6-1), we see

FIGURE 6-1. STORMWATER RUNOFF PROCESS



that there are several components that determine the ultimate stormwater quantity and quality. Most of these remain to be usefully described so that the impact of control measures can be easily discerned.

We must start, then, at the beginning of the process with a better characterization of accumulation rates on the land. How do these rates vary with different land uses, climates, street cleaning? What is the uncertainty associated with estimating residuals accumulation? The first section of the paper addresses these questions by reanalyzing a large body of existing accumulation data.

* Meta Systems Inc. Cambridge, Massachusetts 01238

At the other end of the runoff picture is the effect of the measures. Even if we could predict exactly the quantity and quality of runoff from a given storm, the selection of structural (porous pavement, swales, . . .) and nonstructural (vegetative cover) controls is guided by their unit costs. In many cases cost estimates are as uncertain as measures of stormwater control effectiveness; this makes the selection of control alternatives doubly unsure. The second part of the paper develops regression models based on general runoff, flow, and cost relationships. The resulting equations may be used to estimate control costs as a function of the natural physical features of an area, level of development, and precipitation.

ANALYSIS OF DATA FROM PREVIOUS STUDIES

Two basic strategies have been adopted in gathering data for use in quantifying urban stormwater pollutant loadings. One strategy has focused on measuring the accumulation and composition of dust and dirt on street surfaces. Another has focused on "end-of-pipe" or waterway measurements of flow and concentration during storm periods. While data from both types of studies have been combined and analyzed (URS, 1974), it is unfortunate that only a few ongoing studies (Pitt, 1974; Woodward-Clyde Consultants, 1977; Envirex-DOT, 1977) are being performed in which both types of measurements are employed simultaneously in the same study area. This would provide a basis for development of a mass balance for quantitative comparisons of the two measurement strategies.

Street accumulation measurements have been taken to provide bases for evaluating street sweeper effectiveness and for calibrating the accumulation/washoff functions typically included in various urban runoff models. These studies are plagued by the difficulties involved in estimating the desired deposition rates from street solids measurements, in the presence of a variety of wet- and dry-weather deposition and removal mechanisms. Uniform sampling and data reduction procedures for measuring street solids have not been used. Pollutant loadings scoured from off-street surfaces during rainstorms and possible non-conservative behavior of some pollutants in stormwater collection and transport systems are both ignored when street solids measurements are used to estimate loadings to urban waterways.

Measurement of flow and concentration in collection systems or waterways during storms provides a more direct basis for estimation of loadings. Difficulties are associated with the temporal variability of flow and concentration typical of urban storm events. Such variability can cause sampling, measurement, and data interpretation problems. Possible seasonal or longer term effects suggest that such studies should be carried out for long periods to provide a basis for estimating average loadings. Envirex, Inc. (for the Department of Transportation) is conducting long-term, simultaneous monitoring of this kind.

In general, however, most investigations of urban pollutant loadings from nonpoint sources are considered to have given disappointing results to date. For example, Singh concluded at the 1977 ASCE meeting:

> The main objective of this study was to determine the rates at which solids accumulate on street surfaces.... Initial efforts in fitting simple conceptual models and use of regression analysis were unsuccessful due to extreme scatter in data....

The misconception is, one can argue, to look at "extreme scatter in data" as a nuisance rather than the interesting property of loading rates. Why is there such scatter? Or is there really such scatter? Further, most investigators have concentrated on developing regression models to describe loading rates and have largely commented on their "failure." For example, Singh (1977), as well as Colston (1974), Whipple (1976), and Hammer (1976), find no correlation between elapsed days of accumulation and pollutant concentrations; Sutherland and McCuen (1975) claim the reverse. In part, these contradictions are due to sloppiness in the definition of "concentrations" and lack of consideration of collinear effects between independent variables; but there is also real uncertainty in the physical system being modeled. Given the current lack of a thorough understanding of the accumulation/washoff processes, any kind of straightforward regression technique is doomed to failure. Therefore, methods designed to discover something about loading rates should be based on the following two premises:

(1) Exploratory, rather than predictive data analysis; and
(2) Stochastic, rather than deterministic, model building.

The exploratory process (partially) relieves us from the burden of having a specific explanatory model of the data at hand; the stochastic part is similar -- it throws everything we cannot specifically describe into a lumped box of noise. Actually, the two premises are not distinct; we use either one to guide the other.

EXPLORATORY DATA ANALYSIS

The commentary on loading rates (e.g., Sartor and Boyd, 1972; Hammer, 1976; URS, 1974) invariably mentions the extreme range in urban loading rates. As a typical example, the URS data for residential loading rates (lbs/curb-mile-day) shows a range of 8-770, with a standard deviation of 195. Indeed, for most categories of land uses, climates, and traffic patterns, the coefficient of variation (standard deviation/mean) is greater than one, indicating large variability. Does this mean that the situation is so random as to be unusable? More precisely; is this why regression methods fail? This can be tested by using the URS data which are a summary of all major studies on loading rates done before 1974. All raw data are normalized to lbs/curb-mile-day and we organize them in stem-and-leaf displays.

The basic purpose of such displays* is to arrange raw data into roughly numerical order -- like a histogram -- so that one can easily

---

* The reference for this method and the two way table procedure presented below is Tukey, J.W., Exploratory Data Analysis, Addison-Wesley, 1977.

answer questions like: what is the largest value? the smallest? what does the distribution of values look like? Unlike a histogram, however, the display retains some of the identity of the original data by using the actual digits of the data values to construct the display. This makes it easier to see which data value is located where in the histogram.

How then do we construct the display? We first scan the list of the original data (see Figure 6-2) to determine a suitable scale for the values; in this case, units of tens seem appropriate. Taking the first data value: 400 lbs/curb/mile/day, we cut it to units of tens (the ones place is truncated). Four hundred becomes 40 (tens). Next, 40 is separated into two parts: the right-most digit 0, and the rest, 4. The 0 part is called the leaf (because it hangs out from the left-hand-side); 4, the stem. We show the separation of the stem from the leaf by drawing a vertical line between them: 4**|0, the asterisks indicate that there are really two digits in the leaf (400 really could become 4|00), but we show only one of them -- the tens digit. The display is built by writing down all the stems on the left and the leaves on the right -- see Figure 6-3 which shows the step-by-step construction of the display. At each step, the number placed into the display is marked with an arrow. The next data value is 600; it becomes 6**|0 and step 2 of Figure 6-3 shows its placement on the display. And so it goes; value 390 becomes 3**|9 (step 3); 210 becomes 2**|1; 170 becomes 1**|7; 19 becomes 0**|1 32 goes on the same line as the latter and we have 0**|13.

When there are too many values to fit on one line, stems can be separated. For example, we could have one stem for leaves 0, 1, 2, 3, and 4 and the other for 5, 6, 7, 8, and 9. If we were to do this for the data points represented by 0**|138,

we have

```
  .|8
0**|13
```

Figure 6-4 gives the completed stem-and-leaf display of the residential loading rates. In the spirit of exploratory analysis, we make two remarks about the figure:

1. The data are skewed; this immediately makes the use of the standard deviation as a confidence-interval estimate (as was done in Singh, 1977), or of regression analysis without data transformation, suspicious. Specifically the shape is suggestive of a log-normal distribution (see below).

2. If we calculate the median of the data (to avoid the skewness problem), it is 70. URS reported a mean of 149 -- revealing the skew of the data. (As a comparison, the geometric mean -- essentially a log transform of the data -- gives a value of 84.) Reporting only this arithmetic mean plus a standard deviation of 195, is misleading: it says that about 67 percent of the data was between 149 ± 195. In fact, we can do much better than this: if we calculate the hinges (halfway between the median and each extreme value), we get values of 20 and 185.5 which tells us that about

one-half of all data falls within this range. In general, an analysis by medians is much less sensitive (i.e., the analysis is robust) to extreme fluctuations in data; a simple arithmetic mean can be extremely misleading. Notice also, that the spread (equals difference between hinge values), is 165 close to the URS standard deviation of 195.

FIGURE 6-2. RAW DATA: RESIDENTIAL LOADING RATES URS LBS/CURB-MILE/DAY

400	600	390	210	170	019	032
121	148	081	062	121	135	148
019	020	096	153	060	022	
032	035	024	033	041	028	
070	092	2700	690	260	860	
220	372	659	418	70	85	24
77	238	18	34	103	93	40
770	950	205	950	100	67	93
33	11	8	3	295	31	165
13	69	17	27	18	6	8
39	45	22	12			

Number of observations = 71

FIGURE 6-3. STEPS IN CONSTRUCTING STEM-AND-LEAF DISPLAY

Step 1		Step 2		Step 3		Step 10	
6**		6**	0	6**	0	6**	0
5		5		5		5	
4**	0	4**	0	4**	0	4**	0
3		3		3	9	3	9
2**		2**		2**		2**	1
1		1		1		1	247
0**		0**		0**		0**	138
(value placed = 400)		(value placed = 600)		(value placed = 390)		(value placed = 081)	

Figure 6-4

Residential Loading Rates
(lbs/curb-mile-day)
(URS, 1974)

27**	0	(Bucyrus, Ohio)

10**	
	55
9**	
•	6
8**	
•	7
7**	
•	95
6**	0
•	
5**	
•	
4**	01
•	97
3**	
•	69
2**	1230
•	756
1**	2423400
•	8696797879696
0**	131223323422134210031121003421

(Note: stems separated into two pieces.)

Results: upper hinge: 185
median: 70
lower hinge: 20
spread: 165

Note: median: the data value half-way in from either end; half the data lies above, or below this data point.

hinge: upper (lower) data value -- 75% (25%) of data are below this value.

spread: difference between upper and lower hinges, i.e., contains about one-half of the data.

Given the results of this simple analysis, one should be suspicious about all of the reported summary data on loading rates. We will, however, postpone the analysis of other loading data for now because we want to proceed to investigate the log-normal transformation of the data of the residential areas.

If we take logarithms of the loading rates, the resulting stem-and-leaf display is more normal-looking (though still slightly skewed) (Figure 6-5). Analysis of the data divided into classes, such as by climate (northeast, southeast, southwest, northwest), traffic (<500 average daily traffic volume (ADT); 500-1500 ADT; 1500-5000 ADT; >5000 ADT), land use (commercial, light industrial) reveals the same general results:

(1) The reported arithmetic means are skewed to the high side.
(2) The data are close to log-normal.

Now, what does a log-normal distribution suggest? The standard explanation is this: suppose the increase in loading, Δx, is some multiple, k, of the increase in a large number of other variables (z_i), most of which we don't know about, i.e., $\Delta x = kz_i x$; and suppose the z_i's are random variables. Then the sum of the z's is the sum of these changes in x:

$$z = \Sigma z_i = \frac{1}{k} \int_{x_1}^{x} \frac{dx}{x} = \frac{1}{k} \log \frac{x}{x_1}$$

The random variable z, being the sum of many random variables is by the central limit theorem, normally distributed. So x is log-normal -- reasonable behavior for loading rates. Put another way, once we have a given rate of load, say x_0, it seems likely that those factors that made it high (say, a surrogate like employment density) are associated with a number of other factors that will only increment the loading. Like many other regional influences, effects are multiplicative. (A further speculation: loading rates are area effects -- population, employment, etc. -- funneled into a more or less linear collection system (a street) and, hence, increase multiplicatively.) It would seem that the loading data are not so inexplicably "extreme" as stated. Rather, it follows from what one would expect if there were many random variables contributing to their ultimate values.

The kind of distribution we have described indicates that we must model a lot of what is going on in the loading process as the combined effect of many factors -- a black box. For the process we do know, however, other methods can be used. One approach is to dissaggregate the data into categories -- as in the URS study. As mentioned in URS, the scatter of the data results in a weak relationship between categories such as land use, climate and the loading rate. Also, the URS study used t-tests to try to separate categories; but, as we have seen, without first transforming the skewed data, this approach is weak.

Another approach is to use medians instead of a least squares analysis. With medians we can explore the data in order to describe observed differences

Figure 6-5

Log_{10} (Residential Loading Rates)
(lbs/curb-mile-day)
(URS, 1974)

```
loading rate x 100
     |
 34* | 3
     |
 32* |
     |
 30* |
     | 388
 28* | 429
     | 8
 26* | 02
     | 97
 24* | 17
     | 2481
 22* | 32
     | 7378
 20* | 8810
     | 186377
 18* | 55934
     | 98
 16* | 105
     | 1142329
 14* | 593
     | 04884
 12* | 88636
     | 1
 10* | 48
     | 00
  8* |
     | 8
  6* |
     |
  4* | 8
```

<u>Results</u>:	<u>log scale</u>	<u>original</u>
upper hinge:	2.27	(187.5)
median:	1.79	(70)
lower hinge:	1.43	(27)

between the categories used by URS. Medians are insensitive to extreme fluctuations in the data. To perform the analysis we disaggregate the data into a model of the form:

LOADING = common value (over all categories)	+ land use effect
	+ traffic effect
	+ climate effect
	+ residual

where the residual is a normally distributed source of error. This will enable us to place confidence limits on the estimated loading for a given area. We are in effect making the categories into "dummy variables," using the standard regression parlance, but we are NOT using least squares.* We present, as an illustration of the technique, "two-way" tables in log scale; the first is median loading rate by climate and land use, the second, median loading rate by traffic volume and land use.†

The model is:

LOADING = common value + row effect + column effect + residual
where residual = value left in a cell.

We begin by looking at data in Table 6-1.

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* One problem is that there are scant data for some categories -- for example, northwest region with traffic less than 500/day. No matter; a resistant analysis proceeds with the data available, using medians to estimate the missing data. However, it is a good idea to keep this in mind when attempting to <u>predict</u> loadings.

† The classic statistical approach to fitting a linear regression model to observed data is to use a "least squares" criterion for the fit. If we displayed the data in categories (as in Tables 6-1 and -2), we could follow this technique by subtracting out the row and column <u>means</u> for each category, and arriving at a formula like

fit = grand mean + row effect + column effect + residual

It can be formally shown that the linear decomposition in a model of this kind is equivalent to a least squares analysis.

However, least squares is ineffective with highly variable data, because (as a glance at the least squares derivation shows) points farther away from the grand mean ($\bar{X}$) contribute more to the placement of the least squares line than points closer to the mean (the exact factor is $\partial f(X_i - \bar{X})/\partial X$ where X_i is the particular observation and $\bar{X}$ the mean). But why should "untypical" points contribute more to where the line goes? Shouldn't it be rather the "typical" points that determine the line? In the face of these arguments, it has been suggested to subtract out <u>medians</u> instead of means, thus providing an inherently "typical" value, not one influenced by extreme values. This gives us Tables 4 and 5 as presented in the text.

TABLE 6-1. TRANSFORMED LOADING DATA

Land Use	Climatic Region Northeast	Southeast	Northwest	Southwest	Row Median
residential	2.13	1.70	1.59	1.43	1.65
commercial	2.01	1.61	1.20	1.61	1.61
light industry	2.13	2.02	1.77	1.93	1.98
industry	2.71	1.64	1.59	1.87	1.76

The first step in applying the technique is to find the effect of being in a particular land use -- for example, across all climates. For residential land uses only, what is the typical loading? For this row the typical value, as represented by the median, is (1.70 + 1.59)/2 = 1.65. Similarly, the commercial, light industry, and industry row medians are 1.61, 1.98, and 1.76 respectively. Proceeding, we subtract the row medians from each of the data values to obtain a new table.

TABLE 6-2. LOADING DATA MINUS ROW MEDIAN

Land Use	Climatic Region Northeast	Southeast	Northwest	Southwest	Row Median
residential	0.48	0.05	-0.06	-0.22	1.65
commercial	0.40	0	-0.41	0	1.61
light industry	0.15	0.04	-0.21	-0.05	1.98
industry	0.95	-0.12	-0.17	0.11	1.76
column medians	0.440	0.020	-0.190	-0.025	1.705 Overall Value

We now find the climate fit, just as with land uses, only this time we naturally look down each column. For a particular climate, what is the typical value? (This time we are not using the original data values, but those with land use effects subtracted out.) In this way we find the values labelled "column medians" in Table 6-2. Notice that we also find the median of all land use effects themselves by looking down the column labelled "Row Medians." This is the "median of the row medians" -- the overall loading common to all land uses and climates. Again, we subtract the column medians from the respective table values above them (and subtract the overall effect from each of the row medians). The results are presented in Table 6-3.

TABLE 6-3. LOADING DATA WITH ROW AND COLUMN MEDIANS SUBTRACTED

Land Use	Climatic Region Northeast	Southeast	Northwest	Southwest	Row Fit	Row Median
residential	0.040	0.030	0.130	-0.195	-0.055	0.035
commercial	-0.040	-0.020	-0.220	0.025	-0.095	-0.030
light industry	-0.290	0.020	-0.020	-0.025	0.275	-0.023
industry	0.150	-0.140	0.020	0.135	0.055	0.078
Column fit	0.440	0.020	-0.190	-0.025	1.705 Common Value	-0.003

If we had used means as typical values, we would now be done. However, since we are using medians, the process is not quite complete. If we now calculate row medians, we see they are not quite zero; this indicates that we should go through the whole process once more, but this time "polishing" the fits by subtracting only the new, small row medians. Then we must do the same for the column medians. We iterate, moving from polishing rows to columns, until the medians for both are relatively (within round-off error) close to zero. We are left (in this case, after four steps) with row effects along the right-hand side, column effects along the bottom, and the common, overall value in the lower right-hand corner (Table 6-4). What numbers are left in the cells? Those not accounted for by climate, land use, or overall value -- the residuals. The results for land use and traffic density appear in Table 6-5.

The "common value" is an overall measure of the load. The "effects" are what they say they are -- in the case of land use and region, the effect of a particular value of land use or climatic region. For example in Table 4, being in the northeast adds 0.446 to the common value of 1.76 being residential adds -.05. The URS "overall" value is 156 -- too high, compared to the model above (anti-log 1.76 = 57.5). Notice that this model is linear in a log scale, hence actually multiplicative.

The basic results are these:

1. Differences in land use do not account for much of the observed differences in loadings; the effects are quite small, on the order of the residuals. An exception is light industrial, with a +0.34 value; however, there were very few observations for this land use.

2. There is a good sized positive effect for the northeast region (0.446), and negative effect for the northwest; the other two regional effects contribute little. While the northwest effect is based on just a few samples, the sizeable northeast effect is probably due to associated "infrastructure" -- old housing, density, etc.* The URS study concludes that loadings are lowest in commercial areas of the northwest -- true by

* Compare this with Hammer's 1976 discussion of his regression results.

TABLE 6-4. LOG_{10} LOADING MEDIAN LBS/CURB-MILE/DAY URS (1974)

Original Data	Region				
Land Use	Northeast	Southeast	Northwest	Southwest	Row Median
Residential	2.13	1.70	1.59	1.43	1.65
Commercial	2.01	1.61	1.20	1.61	1.61
Light Industry	2.13	2.02	1.77	1.93	1.98
Industrial	2.71	1.64	1.59	1.87	1.76

The Two-Way Fit: Loading = Common Value+ Land Use (row) Effect + Region (column) Effect + Residual

					Row Effect
Residential	0.01	-0.01	0.12	-0.26	-0.05
Commercial	-0.01	0.01	-0.16	-0.03	-0.02
Light Industry	-0.26	0.04	0.03	-0.03	0.34
Industrial	0.435	-0.22	-0.03	0.03	0.02
Column Effect	0.446	0.03	-0.16	-0.05	1.76 (common value)

TABLE 6-5. LOG_{10} MEDIAN LOADING LBS/CURB-MILE/DAY URS (1974)

Original Data	Traffic Density (ADT)			
Climate	<500	500-5000	5000-15000	>15000/day
Northeast	2.55	2.08	2.16	2.32
Southeast	1.76	1.83	1.66	1.23
Southwest	1.15	1.72	1.41	1.55
Northwest	absent	1.34	1.62	1.20

The Two-Way Fit					Row Effect
Northeast	0.215	-0.298	-0.042	0.060	0.623
Southeast	0.000	0.027	0.033	0.455	0.138
Southwest	-0.425	0.102	-0.032	0.050	-0.137
Northwest	absent	-0.028	0.428	-0.050	-0.167
Column Effect	0.045	0.022	-0.042	-0.060	1.623 (common value)

our table; but what is important from our analysis is that the residuals for commercial northwest areas is -0.16 -- as large as the northwest effect itself. That means there is more going on in this cell than we can explain with a "2-way" table.

3. There are large residuals for residential southwest (-0.26), northeast light industrial and industrial (-0.275, 0.435) -- and southeast industrial (-0.22) -- as large as any land use or climate effect! These all suggest details unaccounted for by the gross categories of land use and climatic region, particularly for the northeast. Are these local disturbances?

4. The simple model does a better job than the URS study: the residential northeast mean given by the URS t-test method is 291, whereas the reported value is 197. That is an error of 48 percent. Our method predicts a median value of 143 compared to an actual value of 135 -- an error of 6 percent. Of course, in an area with large residuals -- the northeast -- neither method can be expected to do well.

5. Examination of residuals in a stem-and-leaf display (Figure 6) indicates that the median is 0, as it should be. The residuals look about normal with some high and low values as noted. A diagnostic plot of residuals versus (row effect x column effect)/common value (Figure 7) shows that the residuals are well-scattered as they should be.

For Table 6-5, the following points are of interest:

1. There are fewer data points with simultaneous traffic-climate measures; than with land-use/climate (Table 6-4). In fact, the number of data values limits our extension of the method (and, of course, a "dummy-variable" regression technique) to a 3- or 4-way table. There are too few observations to fill a multiplying number of tables entries. For example, Table 6-5 has four levels of traffic and four levels of climate effects, which makes 4 x 4 = 16 cells. For a 4-way table, with an additional four levels of "land uses" and four levels of "landscaping beyond the sidewalk," we have 4 x 4 x 4 x 4 = 256 cells. But we have only 200 data points, not enough to fill all these cells.

2. We see that there is little or no effect from increased traffic density. Instead, the climatic effect dominates: the northeast is highly positive (as in Table 6-4), the northwest negative. This result agrees with the URS study: "loadings are lowest in areas with greater traffic,...possibly due to...[wind erosion]," but not as strongly as they would suggest. Our table indicates the lower loading is due mostly to climatic region, not traffic; URS considered only one category at a time.

3. The residuals are displayed in Figure 6-6. The median is 0, expected for "random" residuals, and the distribution looks fairly normal. The diagnostic plot of residuals shows well scattered points; the linear model is doing as well as can be expected.

Figure 6-6

Residuals from Two-Way Fit of Land Use versus Climatic Region

residuals x 100

4	3
3	
2	
1	2
+0	113433
-0	1133
-1	6
-2	762
-3	
-4	

median = 0.0

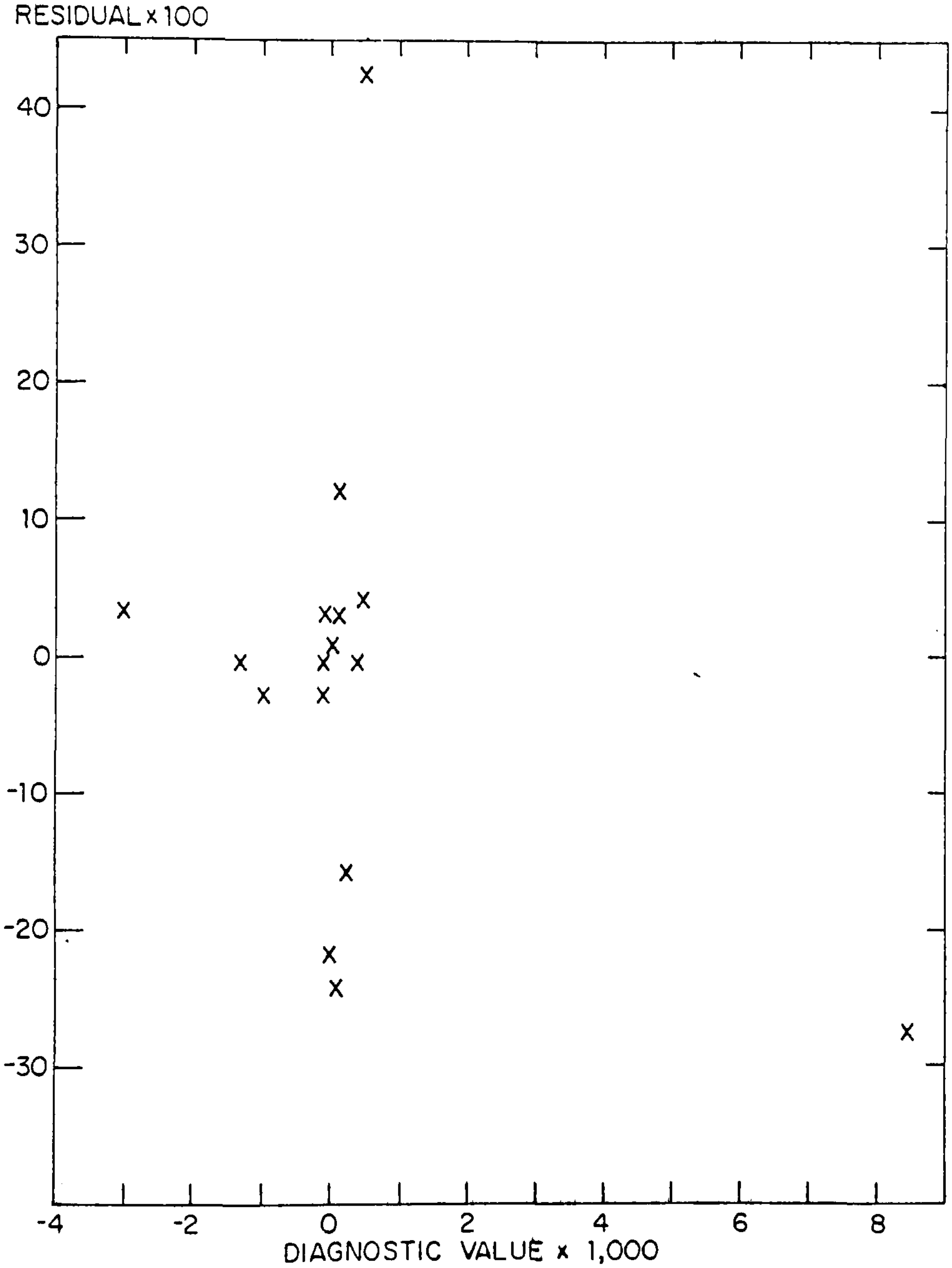

Figure 6-7. RESIDUALS FROM 2-WAY FIT, LAND USE vs CLIMATIC REGION

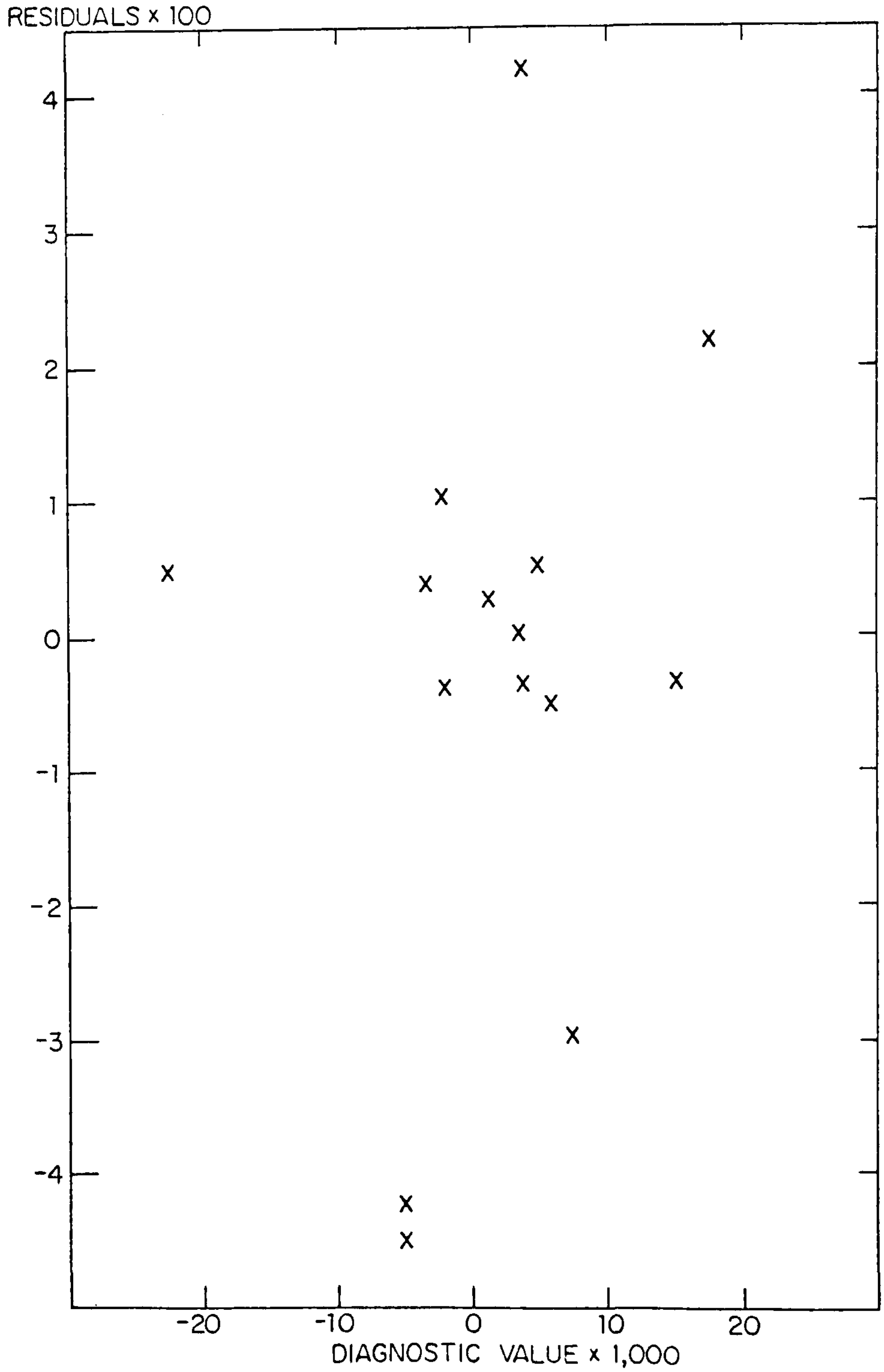

Figure 6-8. RESIDUALS FROM 2-WAY FIT, CLIMATE vs TRAFFIC

Based on the URS data of population density, impervious cover, landscaping beyond the sidewalk, and so forth, we could prepare other tables to delineate differences in loading rates. As we have seen, however, we cannot push limited data to cover too many categories. There is always some residual variation that grows larger and larger as the number of observations in each n-way cell becomes smaller and smaller. Some authors have gone further and claimed (Hammer, 1976) that the observed loadings in receiving waters might not reflect any areal buildup but rather are related to a more local, micro-scale erosion. If true, such an effect would render any more aggregate view (the usual one) generally useless.

NON-LINEAR ACCUMULATION RATES

Let us now turn from data analysis to modeling. The usual procedure to model the buildup of material on streets has been to imagine the observed loading as an equilibrium between accumulation and removal processes. An easy simplifying assumption is to make accumulation and the removal by washoff linear. As Shaheen(1975) and others have noted, this isn't true. Further, the rates probably aren't smooth, but rather, discontinuous.* The real picture is more like a sequence of independent events much like the arrival times and departures at a queue.

The simplest kind of rate model for deposition/removal is:

$$\frac{dL}{dt} = K1 - K2 \times L$$

where L = Load; t = time; and K1 and K2 are two constants. This integrates to

$$\int \frac{dL}{K2 \times L - K1} = - \int dt$$

$$\ln(L - K1/K2) = -K2 \times t + \text{constant}$$

$$L = K1/K2(1 - \exp(-K2 \times t)) \quad \text{(initial conditions: } t=0,\ L=0) \qquad (3)$$

where t is the time since the last washoff event (storm or street cleaning).

If the basic rate equation is generalized we obtain:

$$dL/dt = f(t) - f(L) + \text{random component.} \qquad (4)$$

Suppose the deposition is non-linear, say $a/(1+bt)^2$, so that possible deposition diminishes with time. There is no simple solution to this

---

* Certainly they are as we decrease the observed time interval. For example, at a large time scale, rain events themselves are discontinuous happenings that change the accumulation of material.

differential equation (there is an integral equation solution), but a series solution expanded to two terms gives:

$$
\begin{aligned}
L = -a/(1+bt) &+ ac/b^2(\ln(1+bt)\ \exp - (ct)) \\
&+ c(1+bt)\ \exp - (ct) \\
&+ c^2(1+bt)^2\ \exp - (ct) \\
&+ (a/b+c+c^2/4). \qquad (5)
\end{aligned}
$$

The load approaches its asymptotic value quickly for a selection of a, b, and c, corresponding to the "standard" differential equation (more linear accumulation rate), (3). Figure 6-9 shows a graph with some typical a, b, c values -- the asymptotic limits are just fictional. It is quickly seen that this more general equation can cover the simpler case, equation (3), as well as the empirically found curves of Sutherland and McCuen (1975), Figure 6-10:

$$
\begin{aligned}
\text{load (industrial)} &= 1388(1 - \exp(-.19t)) \\
\text{load (commercial)} &= 500(1 - \exp(-.535t)) \\
\text{load (residential)} &= 1089t(1+1.3t)).
\end{aligned}
$$

The Sutherland and McCuen model added dummy variables for traffic and street condition. We have incorporated these through the table categories and added the stochastic component.

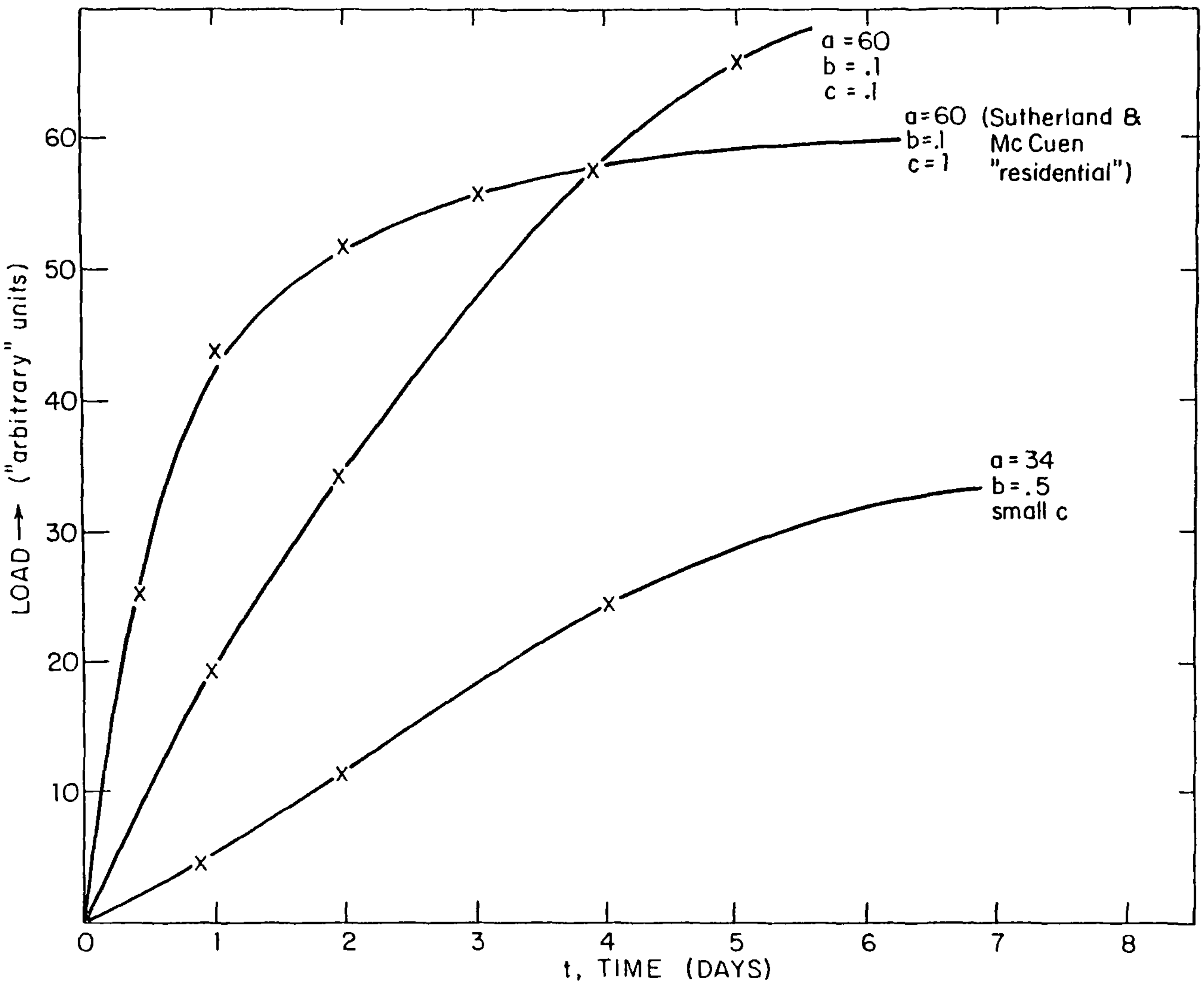

Figure 6-9. ACCUMULATION LOAD vs TIME; 'COMPLEX' DIFFERENTIAL EQUATION

Figure 9

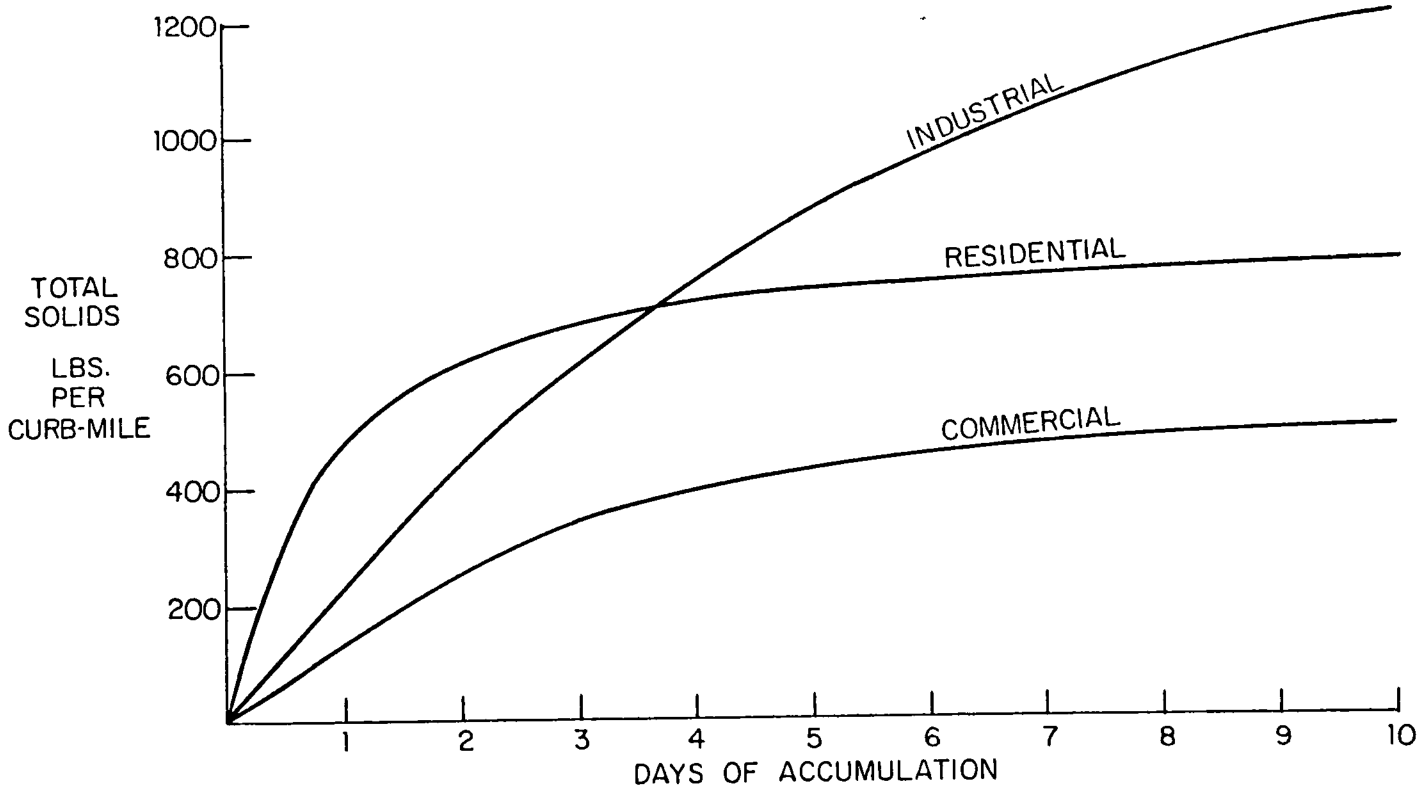

Figure 6-10. SUTHERLAND AND McCUEN EMPIRICAL CURVES

ESTIMATING COSTS OF ON-SITE CONTROL MEASURES

DEVELOPING A BASELINE

We have begun analyzing methods for estimating the capital cost requirements of stormwater control measures. Ideally, we would like to develop functional relationships between costs and the factors which are available at the preliminary planning level. These factors typically include general site characteristics, such as slope, size, and intensity of development, and meteorological information such as the design storm. At this level of analysis engineering design data, such as the size of drainage pipes and the maximum discharge, would not be available. Thus cost estimates at this level will, of necessity, be crude relative to estimates made later in the design process. But if properly developed, such preliminary cost estimating relations would be useful in the evaluation of area-wide stormwater management strategies and in the comparison among alternative combinations of measures.

Our analysis has initially focused upon the evaluation of traditional drainage system costs. This choice is based upon two considerations:

1. In most areas conventional storm sewer systems represent the current practice against which options such as on-site storage will inevitably be compared. Thus it makes sense to treat the conventional systems as a baseline for cost evaluation purposes.

2. Since conventional storm sewer systems are widely applied there is more knowledge and data on costs for this system than for other measures. Cost estimating approaches can be tested against a conventional system before extending them to measures for which cost data are limited or unavailable.

PREVIOUS STUDIES

In order to establish cost functions of the type desired it is necessary to establish an appropriate data base and to evaluate functional forms in light of the data base. In each instance two paths might be followed. For developing a data base it is possible to use actual or bid costs of a suitable sample of projects, or to develop a series of synthetic costs, cost estimates of hypothetical projects designed from scratch. Each approach has its own strengths and weaknesses. Utilizing actual project data has the advantage of encompassing, in a suitable sample, the range of conditions and factors found in practice. On the other hand, great care must be taken to insure that reported costs actually incorporate the full range of appropriate project expenses and no others. In addition, there may be a number of project-specific factors which influence costs and add to unexplainable variance in the data set, thereby reducing the precision of estimated cost functions. A data set developed by synthetic costing techniques eliminates this type of random variation in the cost data. But in the process the assumptions and design philosophy of a single individual or group becomes imbedded in the data set. This may bias the data in ways which are not predictable a priori. In addition, synthetic cost studies may often miss cost areas which are important in real applications. For example, procedures developed in a recent cost study of sewage treatment plants predicts costs that are substantially lower than those found in practice (EPA, 1976).

Given any particular data set, cost functions may be developed by empirically fitting the data to arbitrary functional forms so as to produce a best fit by some criteria. Usually the techniques of linear or non-linear regression analysis are employed for this purpose. Alternatively, a particular functional form might be specified based upon the physical relationships of the variables in the system.

It is possible, and often desirable, to combine the approaches discussed above. But in the two papers which have been reviewed here, separate paths are taken to estimate stormwater drainage costs. Grigg and O'Hearn (1976) developed a cost function based upon a simplified model of the hydraulics of runoff and estimated the parameters of the model from synthetic costs developed for a single drainage area. Rawls and Knapp (1972) developed a data base of actual projects and fit a variety of linear and non-linear models to develop estimating equations.

The Rawls and Knapp data base consisted of 70 projects from 23 areas located across the United States. The project data obtained included the design storm frequency (T), in years; average slope (S); in percent; runoff coefficient (C); number of inlets and manholes (I); smallest pipe diameter (D_B), in inches; largest diameter (D_E), in inches; outlet capacity (Q), in ft^3/sec; total length of drains (L_T), in feet; total drainage area (A), in acres; developed area (A_D), in acres; and total cost, in 1963 dollars (C_T). The individual variables most highly correlated with the total costs were the total and developed drainage area, maximum pipe size, outlet capacity, length of drains, and number of inlets and manholes. For all of these variables $|r| > 0.5$. Noticeably absent from the data set were data on the magnitude of the design storm or soil characteristics, although these are, to some extent, reflected in the other variables.

Using their entire data set, Rawls and Knapp estimated several cost models by nonlinear techniques. Their principal models incorporated engineering design variables such as pipe size, maximum discharge, and number of manholes and inlets as cost determinants. Thus while their fits, as measured by R^2, were good (typically on the order of .9), these models are not appropriate for the preliminary planning level that we have in mind.

Rawls and Knapp also analyzed simpler linear cost functions individually for subsets of data from three separate states and found that they could explain a substantial proportion of the variation in costs in California and Texas. This result indicates that regional effects were important and might be related to the omission of rainfall and soil factors from the data base.

Grigg and O'Hearn developed a functional form for cost estimation from an idealized model of a single storm sewer draining a small basin. Their model combined the rational formula for runoff, Manning's formula for pipe-flow, a rainfall intensity/duration formula and an empirical formula for pipe costs to arrive at a formula relating collection costs to drainage area, design storm frequency, runoff coefficient, slope, storm duration and a correction factor for the extensiveness of the collection network.

The authors did not estimate the function for different areas or topographic and rainfall conditions, but considered only a single site with different degrees of impervious area and different design storm frequencies. Costs for drainage systems were synthesized and fit to a function of the design frequency for each level of impervious area. While the approach yielded good fits for the synthesized costs of the particular area under consideration, the adequacy of their general model for estimating costs for different areas, slopes, and characteristic rainfalls was not really evaluated.

REANALYSIS OF RAWLS AND KNAPP DATA

We have used a generalization of the approach developed by Grigg and O'Hearn to reevaluate the Rawls and Knapp data. This reanalysis has focused on developing a cost relationship suitable for preliminary analysis when no detailed engineering data are available. The model is outlined below:

Rational Formula: $Q = CIA,$ (6)

General Flow Formula: $Q = a_1 D^{a_2} S^{a_3},$ (7)

Cost Per Foot: $C_p = b_1 D^{b_2}$ (8)

Pipe Length: $L_T = C_1 A^{c_2}$ (9)

Combining these expressions yields:

$$C_p = b_1 \left(\frac{CIAS^{-a_3}}{a_1} \right)^{b_2/a_2} \quad (10)$$

and:

$$C_T = c_1 b_1 \left(\frac{CIAS^{-a_3}}{a_1} \right)^{b_2/a_2} A^{c_2}, \quad (11)$$

where C_p = installed cost of pipe in /foot, and other variables are as defined previously.

This last expression can be multiplied by a factor (1 + E) to account for additional costs (engineering, etc.). Finally a factor can be introduced to account for the degree of development in the drainage basin served. We shall use the term $(A/A_d)^{c_3}$ which is the ratio of area served (A) to area developed (A_c). The parameter c_3 is assumed to be less than zero since the larger the ratio the less extensive would be the pipe network required to serve a given area. The expression for total costs becomes:

$$C_T = (1 + E)b_1c_1\left(\frac{CIAS^{-a_3}}{a_1}\right)^{b_2/a_2}\left(\frac{A}{A_d}\right)^{c_3} A^{c_2} \tag{12}$$

which can be simplified to:

$$C_T = d_1 C^{d_2} I^{d_3} A^{d_4} S^{d_5}\left(\frac{A}{A_d}\right)^{d_6} \tag{13}$$

where the d_i's represent the appropriate combinations of parameters and are to be estimated from the data.

Examination of the derivation of the total cost equation leads to the following predictions based on the equation forms and generally recognized properties of the individual coefficients.

$d_2 = d_3$

$d_2, d_3, d_4 > 0,$

and

$d_5, d_6 < 0.$

Moreover, if Manning's equation is the appropriate flow equation, and pipe costs follow the Grigg and O'Hearn cost formula ($b_2 = 1.663$), then

$$d_2 = d_3 = \frac{b_2}{a_2} = \frac{1.663}{2.67} = .62$$

and

$$d_5 = \frac{b_2a_3}{a_2} = \frac{-1.663}{2.67} (.5) = -.31$$

In order to estimate the total cost relationship (13) from the Rawls and Knapp data, it was necessary to include the rainfall intensity, I. This value was approximated by including the 15-minute storm for the appropriate location and design period (Yarnell, 1935). The intensity, I, could be estimated for only 67 of the 70 original projects; thus three projects were dropped from the sample. The total cost equation was estimated by taking the logarithms of each variable and using linear regression. The resulting estimates are presented in Table 6-6.

TABLE 6-6. ESTIMATES OF COEFFICIENTS

Coefficient	d_1	d_2	d_3	d_4	d_5	d_6
Estimate	2,591	.764	.530	.696	-.134	-.356
t Value		(1.86)	(1.48)	(8.00)	(2.00)	(1.86)
Significance level (two tail)		.07	.15	<.001	.05	.07
	$R^2 = .529$					

It may be seen that, while not all the coefficients are highly significant, all signs are in the direction predicted; moreover, the numerical values of d_2, d_3, and d_5 seem reasonably close to those predicted by Manning's equation and the Grigg and O'Hearn cost function, given the sampling errors involved. A restricted regression was also run with $d_2 = d_3 = .62$ and $d_5 = .31$. The results of this regression are reported in Table 6-7.

TABLE 6-7. RESTRICTED REGRESSION RESULTS

	d_1	d_2	d_3	d_4	d_5	d_6
Estimate	2,073	.62	.62	.681	-.31	-.313
t Value				(7.84)		(-1.69)
Significance level (two tail)				<.001		.1
	$R^2 = .491$					

The reduction in R^2 going from the first to the second regression was not statistically significant at a 5 percent level; thus the predicted values for d_2, d_3 and d_5 cannot be rejected.

The R^2 given in the two regressions is not directly comparable to the Rawls and Knapp values because it is computed on the log of the dependent variable. A comparable measure can be obtained by using actual and predicted cost values after a retransformation to the original form of the data. The value corresponding to Table 6 is $R^2 = .65$ which is still much lower than the Rawls and Knapp results. But given the exclusion of design information from the analysis, the results appear to be quite encouraging.

CONCLUSIONS

At the end of studies of stormwater runoff, one often hears the call for "more data." It would appear to be the duty of every investigator to point out the need for additional information. We find it hard to argue with the observation or, if you will, plea; however, a careful reexamination of data we have on hand can reveal much about the runoff process. The beginning of stormwater runoff is in the residuals accumulation, and here we have found that loading rates can be characterized as log-normally distributed (influenced by multiplicative factors) and non-linear over time. Using resilient statistical techniques we can separate these factors into components like traffic density, climate, and land use, and we can gage the relative importance of each. The results could improve modeling efforts like SWMM in two ways: (1) by giving better initial estimates of loads; and (2) by reproducing more closely observed accumulation changes over time. On the cost side, we have shown how to develop a general predictive model for the cost of conventional sewer systems, given little or no specific engineering design information, but utilizing more of the readily available site and precipitation data.

To make these analyses more useful in the context of stormwater management, we must account for the washoff and transport of pollutants and the impact of control measures. We are proceeding by putting our improved accumulation equation and data estimates into a simulation model that deals with the uncertainty of some hydrologic (washoff) events.

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INTERIM PROGRESS REPORT ON CHARACTERIZATION OF SOLIDS BEHAVIOR IN, AND VARIABILITY TESTING OF SELECTED CONTROL TECHNIQUES FOR COMBINED SEWER SYSTEMS

by

DR. WILLIAM C. PISANO*

FOREWORD

The objectives, scope of work and progress to date of the R&D sewer flushing study currently being conducted in Boston, Massachusetts will be presented. The field experiments were performed on sewer segments in Dorchester which is a community of Boston. The project was initiated in the summer of 1976 under a U.S. Environmental Protection Agency research grant (EPA No. R-804578). The grantee for this project is Northeastern University, Boston.** Energy and Environmental Analysis, Inc., Cambridge, Massachusetts is the prime subcontractor for this work.

BACKGROUND

Solids deposition in sewer lines has always been a plague to effective maintenance. Recently the significance of such loads as a major contribution to first-flush pollution has been recognized. Studies in Buffalo, NY have shown that 20 to 30 percent of the annual domestic wastewater solids settle in the combined sewer system and eventually are discharged during storms. As a result, a large residual sanitary pollution load over and above that normally carried is discharged over a relatively short interval of time, often resulting in what is known as a "first-flush" phenomenon. This can produce shock loadings detrimental to receiving water life.

Another manifestation of first-flush, in addition to the scouring of materials already deposited in the lines, is the first flush of loose solid particles on the urban ground surface that are transported into the sewerage system. These particulates may settle out in the system and be available for flushing during periods of large flows. The main purpose of the project is to evaluate the effectiveness of various sewer flushing techniques in controlling the first-flush pollution problem.

In 1966 a research effort was made by the FMC Corporation through the

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*Technical Director, Energy and Environmental Analysis, Inc., 257 Vassar Street, Cambridge, Massachusetts 02139

**Drs. Frederic Blanc and James O'Shaughnessy are the project managers for Northeastern University.

U.S. EPA, Storm and Combined Sewer Research Program to demonstrate the feasibility of reducing pollution from combined sewer overflows by means of periodic flushing during dry weather. The first phase included a study of the overall flushing concept, small-scale hydraulic modeling, and design and development of cost estimates for constructing test equipment. The second phase was a large-scale flushing evaluation of a 1600 foot, 12 in. and 18 in. diameter above-the-ground test facility at FMC. This phase allowed for adjustments to slope with holding tanks at three points along the test sewers for the flushing experiments. The Phase II 1972 final report* recommended that further studies be made for flushing larger sizes of pipe, of wave sequencing and of solids build-up over long time periods. The current R&D project addresses these recommendations as well as evaluation of various types of flushing devices and methods not looked at during the predecessor FMC work.

CONCEPTUAL VIEWS OF PROGRAM

The solids control demonstration/research program has been developed to address many of the issues relating to the feasibility, cost-effectiveness, and ease of application of upstream solids control program as an integral part of overall combined sewer management. Basically, there are five fundamental issues that must be answered before widespread acceptance of upstream solids control may be considered. The issues include: 1) what are the best flushing methods to use for a given situation; 2) what is the expected pollutant removal efficiency associated with the various methods; 3) what are the costs associated with such programs; 4) how do you screen large systems to identify problem pipes with respect to deposition and; 5) what are the effects of stormwater runoff on such a strategy as applied to combined sewer systems.

PROGRAM OBJECTIVES

1. Test the feasibility of applying various solids control techniques as a method of deposition control in combined and sanitary sewer lines on test segments in the Boston sewerage system.
2. Carefully monitor deposition rates on a number of test segments.
3. Monitor pollutant removal including solids, organics, nutrients and heavy metals associated with the various flushing techniques.
4. Assess solids characteristics (particle distribution) of both the flushed and remaining materials as well as analyze (grit/organic) ratios.
5. Recommend most favorable solids control techniques for operational testing by both automated and manual means.
6. Develop, test and evaluate an automated flushing module in a field operational testing program.
7. Develop, test and evaluate manual sewer flushing techniques utilizing specially equipped water tankers in a field operational testing program.

*FMC Corporation "A Flushing System for Combined Sewer Cleansing" EPA Water Pollution Control Research Series Report No. 11020 DNO 03/72, NTIS No. PB 210 858, March 1972.

8. Assess the operational feasibility and performance of flushing both long and short upstream collection segments and/or networks.

9. Refine existing deposition model and flushing criteria.

10. Compare solids control by flushing versus selected structural options as combined sewer overflow pollution abatement technique using "desk-top" procedures.

11. Develop user guidelines for solids control program as an integral part of combined sewer management schemes.

SCOPE

This project is functionally divided into four major phases. The first three phases are intensive field engineering investigations while the fourth phase is relegated to data reduction and desk-top analytical efforts. All of the first three phases of work are completed.

In the first phase of field work four test segments on different streets in the Dorchester sewerage system were field flushed over an extended period using different flushing methods. External sources of fresh water, as well as sewage were used. The experiments were aimed at quantifying the effectiveness of flushing deposition accumulations from a single pipe segment on a routine basis as well as roughly estimating deposition characteristics within collection system laterials.

The second phase of field work was concerned with the problem of flushing a long flat stretch of combined sewer laterial. The street contains five manholes and is roughly 1000 feet in length. Flushes were injected into the upper most manhole and pollutant levels in the flush wave passing three down-stream manholes were monitored. Work was divided into two subphases. Initially, pollutant removals over the three segments were determined for different flushing conditions established in the first manhole. These results provided insights into flushing effectiveness over three segments of pipe. Next, settleability tests were performed on samples taken from flushes conducted in a similar manner for the purpose of crudely extrapolating how far beyond the flushing monitoring manholes would the materials be carried.

In the final phase of field operation, an automatic sewer flushing module was designed, fabricated, installed and operated on a single segment for an extended period. The purpose of this work was to begin to develop operational experience using automated flushing equipment. The state of the art with respect to operational automated flushing methods, equipment, sensing interfaces, etc. has not been fully demonstrated at this point in time. The effort in this study is viewed as a pilot prototype investigation.

In the fourth phase, various predictive deposition loading and flushing criteria will be generated from the large data base developed during the field programs. These refined formalisms will allow for scanning of large-scale sewer systems to identify problem pipes with respect to deposition. The refined tools will allow for comparative analysis of upstream solids control vs selected structural options to compare program efficiencies. The tools will also allow for rough assessment of first flush phenomena as related to combined sewer systems to better clarify the questions over the use of upstreams "first-flush" collection devices.

PHASE I OVERVIEW-SINGLE SEGMENT FLUSHING

After a careful review and inspection program, four streets were selected for the flushing experiments. These streets are all in Dorchester characterized by high density, 3-story multi-family dwellings. Two of the test segments located on Port Norfolk and Walnut Street are served by flat combined sewer laterals of 12" and 15" circular pipe, respectively. Total tributary population down to and including the test segments are 94 and 71 people, respectively. The other two test segments on Shepton and Templeton Streets, are serviced by separate sewer laterals of 12" and 15" circular pipe, respectively. There are downspouts on both streets connected to the sanitary sewer. Although these two segments are separated, considerable stormwater inflow occurs during storm events. Total tributary population for these two streets are 230 and 221, respectively. The characteristics of the four segments are given in Table 7-1.

The flushing program in this phase is concerned only with the effects of flushing a single manhole to manhole segment. Three different methods of manual flushing were performed. The first method consisted of backing up the upper end of the flushing manhole with an inflatible rubber stopper with quick release. The other two methods were gravity and pressurized dump discharge into the flush manhole with the upper end of the flush manhole blocked off. These flushes were performed using a specially designed water tanker equipped with two 1000 gallon tanks mounted on a steel I-beam skid. The tanker was equipped with a pneumatic system to pressurize the tanks to 30 psi. The operation under gravity conditions provided a controlled flush release of 35 to 50 cubic feet at a rate of 0.25 to 0.50 cfs. Under pressurized conditions the same volumetric range of flush was accomplished at a rate of 0.5 to 1.25 cfs. All flush volumes were measured by a water meter supplied by the City of Boston. The meter was repeatedly calibrated to ensure accurate monitoring of the delivered flush volumes.

A total of 87 separate flushing experiments were performed during the period of August 30, 1976 through November 12, 1976. Roughly 20 flushes on a 3-4 day basis were accomplished for each of the four test segments. The method of flushing was rotated per street so that all methods were applied over the test period. The segments were mechanically cleaned by the City of Boston at the onset of work and then three weeks later by a professional sewer cleaning contractor using high pressure water jets. A thin (1"-2") layer of sand and gravel remained after the cleaning operations.

The sequence of pertinent operations during a given flushing experiment is the following. After the safety equipment was set-up and the segment lamped, several liquid background samples at five minute intervals and depth of flow were taken. Next, the upper end of the flush manhole was blocked-off (in most cases) and sediment samples over a prescribed unit length was taken in both the flush and downstream sampling manholes. The flushing experiment was then conducted either with backed-up sewage or from injection from the flush-truck.

Characteristics	Port Norfolk	Shepton	Templeton	Walnut
Pipe Shape & Size	12" circ.	12" circ.	15" circ.	15"circ.
Service Type	Combined	Separated w/ clear water inflow	Separated w/ clear water inflow	Combined
Length of Flush Segment	247'	226'	187'	136'
Plan Pipe Slope	.0049	.0035	.0032	.0048
Contributing Population	94	230	221	71
General Sediment Appearance	Fresh sanitary depositions & fine sand	Fresh sanitary depositions	Septic sanitary deposits	Septic sanitary deposits & sand/gravel
Dry Weather Flow Appearance	Slight Meandering movement	Slight-good movement	Sluggish	Sluggish
Street Surface Appearance	Good surface w/ considerable surface trash	Good surface, clean	Poor surface dirty	Good surface, clean

TABLE 7-1. DESCRIPTION OF FLUSHING SEGMENT CHARACTERISTICS

Dye was injected in the wave and at the instant of arrival, one-liter aliquots were taken with a specially designed hand scoup for obtaining a reasonable cross-sectional sample of the solids within the flush wave at the downstream sampling manhole. The device specifically excluded bed load materials. Eight grab samples of the flush wave were taken at 10-second intervals. Once the flush wave was noted at the downstream, then an additional 10 samples were taken at 20-second intervals. Wave heights were taken at each interval of time which were later used to determine the instantaneous flow rate for computing mass pollutants removed by the flushing experiment.

Development of the stage/discharge relationships for computing discharge rates during the flushing periods was an extremely difficult task. The flush waves are characterized by non-steady state flow conditions with rapid velocities at and during peak discharge conditions and low velocities thereafter. Initially, Manning's equation with plan and profile map pipe slope was used to estimate flow rates. Comparison of computed flush volumes with truck delivery meters volumes was extremely poor, characterized by large unaccountable biases and nearly zero correlation between truck and computed volumes. Next, the flush truck with hydrant input was repetively used to deliver sustained known rates of flow at several flow depths. Steady state discharge rating curves were established by a least squares curve fitting of Manning's equation to the field data. The computed versus the measured metered volumes on the average showed little bias for Walnut and Shepton Streets while nearly a 45 percent error for the other two streets. The computed versus measured volume correlation coefficients for Walnut and Shepton were less than 0.3 and the variances were extremely large. Mathematical programming optimization techniques were finally used to determine the parameters of a non-steady state (histeresis) model while minimizing the variance between computed and measured flush volumes per street. Four different approaches were investigated before the final procedure was adopted. Special dye injection field experiments using fluorometric methods were conducted to determine in-situ non-steady flow conditions for the purpose of establishing and verifying the flow model parameters. The flow model reproduced with remarkable consistancy flush wave volumes and flow rates conducted during the dye injection experiments. The optimized flow model(s) were then used to compute flow conditions for the 87 flushing experiments. The correlation coefficients between computed and measured volumes ranged from 0.75 to 0.99. Bias is nearly zero for all cases. This procedure was also used to establish the stage/discharge relationships for experiments conducted in the next two phases of work.

Average pollutant removal results for the Phase I flushing experiments are given in Table 7-2. These results are final and supercede all previously reported findings. Part A of Table 7-2 presents the average quantities in kilograms of pollutants transported from each of the test segments together with the average for all four streets. The average deposited raw loadings do not vary considerably from street to street for a given pollutant. Average deposited pollutant loadings normalized for the number of antecedent days between flushing events, in kilograms/day, are given in Part B of Table 7-2. The results again are remarkedly consistent from street to street. Coefficients of variations for all pollutants and for all streets range from 0.5 up to 1.0, indicating farily consistent deposition accumulation and flushing removal characteristics. Part C of Table 7-2 presents average pollutant removal characteristics normalized by both the number of antecedent days between flushing periods and the contributing population down to the monitoring manhole, in grams/capita/day. Population estimates are accurate since they reflect last year's census tally. The average normalized pollutant removals for the two combined sewered segments exceeded by at least a factor of two the normalized removals from the two separate sewered segments. Heavy metal results are completed but are not presented in this tabulation. Analysis of heavy metals (Ca, Cr, Cu, Pb, Ni, Zu and Hg) of flow-composited flush wave samples, settled for one hour, indicated that nearly all metals were contained in the settled fractions. The average measured mass of metals from the two combined sewer flush experiments were twice the level that they were in the two separate sewer flush experiments. The concentrations of heavy metals, during wet weather were approximately four to five times the level noted for flushes characterized by the dry weather conditions. Flushes made at these locations considerably reduced the amount of heavy metals available for transport during rainfall events.

The Phase I experiments showed that all flush methods provided about the same degree of removal. The best method, as expected, is an external source high volume/high rate flush. Average flush volume during this experimentation period was 350 gallons, amounting to two to five percent of the dry-weather flow. The periodic flushing removed the domestic sewage deposits that accumulated between flushing events and maintained levels of grit, rock, and debris that remained after the professional sewer cleaning operation.

	POLLUTANT						
	TSS	VSS	BOD	COD	TKN	NH_3	P
A. Raw Data (kg)							
Port Norfolk	5.46	3.18	1.37	3.93	.142	.020	.026
Walnut	4.24	2.94	2.75	5.81	.214	.083	.052
Shepton	4.56	3.92	1.64	6.50*	.143*	.037*	.039*
Templeton	5.56	4.15	2.56	3.35*	.094*	.010*	.019*
Average	4.96	3.55	2.08	4.90	.0148	.0375	.034
B. Normalized for Antecedent Days (kg/day)							
Port Norfolk	1.590	.887	.396	1.28	.041	.006	.007
Walnut	1.234	.838	.709	1.63	.057	.024	.014
Shepton	1.270	1.110	.480	1.79*	.041*	.011*	.011*
Templeton	1.550	1.150	.753	1.17*	.031*	.003*	.006*
Average	1.411	.996	.585	1.468	.0425	.044	.0095
C. Normalized for Antecedent Days & Tributary Population (grams/capita/day)							
Port Norfolk	16.91	9.44	4.21	13.62	.436	.063	.075
Walnut	17.38	11.80	9.99	22.96	.800	.338	.197
Shepton	5.52	4.83	2.09	7.78*	.178*	.048*	.048*
Templeton	7.01	5.20	3.41	5.29*	.140*	.014*	.027*
Average	11.705	7.818	4.925	12.413	.389	.116	.0087

*Results of 2-3 flushing experiments; otherwise 18-21 experiments.

TABLE 7-2. AVERAGE POLLUTANT REMOVAL CHARACTERISTICS: PHASE I FLUSH EXPERIMENTS

PHASE II OVERVIEW

A: Serial Flushing - Pollutant Removals

The purpose of these experiments were to ascertain the pollutant removal effectiveness over three consecutive combined sewered segments on Port Norfolk Street (875') by flushing the uppermost manhole using similar flush volumes/rates used in Phase I. The other two test segments on Port Norfolk Street have similar physical characteristics as the Phase I segment which were reported in Table 1.

The experimentation period began in December 1976 and extended through March, 1976, entailing two replicate sets of three flushing rate/volume experiments. Each experiment consisted of three flushes conducted within a short period of each other. The first two flushes on a given day were the same while the final flush was a maximal volume/rate flush meant to remove any remaining pollutant load in the segments. Different combinations of flush volume (35 to 75 cubic feet) and delivery rate (.3 cfs to 1 cfs) were considered. The backup and release method of flushing was not considered in this phase since there was no appreciable contributary population at the flushing manhole. Three crews sampled the flush wave passing the downstream manholes. Samples were taken at the same frequency as in Phase I. Appreciable flush waves were also visually noted at the end of the street roughly 1000 feet downstream.

Results of these experiments are summarized in Table 7-3, showing the average percentage per flush of the total load removed for each of the three segments downstream of the flush injection manhole. These averages were computed using the loads computed per manhole for the six sets of flushing experiments. The results indicate that most of the loads for all three segments were removed during the first flush. For example, 81 percent of the volatile suspended solids load removed from the first flush. The second and third flush removed an additional 19 percent of the total. No appreciable gain is achieved by repeated flushing. Furthermore the experiments indicate that a single flush at the upper end of the street was reasonably effective in removing most of the deposited load along the 875 feet stretch of 12" combined sewer lateral.

	TSS	VSS	COD	BOD
First Sampling Manhole				
Flush 1	76	81	70	88
Flush 2	12	10	20	6
Flush 3	12	9	10	6
Second Sampling Manhole				
Flush 1	72	79	73	63
Flush 2	14	12	15	31
Flush 3	13	9	12	6
Third Sampling Manhole				
Flush 1	67	72	63	-
Flush 2	18	14	31	-
Flush 3	15	13	6	-

TABLE 7-3. AVERAGE PERCENTAGES OF POLLUTANT LOADS REMOVED PER FLUSH FOR EACH PIPE SEGMENT*

Visual inspections in the second and third downstream manholes after the flushing experiments indicated that little organic sediments remained after the flushing experiments on any flushing day. The amount of fixed sand and gravel depositions remained at constant levels during this phase of work.

*Average percentages computed from six sets of flushing experiments.

B. Serial Flushing - Settleability Analyses

The purpose of these experiments on Port Norfolk Street was to roughly assess the transport of flushed pollutants beyond the test segments (875 feet) using information derived from settleability tests. Special equipment was fabricated to obtain "undisturbed" samples of flush waves at the three downstream manholes on Port Norfolk Street. Samples were taken from each manhole and then flow-composited for settleability experiments. External source flush volumes were injected into the uppermost manhole on Port Norfolk Street using similar volumes/rates as used in Phase II-A. Special settling column equipment and procedures were established in order to perform settleability tests of the flush waves. A special yoke-frame installation was devised to permit axial and transverse mixing of the column before settleability experimentation. This ad hoc procedure was necessary since the settling velocities for a considerable portion of the composited flush wave solids were extremely rapid. Gentle mixing using air agitation was initially performed but resulted in solids bulking since the solids content of the flush samples were typically on the order of 6000 to 9000 mg/ℓ.

Six different flushing experiments were performed in the period of April, 1977 through August, 1977 in which settleability tests were performed for samples taken at each of the three sampling manholes. Solids, organics, nutrients and heavy metals (copper, zinc, nickel, cadmium, lead and chromium) were analyzed for 18 samples per settleability test. Initially, samples were withdrawn at 10 minute intervals for the first half hour and 30 minutes thereafter. This schedule was then changed to 5 minute intervals since over half of the solids removal occurred in the first half hour.

The experiment showed definite shifts in suspended solids/settling velocity distribution from the first to the third downstream sampling manholes indicating that heavier grit fractions would quickly resettle leaving the lighter solid fractions in suspension. About 20 to 30 percent of suspended solids would remain in suspension after 30 minutes of settling time. The fractions of volatile solids relative to the suspended solids increased both with settling time during the experiment and with a given manhole and with each progressive downstream manhole. Distribution of COD and BOD versus the settling time showed the similar characteristics as the suspended solids settling behavior. About half of the initial BOD levels would reamin in suspension after 30 minutes of settling. Heavy metals removal characteristics resemble the characteristics of suspended solids removal curve.

PHASE III OVERVIEW-OPERATION OF AN AUTOMATIC FLUSHING DEVICE

The purpose of this period of testing was to determine the feasibility and pollutant removal effectiveness of a simple automated sewer flushing module. An air on oil hydraulic gated device triggered by an automatic time-clock was fabricated and installed in a manhole on Shepton Street. This device backed-up sewage for a prescribed time period, allowing the field crew to be on-site during actual flushing operation to collect flush wave samples. The manhole had to be plastered to minimize infiltration through the manhole walls and table. The device was constructed to permit overflow over the gate during high flow conditions

The device was installed and operated from August 12, 1977 till the end of September, 1977. The device worked reasonably well and was unaffected by frequent and large rainstormes. Samples were taken from seven flushes over this period. The pollutant removal rates are comparable with the Phase I results.

PHASE IV OVERVIEW-ANALYSIS/USER MANUALS

An interim manual entitled, "procedures for Estimating Dry Weather Pollutant Deposition in Sewerage Systems (EPA-600/2-77-120) has been prepared to provide 208/201 planners with procedures for roughly gaging the magnitude of dry weather deposition in combined sewer systems. This document provides a simplified methodology for providing first-cut assessments of the total amounts of solids (lb/day) that deposit in a sewerage collection system; and the extent of the collection system over which the deposition takes place. A complex distribution-parameter dry weather sewage deposition model was roughly calibrated using field data and then was applied to 75 separate and combined sewer collection systems in eastern Massachusetts to generate estimates of solids deposited daily per system (lb/day). These estimated loads were then regressed with selected variables representing the physical characteristics of these collection systems including total pipe length, service area, average collection system pipe slope, average pipe diameter and other more complicated variables representing various points on the lower end of the collection system pipe slope cumulative density function.

Four alternative predictive single term power functions were developed from the regression analysis. The degree of fit of the non-linear functions to the data set were remarkably high. The R^2 values of the alternative models ranged from 0.85 for the simpliest approach requiring little external data analysis and preparation, up to 0.95 for the most complex model requiring substantial external engineering and data reduction analyses.

The simpliest of the alternative models is given in Table 7-4 along with two other empirical relationships derived from the analysis of the data for the 75 collection systems.

The Phase I field results were used to regress other pollutants such as BOD, COD, TKN, Total Phosphorous, NH_3 and VSS with suspended solids, all with high values of R^2, extending therefore the use of the predictive equations for total solids deposited to the estimation of other pollutants.

Extensive statistical analyses of sewerage system pipe slopes in this effort revealed that collection system pipe slopes can be represented by an exponential probability model. Analysis of the distribution of loads deposited versus cumulative pipe length led to the development of generalized curves as a function of collection system mean slope for estimating the total fraction of collection system pipe footage over which a given percentage of the total loads deposit. These findings can be combined to locate segments associated with the required fractions.

ACKNOWLEDGMENTS

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The Division of Water Pollution Control of the State of Massachusetts contributed supporting funds to this project. The Public Works Department of the City of Boston provided metering equipment for the flush tanker and mechancial pre-cleaning of the test segments. Mr. Gerald Aronson and Celso Queiroz of Energy & Environmental Analysis, Inc. supervised the field experiments and the data processing/computational efforts, respectively.

A. Daily Deposition Loads

$$TS = 0.076\ L^{1.063}\ (\bar{S})^{-0.4375}\ q^{-.51} \quad (R^2 = 0.845)$$

where TS = Collection system solids deposition loadings (lbs/day),

L = total length of collection system piping (feet),

$\bar{S}$ = average collection system pipe slope (ft/ft) and,

q = daily per capita waste discharge rate (gpcd).

B. Pipe Length and Service Area

$$1)\quad L = 168.95\ A^{0.928}$$

$$2)\quad L = 239.41\ A^{0.928} \qquad (R^2 = 0.821)$$

where L = Collection system pipe length (feet) and

A = service area (acres).

and 1 = Low population density (10-20 people/acre)

2 = moderate/high population density (30-60 people/acre).

C. Average Pipe Slope and Ground Slope

$$\bar{S} = 0.348\ (\bar{S}_G)^{0.818} \quad (R^2 = 0.96)$$

where $\bar{S}$ = Average collection system pipe slope and

$\bar{S}_G$ = average ground slope.

TABLE 7-4. PARTIAL SUMMARY OF PREDICTIVE PROCEDURES FOR ESTIMATING DAILY DRY WEATHER SEWAGE COLLECTION SYSTEM DEPOSITION LOADINGS

THE POTENTIAL OF STREET CLEANING IN REDUCING NONPOINT POLLUTION

Robert Pitt*

ABSTRACT

This paper briefly describes the conclusions available at this time from a study of nonpoint pollution abatement through improved street cleaning practices. An improtant aspect of the study was development of sampling procedures to test street cleaning equipment performance in real-world conditions.

The paper summarizes accumulation rate characteristics of street dirt in the area studied. The results of performance tests for street cleaning equipment and the factors that are thought to affect this performance are also summarized. These data are used to draw preliminary conclusions about elements that must be considered in designing an effective street cleaning program.

The study of urban runoff yielded information on overall flow characteristics, concentrations and total mass yields of monitored pollutants in the runoff, and street dirt removal capabilities and effects on deposition in the sewer system for various kinds of storms. These data are summarized, and urban runoff water quality is compared with recommended water quality criteria and the quality of sanitary wastewater effluent.

Costs and labor effectiveness of street cleaning, runoff treatment, and combined runoff and wastewater treatment are also compared, and preliminary results from a study of airborne dust losses from street surfaces are summarized.

This paper is based on research conducted under a U.S. Environmental Protection Agency grant to the city of San Jose (EPA Demonstration Grant No. S-804432). Woodward-Clyde Consultants participated in this study under subcontract with the city. The project began in September 1976 and will be completed in March 1978.

*Environmental Engineer, Woodward-Clyde Consultants, Three Embarcadero Center, San Francisco, CA 94111.

BACKGROUND

Past research, notably that conducted for the U.S. Environmental Protection Agency (EPA), by the American Public Works Association (Sullivan 1969), and by URS Research Company (Sartor and Boyd 1972; Pitt and Amy 1973; Amy et al. 1974), has clearly revealed the water pollution potential of street surface contaminants. These projects present strong evidence relating contaminated streets with the contamination of receiving waters. A paper presented at the American Water Works Association annual conference in Boston in 1974 (Pitt and Field 1974) using data from these reports compared the relative importance of untreated nonpoint urban storm runoff with treated sanitary wastewater in their effect on receiving water. In this example it was found that pollutants in stormwater runoff must be reduced in order to significantly reduce the total pollutant load to a receiving water. These reductions in runoff pollutants could be accomplished by treating the runoff and/or by reducing the quantities of pollutants contaminating the runoff.

Although it is clear that pollutants in street dirt have a significant effect on the quality of urban runoff and its effect on receiving water, there are many questions yet to be answered about the nature of this cause and effect relationship. The report on which this paper is based attempts to answer some of these questions. More specific information is needed in order to develop effective control procedures.

The study was designed to measure street cleaning equipment effectiveness in removing pollution from the street surface in a real-world situation. It must be emphasized that the purpose of the project was not to compare specific types of equipment but to determine the range of capabilities for current street cleaning equipment to gain information about the cost and effectiveness of street cleaning programs for removing street surface pollutants.

The study also determined accumulation rates of street dirt in test areas with different characteristics. The pollution characteristics of street dirt are known to vary as a function of particle size (Sartor and Boyd 1972; Pitt and Amy 1973). This study examined specific concentrations of various pollutants in different particle size groups. It also examined the effectiveness of street cleaning equipment in removing different particle sizes from the street, as well as settling velocities and specific gravities for various particle sizes. These data demonstrate the potential quantity of pollutants that may be affected by street cleaning, the relationship of the pollutants to street dirt particle size, and the way various particle sizes may settle out in a water column (in the sewer system or in a treatment process).

Another area of concern is the transport of particulates in sewer systems and the associated mass balance relationships. In a combined sewer system, the sanitary sewage flow velocities are much less during dry weather than during wet weather when the additional urban storm runoff adds to the flow volumes. During dry weather, primary sanitary solids can settle out in the sewer system, to be flushed out during the high flows of wet weather. This increased concentration of solids can greatly add to the pollution load at the beginning of a storm (Burgess & Niple, Ltd. 1969). Storms with low runoff volumes may remove large quantities of road surface particulates and transport them to the sewer system. These particulates may settle out in the sewer system and be available for flushing during periods of larger flows. Stormwater management techniques utilizing in-line storage can also cause large quantities of solids to build up in the system (Lager and Smith 1974; Pisano 1977). Some data are available on the buildup and transport of these solids in separate sanitary sewer systems. This study obtains particulate routing data for a separate stormwater system through tracer studies. Comparisons of the amounts of pollutants in the street dirt and in the runoff from monitored storms also provided information concerning deposition characteristics in the sewer system and the relative quantity of pollutants in the runoff originating in land-use areas other than the street surface.

Metcalf and Eddy (Lager and Smith 1974), in a study conducted for the EPA, summarized the technology available for the treatment and management of urban runoff and costs and effectiveness of treatment. Unfortunately, comparable data for street cleaning programs have not been available. Some information on typical street sweeper performance is available from the earlier EPA-sponsored studies, but these limited data are based on idealized strip test conditions. This study obtained street cleaning performance data from tests in real-world conditions. These data were then used to make cost and labor effectiveness comparisons with some alternative control measures.

The study also examined the effect of street surface particulates on air pollution. Estimates of air pollutant emissions for EPA air quality regions, statewide areas, and specific air basins are very important for continuing air quality control planning. Most utility, industrial, and residential activities (including unpaved roads) have received attention as particulate air pollutant sources. Research by Roberts (1973) and Cowherd et al. (1977) indicates that paved roads should also be considered as important particulate air pollutant sources. Dust from the atmosphere, soil from erosion, and vehicular deposits on paved street surfaces can be disturbed by wind and traffic, causing particulate emissions. Street cleaning may be an effective means of removing these particulates before they can be blown into the air.

Very little quantitative information about particulate emissions from paved street surfaces is available, although some work has been done on related subjects. As part of an overall program to determine the behavior of radioactive fallout, the Atomic Energy Commission (now the Nuclear Regulatory Commission) has funded continuing studies of particulate residence times in the atmosphere, airborne particulate deposition rates, and resuspension of settled particulates. The particle resuspension studies have included research into resuspension from asphalt streets caused by traffic. Their results and theories are useful, but these studies consider only particles that have settled onto the street surface from the atmosphere. This study examines losses from the total particulate loading on the street surface, including that washed into the street through erosion and that tracked or deposited onto the street by vehicles.

It is expected that the study will have a two-fold benefit. First, the data obtained will fill in significant gaps in current knowledge about the role street dirt plays in causing water and air pollution and its control. Second, the carefully developed experimental design and sampling procedures for various portions of the study can be used by others wishing to obtain specific information about street dirt characteristics, its effects on air and water quality in their own cities, and its control.

The information presented here summarizes the data that have been collected and analyzed thus far. This information is subject to change as further data are gathered and analyzed. The effect this information may have on a specific city's street cleaning program is expected to vary widely, depending on conditions in that city. For this reason, the study does not yield a set of specific, how-to instructions. Rather, it indicates the type of information that must be considered in designing effective control measures.

DESCRIPTION OF THE STUDY AREAS

Eight potential study areas were considered within the city of San Jose. Three were selected as being representative of the variety of conditions found in San Jose and many other cities: the Tropicana study area, the Keyes Street study area, and a downtown study area.

The downtown and Keyes Street study areas lent themselves to division into two test areas, while the Tropicana study area was best treated as a single test area. Thus a total of five test areas were used in the initial field activities:

- Tropicana test area
- Keyes Street - asphalt street surface test area
- Keyes Street - oil and screens street surface test area
- downtown - good street surface test area
- downtown - poor street surface test area

Figure 8-1 shows the San Francisco Bay ARea and the general location of the city of San Jose. Figure 8-2 shows the three study areas selected and their location within the city of San Jose.

These areas were selected because they represent the variety of conditions found in San Jose and in many other cities. The combined downtown study area covers about 100 acres and has 7.0 curb-miles and 25 storm drain inlets. Its major land uses are commercial and industrial, with some older single- and multiple-family residential areas, much roadside vegetation, and many vacant lots (previously cleared for redevelopment).

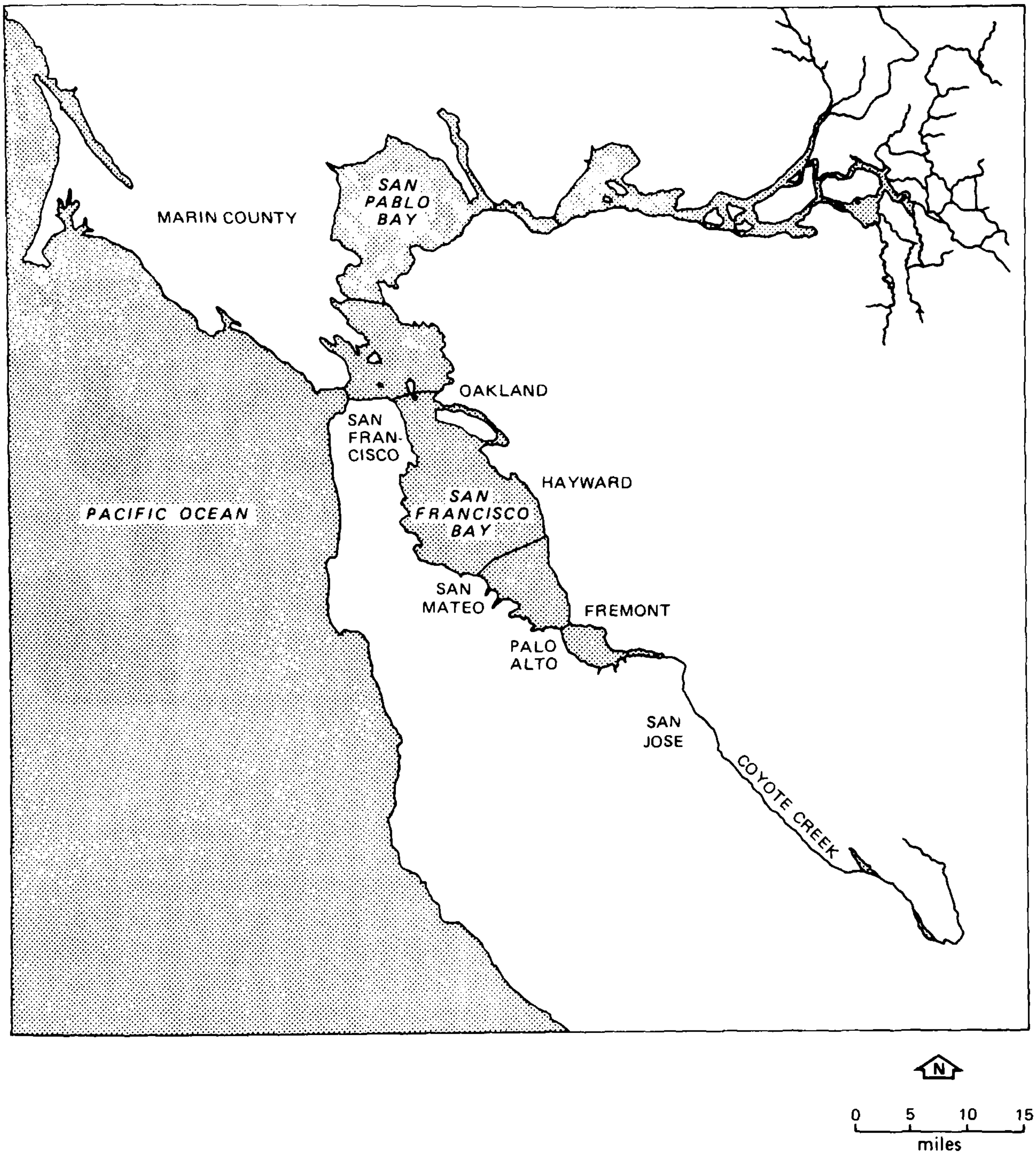

Figure 8-1. San Francisco Bay Area showing the general location of the City of San Jose.

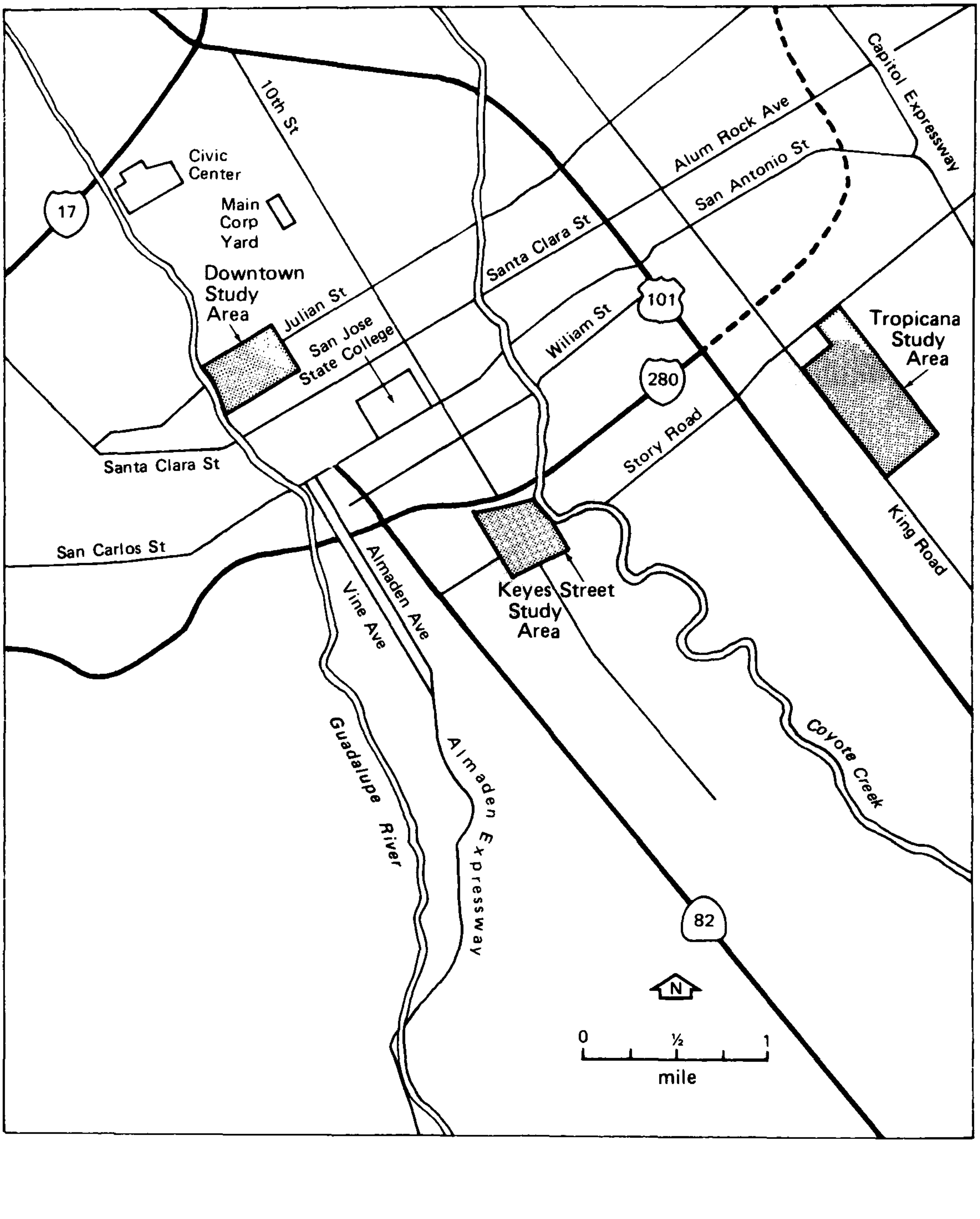

Figure 8-2. Map showing the location of the three study areas.

There was some construction in the area, and several streets have heavy traffic. The stormwater from this area is discharged directly into the Guadalupe River. The downtown commercial part of the study area is normally swept daily, and the remainder of the area is normally swept once every five weeks.

The combined Keyes Street study area covers about 92 acres and has 5.4 curb miles and 17 storm drain inlets. Its major land use is residential, composed of older (early 1900) single-family homes, with some strip commercial use. The study area is adjacent to several schools and playing fields. This area has few vacant lots, many roadside trees, and no construction. Several streets have heavy traffic, but most have light traffic. The stormwater is discharged directly into Coyote Creek. The area is normally swept every five weeks.

The Tropicana study area covers about 195 acres, and has 12.7 curb-miles and 55 storm drain inlets. The most common land use is residential, composed of low-income, single-family homes built around 1960. The area includes a portion of a large shopping center and is adjacent to three schools. There are few vacant lots, some roadside trees, and no construction activities in the area. Again, some streets have heavy traffic, but most carry light traffic. The stormwater is eventually discharged into Silver Creek (a tributary to Coyote Creek). This area is also normally swept every five weeks.

SAMPLING TECHNIQUES

One important aspect of the study is the development of sampling techniques that can be used to monitor accurately the changes in street surface loading for different test areas over a long period. These sampling procedures can be easily used by a public works department to determine the specific loading conditions and street cleaning performance for its city. The equipment can be rented if it is not available within the department. With these procedures, street surface loading conditions over a large area can be sampled in a relatively short time. The experimental design procedures can be used to determine the number of subsamples required for specific project objectives and study area conditions.

STREET CLEANING EQUIPMENT TESTS

Accumulation Rates

The accumulation rate characteristics of street surface contaminants must be known in order to understand the magnitude of the problem a street cleaning program must address and to determine the most effective control methods. This study showed that the accumulation rates varied widely in the different test areas. These variations are thought to be due to street surface conditions and to land-use patterns and activities within the test area such as the presence of vacant lots, commercial development, pedestrian and automobile traffic, and parking. Such variations should be considered in scheduling street cleaning programs for different types of areas.

Samples of each particle size category for each test area and equipment type were analyzed for various pollutants. Calculations were made to average the slopes (the change of street surface particulate loadings as a function of time) of each particle size to determine accumulation rates of each pollutant for each test area and equipment test phase. These calculated pollutant accumulation rates are shown in Table 8-1. This table presents the accumulation rates expressed as pounds of pollutant per curb-mile per day for each of the five test areas. The Tropicana, Keyes - asphalt, and downtown - good street surface test area values are divided into several accumulation time periods. The Keyes - oil and screens and the downtown - good street surface test area accumulation rates are only shown for a combined time period. Initially, accumulation rates were calculated for different effective accumulation periods for all study areas.*

Statistical tests were then conducted to determine which of these accumulation period values were important when compared with the value obtained from combining all the data together for each study area. It was expected that the accumulation rate measured over a short period of effective accumulation (near to the street cleaning date) would be greater than an accumulation rate measured over a longer period of effective accumulation. However, there was significant scatter in the data, and only a few subcategories of accumulation periods were found to be important. In most cases, the accumulation rates derived from the shorter accumulation periods are smaller. That would be portrayed with a sawtooth pattern of accumulation in which loading values level off with time. This effect is thought to be caused by wind and automobile-related air turbulence suspending the particles in the air. This pattern should be considered in establishing optimum street cleaning frequencies. However, it should be remembered that although longer periods between street cleaning may not result in significantly increased loadings, they could cause increased roadside airborne particulate concentrations. More important, significant differences in accumulation rates were found between the different test areas. These analyses will be repeated when all the data are available.

It is interesting to note that the overall pollutant accumulation rates in the oil and screens test area were smaller than for any of the other test areas, and yet the oil and screens test area always had the greatest street surface loadings observed. Because of the increased surface roughness and generally larger particle sizes in the oil and screens test area, a large quantity of loose material could stay on the street surface and not be removed significantly by rainfall. The smoother asphalt streets in the Tropicana and downtown - good test areas had accumulation rates that were about equal and reflect a large increase in street surface loadings with time (large accumulation rates). The smoother streets also allowed a more uniformly effective removal of street surface contaminants by the street cleaning equipment. The downtown - poor street surface test area had the largest accumulation rates

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*These periods were 0 to 2.0, 2.1 to 4.0, 4.1 to 9.0, 9.1 to 15.0, and greater than 15.0 days and for all time periods combined.

TABLE 8-1. ACCUMULATION RATES (lbs/curb-mile day)

Study Area	Effective Accumulation Period (days)	Total Solids	COD	Kjeldahl Nitrogen	Ortho-phosphates
Tropicana	0→2.0	86.3	8.1	0.17	0.015
	4.1→9.0	6.0	0.34	0.0078	0.00075
	All combined	54.1	4.92	0.10	0.0093
Keyes - oil and screens	All combined	11.3	2.92	0.046	0.0030
Keyes - asphalt	2.1→4.0	58.6	7.8	0.099	0.011
	4.1→9.0	18.6	2.6	0.042	0.0032
	All combined	73.9	9.54	0.12	0.012
Downtown - good street surface	All combined	58.6	8.19	0.11	0.0087
Downtown - poor street surface	0→2.0	729	80.1	1.7	0.11
	All combined	312	34.9	0.72	0.046

Study Area	Effective Accumulation Period (days)	Lead	Zinc	Chromium	Copper	Cadmium
Tropicana	0→2.0	0.37	0.040	0.043	0.092	0.00021
	4.1→9.0	0.017	0.0024	0.0047	0.010	0.000011
	All combined	0.22	0.024	0.026	0.057	0.00013
Keyes - oil and screens	All combined	0.068	0.010	<0.001	<0.001	0.00004
Keyes - asphalt	2.1→4.0	0.29	0.037	0.029	0.048	0.00017
	4.1→9.0	0.075	0.010	0.0098	0.017	0.00005
	All combined	0.29	0.040	0.038	0.066	0.00018
Downtown - good street surface	All combined	0.36	0.052	0.033	0.064	0.0018
Downtown - poor street surface	0→2.0	1.4	0.36	0.33	0.66	0.0019
	All combined	0.60	0.15	0.14	0.29	0.00082

of any of the test areas. These largest rates are thought to be caused by the poor condition of the streets and the character of the area, which cause a larger erosion of the street surface and accumulation of material from outside the street environment. Street cleaning performance is closely related to the accumulation rates and the initial contaminant loading values on the streets before street cleaning.

Concentrations of Street Surface Contaminants as a Function of Particle Size

Previous studies (Sartor and Boyd 1972; Pitt and Amy 1973) have demonstrated the importance of chemical analyses of different particle sizes instead of the total sample. The chemical character of each size is relatively constant (within a specific test area and time frame), but the percentage composition of the different sizes can vary significantly. Therefore, analyses of different particle sizes yield more useful information than total sample analyses.

Each collected sample was divided into eight particle sizes ($<45\mu$, $45 \rightarrow 106\mu$, $106 \rightarrow 250\mu$, $250 \rightarrow 600\mu$, $600 \rightarrow 850\mu$, $850 \rightarrow 2000\mu$, $2000 \rightarrow 6370\mu$, and $>6370\mu$). All of the samples collected in each test area for each equipment type were combined for chemical analyses by particle size. These chemical analyses were used to calculate total pollutant loadings for all of the samples collected. Almost all of the parameters for all of the test areas show higher concentrations with decreasing particle size. Mercury, cadmium, zinc, lead, Kjeldahl nitrogen, and total orthophosphates show highest concentrations with smaller particle sizes. However, copper and chromium show the lowest concentrations with the smallest particle size.

These data indicate that a control measure (such as conventional street cleaning methods) that is most effective in removing large particle sizes may be unable to remove enough of those pollutants found largely in the less abundant, smaller particle sizes to completely meet objectives unless extra effort is expended. Street cleaning can remove important amounts of these pollutants because they are also found in the more abundant larger particle sizes. The effectiveness of street cleaning therefore depends on the specific service area characteristics and program objectives.

The asbestos information obtained was subject to wide variation because of the small number of fibers counted in each sample aliquot. The lengths of the fibers observed ranged from 5 to 250 microns in length. Generally the smallest particle sizes had the shortest observed maximum fiber lengths.

No specific test area or test period had significantly different pollutant strengths.* The pollutant strengths observed during the first test phase were all within the range of strengths reported in previous investigations (Sartor and Boyd 1972; Pitt and Amy 1973). This information was

*Pollutant strengths are expressed as mg pollutant per kg total solids, which is equivalent to ppm on a weight basis.

used to determine the accumulation rates and street cleaning equipment performance for the different pollutants, based on particle size measurements.

Street Cleaning Equipment Performance

The design of an effective street cleaning program requires not only an understanding of the service goals,* but also a determination of accumulation rates and an assessment of specific street cleaning equipment performance in the actual conditions encountered. For this study, several street cleaning programs using various types of equipment and levels of effort were evaluated. This evaluation was the major element of the demonstration project. The following types of street cleaning equipment were studied under various operating conditions and cleaning frequencies:

- 4-wheel mechanical sweeper
- state-of-the-art mechanical sweeper
- vacuum-assisted sweeper

The purpose of this project was not to compare these specific types of street cleaning equipment, but to determine the range and capabilities of street cleaning equipment in general. These specific pieces of street cleaning equipment were selected for study because they represent three different generic types and were available for testing. It must be stressed that the performance, as measured in these tests, may not be an accurate indication of the ability of this equipment under other operating conditions. The scope and intent of this project was to demonstrate the range of possible cleaning effectiveness of different types of street cleaning equipment under a variety of real-world operating conditions. The available resources for the project required that the test be conducted in one city with a limited selection of the available equipment.

The cleaning frequencies used in this study ranged from two passes every day to one pass every week. Each piece of equipment was evaluated in the field in two different seven-week periods: once during the winter and once during the summer phase (with the exception of the vacuum-assisted sweeper). The first two weeks of each seven weeks of equipment evaluation used daily cleaning. One week used a single pass every weekday, and two passes were made each weekday during the other week. The last five weeks of each test period used weekly cleaning intervals. The equipment was rotated through the different testing areas at the end of each cleaning period. This schedule allowed the different characteristics and long-term seasonal differences in the test areas to be included in the evaluation of the range of equipment effectiveness. In addition to sweeping the specific test area, an adjacent buffer zone up to three times the size of the test area was also swept in order to reduce potential edge effects (tracking of particulates into the test areas from the adjacent areas).

*Service goals consider effects on water quality, air quality, public safety, aesthetics, and public relations. For a more complete discussion of this topic, see Pitt, Ugelow, and Sartor (1976).

The long-term and frequent sampling in the test areas made it possible to directly measure accumulation rates of street surface contaminants, which are discussed earlier in this paper. Street surface samples were collected within a few hours before and after street cleaning. The data collected in these tests were also used to identify the range of performances that may be expected from currently available street cleaning equipment. Differences of removal values (lbs/curb-mile removed) instead of percentage removals (percentage of initial loading removed) for the various test conditions were used as a more meaningful measure of equipment performances.

Table 2 presents the initial test phase street cleaning equipment performance data. Fourteen different test conditions are identified representing the different test areas, equipment types, number of passes, and the approximate cleaning intervals. The information presented for each of the before and after test samples includes the median particle size, the bulk density, and the street surface loading conditions. Under the column heading, the residual street surface loading values (lbs/curb-mile) are shown; these are generally the lowest street surface loading values that occur under each of the test conditions. Also shown is the amount removed, the percentage of the before loading removed, and the hopper content median particle size. The values shown are the mean ($\bar{x}$) plus or minus the standard deviation (σ).

Street cleaning performance depends on many conditions. These include the character of the street surface, the street surface initial loading characteristics (total loading value and particle size distribution), and various environmental factors. Equipment variables that affect street cleaning performance include the specific type of street cleaning equipment and its subsystems (types and adjustments of brooms, etc.), the number of passes that it makes, and the street cleaning interval. The most important measure of cleaning effectiveness is <u>pounds per curb-mile removed</u> for a specific program condition. This removal value, in conjunction with the unit curb-mile costs, allows one to calculate the cost for removing a pound of pollutant for a specific street cleaning program. The <u>percentage of the before loading removed</u> has often been used as a measure of street cleaning equipment performance. It is very misleading, however, because it is not a measure of the magnitude of the amount of material removed. A street cleaning program may have a very low percentage removal value, but a high total amount removed, if the initial loading is high. That occurred in the tests conducted in the oil and screens area.

Student <u>t</u> statistical tests were conducted with the data shown in Table 8-2 to determine improtant similarities and differences in street cleaning equipment performance under the various test conditions. These tests showed that initial loading values <u>in any one test area</u> varied depending on the street cleaning program (number of passes and cleaning intervals). However, the differences in the initial loading values <u>in various test areas</u> were controlled by differences in test area conditions (largely street surface conditions and accumulation rates), irrespective of the type of equipment being used and the number of passes.

TABLE 8-2. INITIAL TEST PHASE - STREET CLEANER PERFORMANCE (with mean ± standard deviation)

				Before Street Cleaning			After Street Cleaning			Cleaning Effectiveness		
Study Area	Equipment Type*	Number of Passes	Approx. Cleaning Interval	Median Particle Size (μ)	Bulk Density	Street Surface Loading (lbs/ curb-mile)	Median Particle Size (μ)	Bulk Density	Street Surface Loading (lbs/ curb-mile)	Amount Removed (lbs/ curb-mile)	Percentage of Before Loading Removed	Hopper Contents Median Particle Size (μ)
Tropicana	A	1	Daily	965±1160	1.30±0.14	115±38	430±57	1.20±0.08	98±45	17±22	13±25	3170±1410
	A	1	Weekly	510±63	0.98±0.05	164±65	450±78	1.15±0.06	87±38	77±38	47±11	5750±4380
	B	1	Daily	430±130	1.12±0.11	350±274	300±46	1.10±0.17	165±64	185±225	53±19	2090±850
	C	2	Daily	410±95	0.82±0.13	328±93	320±29	1.06±0.09	132±56	196±131	60±32	3190±1030
Keyes - oil and screens	B	1	Weekly	670±35	1.37±0.06	2370±110	600±32	1.3±0.17	1860±104	510±44	22±2	4550±1100(1)
	C	1	Weekly	930±350	1.15±0.13	2200±102	660±21	1.20±0.20	2030±293	171±258	8±12	4460±2500(2)
	B	2	Daily	650±250	1.3±0.14	1830±378	570±24	1.28±0.10	1930±403	-98±300	-6±17	5940±2390(3)
	A	2	Daily	560±19	1.35±0.06	2654±797	600±36	1.38±0.05	2208±375	445±461	17±11	940±380(4)
Keyes - asphalt	B	1	Weekly	520±67	0.87±0.06	381±29	390±28	0.97±0.12	294±67	87±45	23±14	4550±1100(1)
	C	1	Weekly	510±120	0.78±0.15	459±57	390±25	0.90±0.18	295±73	165±34	36±10	4460±2500(2)
	A	2	Daily	480±36	0.98±0.05	401±122	460±69	1.1±0.08	258±81	144±155	36±27	940±380(4)
	B	2	Daily	450±175	1.00±0.17	173±61	340±45	1.07±0.15	142±16	32±49	19±22	5520±2740(3)
Downtown - good street surface	C	1	Daily	430±62	0.99±0.06	243±32	380±54	1.03±0.05	160±15	83±18	34±3	2660±1200(5)
Downtown - poor street surface	C	1	Daily	570±27	0.98±0.18	1350±394	530±66	0.98±0.08	808±189	543±429	40±24	2660±1200(5)

*Equipment types are designated in the following way: A = 4-wheel vacuum-assisted mechanical sweeper; B = state-of-the-art 4-wheel mechanical sweeper; C = 4-wheel mechanical sweeper.

Note: Adjacent test areas at (1), (2), (3), (4), and (5) were swept with the same equipment and the hoppers were not cleaned out between these tests; the study areas overlapped. As a result, these pairs have the same hopper content median particle sizes.

When the residual loading values were statistically examined, the findings were similar. Differences in test area conditions were much more important than differences in equipment type. Similarly, the amount removed under each of the test conditions was more a function of the test area than the street cleaning program. In many cases, two passes with the same piece of equipment removed a larger quantity of material from the street than a single pass (as expected). An exception was found in the tests in the oil and screens test area. Here two passes per day of the state-of-the-art mechanical sweeper actually resulted in a higher residual loading on the street surface than before the test. This result is thought to be due to the extra erosion caused by excessive mechanical action of the broom on the weaker street surface. During a single pass, any extra material loosened from the street surface was removed along with the initial dust and dirt on the street.

The selection of the specific type of street cleaning equipment is less important than the characteristics of the area to be swept. Also, in most cases, the street cleaning interval and number of passes were more important than the specific type of equipment used. Other considerations, such as maneuverability, life-cycle costs, hopper capacity, etc., may be more important from an equipment selection viewpoint. There are, however, expected to be situations not studied as part of this demonstration project in which one type of street cleaning equipment would perform significantly differently from the others.

The median particle size of the material collected in the equipment hopper can reflect differences in equipment performance as a function of particle size. A larger median particle size of the hopper material signifies that not as many smaller particles were removed from the street. Similarly, a smaller median particle size of the hopper material signifies a relatively greater removal of small particle sizes under the same conditions. In all cases, the hopper median particle sizes are much larger than the median particle sizes on the street surface before street cleaning, and the street surface median particle size decreases with street cleaning. Thus, there is a larger percentage of smaller particles on the street after street cleaning than before, with the street cleaning equipment being most effective in removing the larger particle sizes. The vacuum-assisted mechanical street sweeper had a smaller median hopper sample particle size than the other types of test equipment in the Keyes test areas. However, this difference observed in the hopper did not significantly change the median particle size of material found on the street.

PARTICULATE ROUTING AND POLLUTANT MASS FLOW CHARACTERISTICS OF URBAN RUNOFF

Flow Characteristics

The purpose of measuring the urban runoff flows was to calculate pollutant mass yields* from the concentration values found in the sampling program. Information as to these mass yields is important in determining the effect these pollutants may have on receiving waters. The general

*As measured in lbs/hr or total lbs.

hydrographic information from the study may also be useful in verifying urban runoff models, particularly because the characteristics of the study area resulted in a relatively quick response of runoff flow to precipitation.

The hydrographs of the monitored flows showed a lag of 1 to 6 hours between the beginning of precipitation and the start of measurable flow. The most common lag was about 1 hour. The flows also continued for 3 to 8 hours after precipitation, and peak recorded flows lagged peak precipitation by 1 to 2 hours. In most cases, a precipitation total of 0.01 inch caused a measurable flow at the outfalls. Concentrations of some parameters were also monitored at different times during the period of flow; as could be expected, most decreased in concentration with lapse of time.

Pollutant Concentrations

The COD concentrations measured in the runoff were about three to ten times greater than the BOD_5 values, and the TOC concentrations were ten times the BOD_5 concentrations. For a normal waste, having low toxicity and sufficient nutrients, the COD values should only be slightly greater than the BOD_5 values.

Figure 8-3 presents BOD values as a function of incubation time. Selected composite samples representative of each storm were incubated for up to 20 days and BOD values were measured at increments of approximately 1, 3, 5, 10, and 20 days. The relative BOD values shown in the time interval from 0 to 10 days are about what was expected. The 5-day BOD values are about two-thirds the 10-day BOD value. The largest rate of BOD increase in the first 10 days occurred usually on the first day, with 1-day BOD values of about 20 mg/L (for two of the three samples). This value remained relatively constant until about the fifth day when it gradually rose to the 10-day value. The most unusual character of the BOD value is shown in the period of time from 10 to 20 days when the BOD values typically increased by a factor of two or more. These results show that the initial oxygen demand is rapid and may have possible deleterious effects on certain receiving waters close to the time of discharge (within the first day). However, as the material settles out, it can exert a much larger, long-term oxygen demand. Therefore, the oxygen depletion caused by urban runoff is important both immediately after discharge and at periods of time longer than 10 days after discharge. The period after the first several days and before 10 days may not pose as great a problem. (These time factors are all dependent on water temperature and other physical and chemical characteristics of the receiving water.)

This apparent long-term increase in oxygen demand may be caused by some of the inherent problems in the standard bottle BOD test when analyzing toxic and/or low nutrient samples. Because urban runoff has relatively high concentrations of heavy metals and low concentrations of nutrients, the seed bacteria require a longer time for acclimatization than normal. The initial oxygen demand could be caused by the relatively easily assimilated organics being consumed by the standard seed bacteria before significant bacteria die-offs occur from heavy

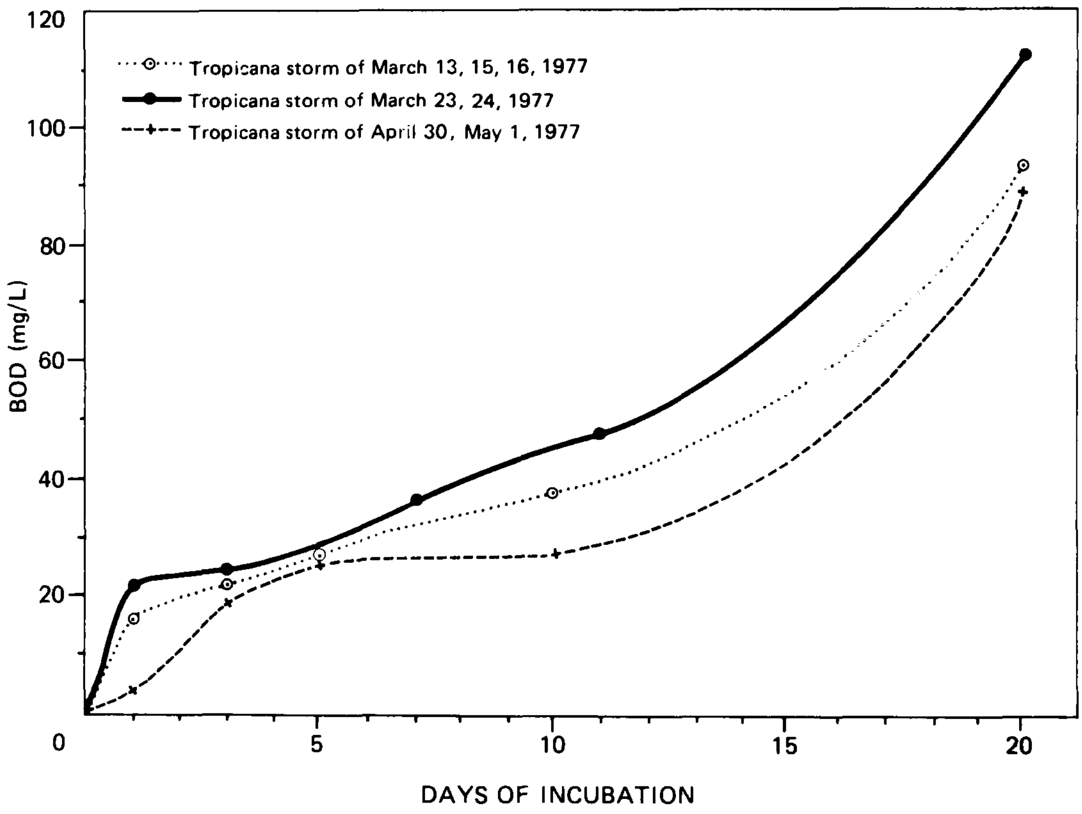

Figure 8-3. BOD values as a function of incubation time.

metal toxicity. The lag period of several days could be required for the surviving seed bacteria to become acclimated and reestablished so as to assimilate the remaining organics. Colston (1974) has developed an alternative BOD procedure for urban runoff based on measurements of COD with time in an aerated and mixed sample, using typical receiving waters for dilution. This alternative procedure will be used in analyzing the BOD of the street surface particulates, and the results will be included in the final report.

When the runoff pollutant strengths (mg pollutant/kg total solids) are compared with the street surface contaminant pollutant strengths, notable differences are found. The relative concentrations in the runoff for COD, Kjeldahl nitrogen, and orthophosphates are much greater than the relative concentrations observed in the street dirt (about 3 to 180 times greater in the runoff).

Some of the zinc and cadmium relative concentrations were also greater in the runoff than in the street dirt. However, the relative concentrations of lead, chromium, and copper in the runoff were all much smaller than those measured on the street. These differences ranged from about 2 to 20. If the erosion products have lower concentrations of heavy metals, the resultant runoff concentrations of heavy metals would be diluted when compared to the higher concentrations in the street dirt. Therefore, it may be that much of the organic and nutrient material in urban runoff originates not from the street surface or from automobile activity, but from the surrounding areas through erosion. Similarly, most of the heavy metals in urban runoff are expected to be associated with street surfaces and automobile activity. A similar conclusion was also identified by Amy et al. (1974). In that study, the authors analyzed existing runoff and street surface loading data in an attempt to determine a loading model as a function of various influencing characteristics (such as geographical area, land use, traffic conditions, etc.). They found that when the street surface loading data were compared with the runoff data, the only major differences in loading predictions were for nutrients. In that case, the nutrient values predicted for runoff data were greater than for street loading data, reflecting the fact that most of the nutrients originate in off-street areas.

Pollutant Removal Capabilities of Monitored Storms

The monitored rains had a much smaller effect on removing materials from the oil and screens test area as compared with the asphalt test areas. It is thought that the increased roughness of the street surface in the oil and screens area trapped much of the erosion material from the surrounding areas on the street and prevented it from reaching the storm sewer system. The Keyes - asphalt and Tropicana test areas, both having relatively smooth asphalt streets, showed large removals of material. The first storm showed a smaller absolute removal as compared to the latter two storms, possibly because of its increased intensity and larger erosion yields from surrounding areas that found their way onto the street during the rain.

The runoff removals in both the Keyes - asphalt and Tropicana study areas for the March 23 and 24 storm and for the April 30 to May 1 storm were very similar. These last two relatively small storms were capable of removing significant quantities of material from the street surface, yet did not cause large amounts of erosion products in the runoff.

Table 8-3 summarizes the pollutant street surface loading changes for the different rain storms on a curb-mile basis and also on a total pounds basis for the two study areas. These runoff yields, as measured on the street surface, are compared to the total pollutant yields of the storms. The observed ratios between street surface loading differences of the pollutants as measured on the street and the runoff yield as measured by analyzing runoff vary. Values smaller than 1 possibly signify that more of that pollutant originated in the surrounding areas than on the street surface. Values greater than 1 possibly indicate that much of the material washed off from the street surface accumulated in the storm sewer.

These ratios appear to vary as a function of the rainstorm characteristics, the study area, and the specific pollutants. The March 15 and 16 storm generally had ratios less than 1 for all of the pollutants in both study areas, while the last two storms shown in Table 3 had many values greater than 1. Again the initial storm was of much greater intensity and volume, possibly causing greater erosion in the surrounding areas and increased sewerage velocities that would keep the particulate material from settling in the storm drainage. The last two storms, however, were of relatively small intensity and showed almost complete removal of street surface contaminants from the street surface. That is probably due to the extra energy imparted to the street surface materials from automobile traffic and the sufficient rain available to wash the loosened materials from the street surface to the storm drain inlet. However, the smaller flows in the sewerage were not capable of preventing the material from depositing in the sewerage.

Comparison of Runoff Water Quality with Recommended Receiving Water Quality Criteria

Table 8-4 compares the overall ranges and average concentrations of the monitored runoff to the recommended water quality criteria for various beneficial uses. The water quality criteria values shown for these uses are recommended maximum limits designed to protect the beneficial uses with a reasonable amount of safety. If a monitored concentration exceeds these criteria, it does not mean that a problem exists but that a problem may occur and additional monitoring may be necessary to define the relationships between water quality and impairment of the beneficial uses for the specific receiving water.

The following list summarizes those parameters that exceeded the recommended beneficial use criteria.

TABLE 8-3. STREET SURFACE POLLUTANT REMOVALS COMPARED WITH RUNOFF YIELDS

	Keyes Street Study Area							Tropicana Study Area			
	Oil and Screens		Asphalt								
Parameter	Lbs/Curb-Mile Difference	Lbs Difference (2.2 curb-mile)	Lbs/Curb-Mile Difference	Lbs Difference (2.7 curb-mile)	Total Keyes Area Lbs Difference	Runoff Yield (lbs)	Street Surface Difference to Runoff Yield Ratio	Lbs/Curb-Mile Difference	Lbs Difference (11.1 curb-mile)	Runoff Yield (lbs)	Street Surface Difference to Runoff Yield Ratio
MARCH 15-16, 1977, STORM											
Total solids	120	260	29	78	340	942	0.36	120	1300	8099	0.16
COD	24	53	3.0	8.1	61	859	0.071	11	120	2267	0.05
KN	0.33	0.73	5.2	14	15	51.8	0.28	0.22	2.4	90.2	0.03
$OrthoPO_4$	0.023	0.051	0.0049	0.013	0.064	21.1	0.003	0.020	0.22	65.8	0.003
Pb	0.40	0.88	0.19	0.51	1.4	1.75	0.79	0.47	5.2	6.5	0.80
Zn	0.067	0.15	0.022	0.059	0.21	0.71	0.29	0.054	0.60	2.9	0.21
Cr	-0.0084	-0.018	0.014	0.038	0.020	0.065	0.31	0.059	0.66	0.4	1.6
Cu	-0.014	-0.031	0.024	0.065	0.034	0.13	0.26	0.13	1.4	0.45	3.2
Cd	0.00031	0.001	0.0001	0.0001	0.001	0.026	0.038	0.0003	0.003	0.055	0.06
MARCH 23-24, 1977, STORM											
Total solids	-130	-290	430	1200	910	134	6.8	300	3300	1260	2.6
COD	8.8	19	58	160	180	68	2.6	27	300	740	0.41
KN	0.21	0.46	0.97	2.6	3.1	0.70	4.4	0.57	6.3	17	0.37
$OrthoPo_4$	0.016	0.035	0.076	0.21	0.25	--	--	0.053	0.59	2.1	0.28
Pb	0.47	1.0	2.0	5.4	6.4	0.15	43	1.3	14	0.90	16
Zn	0.037	0.081	0.26	0.70	0.78	0.063	12	0.14	1.6	0.53	2.9
Cr	-0.14	-0.31	0.22	0.59	0.28	0.0059	47	0.16	1.8	0.042	42
Cu	-0.32	0.70	0.37	1.0	1.7	0.0079	210	0.34	3.8	0.060	63
Cd	0.0001	0.0001	0.0012	0.003	0.003	0.0008	3.8	0.0007	0.008	0.009	0.86
APRIL 30 - MAY 1, 1977, STORM											
Total solids	-540	-1200	650	1800	600	11.6	52	260	2900	1850	1.6
COD	20	44	88	240	200	--	--	24	270	1250	0.21
KN	-0.24	-0.53	1.4	3.8	3.3	--	--	0.49	5.4	72	0.076
$OrthoPO_4$	-0.018	-0.040	0.11	0.30	0.26	0.13	2.0	0.045	0.50	29	0.017
Pb	-0.075	-0.17	2.6	7.0	6.8	--	--	1.1	12	3.2	3.8
Zn	-0.089	-0.20	0.36	0.97	0.77	--	--	0.12	1.3	1.3	1.0
Cr	-0.35	-0.77	0.34	0.92	0.15	--	--	0.13	1.4	0.1	14
Cu	-0.62	-1.4	0.59	1.6	0.20	--	--	0.28	3.1	0.23	14
Cd	-0.0007	-0.002	0.0017	0.005	0.003	--	--	0.0006	0.007	0.009	0.74

TABLE 8-4. RUNOFF WATER QUALITY COMPARED TO BENEFICIAL USE CRITERIA

Parameter[a]	Overall Observed Range	Overall Observed Average	Beneficial Use Criteria[b] Irrigation	Livestock	Wildlife	Aquatic Life	Marine Life	Recreational Uses	Freshwater Public Supply
pH (pH units)	6.0→7.6	6.7	4.5→9.0 desired	--	6.0→9.0 desired	6.0→9.0 desired	6.5→8.5 desired	5.0→9.0 desired	5.0→9.0 desired
Temp. (°C)	14→16.5	16	Narrative	--	Maintain natural pattern	Narrative	Narrative	86°F	Narrative
DO	5.4→12.8	8.0	--	--	--	Usually 5.0 mg/L min.	6.0 mg/L min.	--	Narrative
Turbidity (NTU)	4.8→130	49	--	--	--	Small change	--	4 ft (secchi)	Narrative
TDS	22→376	150	500→5000 mg/L max.	--	--	Narrative	--	--	Narrative
SS	15→845	240	Narrative	--	--	80 mg/L	--	--	--
NO_3	0.3→1.5	0.7	Narrative	450 mg/L (including NO_2)	--	--	--	--	45 mg/L
PO_4	0.2→17.6	2.4	--	--	--	--	0.0003 mg/L	0.3 mg/L for streams; 0.08 for lakes	Narrative
Cl	3.9→17.6	12.1	--	--	--	--	--	--	250 mg/L
SO_4	6.3→27	18	--	--	--	--	--	--	250 mg/L
Na	2.1→26.8	15	Narrative	--	--	--	--	--	Narrative
Cd	<0.002→0.04	0.01	0.01→0.05 mg/L max.	0.5 mg/L	--	0.004→0.03 mg/L max. for soft→hard water	0.01 mg/L	--	0.01 mg/L
Cr	0.005→0.04	0.02	0.1→1.0 mg/L max.	1.0 mg/L	--	0.03 mg/L	0.1 mg/L	--	0.05 mg/L
Cu	0.01→0.09	0.03	0.2→5.0 mg/L max.	0.5 mg/L	--	Narrative	0.05 mg/L	--	1 mg/L
Pb	0.10→1.5	0.4	5.0→10.0 mg/L max.	0.1 mg/L	--	0.03 mg/L	Narrative	--	0.05 mg/L
Hg	0.0001→0.00006	<0.0001	--	0.001 mg/L	Narrative	0.00005 mg/L	0.1 mg/L	--	0.002 mg/L
Zn	0.06→0.55	0.18	--	25 mg/L	--	Narrative	0.1 mg/L	--	5 mg/L
BOD_5	17→29.8	24	--	--	--	10 mg/L	Narrative	--	Narrative

[a]Parameters are measured in mg/L unless otherwise noted.

[b]Maximum limits unless stated as desired range or minimum values.

Livestock : Pb*
Wildlife: none
Aquatic life: Cr, Cd,* Pb,* Hg,* BOD_5, turbidity,* suspended solids*
Marine life: PO_4,* Cd, Cu, Zn
Recreational uses: PO_4*
Freshwater public supply: Cd, Pb*
Irrigation: Cd

The heavy metals - cadmium, chromium, lead, mercury, and zinc - along with phosphates, BOD_5, suspended solids, and turbidity can exceed the recommended criteria. Drinking water standards are not presented because the water would be treated before use and the freshwater public supply criteria applies to the water source. The high turbidity of the runoff water is expected to exceed the narrative criterion for aquatic life. Observed average and maximum suspended solids runoff concentrations exceeded the aquatic life criterion. All of the runoff phosphate concentrations exceeded the recreation criterion by a large amount. The phosphate recreation criterion is designed to prevent eutrophication** in receiving water. Average and maximum cadmium concentrations exceeded the irrigation, aquatic life, marine, and freshwater supply criteria. Maximum copper and chromium concentrations in the runoff also exceeded the aquatic life and marine criteria. All of the lead concentrations in the runoff exceeded the livestock, aquatic life, and freshwater supply criteria by large amounts. The maximum runoff mercury concentrations exceeded the aquatic life criterion by a large amount. The average and maximum zinc runoff concentrations exceeded the marine life criterion. All of the observed BOD_5 concentration values in the runoff exceeded the aquatic life criterion. As these data show, those parameters most potentially responsible for water quality impairment are solids, cadmium, lead, and mercury for aquatic life uses; orthophosphates for marine life; orthophosphates for eutrophication (recreational use); and lead for freshwater public suply. Street cleaning operations can remove portions of these pollutants from the source area before rains can wash them into the receiving water.

Comparisons of Runoff Water Quality with Sanitary Wastewater Effluent Water Quality

Table 8-5 presents a comparison between sanitary wastewater treatment facility effluent and urban runoff for the study areas. The average and peak one-hour runoff concentrations observed and average sewage treatment plant effluent concentrations are shown along with the ratios between them. The sewage treatment facility is a modern, advanced secondary treatment plant serving the study areas. The short-term effects of urban runoff on a receiving water are most important (by definition) during and immediately following a runoff event: short-term effects are associated with instantaneous concentrations. A comparison between the urban runoff average concentrations and the sewage treatment plant effluent average concentrations shows that the concentrations of lead, suspended solids, COD, cadmium, TOC, turbidity, zinc, chromium, and BOD_5 are all higher in the runoff than in the sewage plant effluent. Copper and Kjeldahl nitrogen, in addition to the previously listed parameters, have greater runoff peak concentrations than the sewage

*Greater than ten times the recommended criterion.
**Excessive algae growth that may become a nuisance.

TABLE 8-5. COMPARISON OF URBAN RUNOFF AND WASTEWATER TREATMENT PLANT EFFLUENT

Parameter	Runoff Concentration Avg (mg/l)	Runoff Concentration Peak (1-hr) (mg/l)	Average STP[a] Effluent Concentration (mg/l)	Ratio of Avg Runoff to STP	Ratio of Peak Runoff to STP	Annual Runoff[b] (tons/yr)	Annual STP Effluent[c] (tons/yr)	Ratio of Runoff to STP Annual Yields
Ca^{++}	13	19	65	0.20	0.29	—	8,790	—
K^{+}	2.7	3.5	23.8	0.11	0.15	—	3,220	—
Mg^{++}	4.0	6.2	35	0.11	0.18	—	4,690	—
Na^{+}	15	26.8	218	0.07	0.12	—	29,500	—
Cl^{-}	12.1	17.6	330	0.04	0.05	—	44,600	—
$SO_4^{=}$	18	27	148	0.12	0.18	—	20,000	—
HCO_3^{-}	54	153	233	0.23	0.66	—	31,500	—
NO_3	0.7	1.5	4.9	0.14	0.31	—	663	—
BOD_5	24	29.8	21	1.1	1.4	—	2,840	—
COD	196	350	35[d]	5.6	10	8,900	4,730[d]	1.9
KN	6.7	25	23.9	0.28	1.1	160	3,230	0.05
$OrthoPO_4$	2.4	17.6	19.2	0.13	0.92	12	2,600	0.005
Total solids	350	952	1,040	0.34	0.92	74,000	141,000	0.53
TDS[e]	150	376	1,010	0.15	0.37	—	137,000	—
Suspended solids	240	845	26	9.2	32	—	3,520	—
Cd	0.01	0.04	0.002	5	20	0.41	0.27	1.5
Cr	0.02	0.04	0.016	1.3	2.5	34	2.2	15
Cu	0.03	0.09	0.081	0.37	1.1	69	11.0	6.3
Pb	0.4	1.5	0.0098	41	150	230	1.3	180.
Zn	0.18	0.55	0.087	2.1	6.3	42	11.8	3.6
Hg	<0.0001	0.0006	0.0019	<0.05	0.32	—	0.26	
Specific conductance (μmhos/cm)	118	660	1850	0.06	0.36	—	—	—
Turbidity (NTU)	49	130	20	2.5	6.5	—	—	—
pH (pH units)	6.7	7.6	7.6	0.88	1.0	—	—	—
TOC[f]	106	290	30	3.5	9.7	—	4,060	—

[a]Sewage treatment plant.

[b]About 200 people correspond to one curb-mile (2,880 curb-miles in San Jose/575,000 population). Therefore a population of 850,000 corresponds to about 4,250 curb-miles, with about 1,100 curb-miles of streets surfaced with oil and screens. These annual runoff values were calculated based on a year of the appropriate accumulation rates and these mileage estimates.

[c]An estimated population of 850,000 is served by the sewage treatment facility.

[d]Estimated. [e]Total dissolved solids. [f]Total organic carbon.

plant effluent average concentrations. Therefore, urban runoff may have more important short-term effects on receiving waters than average treated sanitary wastewater effluent.

The annual yield for the different sources gives a measure that indicates the long-term problems. Table 5 shows the annual sewage treatment plant effluent yield expressed as tons per year (derived from monthly average concentrations and effluent quantities) and a calculated annual urban runoff yield expressed in tons per year for a similar service area. On an annual basis, the total orthophosphates associated with the street dirt are less than 1 percent of the total sewage treatment plant plus urban runoff yield. The Kjeldahl nitrogen contribution from urban runoff is about 5 percent of the total urban runoff plus sewage treatment effluent. The total solids contribution from urban runoff is about 35 percent of the total. Lead, chromium, copper, zinc, COD, and cadmium all have contributions from urban runoff greater than 50 percent of the total (99, 94, 86, 78, 66, and 60 percent, respectively).

These data show that for a receiving water getting both secondary treated sewage wastes and untreated urban runoff, additional improvements in the sanitary sewage effluent may not be as cost-effective as some treatment of urban runoff (except for nutrients). That is especially true for lead and chromium, where more than 90 percent of the total wasteload is due to urban runoff. As an example, if all of the lead were removed from the sanitary wastewater effluent, the total annual lead discharge would only decrease by about 1 percent, because the urban runoff accounts for approximately 99 percent of the total long-term lead yield.

TREATABILITY OF NONPOINT POLLUTANTS AND COST AND SELECTION OF CONTROL MEASURES

Street Cleaning Costs

Table 8-6 presents San Jose street cleaning costs for the year ending September 30, 1977. Labor accounts for about 65 percent of the total costs. Those categories that may be affected by a significant change in street cleaning equipment (maintenance costs) make up 30 percent of the total costs. During this same period, the Public Works Department of San Jose swept 55,761 miles. The unit cost was therefore about \$16 per mile swept, and the labor requirement was about 0.9 man-hour per mile. Initially, these costs appear high, but it must be realized that most other evaluations of street cleaning costs do not include all of the actual costs of the street cleaning program. Most other street cleaning cost evaluations include only maintenance and operation supplies and operator labor expenses. Few other jurisdictions have all the other cost information available.

Table 8-7 presents the unit cost effectiveness for the street cleaning operations based on preliminary information. The unit costs (dollars per pound removal for a pollutant) range from 8¢ per pound of total solids removed to \$8,000 per pound of cadmium. The average unit labor needs are also shown in this table and range from 30 seconds per pound of total solids removed to 450 hours per pound of cadmium.

TABLE 8-6. SAN JOSE ANNUAL STREET CLEANING COSTS

Item	Cost ($)	Percentage of Total Cost	Labor (person-days)
Maintenance supplies	93,000	10	--
Operation supplies[a]	29,000	3	--
Debris transfer and disposal	171,000	19	780
Equipment depreciation	31,000	3	--
Labor[b]			
Sweeper operators	326,000	36	3,400
Maintenance personnel	176,000	20	1,200
Supervisors	80,000	9	650
Total annual costs	$906,000	100%	6,030 days
Total annual curb-miles swept	55,761 miles		
Unit effort	$16/mile swept	0.9 hour/mile swept	

[a]Tires, fuel, and oil.

[b]These labor costs include administration, warehouse, secretary, and overhead costs.

TABLE 8-7. PRELIMINARY ESTIMATES OF COST EFFECTIVENESS FOR SAN JOSE STREET CLEANING OPERATIONS

Parameter	Average Removal (lbs/curb-mile)	Average Unit Cost ($/lb removed)	Average Unit Labor (hrs/lb removed)
Total solids	200	0.08	0.005
Suspended solids	100	0.16	0.01
COD	24	0.67	0.04
OrthoPO_4	0.032	500	28
Kjeldahl nitrogen	0.42	38	2.1
Lead	0.80	20	1.1
Zinc	0.12	130	7.5
Chromium	0.10	160	9.0
Copper	0.20	80	4.5
Cadmium	0.002	8,000	450

Urban Runoff Treatment Costs

Table 8-8 presents estimated costs for treating urban runoff and the various runoff treatment operations and processes. When flow equalization (storage) and collection facility costs are excluded, the unit costs are all significantly less than the unit costs for street cleaning operations. However, when flow equalization costs are included, the unit costs for removal of a pound of the various pollutants are all much larger than similar costs for street cleaning operations. If collection facilities are also necessary (such as collection trunklines), these unit costs would be much greater. The costs utilized in these calculations include the annual operation and maintenance costs, depreciation costs, and interest costs over the expected life of the project. Estimated average cost and labor effectiveness values are also shown in this table. The operation and maintenance labor unit effectiveness for these runoff control processes are all about one-half to one-hundredth of the unit labor requirements for street cleaning operations.

Combined Sanitary Wastewater and Runoff Treatment Costs

Table 8-9 presents cost information for the San Jose-Santa Clara Water Pollution Control Plant. Unit costs and unit labor requirements are also shown. It is assumed that these costs and labor requirements would remain approximately the same if the facility began treating combined urban runoff and sanitary wastewater. These costs are for the most part less than the unit costs for the special treatment facilities without flow equalization and collection processes. Unfortunately, there are no adequate data to compare the unit removal costs and labor effectiveness for treating heavy metals in the runoff systems. It is expected that these unit requirements for the important heavy metals (Pb, Zn, Cu) would be much greater than requirements for street cleaning programs.

Decision Analysis

The types of information summarized here can be used to recommend a specific set of treatment and removal systems necessary to meet multiple objectives. The objectives to be considered may include runoff and receiving water quality, air quality (as measured by fugitive dust concentrations alongside roadways), aesthetic consideration, public safety objectives (reducing accumulations of loose debris in intersections), least cost, maximum labor use, and public relations objectives (demonstrating to the taxpayers how their tax dollars are being spent). A decision analysis procedure that considers these multiple objectives and the partial fulfillments of each alternative should be used in determining how much of the total urban runoff control program should be addressed by street cleaning. A candidate procedure will be described in the final report.

AIRBORNE FUGITIVE PARTICULATE LOSSES FROM STREET SURFACES

As stated previously, the street surface particulate accumulation rate is greatest shortly after street cleaning when the streets are relatively clean. Particulate loading values then level off with the passage

TABLE 8-8. ESTIMATED COSTS FOR TREATING URBAN RUNOFF

Process	Unit Costs, Excluding Flow Equalization and Collection ($/lb)					Unit Costs, Including Flow Equalization, Excluding Collection ($/lb)				
	Suspended Solids	BOD_5	COD	N	PO_4	Suspended Solids	BOD_5	COD	N	PO_4
Swirl concentrator	0.003	--	--	--	--	No flow equalization needed				
Sedimentation	0.036	--	--	--	--	2.00	--	--	--	--
Dissolved air flotation	0.032	0.42	0.06	4.00	2.00	1.00	14	2.00	130	70
Micro-straining	0.004	0.08	--	--	--	1.00	23	--	--	--
Filtration	0.026	0.31	0.03	--	--	0.90	17	1.50	--	--
Contact stabilization	0.04	0.38	--	2.00	5.00	0.90	9	--	48	110
Trickling filters	0.07	0.59	--	--	--	1.30	11	--	--	--
Rotating biological contactors	0.04	0.33	0.06	2.00	4.00	1.10	9	1.70	60	110
Aerated lagoons	0.03	0.29	--	--	--	1.00	8	--	--	--
Physical-chemical	0.12	1.00	0.17	3.50	8.00	0.90	8	1.30	27	65
Average cost ($/lb removed)	0.04	0.40	0.08	2.90	5.00	1.10	12	1.60	66	90
Estimated labor (hrs/lb removed)	0.007	0.70	0.01	0.30	0.50	0.007	0.70	0.01	0.30	0.50

TABLE 8-9. SAN JOSE-SANTA CLARA WATER POLLUTION CONTROL PLANT, EFFLUENT CONDITIONS (1975-76 data)

Parameter	Influent Concentration (mg/L, except as noted)	Effluent Concentration (mg/L, except as noted)	Percentage of Removal	Tons/ Year Removed	Tons/ Year Effluent	\$/Lb Removed	Man-Hours/ Lb Removed
Flow	89×10^6 gal./day*	--	--	--	--	--	--
Total solids	--	1040	--	--	141,000	--	--
Suspended solids	610	26*	93.8*	53,300	3,520	0.01	0.003
Settleable solids	24	0.05	99.8	3,390	6.8	0.65	0.04
Total dissolved solids	--	1010	--	--	137,000	--	--
Specific conductance	--	1850 μmhos/cm	--	--	--	--	--
Turbidity	--	20 JTU	--	--	--	--	--
pH	--	7.6 pH units	--	--	--	--	--
Alkalinity (as HCO_3)	312	233	25	10,500	31,500	0.21	0.014
Hardness (as $CaCO_3$)	--	289	--	--	39,100	--	--
BOD_5	395	21*	94.2*	46,100	2,840	0.05	0.003
TOC	--	30	--	--	4,060	--	--
Oil and grease	73.0	3.1*	96	10,100	419	0.22	0.015
Total phosphate (PO_4)	42.6	19.2*	55	3,180	2,600	0.69	0.047
Organic nitrogen	26.8	5.1*	81	2,940	690	0.75	0.051
Ammonia (NH_3)	28.0	18.8*	33	1,250	2,540	1.76	0.12
Kjeldahl nitrogen	54.8	23.9*	56	4,110	3,230	0.52	0.037
Nitrates (NO_3)	1.5	4.9*	--	--	663	--	--
Nitrites (NO_2)	1.3	1.4*	--	--	189	--	--
Total coliform bacteria	--	108 organisms/ 100 mL	--	--	--	--	--
Fecal coliform bacteria	--	8 organisms/ 100 mL	--	--	--	--	--
Sulfates (SO_4)	105	148	--	--	20,000	--	--
Chlorides (Cl)	--	330	--	--	44,600	--	--
Silica (SiO_2)	36	31	14	680	4,190	3.22	0.22
Sodium (Na)	215	218	--	--	29,500	--	--
Potassium (K)	18.4	23.8	--	--	3,220	--	--
Calcium (Ca)	59	65	--	--	8,790	--	--
Magnesium (Mg)	37	35	6	300	4,690	7.34	0.50
Phenols	195	2.9	99	38,600	390	0.06	0.004
Cyanide (CN)	0.06	0.06	--	--	8.1	--	--
Fluoride (F)	2.0	1.3	35	95	176	23	1.6
Boron (B)	--	0.9	--	--	122	--	--
Arsenic (As)	--	0.0004*	--	--	0.05	--	--
Cadmium (Cd)	--	0.002*	--	--	0.27	--	--
Chromium (Cr)	--	0.016*	--	--	2.2	--	--
Copper (Cu)	--	0.081*	--	--	11.0	--	--
Lead (Pb)	--	0.0098*	--	--	1.3	--	--
Mercury (Hg)	--	0.0019*	--	--	0.26	--	--
Nickel (Ni)	--	0.038*	--	--	5.1	--	--
Silver (Ag)	--	0.002*	--	--	0.27	--	--
Zinc (Zn)	--	0.087*	--	--	11.8	--	--

*These values are from routine analyses (several grab samples per month). The remaining values are from only a few data points (1 to 4) collected during the spring of 1977.

of time. It is assumed that the deposition rate is constant and that the increasing difference between the deposition rate and the accumulation rate as time passes is caused by particulate losses to the air. Therefore, if the effects of rain and street cleaning operations are eliminated, it is possible to estimate these dust losses from the accumulation rates. It is assumed that the initial high accumulation rate value approximates the constant deposition rate. The Tropicana test area may lose an average of about 80 lbs per curb-mile per day during days 4 through 9, or about a week after street sweeping or a significant rain. About 20 percent of the total particulates on Tropicana streets are smaller than 106μ in size and could therefore remain suspended. Thus, in the Tropicana test area, about 16 lbs per curb-mile per day of particulates may be suspended owing to winds and automobile traffic. About 80 percent of the total particulates are larger than 106μ and would not remain suspended because of their large size. Thus, the remaining 64 lbs per curb-mile per day would rapidly settle out to the ground near the street.

The average automobile traffic density in the Tropicana area is about 1000 cars per day. Since there are 2 curb-miles per street-mile, the 15 lbs per curb-mile per day value would give an automobile use emission factor of about 0.03 lb per car-mile (10g per car-km) if all of the losses to the air were the result of automobile-caused turbulence. It is important to note, however, that winds by themselves can cause significant losses.

This emission factor would increase if the street cleaning interval or the interval between significant rains were longer. It would also vary for different street, land-use, traffic, and wind conditions.

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16. ABSTRACT

The workshop on urban stormwater management technology was held on December 1, 1977 at the offices of the USEPA in Edison, New Jersey. The purpose of the workshop was to exchange and disseminate the most up-to-date- research results and technical information from projects sponsored under USEPA Urban Runoff Control Research Development and Demonstration Program. The proceedings contained herein represent the contributions from participating lecturers and include the following topics:

a. Urban stormwater management and technology manual (update),
b. Comprehensive planning for control of urban storm runoff and combined sewer overflows,
c. Low cost-effective alternative and comparative analysis from 208 areawide assessment study on combined sewer overflow and urban stormwater pollution control,
d. Statistical characterization of runoff loading rates and cost functions of control measures,
e. Dry weather pollutant deposition in sewerage systems and associated first flush combined sewer overflow pollution control by dry weather sewer flushing,
f. Nonpoint pollution abatement through improved street cleaning practices.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Water quality, Runoff, Urbanization, Stream pollution, Sewers, Water pollution, Mathematical models, Surface water runoff, Cost analysis, Drainage, Hydraulics, Waste treatment	Best management practices, Sewer flushing, Street sweeping, Nonstructural control, Urban hydrology, Computer models, New residential development, Water quality control	13B

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS *(This Report)* UNCLASSIFIED	21. NO. OF PAGES 130
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