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Short Course Proceedings
APPLICATIONS OF STORMWATER MANAGEMENT MODELS - 1976

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SHORT COURSE PROCEEDINGS

APPLICATIONS OF STORMWATER MANAGEMENT MODELS

1976

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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 problems, 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 effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

The Short Course Proceedings contained herein include discussions of various computer assisted models applied to stormwater management, criteria for selecting models either as planning or design tools and methods for collecting field data for model verification. A case study designed for instructional use highlights application of the EPA Stormwater Management Model (SWMM).

Francis T. Mayo
Director
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PREFACE

This Short Course was first held at the University of Massachusetts (August 19-23, 1974) and then revised and offered at the University of Massachusetts (July 28-August 1, 1975), Asilomar Conference Grounds, Pacific Grove, California (January 5-9, 1976) and the University of Chicago (July 18-23, 1976). Registration totaled close to 300 with representation by consultants; Federal, State and Municipal agencies, including those of the Canadian government; and university researchers.

Based upon responses to a questionnaire completed by almost all attendees, the following profile of professional backgrounds and interests emerged:

- professional training areas represented were sanitary engineers (48%): hydraulic engineers (24%); planners (6%) and other disciplines (22%).
- highest degrees earned were BS (35%); MS (52%); PhD (11%) and other (2%).
- interests in stormwater management models were Section 208 implementation (32%); sewer system design (23%); research (9%); environmental impact assessment (8%); and various combinations of the above (28%).
- knowledge of stormwater management models prior to attending the Short Course was indicated by 55% of the registrants.
- as a result of attending this Short Course, 87% of the registrants indicated that use of models will be encouraged in their firms and agencies.
- of the models presented, 68% of the registrants indicated that applying a combination of EPA SWMM with other, more simple models would best serve their needs.

ABSTRACT

This Short Course on applications of stormwater management models is a follow-up to a course sponsored by the U.S. EPA and now available as EPA Report 670/2-75-065. The proceedings contained herein represent an entirely new set of contributions from participating speakers. The objective of this Short Course is to provide practitioners with the capability to apply specific models directly. Toward this goal, a discussion of the common components of stormwater management models first gives an overview of modeling needs. The U.S. EPA Stormwater Management Model (SWMM) is then described in detail and an illustrative case study presented. The methodology for data preparation is outlined and sample input and output data given for the Rainfall-Runoff, Transport, Storage/Treatment and Receiving Water Blocks of the EPA SWMM. A discussion of criteria for selecting models for application as either planning or design tools is then presented along with illustrations of the use of two simplified models. Finally, the techniques for collecting field data for model calibration are presented and the performance of commercially available sampling equipment assessed.

This report was submitted in partial fulfillment of Grant Number 803069 by the Department of Civil Engineering at the University of Massachusetts, under the sponsorship of the Environmental Protection Agency. This report covers the period June 3, 1975 to August 31, 1976 and work completed as of August 31, 1976.

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I would also like to thank the participating speakers. Special recognition is to be given to Mr. Thomas K. Jewell, graduate research assistant and Mr. David Gaboury, undergraduate research assistant, for their tireless effort in applying the EPA SWMM to the UMASS computer system and in developing a case study for the workshop sessions.

I am also grateful to the staff of Conference Services at the University of Massachusetts for handling all of the Short Course details. Finally, I acknowledge the help of our Environmental Engineering secretary, Miss Dorothy A. Blasko, for typing the workshop materials.

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INTRODUCTION

By

Richard Field* and Anthony Tafuri

Welcome to this fourth short course on the application of stormwater management models. As many of you know, the University of Massachusetts and the U.S. Environmental Protection Agency have jointly sponsored three similar courses in the past. Each proved to be very successful and we're looking forward to another success this week.

Before getting into the course, I would like to briefly introduce you to EPA's Stormwater Program and the importance of mathematical modeling to the Program. The Program started in 1965 under the U. S. Public Health Service and is presently assigned to EPA's Municipal Environmental Research Laboratory, Cincinnati; however, it is physically located in Edison, New Jersey. A primary objective of the Program has been to advance the technology for urban stormwater and combined sewer overflow treatment, control and management. Part of this objective involves the development and refinement of mathematical models for the total management of sewer systems considering the full range of wet-and dry-weather flow pollution control.

Traditional analytical methods are inadequate to evaluate the dynamic reaction of the urban drainage system to storm events. The Program recognized that a mathematical computer simulation program to model urban runoff quantity and quality during the storm process would provide an invaluable tool for engineers. Such a simulation system was completed in 1971 and is known as the EPA Stormwater Management Model (SWMM).

SWMM is capable of being used in detailed master planning as it enables the decision-maker to learn the consequences of alternative courses of action. For example, if storage facilities are being considered, the planner can calculate the hydraulic and the pollutant load that the facility will receive. Similarly, the pollutional load imposed on streams by storms can be calculated. SWMM is also useful in predicting and planning required changes in the hydraulic conveyance capacities of the collection system brought about by local overloading. For design purposes, the model is useful in determining

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the anticipated flowrates that a specified sewer needs to convey. The size and slope of the sewer can then be specified. Although the model's operational capability has not yet been applied, there is a likelihood it, or other versions, will be used in San Francisco in the near future, to demonstrate real-time automatic control of combined sewer systems. The model's operational capability lies in its ability to provide information on what to expect at various points in a stream in terms of flow from a rainfall event.

Since its release, SWMM has been refined and expanded. Program "bugs" have been eliminated. We are now able to demonstrate the ability of computer-assisted mathematical models to enhance urban stormwater management by analyzing a major combined sewer system, by selecting and designing control and treatment approaches based on cost effectiveness, and by designing a computerized means of overall management of the system during storm flows. It is the eventual goal to handle all wet-and dry-weather flows in this manner.

A simplified mathematical model approach to the urban stormwater management plan is presently being perfected. This approach determines overflow occurrences on a probabilistic basis according to historical rainfall records. Subsequently, the overflow problems within a drainage system can be pinpointed with the simplified model and then defined in greater detail with SWMM for specific locations and storms. Detailed discussion on SWMM versus the simplified model will be presented later on this week along with information on other stormwater management models that are presently available.

EPA FILM "STORMWATER POLLUTION CONTROL: A NEW TECHNOLOGY"

The film portrays a complete overview of the U.S. EPA's involvements in developing countermeasures for pollution from combined sewer overflows and stormwater discharges.

It starts with a background of sewer construction leading into the pollution problems caused by urban runoff. Early drainage plans made no provisions for storm flow pollutional impacts. Various countermeasures are shown that are ready for implementation by municipalities. The majority are full-scale operations and include (a) control techniques, e.g. infiltration prevention, street cleaning, porous pavement (for runoff attenuation), in-sewer routing and storage, improved overflow regulators, and off-line storage; (b) physical-chemical disinfection and biological treatment as shown adjuncts to the sanitary plant or as remote "satellite" facilities at the outfall; (c) and various schemes which reclaim stormwater for beneficial purposes including enhancement of visual aesthetics, recreation and water supply.

A couple of years ago, we felt that the time was right to make an evaluation of previous program efforts, and we implemented a contract to develop the state-of-the-art and assess techniques available to manage and treat combined sewer overflow and stormwater. A text on this subject has been completed and is available. Expanding on the cliche "a picture is worth a thousand words", we felt a film would be worth one hundred thousand--so a film was included as part of the work.

The specific reasons for this film were to alert the environmental engineering community to the stormwater problem and the immediate need to apply available technology to counteract it.

ROLE OF MODELS IN URBAN STORMWATER MANAGEMENT PLANNING

By
James P. Heaney*

INTRODUCTION

Section 208 of the Federal Water Pollution Control Act Amendments of 1972 requires that plans be developed to abate and control point and non-point waste sources in urban areas throughout the country. This section has brought about an unprecedented amount of activity in devising integrated urban environmental management plans. In addition to multi-purpose water related programs, attention is being given to air pollution and solid waste as they relate to the water pollution control program.

Model developments during the past 10-15 years have produced a wide variety of tools which are available to assist the engineers and planners in conducting these studies. Many of these models simulate the relevant physical, chemical, and biologic processes. Various optimization techniques are available to assist in making decisions regarding the "best" solution. Literally hundreds of papers have been written describing these simulation and optimization models. The balance of this paper discusses how I perceive that some of these models could fit into the 208 planning process as it is currently envisaged. Because 208 planning is still in the formative stages of development, it is reasonable to expect that others might view the "problem" quite differently. Brandstetter (1975) and Brown et al. (1974) have developed assessments of urban water management models.

At a recent workshop in stormwater management (Wanielista, 1975), Dowling presented a preliminary work plan for 208 planning in the Orlando, Florida, area. This outline will be used as a format for discussing the relevance of various types of models to the various phases of the study.

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1.0 PUBLIC PARTICIPATION AND LOCAL GOVERNMENT INVOLVEMENT

1.1 PUBLIC CONTACT SECTIONS
1.2 RESPONSIVE COMMITTEES
1.3 PUBLIC PARTICIPATION
1.4 REGIONAL CLEARINGHOUSE REVIEWS

No significant modeling efforts are envisaged for this phase of the study.

2.0 ORGANIZATION

2.1 ADMINISTRATION
2.2 RELATIONSHIPS TO OTHER PROGRAMS

Existing models would not have much relevance to this phase of the work plan.

3.0 DATA COLLECTION

3.1 EXISTING DATA SOURCES
3.2 PROJECTED DATA COLLECTION EFFORTS
3.3 AGENCIES WITH POTENTIAL DATA COLLECTION FUNCTIONS
3.4 ASSEMBLY AND REVIEW OF EXISTING DATA
3.5 DETERMINE ADDITIONAL DATA NEEDS AND APPROPRIATE AGENCIES
3.6 ADDITIONAL DATA COLLECTION

If one knows the type of models needed to conduct the study, then a relatively good idea of data needs is available. Of course, the converse is true, i.e., the availability of data and resources to collect additional data strongly influences the selection of models. Decision theoretic models can be used to assess the value of data if the decision problem is well specified. Unfortunately, 208 planning activities tend to have fuzzy decision criteria since it is not clear, a priori, what the "problem" is. In particular, we have yet to concur on performance criteria for wet weather pollution. Thus, one still needs to rely heavily on "engineering judgment" in the data gathering phase of the study.

4.0 NATURAL AND PHYSICAL SYSTEMS ANALYSIS

4.1 DESCRIPTION OF THE PLANNING AREA
4.2 IDENTIFICATION AND USE CLASSIFICATION OF RECEIVING WATERS

4.3 IDENTIFICATION OF HYDROLOGICAL, GEOLOGICAL AND CLIMATOLOGICAL FACTORS
4.4 IDENTIFICATION OF TOPOGRAPHICAL FACTORS
4.5 IDENTIFICATION OF AQUATIC, TERRESTRIAL AND CULTURAL FACTORS
4.6 IDENTIFICATION AND EVALUATION OF ENVIRONMENTALLY SENSITIVE AREAS

This phase of the study deals with an inventory and characterization of the above factors. I would not envisage any significant modeling effort during this phase of the study.

5.0 SOCIAL, ECONOMIC AND LAND USE ANALYSIS

5.1 EXISTING POPULATION CHARACTERISTICS
5.2 EXISTING ECONOMIC BASE
5.3 EXISTING LAND USE
5.4 FUTURE POPULATION, ECONOMIC BASE AND LAND USE
5 5 IMPACT

There are several land use based urban planning models which could be useful in this phase of the study. Kilbridge et al. (1970) have reviewed many of these models. A key question to be addressed in such modeling efforts is to what extent might environmental considerations affect future land use. Similar types of impact analyses have been done in urban transportation studies for a number of years. Relatively well documented versions of these models are available from the US Department of Transportation. However, it would be reasonable to expect that the city already has a model available from their transportation studies.

6.0 WASTEWATER SOURCE ANALYSIS

6.1 DOMESTIC WASTE DISCHARGES
6.2 INDUSTRIAL WASTE DISCHARGES
6.3 JOINT TREATMENT SYSTEMS
6.4 STORM SEWER DISCHARGE LOCATIONS
6.5 TOTAL POINT SOURCE WASTE DISCHARGES
6.6 IDENTIFICATION OF NON-POINT WASTE SOURCES
6.7 TOTAL NON-POINT WASTE LOADS
6.8 TOTAL WASTE LOADS
6.9 FUTURE WASTE LOADS-SOURCES AND CHARACTERISTICS

Discharges of domestic wastewater and industrial wastes are relatively easy to estimate since they have been monitored for a number of years and exhibit steady state behavior in most cases. FILTH in the SWMM model can be used to estimate domestic wastewater loads. The wet weather aspects of the wastewater source analysis are the main problem.

One could expect to have a relatively large number of storm sewer discharge locations. A problem arises in defining receiving waters for cases where the original creek or river system has been improved to accommodate stormwater runoff. Is it a receiving water or is it part of the drainage network?

Wet weather sources can be estimated using a model such as STORM (HEC, 1975) which provides an hourly accounting of runoff and pollutant discharges. Local data are needed to calibrate the pollutant loading factors. The use of literature values could lead to very misleading results. The Universal Soil Loss Equation (USLE) is gaining growing acceptance as a tool for doing nonpoint source analysis. This equation was incorporated in SWMM (Heaney, Huber et al., 1975) a few years ago and has recently been added to STORM. Using the USLE, other pollutants are expressed as a function of the soil loss.

7.0 WASTEWATER TREATMENT SYSTEMS

7.1 DESCRIPTION OF EXISTING FACILITIES
7.2 IDENTIFICATION OF PROPOSED FACILITIES
7.3 EXISTING AND PROPOSED SERVICE AREAS
7.4 IDENTIFICATION OF EXISTING PLANS OF FEDERAL, STATE AND LOCAL GOVERNMENTAL AGENCIES AND PRIVATE ORGANIZATIONS
7.5 SLUDGE GENERATION FORECASTS
7.6 INFILTRATION/INFLOW AND OVERFLOW ANALYSIS

This phase of the study does not appear to rely heavily on models.

8.0 NON-POINT SOURCES

8.1 ASSEMBLY OF AVAILABLE NON-POINT SOURCE DATA
8.2 EVALUATION OF NON-POINT POLLUTION SOURCES

See Section 6.0 for a discussion of non-point source analysis.

9.0 WATER QUALITY ANALYSIS

9.1 PRESENT WATER QUALITY STANDARDS
9.2 DETERMINATION OF EXISTING WATER QUALITY
9.3 VERIFICATION OF PRESENT STREAM STANDARDS
9.4 EVALUATION OF ACHIEVABLE WATER QUALITY
9.5 WATER QUALITY PROBLEM AREAS
9.6 RELATIONSHIP OF EXISTING WASTE SOURCES TO PRESENT WATER QUALITY
9.7 PROJECTED WATER QUALITY

A wide variety of receiving water models are available. The receiving water model in SWMM is very popular. The vexing question in this phase of the study is how to relate receiving water quality to wet weather flows. One needs to extend the traditional event analysis to a continuous simulation of an extended period of time (year or more).

10.0 GROUNDWATER PROTECTION AND ENHANCEMENT

10.1 GROUNDWATER QUALITY
10.2 SOURCES OF POTABLE WATER
10.3 SOURCES OF RECHARGE
10.4 SOURCES OF DEGRADATION

As with receiving water models, there are several groundwater models available. Perez et al. (1974) have developed a conjunctive surface groundwater model which examines both quantity and quality. Unfortunately, little data exist to support a very sophisticated groundwater model especially with regard to water quality.

11.0 DEVELOPMENT OF NON-POINT SOURCE CONTROL STRATEGIES

11.1 DEVELOP ALTERNATIVES
11.2 RANKING AND PRIORITY OF NON-POINT SOURCE CONTROL STRATEGIES

A recent report by Metcalf and Eddy, Inc. (1974) provides a nice summary of the available wet weather controls and their effectiveness. Murphy (1975) presents a methodology for doing this analysis using the appropriate cost data and storage/treatment isoquants generated by STORM. One critical assumption in STORM and SWMM is the assumed rate at which pollutants are washed off the catchment. At present, it is assumed that a uniform runoff of one-half inch per hour would wash away 90 percent of the pollutants in one hour. By varying this assumption, one changes the significance of the "first flush" effect. If the first flush is significant one would concentrate on controlling the initial part of each storm event. This would imply a reservoir operating policy of storing the first part of the storm and bypassing the rest of it. Here again, local data are needed.

12.0 LAND USE/WATER QUALITY RELATIONSHIP

12.1 EVALUATION OF EXISTING LAND USE PLANS
12.2 ASSESSMENT OF LAND USE CONTROLS
12.3 LAND USE/WATER QUALITY RELATIONSHIP
12.4 DEVELOPMENT OF RECOMMENDED LAND USE GUIDELINES
12.5 RECOMMENDED LAND USE PLAN MODIFICATIONS

SWMM and STORM can be used for this phase of the study since the models generate runoff and pollutants as a function of land use.

13.0 PUBLIC WORKS/WATER QUALITY RELATIONSHIP

13.1 DRAINAGE
13.2 STORMWATER COLLECTION AND DISPOSAL
13.3 SOLID WASTE DISPOSAL
13.4 LAKE RESTORATION

Our experience to date indicates that it is vital to integrate wet weather quality control with quantity control. Specifically, joint utilization of storage facilities is a particularly promising alternative. SWMM provides output regarding the hydrograph attenuation and treatment occurring in storage facilities. Also, on-site storage can reduce stormwater collection costs considerably. It does not appear that an adequate data base exists to relate solid waste incinerator discharges to wet weather quality. The groundwater models discussed in Section 10 can be used to examine the impact of land disposal of solid wastes.

14.0 INSTITUTIONAL ANALYSIS

14.1 PRESENT FEDERAL, STATE, REGIONAL, LOCAL AND PRIVATE PROGRAMS AND ORGANIZATIONS
14.2 EVALUATION OF INSTITUTIONAL ARRANGEMENTS
14.3 RECOMMENDATIONS OF INSTITUTIONAL ALTERNATIVES

The overall problem needs to be viewed as a multi-purpose, multi-group decision situation wherein one is attempting to find an equitable and efficient solution. A disproportionate amount of the effort to date has addressed only the efficiency question. Burke and Heaney (1975) present a broad overview of the models which are available at this time. For example, it is possible to assess the relative voting power of the various interest groups to evaluate the fairness of various voting rules. Also, simulation models exist to aid in assessing the bargaining arena, interest groups, salient issues, etc. (Bulkley and McLaughlin, 1966). This important phase of the study is often overlooked entirely.

15.0 WASTE TREATMENT MANAGEMENT TECHNIQUES

15.1 POINT SOURCE SUBPLANS
15.2 NON-POINT SOURCE SUBPLANS
15.3 INSTITUTIONAL (FINANCE AND MANAGEMENT) SUBPLANS

Given a multi-purpose,multi-interest group situation one needs to know, for each plan, the incidence of benefits and costs to each purpose and each group (James and Lee, 1971). Coordinated plans reap savings by taking advantage of economies of scale, interdependencies in production functions, and more efficient use of assimilative capacity. Deininger and Loucks (1972) present a nice summary of optimization procedures for point source analysis. Heaney, Huber et al. (1975) describe a procedure whereby equitable financial arrangements can be determined.

16.0 EVALUATION AND COMPARISON OF ALTERNATIVE PLANS

16.1 ACHIEVEMENT OF TREATMENT STANDARDS (BAT, BPWTT, ETC.) AND WATER QUALITY OBJECTIVES
16.2 MONETARY COSTS AND ADMINISTRATIVE FEASIBILITY
16.3 TECHNICAL RELIABILITY
16.4 LAND USE IMPACT
16.5 ENVIRONMENTAL FACTORS
16.6 IMPLEMENTATION FEASIBILITY AND RELIABILITY
16.7 PUBLIC ACCEPTABILITY

Models from Section 5 (land use), 14 (institutional analysis) and 15 (waste treatment management techniques) can be integrated to assist in the overall evaluation. Standards and environmental factors can be included as additional constraints.

17.0 SELECTION OF WASTE TREATMENT MANAGEMENT PROGRAM

17.1 POINT SOURCE TREATMENT WORKS
17.2 NON-POINT SOURCE TREATMENT WORKS
17.3 NON-STRUCTURAL PROCESSES
17.4 MANAGEMENT
17.5 IMPLEMENTATION RATIONALE AND PRIORITY CRITERIA
17.6 PRESENTATION OF COMPREHENSIVE PROGRAM

This selection would be based heavily on Section 16 investigations. No significant additional modeling appears necessary.

18.0 ENVIRONMENTAL, SOCIAL AND ECONOMIC IMPACT ASSESSMENT

18.1 ECOLOGICAL INVENTORY
18.2 EVALUATION OF PROPOSALS, STRATEGIES, ALTERNATIVES AND PROGRAM
18.3 PREDICTED EFFECTIVENESS - IMPACT
18.4 CONSISTENCY WITH, IMPACT ON OR ADJUSTMENT REQUIRED FOR RELATED PLANS

A wide variety of procedures are now being used for environmental impact assessment, e.g., Dee et al., 1973. Techniques of multiple objective analysis can be used to evaluate incommensurables such as dollars and environmental quality. Cohon (1973) presents an excellent survey of available techniques. We have applied this approach to the Upper St. Johns River Basin in Florida using natural energy as a surrogate for environmental quality (Butkovich and Heaney, 1975). The results are presented in Figure 1. In multiple objective analysis one replaces notion of optimality with determining a set of undominated or non-inferior solutions. This set is displayed to the decision makers so that they might select a reasonable compromise solution.

19.0 PLAN PREPARATION AND COMPLETION

20.0 GRAPHICS AND PRINTING

21.0 COORDINATION ACTIVITIES - COMPLETED AND CERTIFIED 208 PLAN

Little modeling seems relevant to these last three phases of the study.

CONCLUSIONS

The purpose of this discussion has been to present some very preliminary ideas regarding how models can be used in the 208 planning process. Following the format of a preliminary work plan for Orlando, Florida, various types of models were described with a heavy bias towards models we have worked with or are familiar to us. No attempt was made to survey all of the available simulation and optimization models; rather, the purpose was to provide some preliminary suggestions on how models might fit into the 208 planning process. An important consideration in successful modeling is to

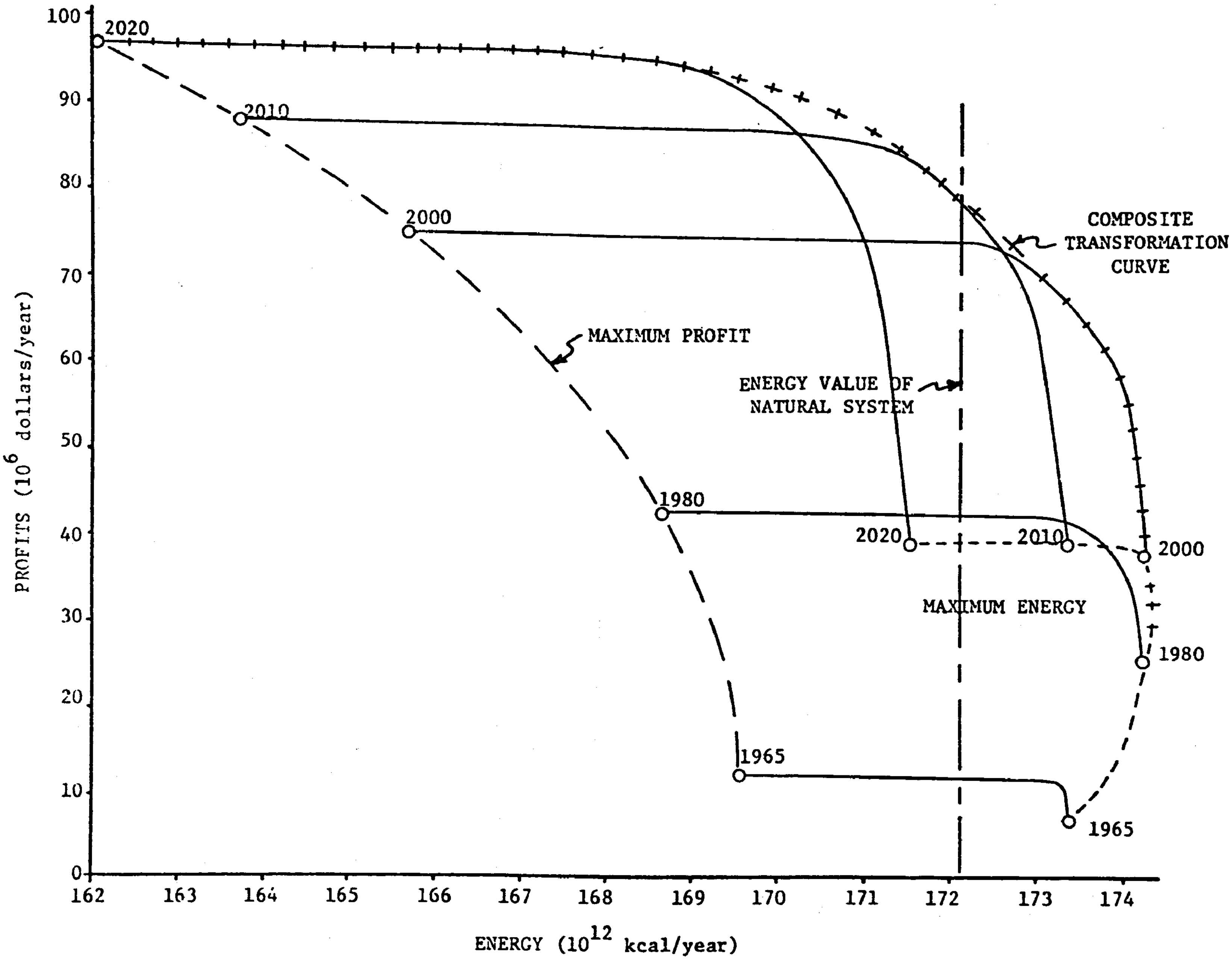

FIGURE 1. TRANSFORMATION CURVES FOR VARIOUS LEVELS OF DEVELOPMENT, UPPER ST. JOHNS RIVER BASIN.

interact frequently with the decision makers. In view of the existing uncertainty in wet weather pollution, the problem definition phase of the analysis is of vital importance.

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METHODOLOGY FOR EVALUATING THE COST OF URBAN STORMWATER QUALITY MANAGEMENT

By

James P. Heaney and Sheikh M. Hasan*

INTRODUCTION

Traditionally, cities have utilized a single sewer network (called combine sewers) to carry away domestic wastes and urban stormwater. Treatment facilities were installed with capacity sufficient to treat two to five times the average rate of dry weather sewage flow. If the rainfall was larger than that treatment capacity, the excess flow containing a mixture of untreated sewage and storm water was bypassed directly to the receiving water.

Following World War II, most cities installed separate sewer systems in newly developing areas. Also, programs were initiated to separate existing combined sewer systems. However, preliminary cost estimates indicated that this separation program would be very expensive. In addition, stormwater quality sampling programs indicated that this water contains significant quantities of pollutants which it picks up as it moves through the urban area. Thus, the current thinking indicates that remedial measures to control combined sewer overflows are more cost-effective than sewer separation.

During the same period, cities installed improved sewage treatment facilities capable of removing about 85 to 90 percent of the biochemical oxygen demand (BOD). During the late sixties and early seventies, numerous cities were investigating tertiary treatment of dry weather sewage. However, tertiary treatment is quite expensive. Some people argued that it may be more cost-effective to treat combined sewer overflows and/or urban stormwater. This question remains unresolved.

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Dry weather sewage flows are continuous and relatively small in volume and the technology for their control is fairly well established. On the other hand, the wet weather problem involves control of massive and dynamically variable "wet weather" periods of short duration. The quantity and quality of these flows depend upon many factors such as the degree of urbanization, amount of rainfall, size of the area, type of land use, soil types, etc. Thus, control of wet weather flows involves both quantity and quality. Therefore, the three facets of urban water management to be examined are control of:

1. dry weather sewage flow,
2. stormwater quality, and
3. stormwater quantity.

Since the mid-sixties, the federal government has played an increasing role in the management of urban water quality. The states have been required to establish water quality standards and federal subsidies have been provided for the construction of publically owned waste treatment works. The 1972 Amendments to the Federal Water Pollution Control Act establish the year 1983 as the goal for zero discharge. Towards achieving this goal, comprehensive guidelines have been issued which require planning on an area wide basis, consideration of dry weather as well as wet weather pollution sources and the installation of the best practicable wastewater treatment technology for publically owned treatment works.

Approximately two years ago, the American Public Works Association and our group at the University of Florida initiated an EPA sponsored study to estimate the nationwide cost of treating combined sewer overflows and stormwater runoff. Results from this effort will be available later this year. This lecture describes the procedures used to conduct this study. Emphasis is placed on the interrelationships among stormwater quality control and the more established programs in wastewater control and storm drainage control.

The next section describes procedures for estimating the costs of stormwater quality control. Then, the net costs for stormwater quality management are determined by recognizing the complementarities among the various purposes. N-person game theoretic concepts are used for cost allocation purposes. Minneapolis, Minn. is used as the test city.

CONTROL TECHNOLOGY AND ASSOCIATED COSTS

A wide variety of control alternatives are available for improving the quality of wet-weather flows (Field and Struzeski, 1972; Lager and Smith, 1974; Becker et al., 1973). Rooftop and parking lot storage, surface and underground tanks and storage in treatment units are the flow attenuation control alternatives. Wet-weather quality control alternatives can be subdivided into two categories: primary devices and secondary devices. Primary devices take advantage of physical processes such as screening, settling and flotation. Secondary devices take advantage of biological processes and physical-chemical processes. These control devices are suitable for treating stormwater runoff as well as combined sewer overflows. However, the contact stabilization process is feasible only if the domestic wastewater facility is of an activated sludge type. The quantities of wet-weather flows that can be treated by this process are limited by the amount of excess activated sludge available from the dry-weather plant. At the present time, there are several installations throughout the country designed to evaluate the effectiveness of various primary and secondary devices. A summary of the design criteria and performance of these devices is presented in Table 1. Based on these data, the representative performance of primary devices is assumed to be 40 percent BOD_5 removal efficiency and that of secondary devices to be 85 percent BOD_5 removal efficiency. No treatment is assumed to occur in storage. Hasan (1976) has synthesized available information regarding stormwater pollution control costs. The results are shown in Table 2.

COSTS OF URBAN STORMWATER QUALITY MANAGEMENT

The evaluation procedure for the nationwide assessment consisted of relatively detailed studies of five cities: Atlanta, Denver, Minneapolis, San Francisco, and Washington, D.C. For each city, a single storm event for a selected catchment was simulated using the EPA Storm Water Management Model - SWMM (EPA, 1971a, b, c, d; 1975). Also, one year of hourly precipitation, runoff, and discharage rates were estimated using the HEC STORM model (HEC, 1974). STORM provides estimates of the total volume of storm water which is treated for a specified size of storage unit and treatment rate. Numerous combinations were tested for each of the five cities to derive storage-treatment isoquants as shown in Figure 1 for Minneapolis. Given the storage-treatment isoquants and knowning the relative costs of storage and treatment, one can determine an optimal expansion path in terms of control costs versus percent BOD removal. The optimal expansion

TABLE 1. WET-WEATHER TREATMENT PLANT PERFORMANCE DATA

Device	Control Alternatives	Design Criteria	Reported BOD_5 Removal Efficiency, n
Primary	Swirl Concentrator[a,b]	60.0 gpm/sq ft	0.25 - 0.50
	Microstrainer[c]	20.0 gpm/sq ft	0.40 - 0.60
	Dissolved Air Flotation w/ Chemical Addition[d]	2.5 gpm/sq ft	0.50 - 0.60
	Sedimentation[e]	0.5 gpm/sq ft	0.25 - 0.40
	Representative Performance		0.40
Secondary	Contact Stabilization[f]	Cont 0.25 hrs Stab 3.0 hrs	0.75 - 0.88
	Physical-Chemical[g]	3.0 hrs	0.85 - 0.95
	Representative Performance		0.85

[a]Field and Moffa, 1975

[b]APWA, 1974

[c]Maher, 1974

[d]Lager and Smith, 1974

[e]Performance data based on domestic wastewater treatment

[f]Agnew, R. et al., 1975

[g]Estimate based on performance of these units for domestic wastewater

TABLE 2. COST FUNCTIONS FOR WET-WEATHER CONTROL DEVICES[a,b,i]

		Annual Cost: $/yr					
		Amortized Capital		Operation and Maintenance		Total	
		$CA = lT^m$ or lS^m		$OM = pT^q$		$TC = wT^z$ or wS^z	
Device	**Control Alternative**	l	m	p	q	w	z
Primary	Swirl Concentrator[c,d,e]	1,971.0	0.70	584.0	0.70	2,555.0	0.70
	Microstrainer[e,f]	7,343.8	0.76	1,836.0	0.76	9,179.8	0.76
	Dissolved Air Flotation[e]	8,161.4	0.84	2,036.7	0.84	10,198.1	0.84
	Sedimentation[e]	32,634.7	0.70	8,157.8	0.70	40,792.5	0.70
	Representative Primary Device Total Annual Cost = $3,000 per mgd						
Secondary	Contact Stabilization[g]	19.585.7	0.85	4,894.7	0.85	24,480.4	0.85
	Physical-Chemical[e]	32,634.7	0.85	8,157.8	0.85	40,792.5	0.85
	Representative Secondary Device Total Annual Cost = $15,000 per mgd						
Storage	High Density (15 per/ac)	51,000.0	1.00	---	---	51,000.0	1.00
	Low Density (5 per/ac)	10,200.0	1.00	---	---	10,200.0	1.00
	Parking Lot[h]	10,200.0	1.00	---	---	10,200.0	1.00
	Rooftop[h]	5,100.0	1.00	---	---	5,000.0	1.00
	Representative Annual Storage Cost[j] ($ per ac-in.) = $122 $e^{0.16(PD)}$						

T = Wet-Weather Treatment Rate in mgd; S = Storage Volume in mg

[a] ENR = 2200. Includes land costs, chlorination, sludge handling, engineering and contingencies.

[b] Sludge handling costs based on data from Battelle Northwest, 1974.

[c] Field and Moffa, 1975.

[d] Benjes, et al., 1975.

[e] Lager and Smith, 1974.

[f] Maher, 1974.

[g] Agnew et al., 1975.

[h] Wiswall and Robbins, 1975.

[i] For T ≤ 100 mgd. No economies of scale beyond 100 mgd.

[j] PD = gross population density, persons/acre.

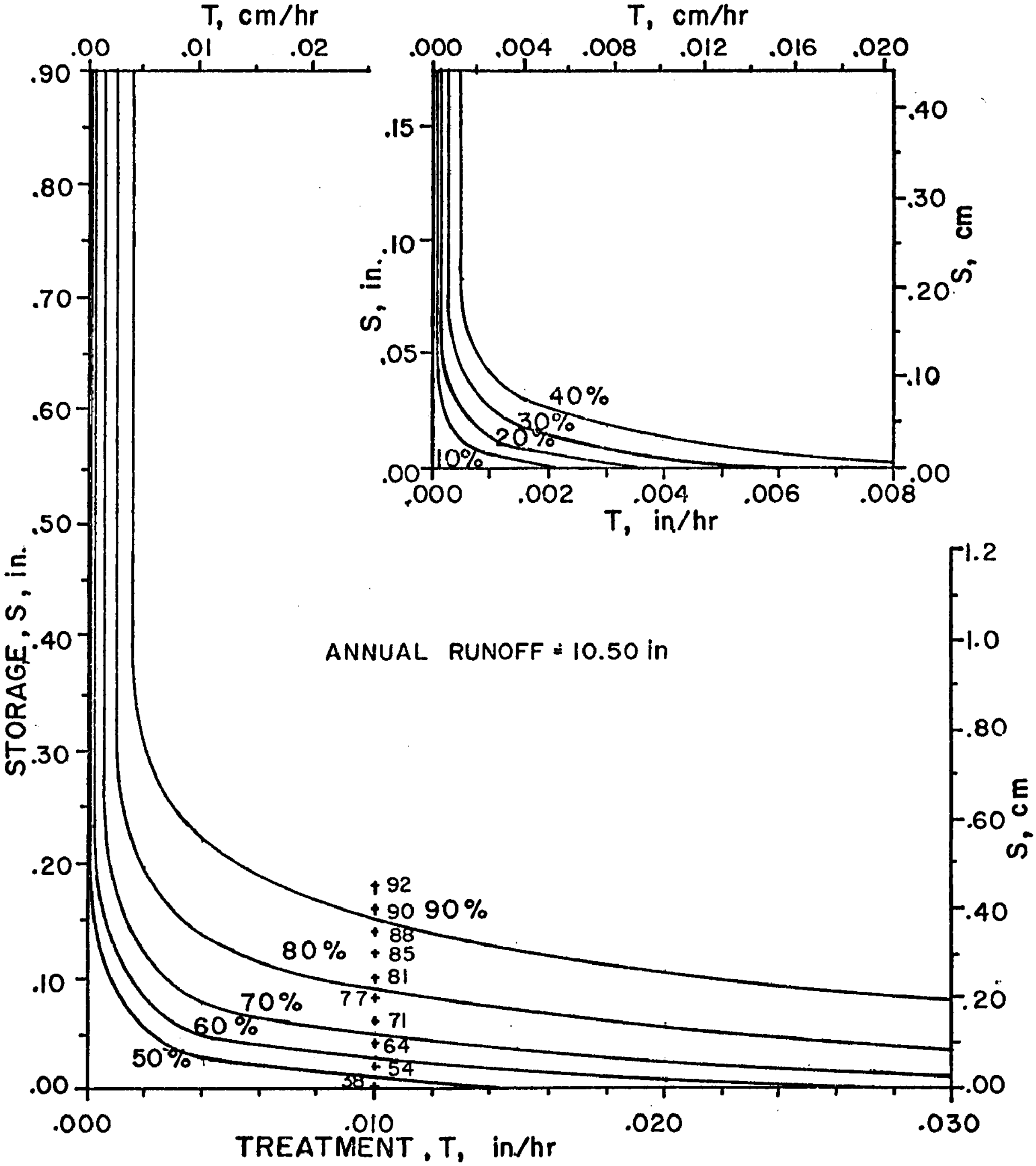

Figure 1. Storage-Treatment Isoquants for Various BOD Control Levels - Region III - Minneapolis

path is determined by comparing the costs of the various alternatives or

$$\frac{c_T}{c_S} = MRS_{ST} \tag{1}$$

where c_T = unit cost of treatment (\$/mgd),

c_S = unit cost of storage (\$/mg), and

MRS_{ST} = marginal rate of substitution of storage for treatment.

The above problem can be expressed in the more compact mathematical form shown below:

minimize

$$Z = c_S(S) + c_T(T)$$

subject to

$$f(R_1;S,T) = 0 \tag{2}$$

$$R_1,S,T \geq 0$$

where Z = total control costs,

$c_S(S)$ = storage costs,

$c_T(T)$ = treatment costs,

S = storage volume,

T = treatment rate,

R_1 = percent pollutant control, and

$f(R_1;S,T)$ = production function relating the level of pollution control attainable with specified availabilities of storage (S) and treatment (T).

Mathematical Representation of Isoquants

The storage/treatment isoquants are of the form:

$$T = T_1 + (T_2 - T_1)e^{-KS} \qquad (3)$$

where T = wet-weather treatment rate, inches per hour,

T_1 = treatment rate at which isoquant becomes asymptotic to the ordinate, inches per hour,

T_2 = treatment rate at which isoquant intersects the abscissa, inches per hour,

S = storage volume, inches, and

K = constant, inch^{-1}.

The value of T_1 occurs at a relatively high storage capacity in combination with a low treatment rate such that the treatment plant operates continuously. T_1 can be found as follows:

$$T_1 = \frac{AR}{8760}\left(\frac{R}{100}\right) = aR \qquad (4)$$

where AR = annual runoff, inches per year, and

R = percent runoff control.

By relating the parameters T_1, T_2-T_1 and K to the level of control R, one equation was developed for Minneapolis. The T_2-T_1 and K terms versus R were found to be:

$$T_2 - T_1 = .00193e^{.100894R} \qquad (5)$$

$$K = 492.0e^{-.0319576R} \qquad (6)$$

Based on this analysis, the following general equation for the isoquants is obtained:

$$T = .0000345R + .00193e^{.100894R-(492.0e^{-.0319576R})S} \qquad (7)$$

<u>Adjusting Isoquants for Treatment Efficiencies</u>

The above isoquants equation (7) characterizes the percentage of total runoff that passes through "treatment." If the concentration of pollutants is constant and "treatment" efficiency, η, is 1.0 then percent runoff control is synonymous with percent pollutant control. Obviously, this is not the case. Thus, these results need to be refined to account for treatment efficiency.

Recall that R is the percent runoff control. Let η equal treatment plant efficiency. If R_1 denotes the percent pollutant control, then to realize R_1, one needs to process R_1/η of the runoff. Note that R_1 may be percent BOD removal, percent SS removal, etc. In Table 1, representative treatment efficiencies, in terms of BOD_5 removal, were derived for primary and secondary devices. Thus, if one desires 25 percent BOD_5 removal with a primary device, then 62.5 percent of the runoff volume must be processed whereas only 29.4 percent of the runoff needs to be processed if a secondary device is selected. Thus, to convert percent runoff control isoquants to percent pollutant control isoquants, one simply uses

$$R = \frac{R_1}{\eta} \qquad (8)$$

in the isoquant equation (7).

<u>Wet-Weather Quality Control Optimization</u>

The wet weather optimization problem, assuming linear costs, may be stated as follows:

$$\text{minimize } Z = c_S S + c_T T \qquad (9)$$

subject to

$$T = T_1 + (T_2 - T_1)e^{-KS}$$

$$T, S \geq 0$$

This constrained optimization problem can be solved by the method of Lagrange multipliers to yield

$$S^* = \max \begin{cases} \frac{1}{K} \ln \frac{c_T}{c_s} [K(T_2 - T_1)] \\ 0 \end{cases} \tag{10}$$

and

$$T^* = T_1 + (T_2 - T_1)e^{-KS^*} \tag{11}$$

Note that T* is expressed as a function of S*, so it is necessary to find S* first. Knowing S* and T*, the optimal solution is

$$Z^* = c_S S^* + c_T T^*. \tag{12}$$

Data needed to estimate T_1, T_2 and K have already been presented.

For a primary device, c_T = \$3,000/mgd = \$1,960/$\frac{\text{acre-in}}{\text{hr}}$

and $\eta = 0.40$.

For a secondary device, c_T = \$15,000/mgd = \$9,800/$\frac{\text{acre-in}}{\text{hr}}$

For storage cost,

$$c_S(\$/\text{acre-in}) = 122\ e^{0.16(PD)} \tag{13}$$

where PD = population density in persons per acre.

The above optimization procedure was programmed to generate curves (Figure 2) showing percent pollutant removed versus total annual costs for primary and secondary treatment in conjunction with storage. Note that, for wet-weather control, marginal costs are increasing because of the disproportionately large sized control units needed to capture the less frequent larger runoff volumes. The curves shown in Figure 2 can be approximated by functions of the form:

$$Z = ke^{R_1 n} \tag{14}$$

where Z = total annual cost, dollars per year

k, n = parameters

R_1 = percent pollutant removal, $0 \leq R_1 \leq \bar{R}_1$ and

$\bar{R}_1$ = maximum percent pollutant removal.

The results for primary and secondary levels are given below.

Unit	Range	k	n
Primary, C_p	$0 \leq R_1 \leq 40$	2.39	0.118
Secondary, C_s	$0 \leq R_1 \leq 85$	4.98	0.049

The level of control at which one is indifferent between primary and secondary levels of control in conjunction with storage i.e. $C_p = C_s$ is 10.6 percent.

POTENTIAL SAVINGS DUE TO MULTIPURPOSE PLANNING

The cost of wet weather quality control can be reduced by integrating this purpose with dry weather treatment plants and/or storage reservoirs for stormwater quantity control. Dry weather sewage treatment

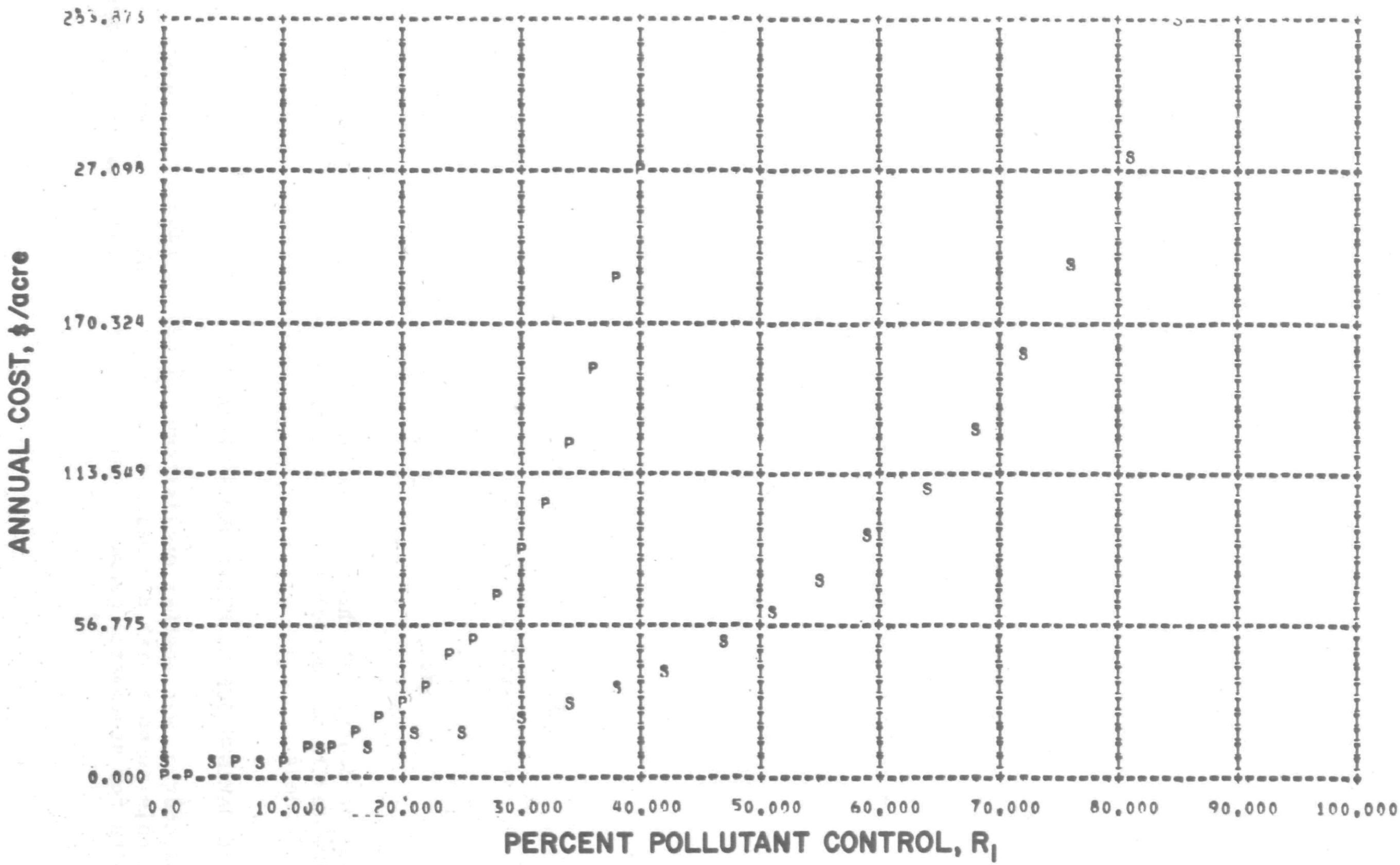

FIGURE 2. CONTROL COSTS FOR PRIMARY AND SECONDARY UNITS AS A FUNCTION OF PERCENT BOD REMOVAL.

plants are designed to handle the peak flow anticipated 10 to 15 years hence. The full capacity of these plants is seldom utilized because peak flows occur infrequently and also because additional capacity is frequently added before the actual flow approaches the design capacity of the plant. Utilization of this excess capacity can reduce the treatment capacity needed for wet weather quality control. Similarly, utilization of storage available for wet weather quantity control can result in reducing the storage and treatment requirements for wet weather quality control. A rough estimate of what the potential savings by integrating might be can be made as follows:

Let Z = cost of dry weather control using a secondary treatment plant (Hasan, 1976), dollars per year

= amortized capital costs + annual O & M costs,

$$= 118{,}300(D+E)^{.7715} + 54{,}800D^{.8345} \qquad (15)$$

D = actual sewage flow, mgd, and

E = excess capacity, mgd.

Assuming D = 10 mgd, E = 10 mgd, and a per capita sewage flow of 100 gallons,

$$Z_1 = 15.67\ PD \qquad (16)$$

where PD = population density, persons per acre.

Also,

Z_2 = annual cost of wet weather quality control (\$/acre)

$$= c_S S^* + c_T[T_1+(T_2-T_1)e^{-KS^*}] \qquad (17)$$

where all terms are as defined earlier;

and

Z_3 = annual cost of wet weather quantity control ($/acre)

$$= c_S V \tag{18}$$

where V = storage volume required for wet weather quantity control (inches)

$$= CR'(i) \tag{19}$$

CR' = runoff coefficent in developed state - runoff coefficient in undeveloped state; and

i = 24 hour rainfall for design frequency (inches).

If dry weather quality control and wet weather quality control are integrated, then

Z_{12} = joint cost of two purposes, dollars per year

$$= 15.67\ PD + c_E E + c_S S^* + c_T[(T_1 - E) + (T_2 - T_1)e^{-KS^*}] \tag{20}$$

where E = excess capacity available at dry weather plant, assumed to be 10 mgd or (1.474×10^{-4}) PD inches per hour

c_E = assumed annual cost of treating E at dry weather plant

= $1,960/inch/hr

If wet weather quality control is integrated with wet weather quantity control, then

Z_{23} = joint cost of two purposes, \$/yr

$$= c_S S_2^* + c_T(T_1 + (T_2 - T_1)e^{-KS_2^*}) \quad (21)$$

where $$S_2^* = \max \begin{cases} V \\ \frac{1}{K} \ln [\frac{c_T}{c_S} K (T_2 - T_1)] \\ 0 \end{cases} \quad (22)$$

It is assumed that dry weather control cannot be integrated with wet weather quantity control. Therefore,

Z_{13} = joint cost of two purposes, \$/yr

$$= Z_1 + Z_3 \quad (23)$$

If all three purposes are integrated, then

Z_{123} = joint cost of three purposes, \$/yr

$$= 15.67\ PD + c_E E + c_s S_2^* + c_T[(T_1 - E) + (T_2 - T_1)e^{-KS_2^*}] \quad (24)$$

To determine the potential savings due to integrating the three purposes, it is first necessary to apportion the joint costs among various purposes. For cost allocation, the Use of Facility method cannot be applied in this case because there is no single facility-utilization parameter that is common to all three purposes. Therefore, some other method must be devised. One may use the game-theoretic procedure (Heaney et al., 1975). The method presented below (called Separable Cost Remaining Benefits Method) yields equivalent results.

Let ϕ_2 = cost apportioned to storm water quality control

Then $$\phi_2 = (Z_{123} - Z_{13}) + \frac{(Z_2 - Z_{123} + Z_{13})(Z_{12} + Z_{13} + Z_{23} - 2Z_{123})}{Z_1 + Z_2 + Z_3 - (Z_{123} - Z_{12}) - (Z_{123} - Z_{13}) - (Z_{123} - Z_{23})} \quad (25)$$

then the potential savings are $Z_2 - \phi_2$

The above procedure was utilized to develop curves for various frequency storms as shown in Figure 3 for Minneapolis.

Cost allocation is seen to have a very significant impact on the costs attributable to wet weather quality control. In the lower levels of control, joint utilization with the dry weather facility yields significant savings. Sharing storage permits significant savings up to much higher levels of pollutant control.

CONCLUSIONS

This study describes the methodology used to assess the cost of controlling stormwater pollution. Cost data are combined with storage-treatment isoquants to determine an efficient expansion path for controlling varying degrees of wet weather control. It is possible to reduce the costs of stormwater pollution control by integrating this purpose with existing sewage treatment plants and storage facilities for storm drainage systems. Sample results were presented for Minneapolis, Minnesota.

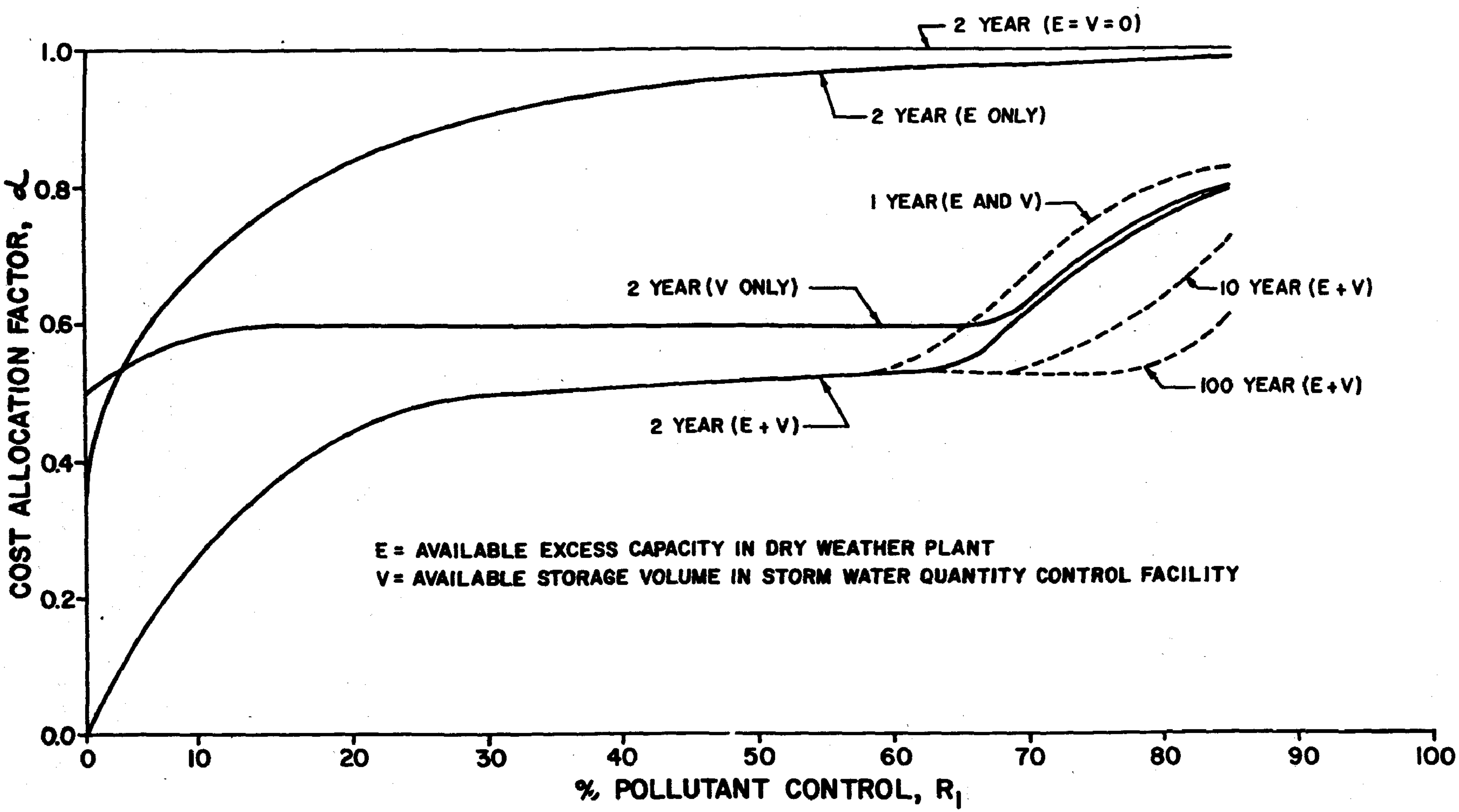

FIGURE 3. EFFECT OF DESIGN STORM AND NUMBER OF PURPOSES ON COST ALLOCATION FACTOR FOR VARIOUS LEVELS OF CONTROL.

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INTRODUCTION TO THE EPA STORM WATER MANAGEMENT MODEL (SWMM)

By Wayne C. Huber* and James P. Heaney*

PROBLEMS OF URBAN RUNOFF

An enormous pollution load is placed on stream and other receiving waters by combined and separate storm sewer overflows (Figure 1). It has been estimated that the total pounds of pollutants (BOD and suspended solids) contributed yearly to receiving waters by such overflows is of the same order of magnitude as that released by all secondary sewage treatment facilities (6,7). The Environmental Protection Agency (EPA) has recognized this problem and lead and coordinated efforts to develop and demonstrate pollution abatement procedures. As shown in Figure 2, these procedures include not only improved treatment and storage facilities, but also possibilities for upstream abatement alternatives such as rooftop and parking lot retention, increased infiltration, improved street sweeping, retention basins and catchbasin cleaning or removal (5,6). The complexities and costs of proposed abatement procedures require much time and effort to be expended by municipalities and others charged with decision making for the solution of these problems.

It was recognized that an invaluable tool for decision makers would be a comprehensive mathematical computer simulation program that would accurately model quantity (flows) and quality (concentrations) during the total urban rainfall-runoff process. This model would not only provide an accurate representation of the physical system, but also provide an opportunity to determine the effect of proposed pollution abatement procedures. Alternatives could then be tested on the model, and least cost solutions could be developed.

The resulting EPA Storm Water Management Model is introduced below. However, since its initial release in 1970, there has been an insurgence of urban runoff modeling, and it is worthwhile to review briefly objectives and options pertinent to management of urban stormwater runoff.

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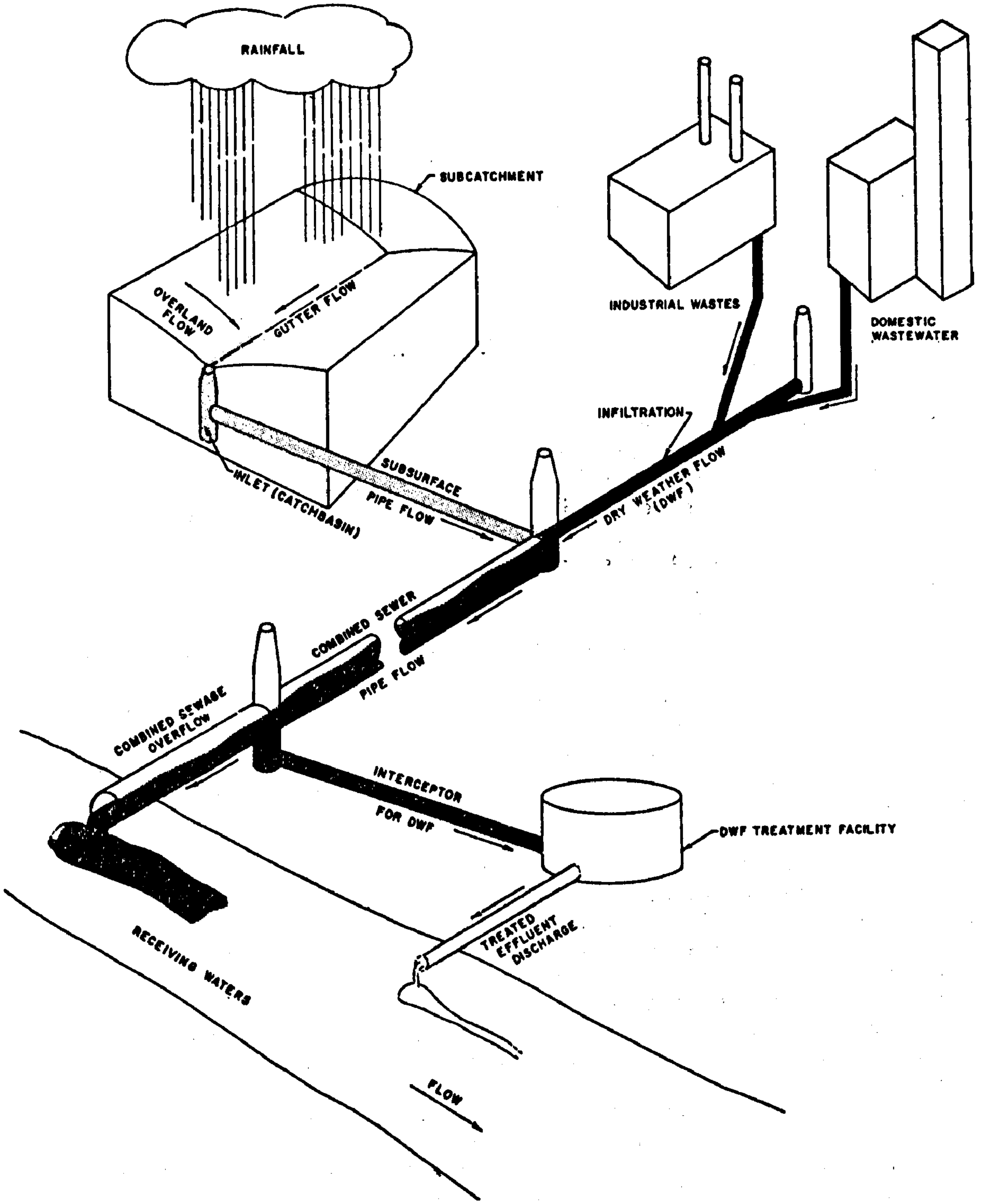


Note: DWF sources actually overlay the subcatchments. They are separated in the figure only to simplify the representation.

Figure 1. SCHEMATIC SYSTEM DRAWING RAINFALL THROUGH OVERFLOW.

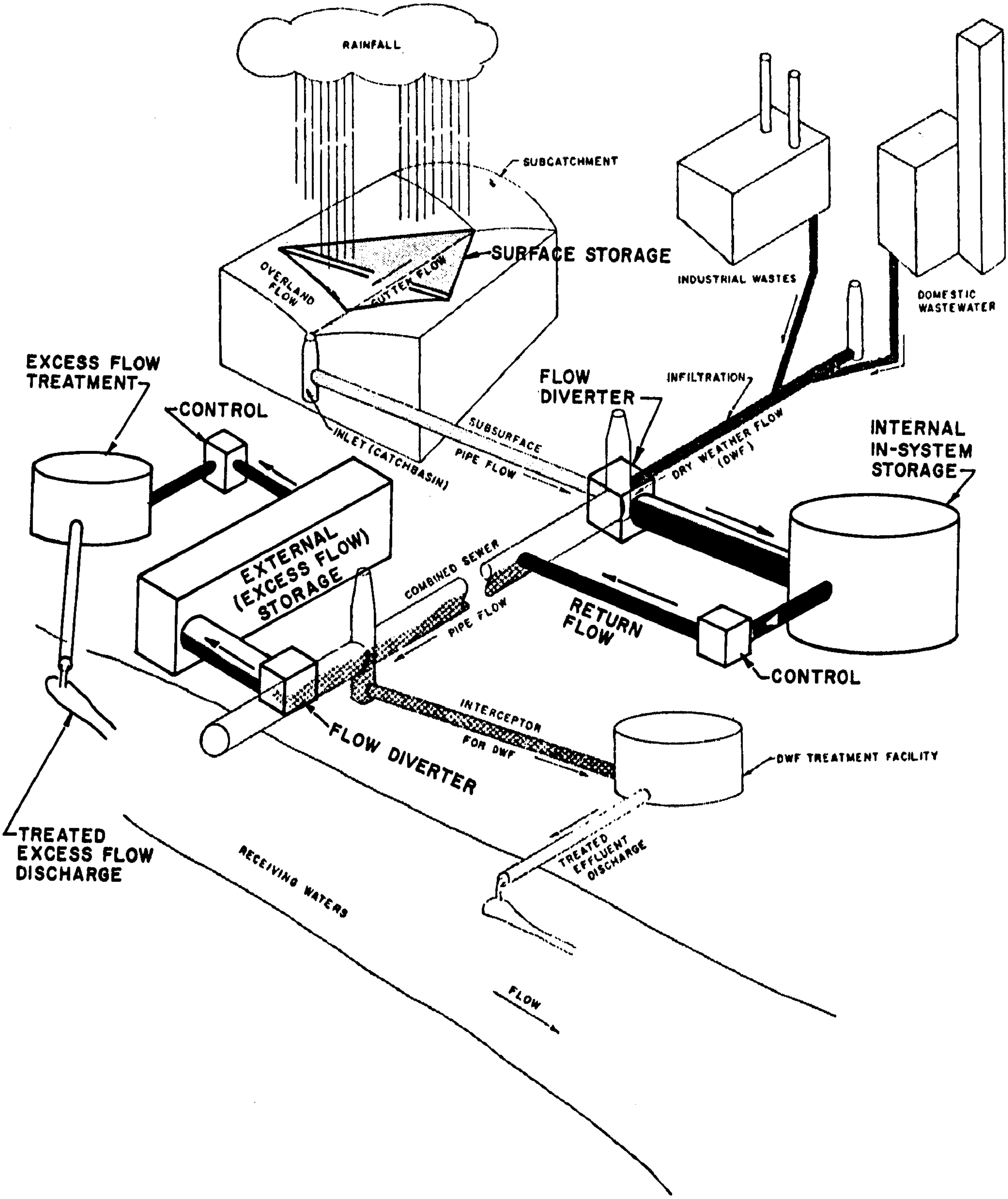


Figure 2. TYPICAL STORAGE-TREATMENT APPLICATIONS TO LIMIT UNTREATED OVERFLOWS.

URBAN RUNOFF MODELS

Objectives

The overall objective of urban runoff modeling is to aid in decision making for control of quantity and quality problems. Within this broad objective, three sub-objectives may be identified: planning, design and operation. Each objective typically produces models with somewhat different characteristics and the different models overlap to some degree. A more complete description of objectives, along with a description of several current models is given by Huber (10).

Planning Models

Planning models are used for an overall assessment of the urban runoff problem as well as estimates of the effectiveness and costs of abatement procedures. They may be used for "first cut" analyses of the rainfall-runoff process and illustrate trade-offs among various control options, e.g., treatment versus storage. They are typified by relatively large time steps (hours) and long simulation times (months and years). Data requirements are kept to a minimum and their mathematical complexity is low.

A current example of such a model is the STORM Model (12,21) developed by the Corps of Engineers Hydrologic Engineering Center (HEC) and Water Resources Engineers, Inc. (WRE) for the City of San Francisco. It utilizes hourly time steps and precipitation inputs and has simple quantity and quality prediction procedures based on such parameters as percent imperviousness and land use. Included are the effects of snow melt and soil erosion as well as treatment and storage options. The output may be used to illustrate, for example, the frequency and/or volumes of discharges to receiving waters of untreated urban runoff for a given treatment-storage combination. STORM has been run for simulation periods of 25 years and longer, depending upon the desired definition of return periods.

A planning model such as STORM may also be run to identify hydrologic events that may be of special interest for design or other purposes. These storm events may then be analyzed in detail using a more sophisticated design model. Planning or long-term models may also be used to generate initial conditions (i.e., antecedent conditions) for input to design models. Although not originally developed as such, as a result of recent modifications (23) the SWMM may also be used as a planning model. More details are presented later.

Design Models

Design models are oriented toward the detailed simulation of a single storm event. They provide a complete description of flow and pollutant routing from the point of rainfall through the entire urban runoff system and often into the receiving waters as well. Such models may be used for accurate predictions of flows and concentrations anywhere in the rainfall-runoff system and can illustrate the detailed and exact manner in which abatement procedures or design options affect them. As such, these models are a highly useful tool for determining least-cost abatement procedures for both quantity and quality problems in urban areas. Design models are generally used for simulation of a single storm event and are typified by short time steps (minutes) and short simulation times (hours). Data requirements may be moderate to very extensive depending upon the particular model employed.

The EPA Storm Water Management Model (16,17,18,19), frequently abbreviated "SWMM", is an example of a model developed specifically for simulation of urban quantity and quality processes and useful for the purposes mentioned above. It is also versatile enough to be used for certain planning studies or adapted to uses other than were originally intended. For instance, the surface runoff portion may be used to simulate natural drainage systems, and the receiving water portion may be applied to a variety of natural configurations independent of the urban runoff context.

Many other urban runoff models have been described in the literature (10) and are too numerous to enumerate here. Examples range from relatively simple models, e.g., RRL (25), ILLUDAS (26), Chicago (14), to highly complex models that utilize the complete dynamic equations of motion to simulate every aspect of the drainage systems, e.g., the WRE version of the SWMM (22), Hydrograph Volume Method (13), and Sogreah (24). These latter three models, for instance, are probably the best available for analysis of surcharging and backwater effects. However, many of these other models lack quality calculations; of the aforementioned ones, quality routing is included only in the WRE version of the SWMM. Furthermore, many are either proprietary or ill-documented. The EPA SWMM is well documented, widely tested and of a fairly high level of sophistication. In addition, through its broad use improvements and updating have been continuous. It is a widely accepted, detailed simulation model.

Operational Models

Operational models are used to produce actual control decisions during a storm event. Rainfall is entered from telemetered stations and the model is used to predict system responses a short time into the future. Various control options may then be employed, e.g., in-system storage, diversions, regulator settings.

These models are frequently developed from sophisticated design models and applied to a particular system. Examples are operational models designed for Minneapolis-St. Paul (3) and Seattle (15).

DEVELOPMENT OF THE STORM WATER MANAGEMENT MODEL

Under the sponsorship of the Environmental Protection Agency, a consortium of contractors -- Metcalf and Eddy, Inc., the University of Florida, and Water Resources Engineers, Inc. -- developed in 1969-70 a comprehensive mathematical model capable of representing urban stormwater runoff and combined sewer overflow phenomena. The SWMM portrays correctional devices in the form of user-selected options for storage and/or treatment with associated estimates of cost. Effectiveness is portrayed by computed treatment efficiencies and modeled changes in receiving water quality.

The project report is divided into four volumes. Volume I, the "Final Report" (16), contains the background, justifications, judgments, and assumptions used in the model development. It further includes descriptions of unsuccessful modeling techniques that were attempted and recommendations for forms of user teams to implement systems analysis techniques most effectively. Although many modifications and improvements have since been added to the SWMM, the material in Volume I still accurately describes most of the theory behind updated versions.

Volume II, "Verification and Testing" (17), describes the methods and results of the application of the original model to four urban catchments.

Volume III, the "User's Manual" (18), contains program descriptions, flow charts, instructions on data preparation and program usage, and test examples. This volume has recently been revised (11).

Volume IV, "Program Listing" (19), lists the entire original program and JCL as used in the demonstration runs. Since many routines in the updated version are similar or identical to the original, it is still a useful reference.

All three original contractors have continued to modify and improve the SWMM, as have numerous other users since its release. As a result, an official "Release 2" of the SWMM has been made in August 1974. Although it has been prepared for EPA by the University of Florida, it also relies heavily upon contributions by Water Resources Engineers and Metcalf and Eddy. The revised User's Manual (11) was written to accompany Release 2. Several of the modifications incorporated in Release 2 are also described in detail by Heaney and Huber et al. (8).

OVERALL SWMM DESCRIPTION

The comprehensive Storm Water Management Model uses a high speed digital computer to simulate real storm events on the basis of rainfall (hyetograph) inputs and system (catchment, conveyance, storage/treatment, and receiving water) characterization to predict outcomes in the form of quantity and quality values. Single event or continuous (e.g., yearly or longer) simulations may be performed, the former for design and the latter for planning purposes.

The simulation technique -- that is, the representation of the physical systems identifiable within the Model -- was selected because it permits relatively easy interpretation, and because it permits the location of remedial devices (such as a storage tank or relief lines) and/or denotes localized problems (such as flooding) at a great number of points in the physical system.

Since the program objectives are particularly directed toward complete time and spatial effects, as opposed to simple maxima (such as the rational formula approach) or only gross effects (such as total pounds of pollutant discharged in given storm), it is considered essential to work with continuous curves (magnitude versus time), referred to as hydrographs and "pollutographs". The units selected for quality representation, pounds per minute, identify the mass releases in a single term. Concentrations are also printed out within the program for comparisons with measured data.

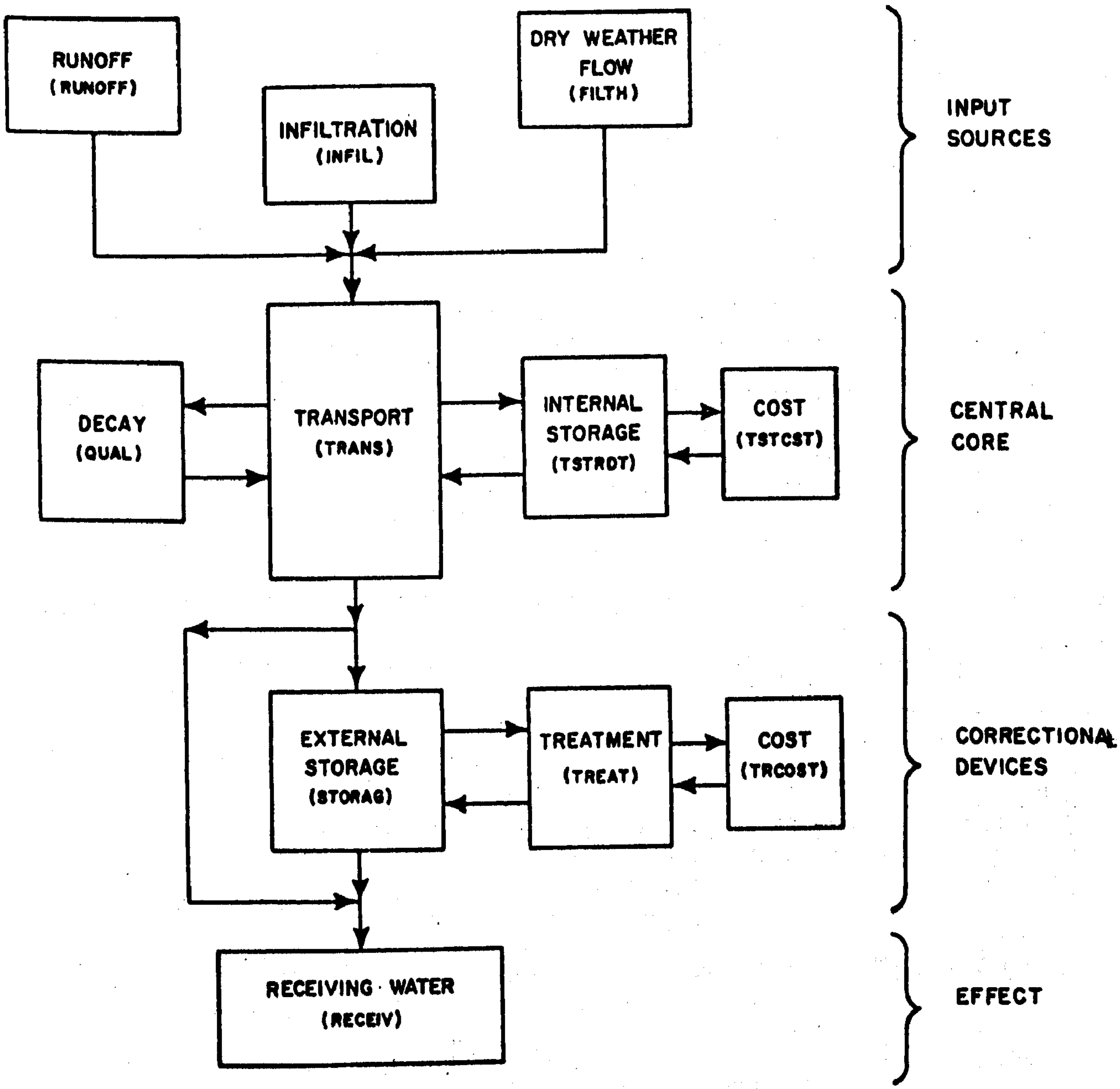
An overview of the Model structure is shown in Figure 3. In simplest terms the program is built up as follows:

1) The input sources:

    <u>RUNOFF</u> generates surface runoff using a kinematic wave approach based on arbitrary rainfall hyetographs, antecedent conditions, land use, and topography.

    <u>FILTH</u> generates dry weather sanitary flow based on land use, population density, and other factors.

    <u>INFIL</u> generates infiltration into the sewer system based on available groundwater and sewer conditions.



Note: Subroutine names are shown in parentheses.

FIGURE 3. OVERVIEW OF MODEL STRUCTURE.

2) The central core:

TRANS carries and combines the inputs through the sewer system using a modified kinematic wave approach in accordance with Manning's equation and continuity; it assumes complete mixing at various inlet points.

3) The correctional devices:

TSTRDT, TSTCST, STORAG, TREAT, and TRCOST modify hydrographs and pollutographs at selected points in the sewer system, accounting for retention time, treatment efficiency, and other parameters; associated costs are computed also.

4) The effect (receiving waters):

RECEIV routes hydrographs and pollutographs through the receiving waters, which may consist of a stream, river, lake, estuary, or bay.

The quality constituents considered for simulation within the total program are 5-day BOD, total suspended solids, and total coliforms (represented as a conservative pollutant). These constituents were selected on the basis of available supporting data and importance in treatment effectiveness evaluation. In addition, the Runoff Block also models COD, settleable solids, total nitrogen, phosphate and grease. However, routing of these parameters through subsequent blocks presently involves special programming efforts. The contribution of suspended solids by urban erosion processes is also simulated by the program.

PROGRAM BLOCKS

Executive Block

The adopted programming arrangement consists of a main control and service block, the Executive Block, a service block (Combine), and four computational blocks: (1) Runoff Block, (2) Transport Block, (3) Storage Block, and (4) Receiving Water Block.

The Executive Block assigns logical units (disk/tape/drum), determines the block or sequence of blocks to be executed, and, on call, produces graphs of selected results on the line printer. Thus, this Block does no computation as such, while each of the other four blocks are set up to carry through a major step in the quantity and quality computations. All access to the computational blocks and transfers between them must pass through subroutine MAIN of the Executive Block. Transfers are accomplished on off-line devices (disk/tape/drum) which may be saved for multiple trials or permanent record.

Combine Block

This block allows the manipulation of data sets (files stored on off-line devices) in order to aggregate results of previous runs for input into subsequent blocks. In this manner large, complex drainage systems may be partitioned for simulation in smaller segments.

Runoff Block

Quantity Techniques --

Runoff hyetographs are generated in two phases: overland flow and small gutter/pipes. Both phases utilize a kinematic wave formulation through the use of Manning's equation and continuity. Gutter/pipes may be arranged in a tree-like network with inflows from upstream gutters and/or surface subcatchments. Outflows are typically to inlet manholes of the Transport Block, although output may be transferred directly to any "downstream" block.

Infiltration from pervious areas is accomplished through the integrated form of Horton's equation. For continuous simulation (23), infiltration capacity is recovered during periods of no runoff, as is detention storage through evaporation.

Data inputs include physical parameters for each subcatchment and/or gutter/pipe (e.g., area, imperviousness, slope, roughness, etc.) and rainfall hyetographs. The latter may be input at short time intervals (e.g., five minutes) for single event simulation for design purposes or at long time intervals (e.g., one hour from US Weather Service tapes) for long term, continuous simulation.

Quality Techniques --

Surface "washoff" quality is generated by assuming a linear build-up of dust and dirt as a function of land use, dry days and street sweeping practices. Available pollutant loads (i.e., BOD, suspended solids, total coliforms, settleable solids, COD, N, PO_4 and grease) are determined as fractions of the dust and dirt load. Values for these parameters (dust and dirt loads and pollutant fractions) are generally based on data from Chicago (1) but should be altered to reflect local conditions if known. For continuous simulation, surface loads are recharged during periods of no runoff.

Surface pollutographs are generated assuming an exponential erosion of available pollutant loads. The decay coefficient is a function of the runoff rate. Pollutographs are routed through gutter/pipes, if used, assuming complete mixing within each conduit. Catchbasins contribute BOD and COD in a similar exponential fashion.

Urban erosion is simulated using the Universal Soil Loss Equation (8). Data required for quality computations include land use definition for each subcatchment (i.e., single family residential, multiple family residential, commercial, industrial and open or park), catch-basin information, curb lengths and soil loss parameters.

Other Information --

Runoff may be run independently of other SWMM blocks to provide "first cut" predictions of quantity and quality parameters for urban catchments. However, more useful results may be generated by linking Runoff directly to Storage/Treatment. The model may then be used in a planning mode with output similar to that of STORM (12,21). This will be discussed later.

Discretization of the urban catchment may be performed in a detailed manner (i.e., many subcatchments and conduits) if necessary. However, recent results indicate that a very coarse discretization (e.g., one subcatchment, no conduits) may produce hydrographs and pollutographs at the outlet that are entirely comparable to those of a detailed schematization (23). Obviously, considerable work may be avoided under the latter option.

Transport Block

Quantity Techniques --

The primary flow and quality routing through the drainage system is accomplished in this block. In addition, dry weather flow (DWF) and infiltration are generated, and internal storage units may be included.

To categorize a sewer system conveniently prior to flow routing, each component of the system is classified as a certain type of "element." All elements in combination form a conceptual representation of the system in a manner similar to that of links and nodes. Elements may be conduits, manholes, lift stations, overflow structures, or any other component of a real system. Conduits themselves may be of different element types depending upon their geometrical cross-section (e.g., circular, rectangular, horseshoe, trapezoidal). A sequencing is first performed to order the numbered elements for computations. Flow routing then proceeds downstream through all elements during each increment in time until the storm hydrographs have been passed through the system.

The solution procedure follows a modified kinematic wave approach in which disturbances are allowed to propagate only in the downstream direction. As a consequence, backwater effects are not modeled beyond the realm of a single conduit, and downstream conditions (e.g., tide gates, diversion structures) will not affect upstream computations. Systems that branch in the downstream direction can be modeled using "flow divider" elements to the extent that overflows, etc., are not affected by backwater conditions. Surcharging is modeled simply by storing excess flows (over and above the full-flow conduit capacity) at the upstream manhole until capacity exists to accept the stored volume. Pressure-flow conditions are not explicitly modeled and no attempt is made to determine if ground surface flooding exists. However, a message is printed at each time step for each location at which surcharging occurs. The Transport routine has proven its ability to model accurately flows in most sewer systems, within the limitations discussed above, and as such it should be more than adequate for most applications. However, it will not accurately simulate systems with extensive interconnections or loops, systems that exhibit flow reversals or significant backwater effects, or systems in which surcharging must be treated as a pressure-flow phenomenon. These problems should be overcome with the inclusion in 1975 of the WRE transport routine (22) as an option in the Transport Block. This routine utilizes the complete, coupled equations of motion and simulates surcharge and pressure-flow conditions as well.

A present option in the program is the use of the internal storage model which acts as a transport element. The model provides the possibility of storage-routing of the storm at one or two separate points within the sewer system (restricted by computer core capacity). The program routes the flow through the storage unit for each time-step based on the continuity equation in a manner analogous to flood routing through a reservoir. Extensive backwater conditions may thus be modeled by treating portions of the sewer system as a storage unit with a horizontal water surface.

Another program option is a modest hydraulic design capacity (8). If requested, the program will size conduits such that surcharging is avoided. Although the entire system will then have adequate hydraulic capacity to pass the input hydrograph, this is sometimes accomplished through the loss of considerable in-system storage and loss of consequent attenuation of the downstream peak flow.

Data input consists of physical parameters of the drainage system (e.g., shapes, dimensions, slopes, roughnesses), linkage information, and data descriptive of special elements (e.g., flow dividers, storage units). The program time step must be the same as that of the Runoff Block (if Runoff is used to generate input).

Quality Techniques --

Pollutants are routed through the transport system by means of complete mixing within each element. They may be introduced into the system by four means:

1) Storm-generated pollutographs (and hydrographs) may be computed by the Runoff Block and transferred on tape/disk devices to enter the system at designated inlet manholes.

2) Pollutographs (and hydrographs) may be input from cards or generated by previous runs of the Transport or Storage/Treatment Blocks.

3) Residual bottom sediment in the pipes may be resuspended due to the flushing action of the storm flows. In this manner, a "first flush" effect is simulated.

4) For combined systems, DWF pollutographs are also entered at designated inlet manholes.

The routing of the pollutants is then done for each time step. The maximum number of contaminants that can be routed is four, although suspended solids, BOD and coliforms are the only ones commonly input from Runoff.

Dry Weather Flow and Infiltration --

Average DWF quantity and quality are generated on the basis of land use, population density and other demographic parameters. Measured flows (e.g., at a DWF treatment plant) are used to calibrate the prediction. Hourly and daily correction factors are used to modify predicted average values.

Total infiltration at the system outlet must be input by the user. The program then apportions this total throughout the system on the basis of conduit lengths and joint distances. If measurements are unavailable, advice is given in the User's Manual (11) on infiltration prediction.

Other Information --

Obviously, the Transport Block may be used for separate systems by not including DWF or infiltration. Overbank flow in a natural channel may be simulated using certain types of flow dividers. Discretization may be extensive or simple as required. However, for some SWMM runs, especially those for planning purposes, Transport may be conveniently omitted entirely.

Storage/Treatment Block

Storage Routing Techniques --

Quantity -- Flow routing through storage units is accomplished using simple level-pool, storage routing techniques (9, p. 356). Geometries may be regular or irregular with alternative inlet and outlet controls such as by weir, orifice or pumping. Only the storage unit itself possesses storage characteristics. Storage effects of treatment devices such as sedimentation, swirl concentrator, dissolved air flotation, etc., are scheduled to be added to SWMM during 1975.

Quality -- Removal of pollutants by sedimentation in storage is computed as a function of detention time as shown in Figure 4. Detention time in turn is computed on the basis of either plug flow or complete mixing (user specified), the latter probably being more appropriate for short detention times and the former for long ones. Resuspension of solids is not modelled.

Treatment Techniques --

Many treatment options are available in this block, as shown in Figure 5. Each option computes removal of BOD, suspended solids and total coliforms as a function of flow rate, based on actual field data as much as possible. Certain options require other design and operational parameters as input data (e.g., particle size distribution, optional addition of chemicals). In addition, DWF treatment may be simulated prior to storage or treatment using constant removal functions, as indicated in the overall storage/treatment schematization shown in Figure 6.

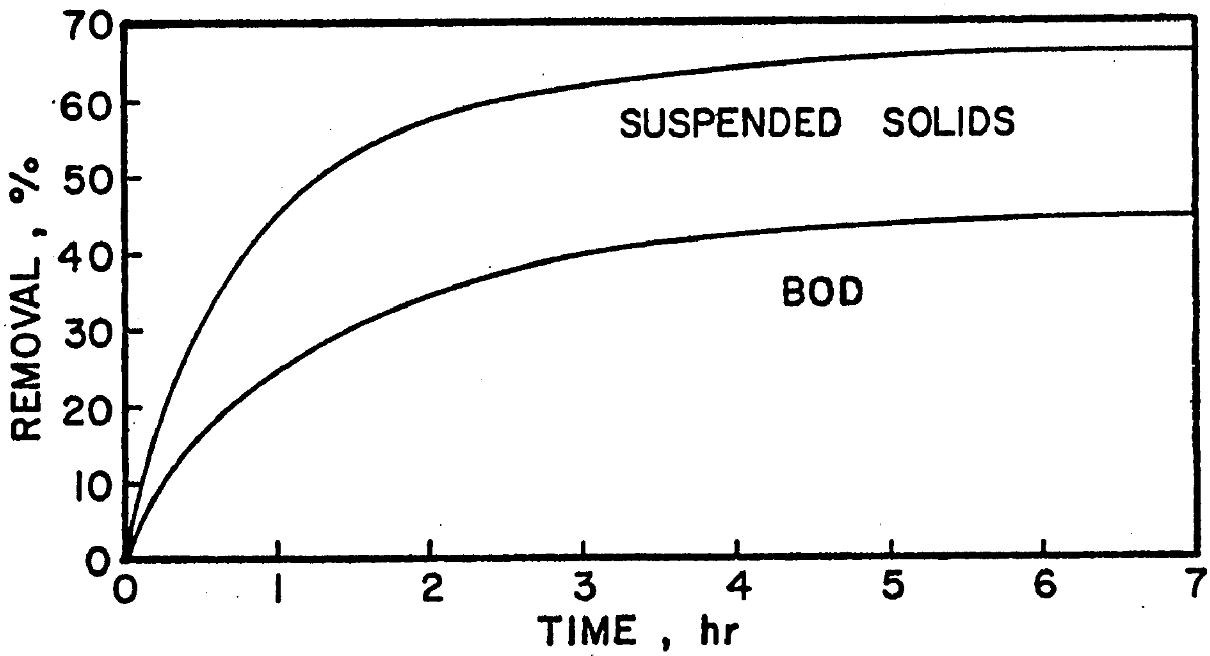


FIGURE 4. POLLUTANT REMOVAL IN A STORAGE UNIT AS A FUNCTION OF DETENTION TIME.

Source: Fair, G. M., J. C. Geyer and D. A. Okun, Water and Wastewater Engineering, John Wiley and Sons, Inc., 1968, p. 25-25.

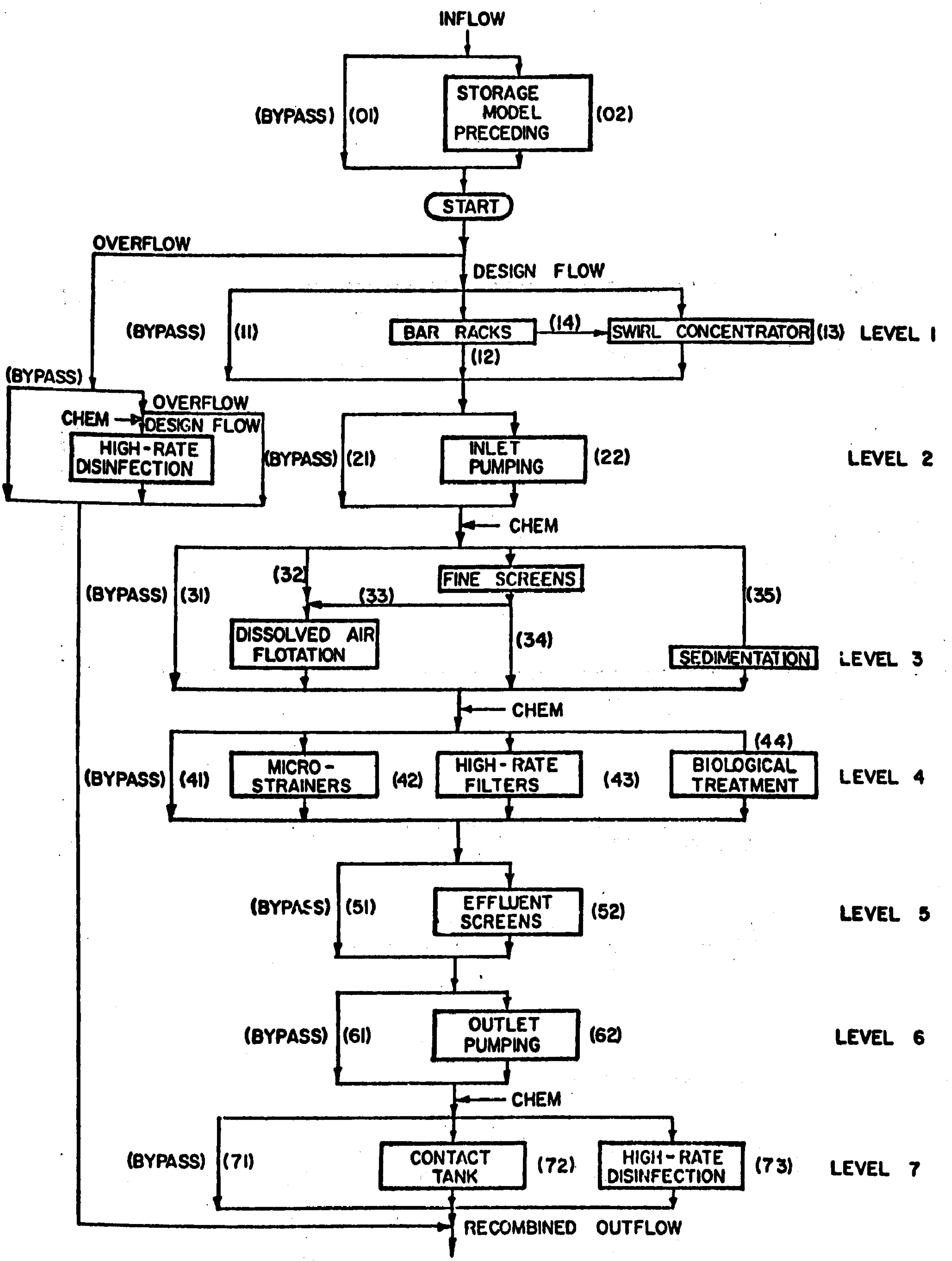


FIGURE 5. OPTIONS AVAILABLE IN THE WET WEATHER TREATMENT STRING IN THE SWMM.

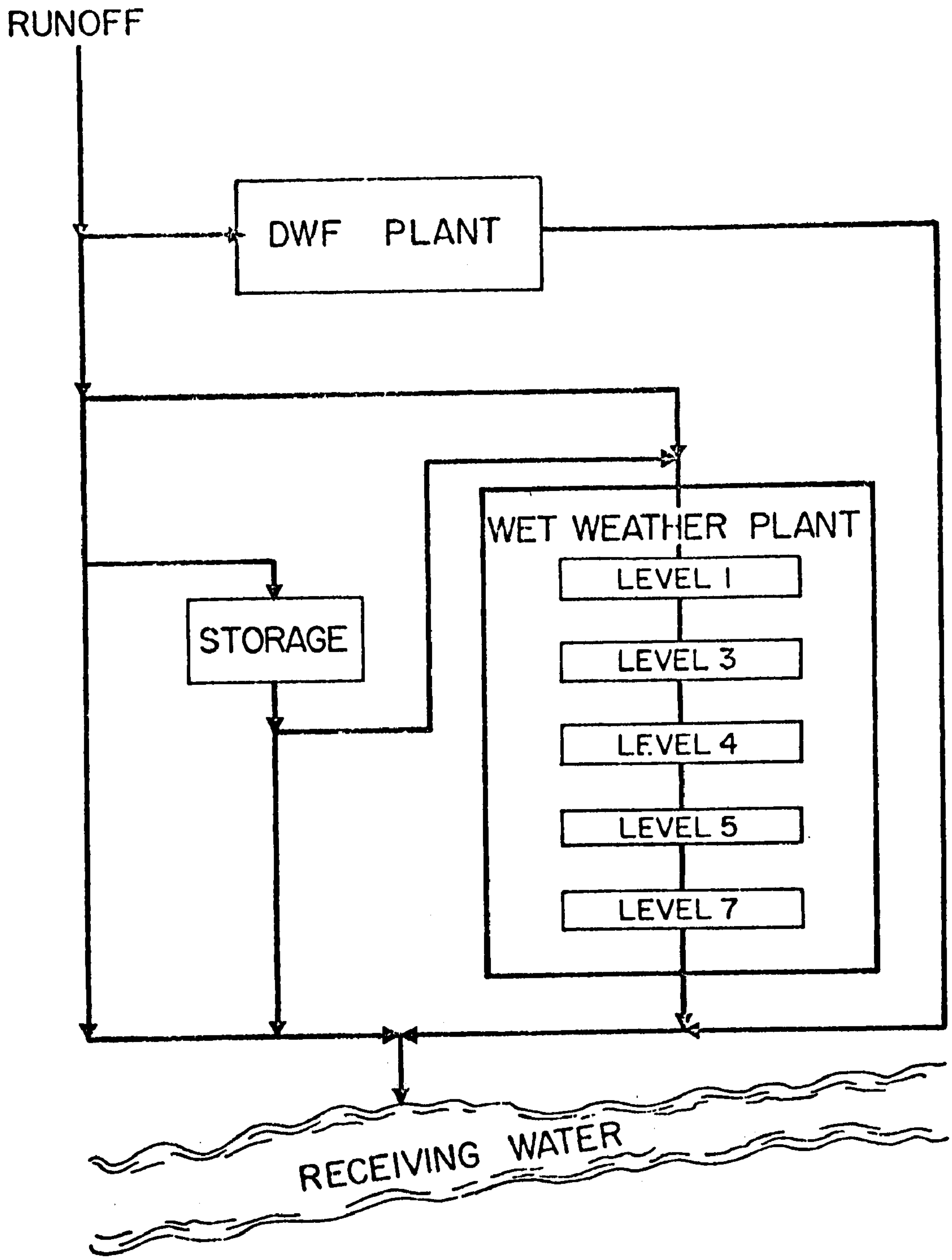


Figure 6. OPTIONS AVAILABLE IN THE SWMM STORAGE/TREATMENT BLOCK.

Cost Computations --

Cost of storage is based on land and excavation costs. Treatment costs are calculated as a function of design flow rate. These unit costs in turn are included in the model on the basis of the most recent demonstration grant or other installations. Costs are localized using ENR factors and amortized over a specified period.

Other Information --

The overall storage/treatment schematization shown in Figure 6 is well suited to both planning and design oriented simulations. The former is a relatively new application of the SWMM (23), but is advantageous because of the more exact representation of the effect of storage in a long term simulation. In particular, SWMM accounts for treatment that occurs in storage due to sedimentation, and does not treat it simply as a holding tank. This is highly significant when assessing the relative effect of treatment versus storage as control options. For instance, if flow is allowed through a full storage unit, some treatment by sedimentation may still occur if sufficient detention time is planned (e.g., a minimum of 15 minutes). Thus, when combined with Runoff, a powerful planning tool is available. Any of the several pathways shown in Figure 6 may be utilized to simulate better the actual effects of control options.

Receiving Water Block

Quantity Techniques --

The Receiving Water Model simulates the behavior of estuaries, bays, reservoirs, lakes and rivers. The program has two distinct phases which may be simulated together or separately. In Phase A, the time history of stage, velocity, and flow is generated for various points in the system. In Phase B, the computed hydrodynamics are utilized to model the behavior of conservative and nonconservative quality constituents.

The receiving water is schematized by cutting the continuous system into a series of discrete one- and two-dimensional elements which connect node points. For the purpose of this analysis, the velocity of flow is assumed constant with depth, one-dimensional elements represent rivers and specific channels, and two-dimensional elements represent areas of continuous water surface. For each time-step, the complete equations of motion and continuity are applied to all nodal points to derive the hydrodynamics for the system. As a result, the input and output of the model represent completely transient phenomena. Of course, the spatial distribution is determined by observation of computed flows and velocities along channels and stages at nodes (junctions).

Input data consist primarily of a description of bathymetry and hydraulic characteristics of junctions and channels (e.g., surface areas and widths, initial stages and depths, roughnesses and geometry). Tidal or other boundary condition data may be input for up to 10 junctions.

Quality Techniques --

Given channel flows and junction volumes, quality parameters are routed by advection, with fluxes flow-weighted if more than one channel flows into a junction. Diffusive fluxes are not modelled explicitly although some "numerical diffusion" results from the computations.

Non-conservative parameters (e.g., BOD, DO) are simulated assuming first-order decay and reaeration. Under these restrictions, any water quality parameters may be modeled, and pollutographs (and hydrographs) may be conveniently entered from cards as well as (or in place of) from tape/disk files generated by other SWMM blocks.

Quality input data include information on sources, sinks and decay rates and a tidal exchange factor. The qualityportion may be run several times after just one run of the quantity portion.

Other Information --

Receiving is a useful "stand-alone" model. Its many options and easy discretization allow its use in a variety of receiving waters including swamps and marshes, in applications independent of the rest of the SWMM. Its primary limitations are its psuedo representation of two-dimensional systems and lack of diffusive fluxes in quality calculations. The Receiving Block has been considerably expanded with regard to available water quality parameters and hydrodynamic options by recent work of the Raytheon Corporation (20).

Total Simulation

In principle, the capability exists to run all blocks together in a given computer execution, although from a practical and sometimes necessary viewpoint (due to computer core limitations), typical runs usually involve only one or two computational blocks, together with the Executive Block. Using this approach avoids overlay and, moreover, allows for examination of intermediate results before continuing the computations. Further, it permits the use of intermediate results as start-up data in subsequent execution runs, thereby avoiding the waste of repeating the computations already performed.

USER REQUIREMENTS

Computer Facilities

A large, high-speed computer is required for operation of the SWMM such as an IBM 360, UNIVAC 1108 or CDC 6600. The largest of the blocks requires on the order of 90,000 words of storage. Through considerable efforts, users have been able to adapt portions of the program to small-core machines such as the IBM 1130, but only with extensive use of off-line storage and a large increase in execution time.

Data Requirements

As can be surmised, the data requirements for the SWMM are extensive. Collection of the data from various municipal and other offices within a city is possible to accomplish within a few days. However, reduction of the data for input to the Model is time consuming and may take up to two man-weeks for a large area (e.g., greater than 2000 acres). On an optimistic note, however, most of the data reduction is straight forward (e.g., tabulation of slopes, lengths, diameters, etc., of the sewer system). The SWMM is flexible enough to allow different modeling approaches to the same area, and a specific, individual modeling decision upstream in the catchment will have little effect on the predicted results at the outfall. Moreover, a highly detailed schematization of the urban catchment (requiring extensive data reduction) is often unnecessary, and reasonably accurate results (compared to the detailed schematization) may be obtained with a relatively "coarse" analysis, as mentioned previously while discussing the Runoff Block.

Verification and Calibration

The SWMM is designed as a "deterministic" model, in that if all input parameters are accurate, the physics of the processes are simulated sufficiently well to produce accurate results without calibration. This concept fails in practice because neither the input data nor the numerical methods are accurate enough for most real applications. Furthermore, many computational procedures within the Model are based upon limited data themselves. For instance, surface quality predictions are based almost totally on data from Chicago, and are unlikely to be of universal applicability.

As a result it is essential that some local verification/calibration data be available at specific application sites to lend credibility to the predictions of any urban runoff model. These data are usually in the form of measured flows and concentrations at outfalls or combined sewer overflow locations. Note that quality measurements without accompanying flows are of little value. The SWMM has sufficient parameters that may be "adjusted", particularly in the Runoff Block, such that calibrating the model against measured data is usually readily accomplished.

EXAMPLE: RUNOFF, TRANSPORT AND STORAGE/TREATMENT

Introduction

Details of SWMM methodology and operation can only be learned through a more lengthy study of the documentation, coupled with experience. As an alternative, two examples are presented that illustrate various SWMM capabilities and features. The first will present an application of the surface runoff and quality routines, as tested in Lancaster, Pennsylvania. Since the receiving water application at that location is relatively uninformative, a different receiving example is given later. Further details of the example presented below are given by Heaney and Huber et al. (8).

The Study Area

The City of Lancaster, Pennsylvania, population 79,500, is situated in a drainage area of about 8.24 square miles (21.34 km^2). The receiving stream in the Lancaster area is the Conestoga River which drains an area of approximately 473 square miles (1225 km^2) into the Susquehanna River. The average flow is 387 ft^3/sec (11 m^3/sec) with a maximum recorded flow of 22,800 ft^3/sec (638 m^3/sec).

There are two sewage treatment plants within the City, both of which discharge into the Conestoga River. The North Plant with a capacity of 10 mgd (3.80×10^4 m^3/day) serves a population of 36,000 people, and the South Plant, recently expanded from 6 mgd (2.28×10^4 m^3/day) to 12 mgd (4.56×10^4 m^3/day), is designed to serve 69,000 people. Both plants provide secondary treatment. About one third of the flow to the North Plant is derived from areas with separate sewers outside the City serving an estimated population of 17,500 people and some industries. The remaining two thirds of the sewage flow to the North Plant is derived from the combined sewers serving the north part of the City plus about 250 suburban acres estimated to have 18,500 people and many water-using industries. In addition, most of the year the water table is high resulting in considerable infiltration. An overflow line diverts excess flow to the Conestoga during wet weather. The North Plant drainage area is estimated at 3.72 square miles (9.63 km^2).

The South Plant is designed to handle a population of 34,500 served by combined sewers and, in addition, up to an approximately equal amount from separated sewers throughout the surrounding area. The South Plant drainage area encompasses 4.52 square miles (11.71 km^2) and is comprised of four districts. Stevens Avenue district which is the subject of an EPA demonstration grant is one of the four districts connected to the South Plant. Three of the districts, including Stevens Avenue, pump the sewage from a receiving station within the district to the South Plant. All locations have overflow arrangements that discharge into the Conestoga River when the capacity of the system is exceeded.

The total drainage area of the Stevens Avenue district is 227 acres (92 ha) which, while only about 4.3 percent of the total Lancaster drainage area served by North and South treatment plants, is 17 percent of the drainage area designed to flow into the South Plant from combined sewers. The population within the Stevens Avenue district is estimated at 3,900. Figure 7 illustrates various drainage districts within the City.

Demonstration Grant Description

In order to remedy the situation resulting from combined sewer overflows, the City of Lancaster decided to explore means other than sewer separation.

Construction of several underground silos at various locations within the City is contemplated for retention of overflow during wet periods and subsequent pumping to the treatment plants during low flow periods.

Stevens Avenue district was selected as the demonstration site for evaluation of the effectiveness of a silo in combating combined sewer overflows. The sewer layout for Stevens Avenue district is shown in Figure 8. During normal dry weather periods, the dry weather flow is pumped to the South Treatment Plant. During wet periods, when the incoming flow to the pump station exceeds the capacity of the station, the overflow discharges directly into the Conestoga River through a 60-inch (1520 mm) sewer located at point 6 on Figure 7.

The City of Lancaster also authorized APWA to develop design parameters for a full-scale swirl concentrator for removal of solids prior to the retention of flow in the underground silo (2). Location of the demonstration site is shown in Figure 8. A flow diagram of the pro-

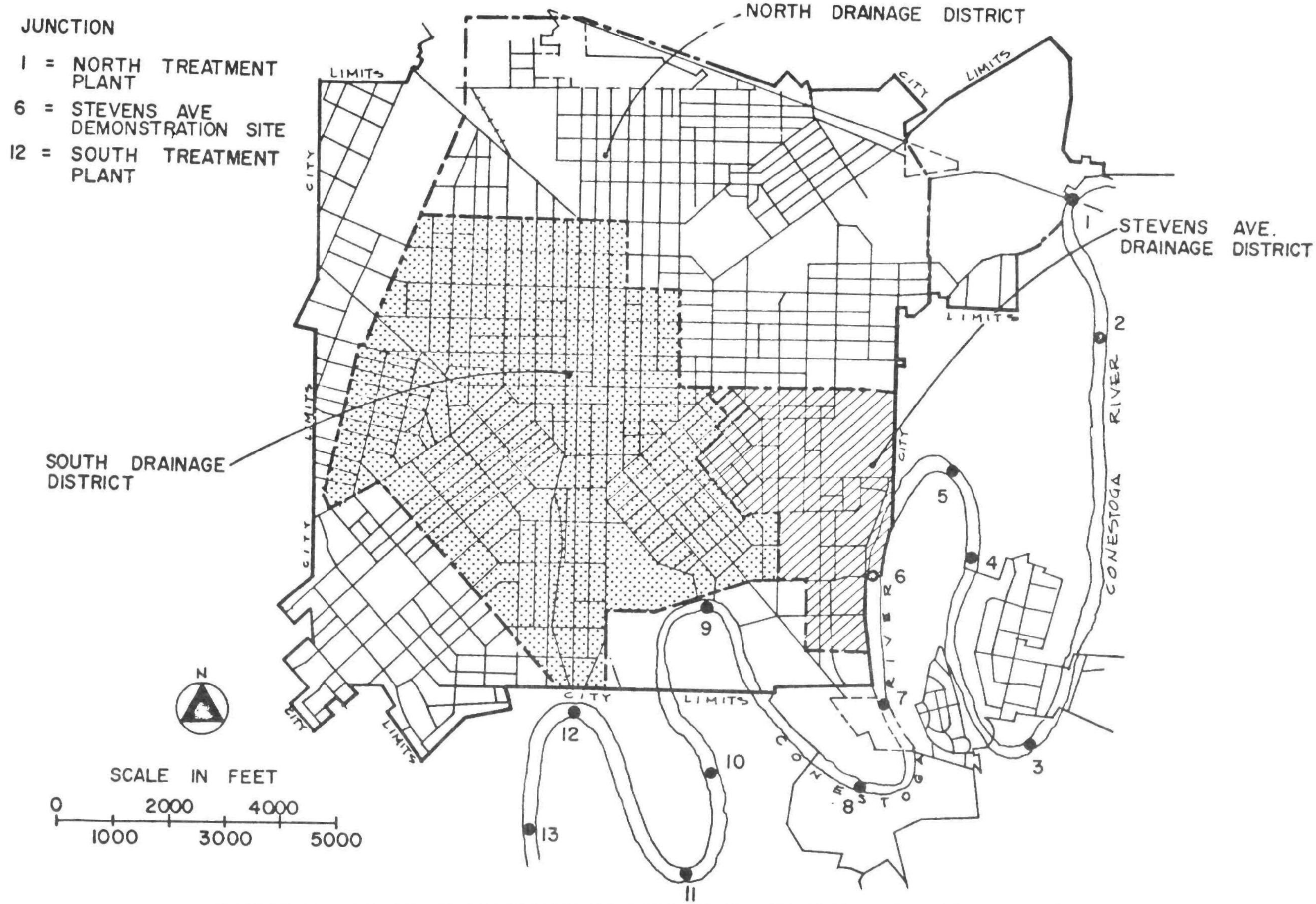


FIGURE 7. DRAINAGE DISTRICTS OF LANCASTER, PENNSYLVANIA AND NUMBERING SYSTEM FOR RECEIVING JUNCTIONS.

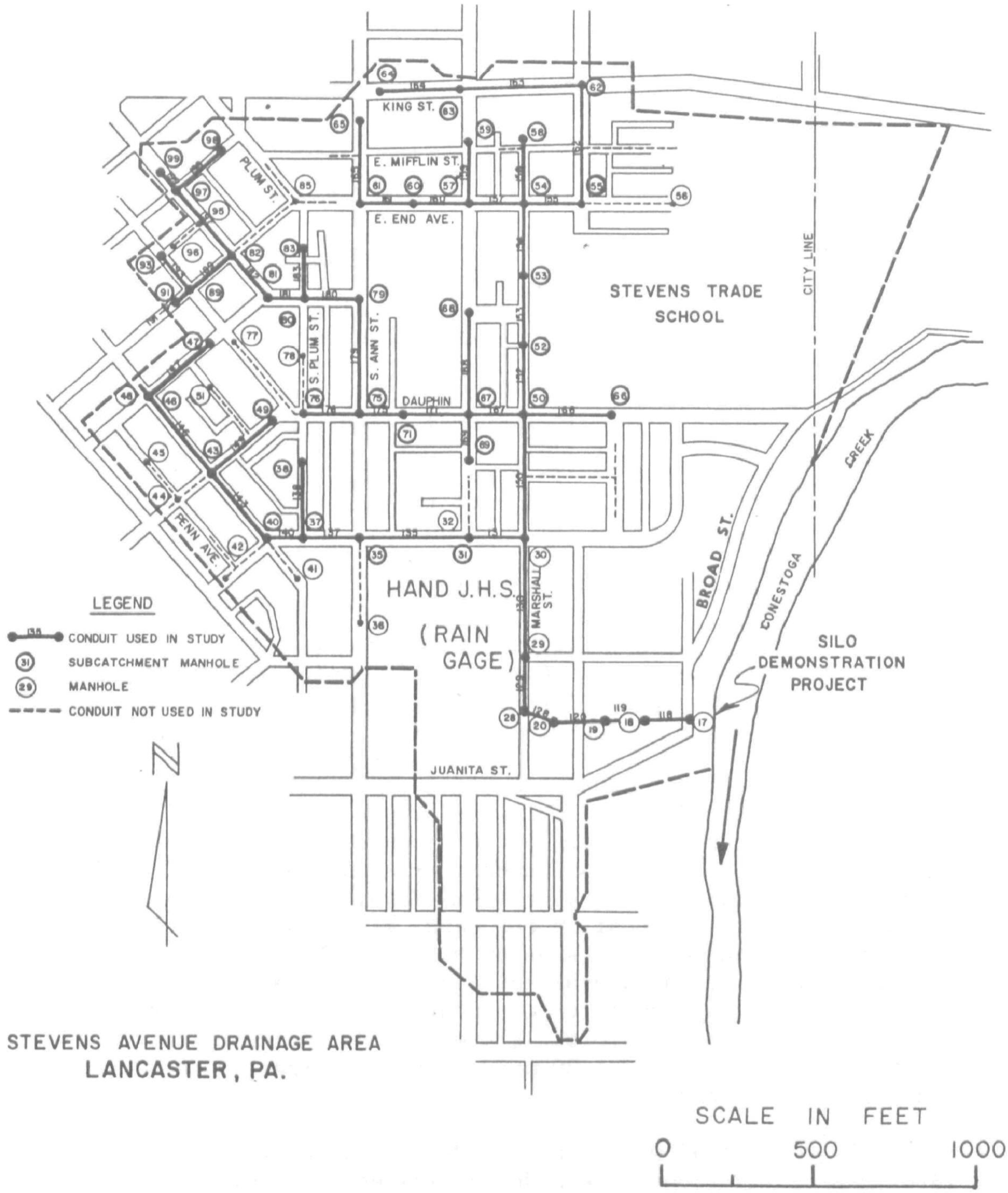


Figure 8. STEVENS AVENUE DRAINAGE AREA WITH RUNOFF-TRANSPORT NUMBERING SYSTEM.

posed swirl concentrator-silo treatment is presented in Figure 9. In order to fully evaluate this treatment, the city decided to include chlorination and microstraining as a part of this demonstration project. The capacity of the silo is expected to be 160,000 ft^3 (4480 m^3).

The tasks assigned to the University of Florida were as follows:

1) conduct further verification and testing of the Storm Water Management Model based on active overflow measurements on selected storm events and to make refinements to the Model;

2) provide results of simulations to the APWA in order for it to develop design criteria and sizing of the swirl concentrator;

3) simulate the effect of the swirl concentrator-underground silo treatment; and

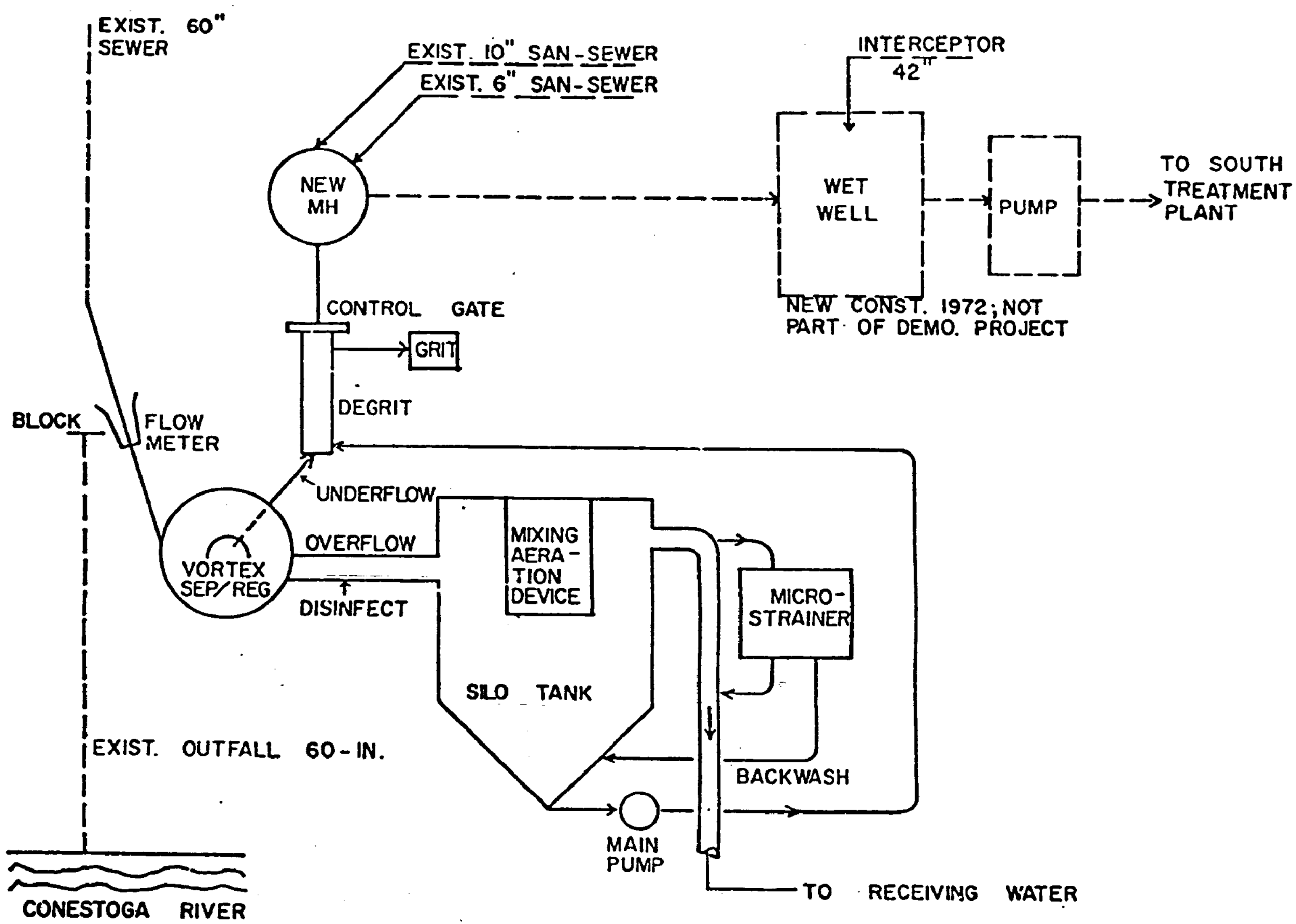
4) simulate the effect of combined sewer overflow from the entire city to the Conestoga River.

Description of the Stevens Avenue Runs

A total of three studies comprising eight storms were simulated, two of which are shown here. The City and its engineers provided input data as well as one overall set of measurements. A description of each study and its results are given below.

Study No. 1.--The first study was based on a series of storms between July 29 and August 3, 1971. The six-day period deposited a record amount of precipitation throughout the Lancaster area (variously measured between 7.3 and 9.46 inches, or 18.54 cm and 24.03 cm respectively). During four of the six days, the storms were very intense over short periods; in one case, being the second heaviest on record. For purposes of simulation, Study No. 1 was divided into six storms. Due to the unavailability of field data for the study, verification of the results of computer simulations for each of these storms was not possible. These runs do indicate that an overflow as high as 400 cfs (11.20 m^3/sec) may be expected for a storm event similar to Storm No. 6.

Results of this study were used by APWA in sizing the swirl concentrator.[3] A design flow of this device was established at 165 cfs (4.62 m^3/sec).



PRELIMINARY LANCASTER FLOW DIAGRAM

FIGURE 9. FLOW OF TREATMENT-STORAGE OPTIONS AT DEMONSTRATION SITE.

Computer simulation studies were also conducted for all six storms to evaluate the effect of the swirl concentrator-underground silo facilities on the combined overflow quality. The results of Storm No. 6 are shown on Figure 10. As illustrated, the quality of all the overflow is significantly improved through the installation of the swirl concentrator and a chlorine contact tank. Figure 10 also shows that as soon as the swirl concentrator operates at full capacity (165 cfs), excess stormwater simply overflows into the silo. The shaded portion of the flow versus time graph indicates the volume of silo storage, 160,000 ft^3 (4530 m^3). Computed flow from transport reflects the stormwater inflow to the storage facility (which is the same as the computed flow to the river without the silo simulation). The silo dewatering rate is limited by existing pumping capacity of the Stevens Avenue station (3.57 cfs or 0.10 m^3/sec). For simulation purposes, pumping was set to begin at a silo depth of 3 feet (0.91 m).

Study No. 3.--This study is based on a relatively minor rainfall event of March 22, 1972. This study is of special importance, however, because it is one of the types most frequently experienced in terms of intensity of rainfall (return period of less than one year). It is also one for which relatively complete verification data such as rainfall, flow readings and samples were collected. The rainfall is shown in Figure 11 along with results of the computer simulation showing overflow quantity and quality. Shown in the same illustration are the actual quantity and qualitymeasurements (points labeled x) of the overflow. It can be seen that agreement between the computer simulation and the actual measurements of flow is only fair. However, considerable doubt exists as to the validity of the flow data, since they were obtained from depths measured with a yardstick inserted into the mouth of the outfall. The agreement between the computed and measured quality parameters is only fair also. It should be noted that no calibration of the SWMM was attempted due to the lack of good verification data.

Computer simulations were also conducted on this study to determine the effect of the swirl concentrator-underground silo system. These results are also shown in Figure 11. With the silo system, the model indicates no overflow into the Conestoga River.

Summary

Testing of the SWMM in Lancaster, Pennsylvania, has revealed the importance of having sufficient and accurate measured data for model calibration. It is recommended that the SWMM user begin his simulation from a coarse analysis of the study area. Subsequent refinement

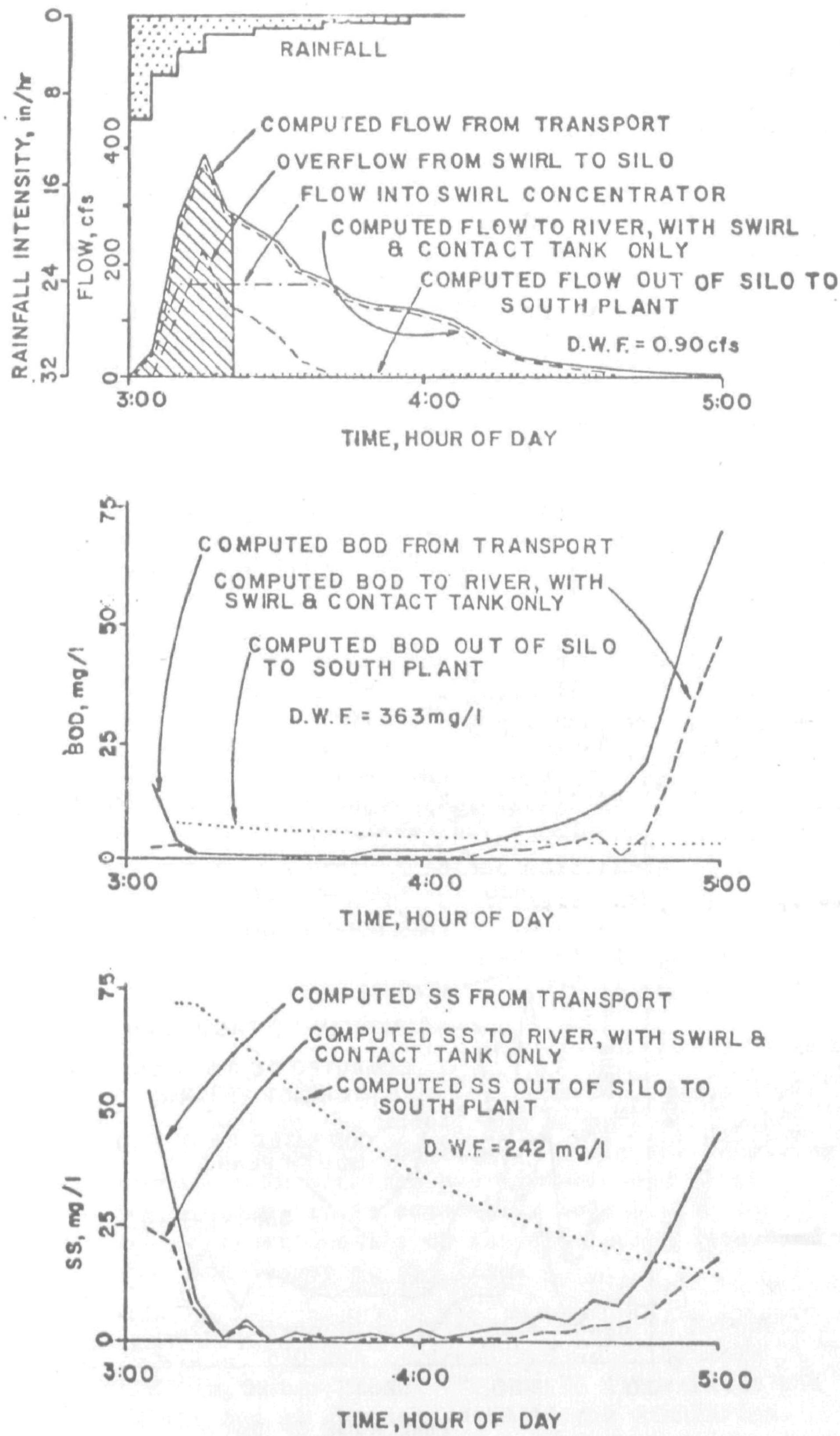


Figure 10. SIMULATION FOR STEVENS AVENUE, STUDY 1, STORM 6. HIGH CONCENTRATIONS OUT OF SILO REFLECT INITIAL SLUSH OF DWF IN SEWER SYSTEM INTO THE SILO.

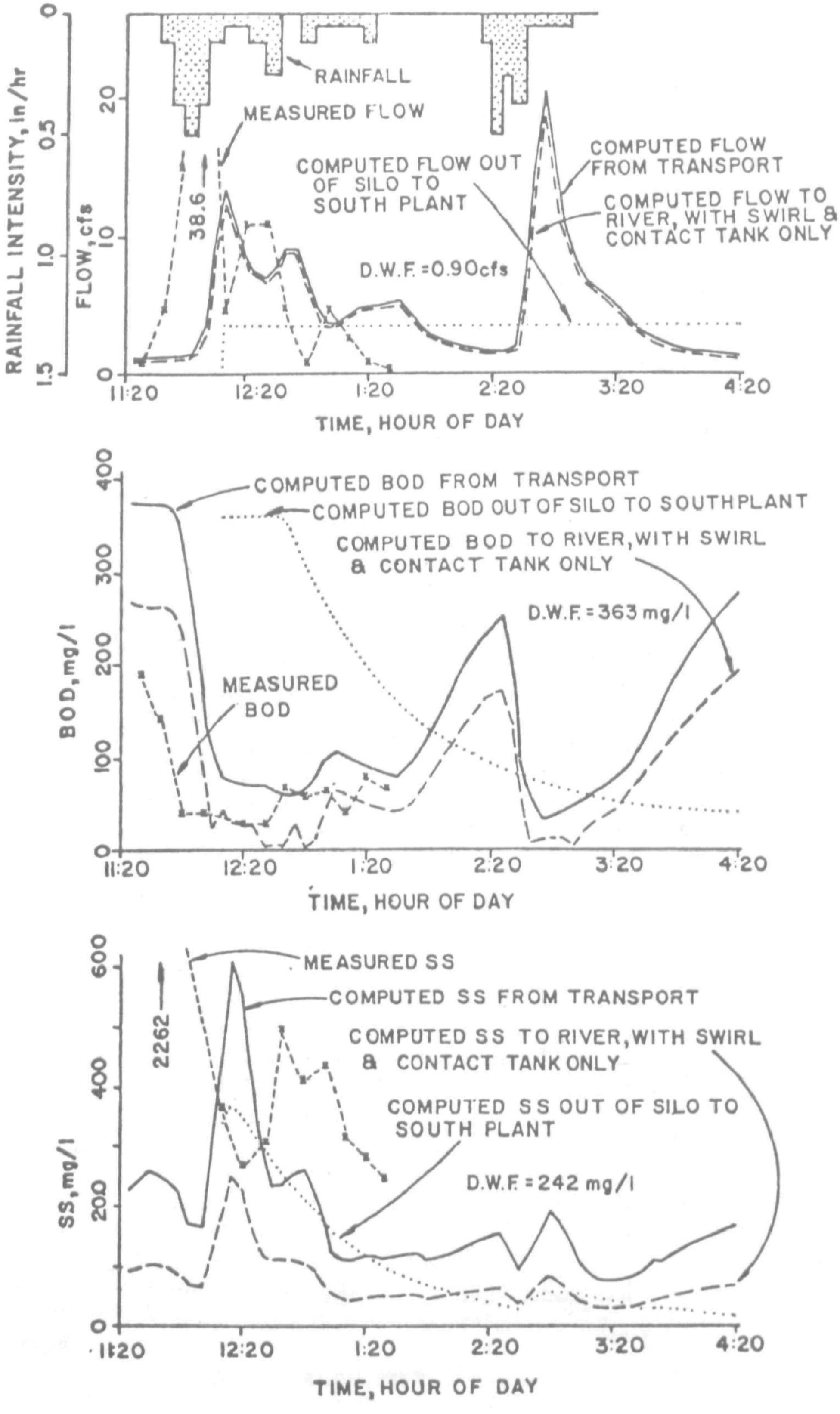


FIGURE 11. SIMULATION FOR STEVENS AVENUE, STUDY 3. HIGH CONCENTRATIONS OUT OF SILO REFLECT INTIAL FLUS OF DWF IN SEWER SYSTEM INTO THE SILO. FOR THIS STORM, SILO WOULD STORE ALL RUNOFF, BUT RESULTS WITHOUT IT ARE ALSO SHOWN.

can be tailored with his knowledge of the more sensitive parameters (impervious area, detention storage, catchbasin BOD content, infiltration and others) and available field measurements. The model is not fully calibrated until satisfactory results have been obtained with more than one storm within the study area. The application to Lancaster suffered from lack of good measured data for several of the storms studied, but it can be generally stated that for the Study No. 3 storm:

1) The SWMM was able to predict only fairly the quantity of the combined overflow for the Stevens Avenue district; however, the only set of measured flows are questionable.

2) Computed suspended solids were universally lower than measured values at the overflow and in the receiving water (Conestoga River) probably because erosion was not modeled and appropriate upstream concentrations were unknown.

3) The installation of the swirl concentrator and silo complex will result in substantial improvement in the quality of the overflow at Stevens Avenue, provided the full-scale performance of the swirl concentrator is comparable to the results obtained in laboratory studies by APWA. This fact should be true for any storm comparable to those described earlier in this report.

EXAMPLE: RECEIVING BLOCK

The Receiving Block of SWMM was applied to the lower St. Johns River for further developmental work on the model, and as part of a study of the waste assimilation capacity of the river. Since the Receiving Model is completely transient in both the hydrodynamic and water quality portions of the program, it is especially well suited for estuarine applications. Further details of the simulation discussed below are given by Huber and Heaney <u>et al.</u> (11).

The St. Johns River, from Palatka, Florida to its mouth at Mayport, a distance of about 75 miles (120 km) is presented as an example application of the Receiving Water Block. Figure 12 illustrates the layout of the 37 junctions and 40 channels used in the simulation. In addition, Figure 13 is a detailed diagram of the area of junctions 13, 14 and 15. All physical data were reduced from US Coast and Geodetic Survey nautical charts 685 and 636-SC and US Geological Survey

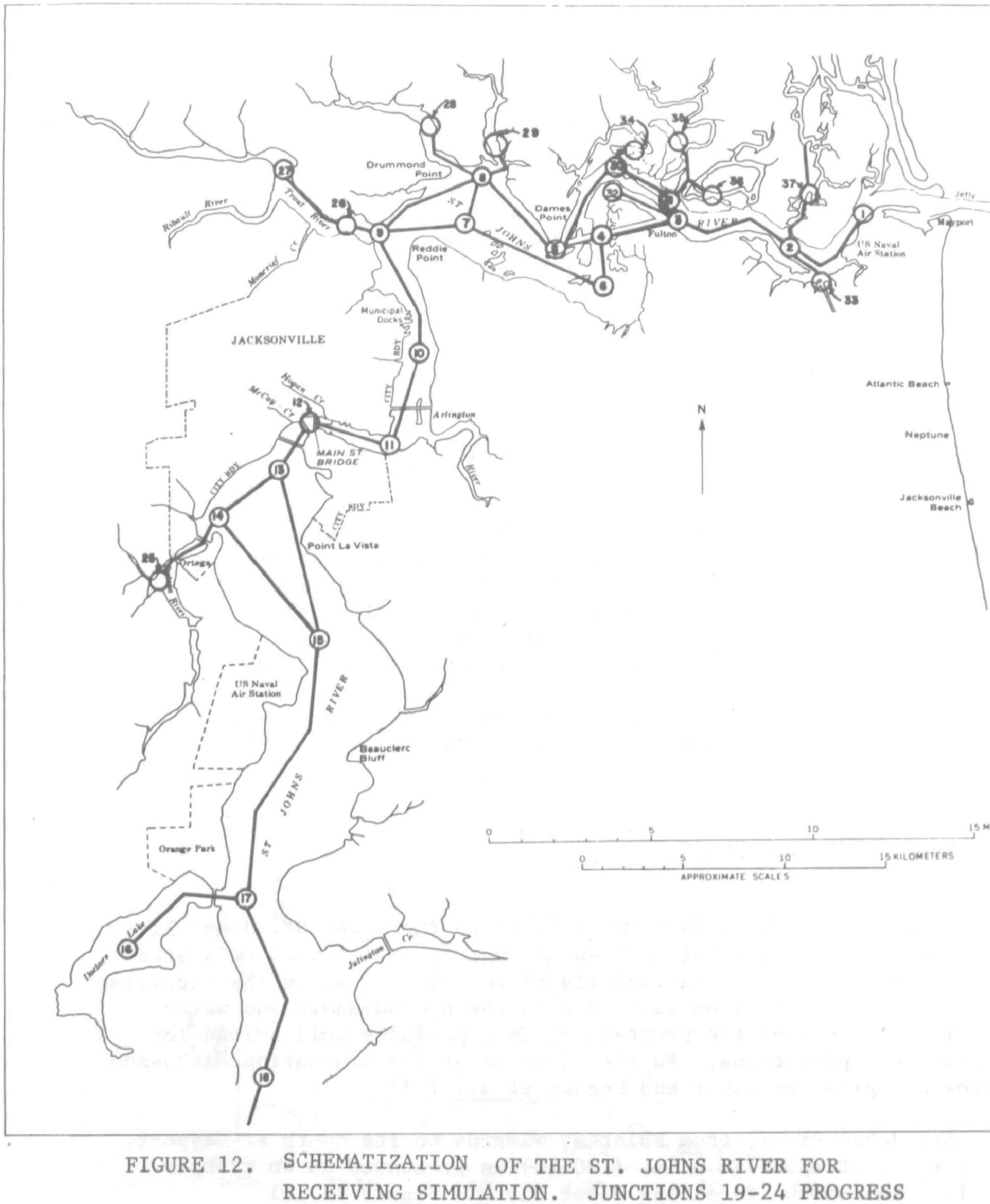


FIGURE 12. SCHEMATIZATION OF THE ST. JOHNS RIVER FOR RECEIVING SIMULATION. JUNCTIONS 19-24 PROGRESS UPSTREAM TO JUNCTION 24 AT PALATKA. JUNCTIONS 34-37 ARE USED TO SIMULATE STORAGE AVAILABLE IN TIDAL MARSHES. THE INTERACTION OF JUNCTIONS 33 AND 37 WITH THE INTRACOASTAL WATERWAY, ON WHICH THEY ARE LOCATED, IS NOT SIMULATED.

FIGURE 13. SCHEMATIZATION OF PORTION OF ST. JOHNS RIVER AT JACKSONVILLE, FLORIDA.

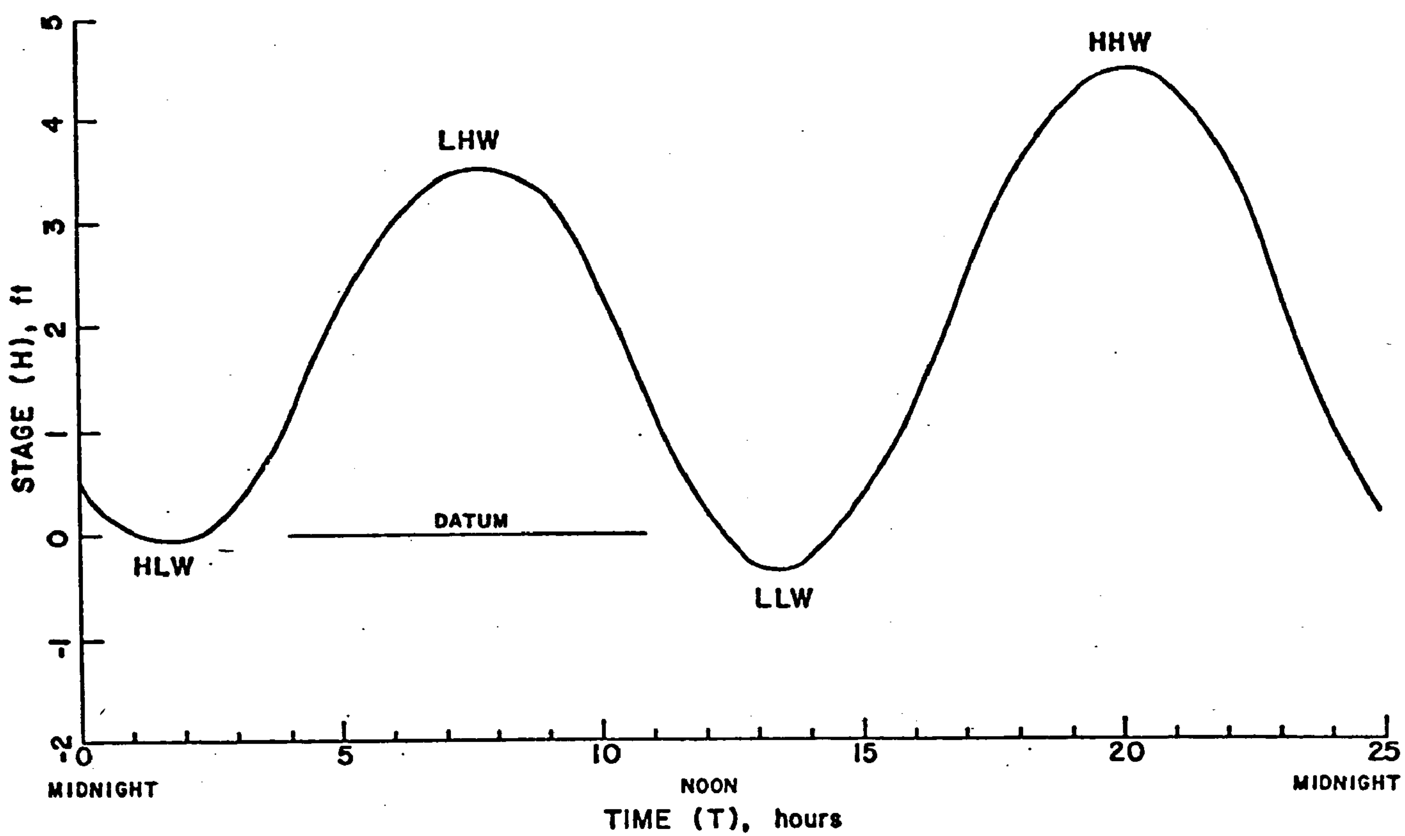


Figure 14. REPRESENTATIVE SEMI-DIURNAL TIDE AT MOUTH OF ST. JOHNS RIVER AT MAYPORT, FLORIDA, AUGUST 1, 1970. DATUM IS MEAN LOW WATER.

quadrangal maps for Mayport and Eastport, Florida. Note that channels represent meandering river segments and need not be straight lines. Lengths and areas are scaled off the maps.

Measured tides at junction 1 were input from the National Oceanographic and Atmospheric Administration tide gage at Mayport. Figure 14 illustrates the type of semi-diurnal tide experienced there. Measured transient river flows are entered at Palatka, junction 24.

Quality input data reflect the base dry weather loads entering the system at several junctions. The purpose of the run was to determine the effect of these loads rather than to simulate a specific stormwater runoff event. Thus, the only "stormwater" input is the river inflow at Palatka and is read in from cards. The BOD loading at Palatka is obtained through a known concentration measured there. This is then multiplied by the flow and converted to pounds per day.

The principal output of the quantity portion of the Receiving Model is stages at junctions and flows and velocities along channels. Samples are shown in Tables 1 and 2. The tidal influence may be clearly seen.

Representative quality output is shown in Table 3. In addition, maximum, minimum, and average concentrations are printed out after each simulated day. In this example, the predicted chloride concentrations compared favorably with measured values along the river, and are considered a useful verification parameter. Verification of predicted BOD and DO values is difficult on the St. Johns because of extensive benthal deposits and other unknown sources and sinks. Identification and quantification of sources, sinks and decay rates of water quality parameters is typically the most difficult single problem in modeling of receiving water quality.

SUMMARY

The EPA Storm Water Management Model (SWMM) is a useful tool for analysis of problems related to urban stormwater runoff. As such, it may be used for both the planning and design phases of investigations. The former utilizes time steps on the order of one hour, and simulation times on the order of years. The latter is oriented toward single event simulation and utilizes time steps on the order of minutes and simulation times on the order of hours.

Although only one of several urban runoff models currently available the SWMM is perhaps the most accessible and widely disseminated of any. It has found considerable acceptance in the engineering community and continues to be at the state-of-the-art by virtue of extensive use and continuous updating and maintenance.

TABLE 1. SAMPLE JUNCTION OUTPUT, DAY 2.

THIS IS A SAMPLE RUN OF QUANTITY AND QUALITY
LOWER ST. JOHNS RIVER. JULY 1972 VERIFICATION DATA.

E.P.A.
RECEIVING WATER HYDRODYNAMICS

JULY 19, 1972 TIDE AT MAYPORT.

USE MEASURED FLOW AT PALATKA, JULY 17-20, 1973.

DAY IS 2

* T I M E H I S T O R Y O F S T A G E *

HOUR	JUNCTION 13 HEAD(FEET)	JUNCTION 14 HEAD(FEET)	JUNCTION 15 HEAD(FEET)	JUNCTION 16 HEAD(FEET)	JUNCTION 17 HEAD(FEET)	JUNCTION 18 HEAD(FEET)
0.0	2.3870	2.4055	2.4838	2.5057	2.5014	2.5157
1.00	2.5760	2.5451	2.5070	2.4459	2.4565	2.4276
2.00	2.8500	2.8021	2.6619	2.4759	2.5040	2.4099
3.00	2.9800	2.9325	2.7755	2.7060	2.6870	2.6008
4.00	3.0152	2.9744	2.8634	2.8169	2.8156	2.7482
5.00	2.9758	2.9574	2.8760	2.8574	2.8538	2.8267
6.00	2.8279	2.8438	2.8438	2.8678	2.8609	2.8613
7.00	2.5274	2.5710	2.6881	2.8187	2.8032	2.8739
8.00	2.3271	2.3784	2.5193	2.6479	2.6498	2.7911
9.00	2.2593	2.3199	2.4932	2.5626	2.5777	2.6709
10.00	2.2356	2.3032	2.4678	2.5541	2.5358	2.5732
11.00	2.2334	2.2852	2.4256	2.4487	2.4638	2.5057
12.00	2.2613	2.2846	2.3708	2.4101	2.3943	2.4047
13.00	2.4469	2.4150	2.3847	2.3154	2.3356	2.3162
14.00	2.7508	2.6975	2.5463	2.3598	2.3850	2.2948
15.00	2.9404	2.8792	2.7002	2.6109	2.5972	2.4962
16.00	3.0506	2.9857	2.8194	2.7489	2.7469	2.6621
17.00	3.1183	3.0610	2.8964	2.8251	2.8297	2.7806
18.00	3.1139	3.0792	2.9536	2.9206	2.9112	2.8668
19.00	3.0036	3.0016	2.9604	2.9639	2.9666	2.9722
20.00	2.7577	2.7997	2.8903	3.0007	2.9873	3.0521
21.00	2.5531	2.6039	2.7437	2.9245	2.9044	3.0253
22.00	2.4791	2.5365	2.7014	2.7489	2.7747	2.8500
23.00	2.4331	2.4952	2.6362	2.7079	2.6985	2.7571
24.00	2.4174	2.4640	2.6026	2.6457	2.6486	2.6868
25.00	2.4562	2.4780	2.5626	2.5874	2.5833	2.6002

TABLE 2. SAMPLE CHANNEL OUTPUT, DAY 2.

THIS IS A SAMPLE RUN OF QUANTITY AND QUALITY
LOWER ST. JOHNS RIVER. JULY 1972 VERIFICATION DATA.

E.P.A.
RECEIVING WATER HYDRODYNAMICS

JULY 19, 1972 TIDE AT MAYPORT.

USE MEASURED FLOW AT PALATKA, JULY 17-20, 1973.

DAY IS 2

* * * * * * * * * * * * * * * T I M E H I S T O R Y O F F L O W A N D V E L O C I T Y * * * * * * * * * * * * * * *

HOUR	CHANNEL 17 18 FLOW (CFS)	CHANNEL 17 18 VEL. (FPS)	CHANNEL 16 17 FLOW (CFS)	CHANNEL 16 17 VEL. (FPS)	CHANNEL 15 17 FLOW (CFS)	CHANNEL 15 17 VEL. (FPS)	CHANNEL 14 15 FLOW (CFS)	CHANNEL 14 15 VEL. (FPS)	CHANNEL 13 15 FLOW (CFS)	CHANNEL 13 15 VEL. (FPS)	CHANNEL 13 14 FLOW (CFS)	CHANNEL 13 14 VEL. (FPS)
0.0	-89679.	-0.41	1468.	0.03	-98874.	-0.45	-55147.	-0.73	-46072.	-0.87	-50755.	-0.55
1.00	-62744.	-0.29	2576.	0.04	-68050.	-0.29	-34465.	-0.45	-25872.	-0.48	-23718.	-0.24
2.00	-12442.	-0.05	-5707.	-0.10	10990.	0.05	13238.	0.18	19316.	0.37	28020.	0.29
3.00	66004.	0.30	-10622.	-0.18	96648.	0.42	53843.	0.70	49795.	0.92	57519.	0.58
4.00	97772.	0.44	-1163.	-0.02	108359.	0.46	62315.	0.80	53800.	0.99	62529.	0.62
5.00	98663.	0.44	-1908.	-0.03	104434.	0.45	55320.	0.71	46660.	0.85	54162.	0.54
6.00	83155.	0.37	1157.	0.02	80346.	0.34	41431.	0.53	31956.	0.58	29978.	0.30
7.00	43522.	0.19	3406.	0.06	25908.	0.11	5150.	0.06	-2490.	-0.06	-12253.	-0.13
8.00	-46285.	-0.22	9892.	0.17	-72550.	-0.32	-38489.	-0.52	-38937.	-0.75	-47464.	-0.51
9.00	-110905.	-0.51	-3026.	-0.05	-114867.	-0.50	-62442.	-0.83	-55028.	-1.05	-61980.	-0.66
10.00	-111318.	-0.51	5637.	0.10	-123407.	-0.54	-66957.	-0.89	-58296.	-1.11	-68548.	-0.73
11.00	-102510.	-0.47	1355.	0.02	-112947.	-0.49	-64389.	-0.86	-55724.	-1.07	-65859.	-0.70
12.00	-88237.	-0.41	4140.	0.07	-100800.	-0.44	-55957.	-0.75	-47329.	-0.90	-53805.	-0.57
13.00	-63788.	-0.30	1619.	0.03	-70098.	-0.31	-36803.	-0.49	-27848.	-0.52	-25276.	-0.26
14.00	-18427.	-0.08	-5087.	-0.09	6894.	0.04	13516.	0.19	20039.	0.39	30049.	0.31
15.00	64243.	0.30	-13506.	-0.23	99831.	0.43	56565.	0.74	53056.	0.98	64141.	0.65
16.00	106640.	0.49	-376.	-0.01	120882.	0.52	70896.	0.91	62166.	1.14	75362.	0.75
17.00	112007.	0.51	-6111.	-0.10	129259.	0.55	71881.	0.92	62270.	1.13	76451.	0.76
18.00	110936.	0.50	-1274.	-0.02	121340.	0.52	68728.	0.87	57726.	1.05	66514.	0.66
19.00	92776.	0.41	-3335.	-0.06	100955.	0.43	53806.	0.68	43632.	0.79	47386.	0.47
20.00	49501.	0.22	1085.	0.02	47234.	0.20	22505.	0.28	13758.	0.24	7728.	0.07
21.00	-30173.	-0.14	5601.	0.10	-54584.	-0.24	-31166.	-0.41	-33544.	-0.63	-40224.	-0.42
22.00	-90876.	-0.41	5955.	0.10	-109774.	-0.47	-61414.	-0.81	-54575.	-1.03	-61051.	-0.64
23.00	-108929.	-0.50	653.	0.01	-115376.	-0.50	-64167.	-0.84	-56230.	-1.06	-67721.	-0.71
24.00	-103139.	-0.47	3528.	0.06	-114135.	-0.49	-63269.	-0.83	-54710.	-1.03	-64784.	-0.68
25.00	-91722.	-0.42	2111.	0.04	-102187.	-0.44	-57094.	-0.75	-47810.	-0.90	-53167.	-0.56

TABLE 3. QUALITY OUTPUT DURING DAY 2.

THIS IS A SAMPLE RUN OF QUANTITY AND QUALITY
LOWER ST. JOHNS RIVER. JULY 1972 VERIFICATION DATA.

EPA STORMWATER MODEL
RECEIVING WATER QUALITY

JULY 19, 1972 TIDE AT MAYPORT.

USE MEASURED FLOW AT PALATKA, JULY 17-20, 1973.

JUNCTION CONCENTRATIONS, MGL (OR MPN/L), DURING TIME CYCLE (DAY) 2, QUALITY CYCLE 2, TIME (SEC) 7200.

CONSTITUENT NUMBER 1 BOD

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	0.02	0.23	0.51	0.60	0.73	0.64	0.89	0.88	0.96	1.06
11 TO 20	0.98	1.02	0.81	0.76	0.70	0.64	0.64	0.64	0.64	0.64
21 TO 30	0.64	0.64	0.64	0.79	0.85	1.29	0.79	2.46	0.78	0.63
31 TO 37	0.56	0.62	0.48	0.64	0.60	0.59	0.39			

CONSTITUENT NUMBER 2 CHLORIDES (MGL)

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	17732.35	14450.17	9961.09	8538.69	6707.11	7892.46	5075.64	5342.50	4382.59	3751.88
11 TO 20	2288.86	1915.75	1461.57	1130.53	873.14	347.06	315.04	218.02	226.80	231.20
21 TO 30	226.76	191.51	186.86	202.22	793.53	3538.49	2375.81	4959.41	4964.31	8078.95
31 TO 37	8980.00	6850.09	11611.25	7791.27	7662.71	8337.59	11875.26			

CONSTITUENT NUMBER 3 BOD (DO)

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	6.42	6.01	5.38	5.27	5.13	5.32	5.10	5.03	4.98	4.95
11 TO 20	4.99	5.04	5.14	5.18	5.13	5.05	5.04	5.07	5.08	5.08
21 TO 30	5.09	5.12	5.11	4.70	5.32	5.00	5.18	5.06	5.24	5.23
31 TO 37	5.30	5.01	5.43	5.30	5.24	5.31	5.57			

COMPUTED OXYGEN SATURATION VALUES(MGL) AT THIS TIME STEP

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	6.42	6.67	7.01	7.12	7.26	7.17	7.38	7.36	7.44	7.48
11 TO 20	7.60	7.62	7.66	7.68	7.70	7.74	7.75	7.75	7.75	7.75
21 TO 30	7.75	7.76	7.76	7.75	7.71	7.50	7.58	7.39	7.39	7.16
31 TO 37	7.09	7.25	6.89	7.18	7.19	7.14	6.87			

COMPUTED REAERATION COEFFICIENTS (1/DAY) AT THIS TIME STEP

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	0.10	0.08	0.08	0.14	0.09	0.65	0.46	0.13	0.11	0.08
11 TO 20	0.07	0.08	0.13	0.17	0.12	0.14	0.12	0.13	0.13	0.14
21 TO 30	0.15	0.16	0.16	0.16	0.37	0.18	0.19	0.31	0.27	0.11
31 TO 37	0.12	0.11	0.14	0.57	0.19	0.27	0.25			

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HYDROLOGY REVIEW

By

Donald Dean Adrian*

INTRODUCTION

A brief discussion of a few hydrologic principles will be presented. The review will consist of a discussion of precipitation, its measurement and description in graph form, its distribution in time and space; infiltration and infiltration capacity; and runoff.

PRECIPITATION

Precipitation is measured by raingages which may be recording or nonrecording. The nonrecording type measures the total amount of precipitation between readings. It is being replaced by the recording raingage which record the rate of precipitation and the time of its occurrence.

Ordinary raingages may be modified with shields to record snowfall. However, due to the snow being carried by the wind inaccuracies in measurement may be introduced. Because snowmelt is not converted immediately into runoff, snowfall has received little attention in the SWMM.

Rainfall records are usually presented in the form of bar graphs called hyetographs. The rainfall intensity serves as the ordinate and time is the abscissa.

Rainfall records are frequently analyzed for intensity-duration relationships. The average rainfall intensity in inches per hour decreases as the duration increases. For example, in Los Angeles, at an average of once each five years the rainfall intensity will be 3.2 in/hr for a five minute duration, 1.6 in/hr for a 20 minute duration, and 0.85 in/hr for a 60 min duration.

If multiple raingages are available their records for any storm will be different, with each raingage being most representative of the area in its immediate vicinity. Rainfall is allocated to the surrounding area by the Theissen Polygon Method. Refer to Figure 1 for the construction of the Theissen Polygon for the area shown. One has three raingages 1, 2, and 3. Straight lines are drawn connecting the raingages, then the perpendicular bisector of each line is drawn. The perpendicular bisectors serve as the boundaries for the area of influence of each raingage. Thus raingage 1 serves area A_1, raingage 2 serves area A_2, and raingage 3 serves area A_3. The method may be extended to define the area served by more raingages.

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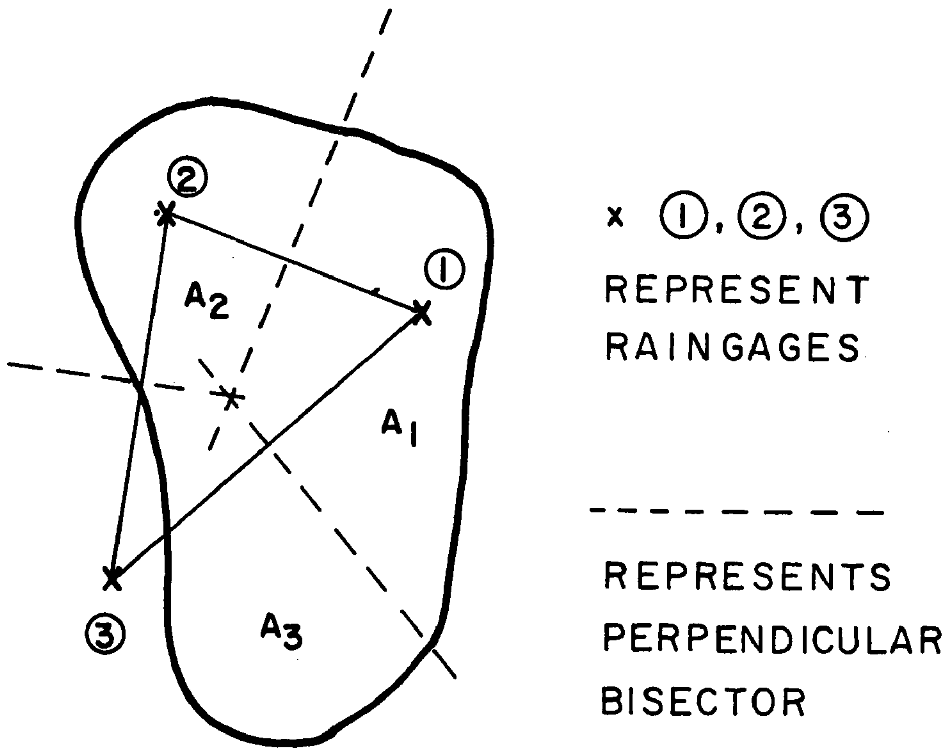

Figure 1. Construction of a Theissen Polygon

INFILTRATION

Infiltration is the rate of movement of water into the ground. In hydrology "infiltration" is used to express the notion that some precipitation does not run off, but soaks into the ground. In environmental engineering "infiltration" is also used to describe the movement of water out of the ground and into sewers through cracks, poorly sealed joints, and other openings. The reader is cautioned to note the distinction between these two usages.

Infiltration rate (in the hydrologic sense) is important in studying runoff because if the rainfall rate is less than the infiltration rate no runoff will occur. The maximum infiltration rate is called the infiltration capacity. It is a function of time and has been expressed in equation form by Horton's equation

$$f_p = f_c + (f_o - f_c)e^{-kt_R}$$

where f_p is the infiltration capacity,

f_c is a constant rate of infiltration observed at the end of a long storm

f_o is the maximum rate of infiltration at the beginning of a storm,

k is a positive constant, and

t_R is the duration of rainfall.

OVERLAND FLOW AND SURFACE RUNOFF

Runoff starts when the precipitation rate exceeds the infiltration capacity and depression storage has been filled. Runoff first flows in trickles and rivulets until it joins larger streams and makes its way to a gutter or channel. Water which is enroute to a channel is known as overland flow. It is known as surface runoff after it enters a flow channel.

Water is stored temporarily while it is engaged in overland flow. The temporary storage results because a certain depth of water builds up as overland flow takes place. Such temporary storage is known as surface depression storage. Depression storage represents water which does not engaged in overland flow, such as water filling a pothole in a street.

It has been difficult to describe in equation form the behavior of overland flow. Difficulties arise because of the complex flow paths which the water follows as it travels to become surface runoff.

HYDRAULICS REVIEW

By

Donald Dean Adrian*

BASIC CONCEPTS

Open channel flow is flow in a conveyance structure which allows the upper surface of the water to be in contact with the atmosphere. An example is flow in an irrigation canal. Another example is flow in a stream. A third example is flow in a partially filled sewer. Gravity is the driving force inducing flow in an open channel.

Pipe flow, or sometimes pressure flow, is flow in a conveyance structure, such as a pipe, in which the water completely fills the pipe. An example of pipe flow, or pressure flow, is flow in a water main. Another example of pipe flow or pressure flow is flow in a surcharged sewer, i.e., a sewer which is completely filled during a storm event.

Unsteady flow is flow which changes with time. An example is the flow in a storm sewer during the initial period of a rainfall event.

Steady flow is flow which does not change with time. An example might be the flow in a force main. (A force main is a sewer which is designed to always operate under pressure, such as a sewer which leads from a pumping station located in a low part of town and discharges to a treatment plant located at a higher elevation.)

Uniform flow is flow in which the velocity stays the same in magnitude and direction throughout the whole of the fluid. Normally one can think of uniform flow as being open channel flow in which the depth remains constant along the channel length of interest.

Non-uniform flow is flow in which the velocity changes in magnitude and direction over the channel length of interest. Again, it is easier to visualize non-uniform flow as open channel flow which changes in depth along the channel length of interest. Sometimes it is advantageous for computational purposes to treat a non-uniform flow as a uniform flow. For example, a long combined sewer may have a significant depth of flow change along its length, thus making the flow non uniform, but if the length of the combined sewer is visualized as being made up of several shorter segments, say 10, the flow in each segment can be approximated as being uniform flow.

Discharge rate or sometimes the discharge, is the volumetric flowrate, usually measured in units such as million gallons per day, cubic feet per second, liters per second, and cubic meters per second.

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CONTINUITY

The principle of conservation of mass, which states that the rate of mass inflow minus the rate of mass outflow is equal to the change in storage in a control volume, leads to a useful concept in hydraulics, the equation of continuity.

The continuity equation is most readily visualized for steady flow. It states that the discharge, Q, is equal to the cross sectional area through which flow takes place, A, multiplied by the flow velocity, V. Thus,

$$Q = AV \tag{1}$$

where A, V, and Q may be measured anywhere along a channel. At the junction of two channels which combine to form a single channel the equation of continuity states

$$Q_1 + Q_2 = Q_3 \tag{2}$$

where Q_1 = discharge of the first upstream channel

Q_2 = discharge of the second upstream channel

Q_3 = discharge of the third channel, the downstream channel.

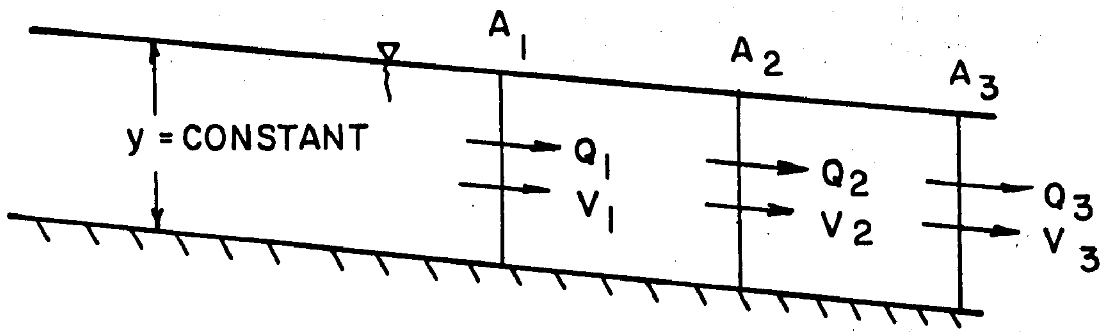
Along a single channel the equation of continuity may be written

$$A_1V_1 = A_2V_2 = A_3V_3 \tag{3}$$

where the subscripts now refer to different locations along the channel. Figure 1 summarizes the continuity relationships for steady flow.

The depth of flow, h, and the cross sectional flow area, A, are functions of X for steady nonuniform flow. Thus $h = h(X)$, $A = A(X)$, and of necessity $V = V(X)$ since $V = Q/A(X)$. Any differential changes in h, A or V will then be expressed as ordinary derivatives. As the equation of continuity states $Q = AV$ one can take the derivative to obtain a differential form of the equation of continuity for steady non-uniform flow

$$\frac{dQ}{dX} = \frac{d}{dX}(AV) \text{ or } 0 = V\frac{dA}{dX} + A\frac{dV}{dX} \tag{4}$$



FOR STEADY FLOW: $Q_1 = Q_2 = Q_3$ AND $A_1V_1 = A_2V_2 = A_3V_3$

FOR STEADY UNIFORM FLOW: $y =$ CONSTANT AND $A_1 = A_2 = A_3$

$\therefore V_1 = V_2 = V_3$

FOR STEADY NONUNIFORM FLOW: $A_1 \neq A_2 \neq A_3$

$V_1 \neq V_2 \neq V_3$

BUT $A_1V_1 = A_2V_2 = A_3V_3$

Figure 1. Equation of Continuity

For "unsteady" flow the continuity principle is not so readily apparent as for steady flow. If one considers a reach of length ΔX the discharge Q may be different at either end of the reach. Thus Q can be changing with distance. Also, since the flow is unsteady Q may be a function of time. Thus $Q = Q(x,t)$ and any differential changes in Q have to be expressed as partial derivatives.

The flowrates Q_1 and Q_2 may be expressed in terms of $Q(X,t)$, which is shown in Figure 2 at the center of the control volume. By use of a Taylor's Series, truncated after the second term,

$$Q_1 = Q(X, t) - \frac{\partial Q}{\partial X}\frac{\Delta X}{2} \tag{5}$$

$$Q_2 = Q(X,t) + \frac{\partial Q}{\partial X}\frac{\Delta X}{2} \tag{6}$$

The change in storage inside the control volume is $\Delta X \cdot B \frac{\partial h}{\partial t}$ where B is the water surface width. From the conservation of mass principle that the rate of inflow minus the rate of outflow is equal to the rate of change of storage, one can write

$$Q_1 - Q_2 = B \frac{\partial h}{\partial t} \Delta X \tag{7}$$

or, writing Q for $Q(X,t)$,

$$Q - \frac{\partial Q}{\partial X}\frac{\Delta X}{2} - \left(Q + \frac{\partial Q}{\partial X}\frac{\Delta X}{2}\right) = B \frac{\partial h}{\partial t} \Delta X \tag{8}$$

which becomes

$$- \frac{\partial Q}{\partial X} \Delta X = B \frac{\partial h}{\partial t} \Delta X \tag{9}$$

and this expression simplifies to

$$\frac{\partial Q}{\partial X} + B \frac{\partial h}{\partial t} = 0 \tag{10}$$

Sometimes it is useful to replace Q by its equivalent AV to obtain

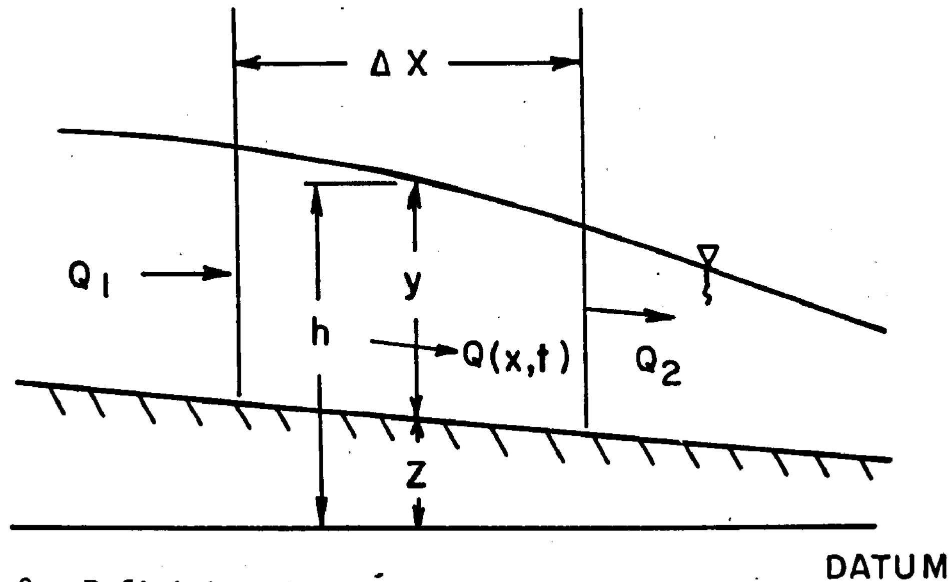


Figure 2. Definition Sketch for Equation of Continuity for Unsteady Nonuniform Flow

$$V \frac{\partial A}{\partial X} + A \frac{\partial V}{\partial X} + B \frac{\partial h}{\partial t} = 0 \qquad (11)$$

as the equation of continuity for unsteady non-uniform flow.

THE ENERGY PRINCIPLE

The energy content of a fluid is expressed in terms of the "head" in hydraulics usage. The energy content is measured in ft-lb units, and by expressing it on a per lb basis one obtains the energy content per lb with the units of ft. Thus, the head available is expressed in terms of ft.

The Bernoulli Equation

The Bernoulli equation

$$\frac{p_1}{\gamma} + Z_1 + \frac{V_1^2}{2g} = \frac{p_2}{\gamma} + Z_2 + \frac{V_2^2}{2g} + h_L \qquad (12)$$

expresses the head relationship between two points, 1 and 2, along a pipe or open channel. In the above expression each of the terms p/γ, Z, $V^2/2g$ and h_L have dimensions of feet. The terms represent:

p = pressure
γ = specific weight of the fluid
Z = elevation above a reference datum
h_L = head loss between points 1 and 2
(h_L represents frictional dissipation).

For steady nonuniform flow one can examine the change in the total head, H, as a function of distance. Let

$$H = p/\gamma + Z + V^2/2g \tag{13}$$

and note that

$$\frac{dH}{dX} = - S_f \quad \text{where } S_f = \text{frictional slope} = \frac{dh_L}{dX} \tag{14}$$

Then (see Figure 3)

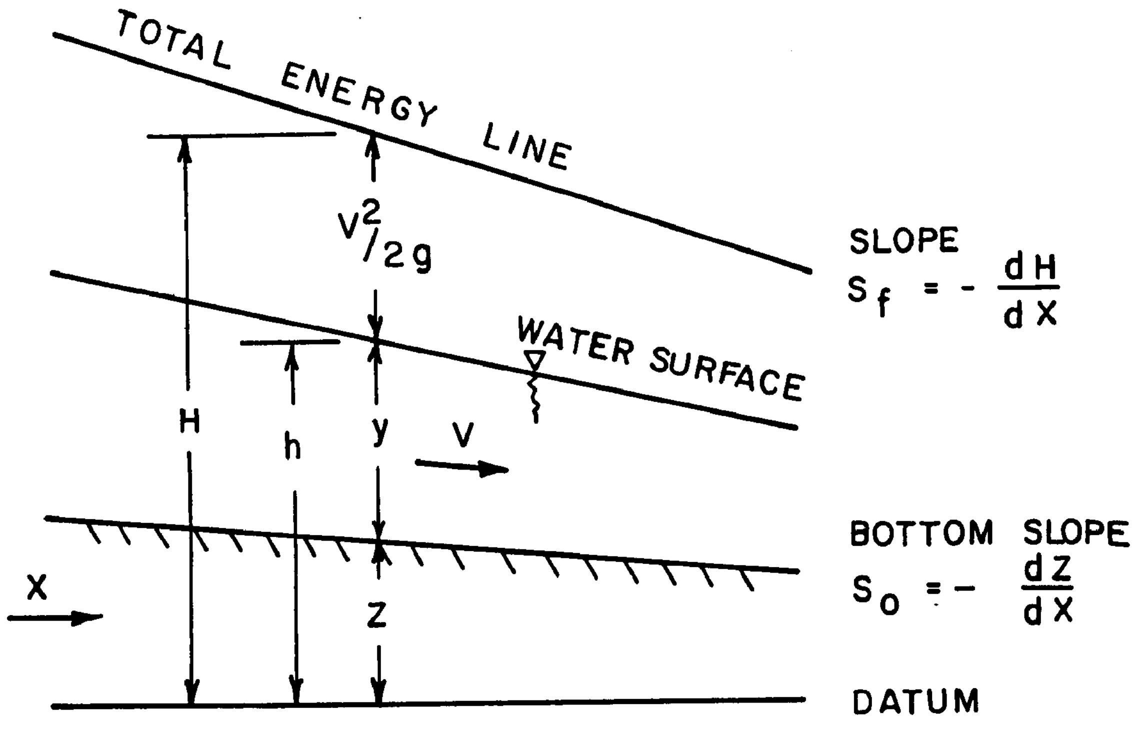


Figure 3. Definition Sketch for Steady Nonuniform Flow

$$\frac{dH}{dX} = \frac{d}{dX}\left(Z + y + \frac{V^2}{2g}\right) = - S_f \tag{15}$$

or

$$\frac{d}{dX}\left(y + \frac{V^2}{2g}\right) = - \frac{dZ}{dX} - S_f = S_o - S_f \tag{16}$$

Letting V = Q/BY and differentiating yields

$$\frac{dy}{dX} - \frac{Q^2}{gB^2Y^2}\frac{dy}{dX} = S_o - S_f \tag{17}$$

which becomes, in terms of the Froude number

$$\frac{dy}{dX}\left(1 - F_f^{\,2}\right) = S_o - S_f \tag{18}$$

where $F_r = \frac{V}{\sqrt{gy}}$, or $F_r = \frac{Q}{By\sqrt{gy}}$.

The above relationship is widely used in developing water surface profiles for non-uniform steady flow.

Critical Depth, Subcritical and Supercritical Flow

A gravity wave will travel through water in an open channel with a velocity given by the expression

$$C = \sqrt{gy} \tag{19}$$

where c = wave velocity
g = acceleration of gravity
y = depth of the water.

The above expression for wave velocity divides open channel flow regime into two parts. By substituting for the wave velocity c = Q/A, and expressing for a rectangular open channel $A = B\ Y_c$, one obtains

$$Y_c = \sqrt[3]{\frac{Q^2}{B^2 g}} \qquad (20)$$

as the expression for the critical depth of the channel.

If the depth of flow in the channel is greater than Y_c the flow is called subcritical. If the depth is less than Y_c the flow is called supercritical. In subcritical flow waves can travel faster than the stream velocity, and are transmitted both upstream and downstream. In supercritical flow waves travel slower than the stream velocity, and are transmitted only downstream. The above property of waves is important in devising numerical flow routing schemes.

A useful indicator of whether the flow regime is subcritical, critical, or supercritical is the Froude number, F_r. It is defined as

$$F_r = V/\sqrt{gy} \quad \text{or } F_r = \frac{Q}{By\sqrt{gy}} \qquad (21)$$

where V is the actual stream velocity. If $F_r < 1$ the flow is subcritical. If $F_r = 1$ the flow is at critical depth, and if $F_r > 1$ the flow is supercritical.

THE MOMENTUM PRINCIPLE

The momentum principle is widely used in hydraulics. In applications one applies the principle by summing all the forces acting on a control volume of water. These forces usually are: hydrostatic pressure forces, momentum flux forces (usually these are expressed as $Q\rho V$ where Q is the flowrate, ρ is the specific density, and V is the stream velocity), frictional resistance forces, and a component of the gravity force in the direction of flow.

An application of the momentum principle is in the hydraulic jump which is a violent transition from supercritical to subcritical flow. If one lets y_1 and y_2 represent the depth of flow before and after a hydraulic jump, and F_{r1} and F_{r2} represent the before jump and after jump Froude number, the momentum principle yields

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1 + 8F_{r1}^{2}} - 1\right) \qquad (22)$$

which is useful when upstream conditions are known and the depth y_2 is to be solved for. An alternative form which will give the upstream depth if the downstream conditions are known is

$$\frac{y_1}{y_2} = \frac{1}{2} \left(\sqrt{1 + 8F_{r_2}^{\ 2}} - 1\right) \tag{23}$$

FLOW RESISTANCE

The resistance of a channel or a pipe to flow results in a relationship between the flow velocity and channel properties. Over the years a number of equations describing the relationship have been developed. One of the most widely used is the Manning equation. It is written

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{24}$$

where V = the flow velocity
n = a coefficient dependent upon the channel roughness
S = slope of the energy grade line
R = hydraulic radius.

Values of n are tabulated for different materials, for example, $n = 0.014$ for unfinished concrete and $n = 0.025$ to 0.030 for clean straight river channels. S is the slope of the energy grade line (the energy grade line is given from the Bernoulli equation as the sum of $p/\gamma + Z + \frac{V^2}{2g}$) for the channel or pipe. S may be calculated from the Bernoulli equation as h_L/L where h_L is the head loss between two points a distance L apart. R is defined as the ratio of the cross sectional area through which flow takes place, A, to the wetted perimeter, Pw. The wetted perimeter is the length of the contact surface between the flowing liquid and the solid boundary. For a pipe flowing full the wetted perimeter is the pipe circumference $2\pi r$, the cross sectional area is πr^2, resulting in a hydraulic radius of $R = r/2$. If the pipe were flowing half full $P_w = \pi r$, $A = \frac{1}{2}\pi r^2$ and $R = r/2$ again. For other values of flow depth $R = r/2$, but must be calculated or looked up in a table.

The Manning equation is applicable to steady uniform flow, but is frequently used for unsteady and non-uniform flows. For steady uniform flow the value of S may be taken as the slope of the energy grade line or the slope of the channel bottom. For non-uniform flow the slope S for use in the Manning equation is the slope of the energy grade line. Sometimes subscripts are used to differentiate between the two slopes, S_o representing the slope of the channel bottom and S_f representing the frictional slope, or the slope of the energy grade line.

CHANNEL CONTROLS

Certain structures in an open channel serve to establish a unique relationship between the head available and the flowrate. Such structures are called controls. They are often encountered in calculations because they serve as boundary conditions at upstream or downstream reaches of a channel. Two types of control will be discussed here, namely the sharp crested weir and the free overfall. See Figure 4 for sketches of these structures.

The flowrate over a sharp crested weir is given by the relation

$$Q = \frac{2}{3} C_D B \sqrt{2g}\ H^{2/3} \tag{25}$$

where C_D is a discharge coefficient and all other terms have been previously defined. The discharge coefficient varies with the ratio H/W and has a value of 0.611 when W is large. A relationship for other values of the ratio is

$$C_D = 0.611 + 0.08\ H/W \tag{26}$$

The free overflow is another type of control which may occur in practice when a sewer discharges into a river or a drop structure. Critical depth, Y_c, occurs slightly upstream from the brink of the free overfall, establishing a unique relationship between the depth and the flowrate. From the critical depth relation

$$Y_c = \sqrt[3]{\frac{Q^2}{B^2 g}} \tag{27}$$

the depth Y_c can be solved for if Q is known (if B is a function of depth a trial and error solution may be necessary), or if y at the overfall is known Q can be solved for. For some calculational purposes, especially when A and B are functions of depth, another form of the critical depth relation may be easier to use. It is

$$\frac{Q^2 B_c}{g\ A_c^3} = 1 \tag{28}$$

where the subscripts indicate that the relation is applicable only at critical depth.

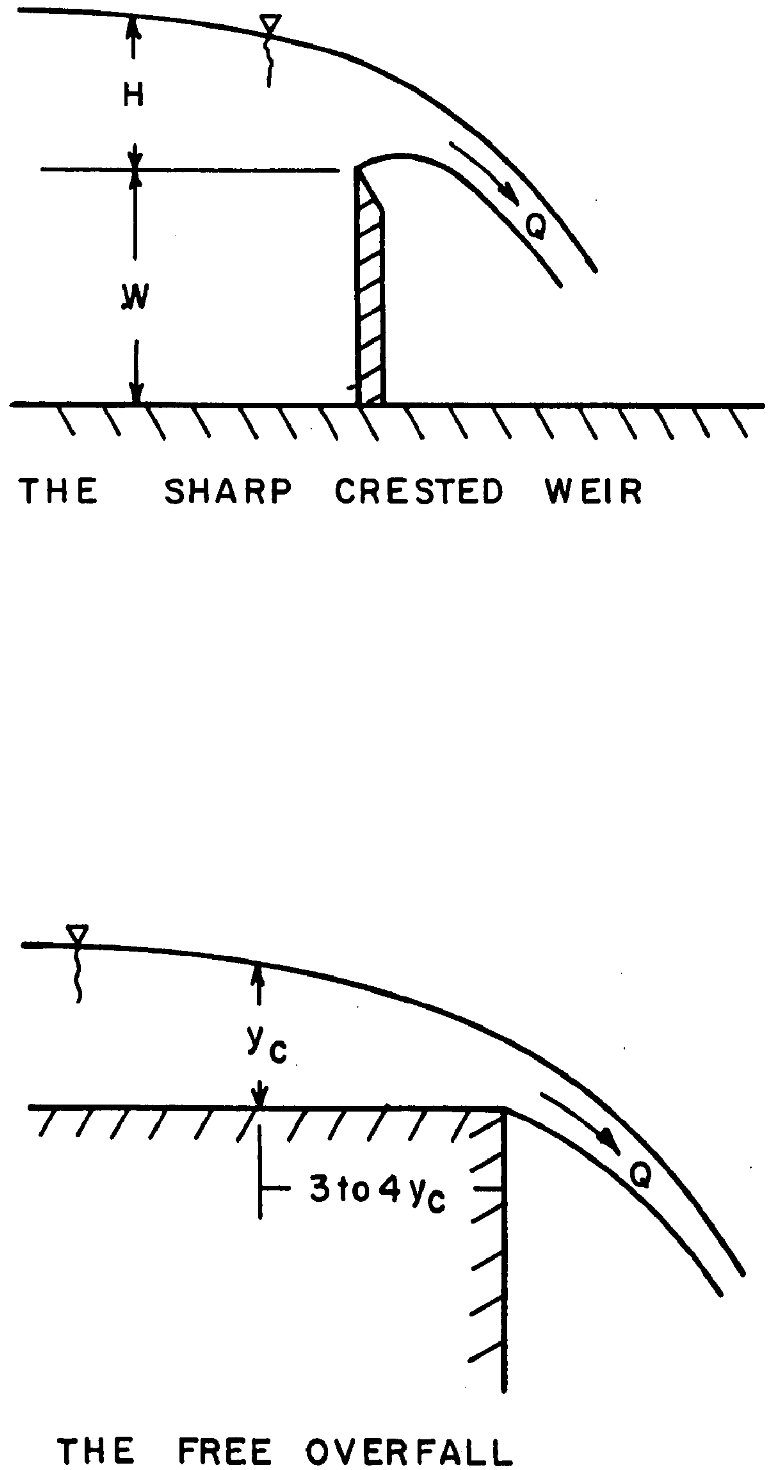


Figure 4. Two Channel Controls

UNSTEADY NON-UNIFORM FLOW

Unsteady non-uniform flow occurs frequently in a st orm or combined sewer system as the storm event continuously changes the flowrate the sewer has to carry. Two basic relations are necessary to develop equations to describe unsteady non-uniform flow: a dynamic equation of motion (that is one which considers the acceleration of the fluid) and an equation of continuity.

The equation of continuity for unsteady non-uniform flow was derived as

$$\frac{\partial Q}{\partial X} + B\,\frac{\partial h}{\partial t} = 0 \qquad (10)$$

or

$$V\,\frac{\partial A}{\partial X} + A\,\frac{\partial V}{\partial X} + B\,\frac{\partial h}{\partial t} = 0 \qquad (11)$$

Refer to Figure 5 in the derivation of the equation of motion for unsteady nonuniform flow. The slope of the total energy line is

$\frac{dH}{dX} = S_f$ and $\frac{\partial H}{\partial X} = \frac{\partial}{\partial X}\,(h + \frac{V^2}{2g})$. Recall that for <u>steady</u> nonuniform flow $\frac{\partial H}{\partial X} = - S_f$. The frictional slope S_f is expressed in terms of a flow equation, such as the Manning equation

$$V = \frac{1.486}{n}\,R^{2/3}\,S_f^{\;1/2} \qquad (24)$$

which can be changed around to yield

$$S_f = \frac{V^2\;n^2}{2.208\;R^{4/3}} \qquad (29)$$

An alternate form of expression for the friction slope is obtained by balancing the boundary shear with the gravity force leading to $\tau_o = \gamma RS_f$.

The energy available to the flow expressed by the slope of the total energy line, $\partial H/\partial X$, can be used to overcome frictional resistance, expressed by S_f, and the excess energy will be available for accelerating the flow.

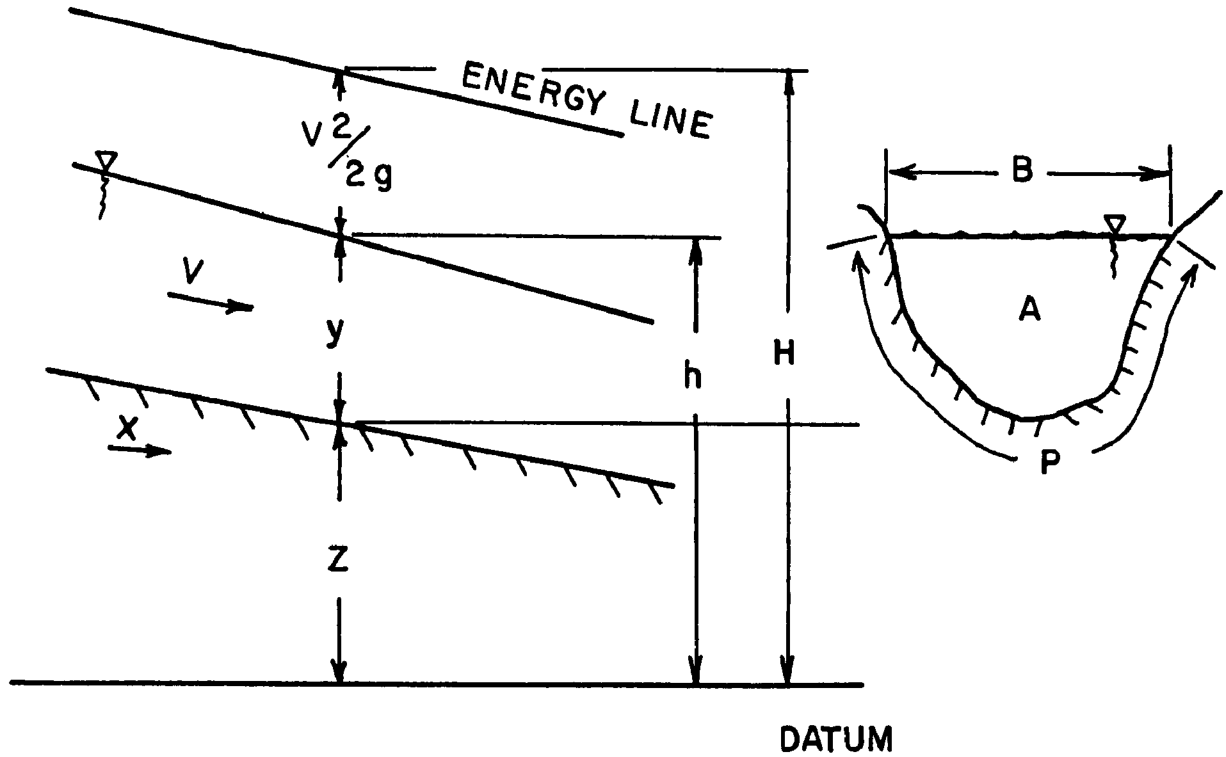


Figure 5. Definition Sketch for Unsteady Nonuniform Flow

One can express the above concept in equation form by stating

$$S_e = -(S_f + S_a) \tag{30}$$

where S_e is the slope of the total energy line and S_a is the acceleration slope. Evaluating the forces acting on a control volume of liquid of length ΔX in Figure 5, i.e., evaluating the pressure, resistance, and gravity forces; results in an unbalanced force of

$$-\gamma A\Delta h - \tau_o P\Delta X \tag{31}$$

which is available to accelerate the mass of fluid in the control volume where the mass is $\rho A\Delta X$. τ_o is the boundary shear resistance and P is the wetted perimeter. Newton's law of motion F = ma can be applied to obtain

$$\gamma A\Delta h - \tau_o P\Delta X = \rho A\ \Delta X\ a_x \tag{32}$$

where a_x is the acceleration. The accelration $a_x = \frac{dV}{dt}$ and by noting that $V = V(x,t)$ and using the chain rule

$$\frac{dV}{dt} = \frac{\partial V}{\partial X}\frac{dX}{dt} + \frac{\partial V}{\partial t} \tag{33}$$

then noting that $\frac{dX}{dt} = V$ the acceleration is

$$\frac{dV}{dt} = V \frac{\partial V}{\partial X} + \frac{\partial V}{\partial t} \tag{34}$$

Then Newton's equation becomes

$$\frac{\tau_o}{\gamma R} = - \left(\frac{\partial h}{\partial X} + \frac{V}{g} \frac{\partial V}{\partial X} + \frac{1}{g} \frac{\partial V}{\partial t}\right) \tag{35}$$

after using the relation $\gamma = \rho g$, and $R = A/P$. Now $\frac{\partial H}{\partial X} = \frac{\partial}{\partial X} (h + \frac{V^2}{2g})$ so that

$$\frac{\tau_o}{\gamma R} = - \left(\frac{\partial H}{\partial X} + \frac{1}{g} \frac{\partial V}{\partial t}\right) \tag{36}$$

which may be written in terms of slopes as

$$S_e + S_a + S_f = 0 \tag{37}$$

The bed slope is introduced into the dynamic flow equation by noting $h = z + y$, then $\frac{\partial h}{\partial X} = \frac{\partial Z}{\partial X} + \frac{\partial y}{\partial X}$ and recalling $\frac{\partial Z}{\partial X} = - S_o$ so that $\frac{\partial h}{\partial X} = - S_o + \frac{\partial y}{\partial X}$. The dynamic flow equation then becomes

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{V}{g} \frac{\partial V}{\partial X} - \frac{1}{g} \frac{\partial V}{\partial t} \tag{38}$$

steady uniform flow

steady nonuniform flow

unsteady nonuniforn flow

The dynamic flow equationand the equation of continuity provide the basis for developing the solution to unsteady nonuniform flow problems. Due to the difficulty in solving these simultaneous equations a number of simplifications have been developed.

SWMM APPLICATION STUDY GUIDE

By

Thomas K. Jewell, Peter A. Mangarella, Francis A. DiGiano
Donald D. Adrian*

INTRODUCTION TO WORKSHOP STUDY GUIDE

This study guide has been prepared to allow the reader to utilize the U.S. Environmental Protection Agency Storm Water Management Model (SWMM) in a typical case study. Information is presented concerning data gathering and preparation for each of the four SWMM functional blocks. The attendee is assigned the task of completing a case study that illustrates typical conditions encountered in practice, for each block. All necessary data for completion of the case study are provided along with sample solutions.

The Runoff and Transport Block case studies have been developed using an urban drainage basin of approximately 300 acres. Figure 1a is a topographic map of this basin on a scale of 1 inch = 660 feet. Figures 1b-1e show the four quadrants of the basin with a scale of 1 inch = 360 feet. A schematic diagram of the basin sewer system (combined sewers) is given in Figure 2. This sytem was designed about 20 years ago when the projected land uses included industry, single and multi-family housing and open space. At that time the interstate highway, large shopping center and drive-in movie were not contemplated. Localized flooding during moderate to heavy rainfalls has been reported.

The objectives of the Runoff and Transport Block case studies are to model the existing sewer system and to identify areas where the system may be inadequate. A ten year return period, two hour duration design storm will be utilized. Derivation of this design storm is discussed in the Study Guide Section entitled Case Study Materials directly following a description of the Runoff Block.

The Runoff/Transport basin drains into a larger 3000 acre basin, the output of which will be utilized in the Storage/Treatment and Receiving Water Block case studies. Evaluation of the effects of treated and untreated combined sewer overflows on receiving waters is the objective of the Storage/Treatment and Receiving Water Block case studies.

*Research Assistant, Assistant Professor, Associate Professor, Professor, respectively, Department of Civil Engineering, University of Massachusetts/ Amherst.

FIGURE 1A. TOPOGRAPHIC MAP OF CASE STUDY BASIN.

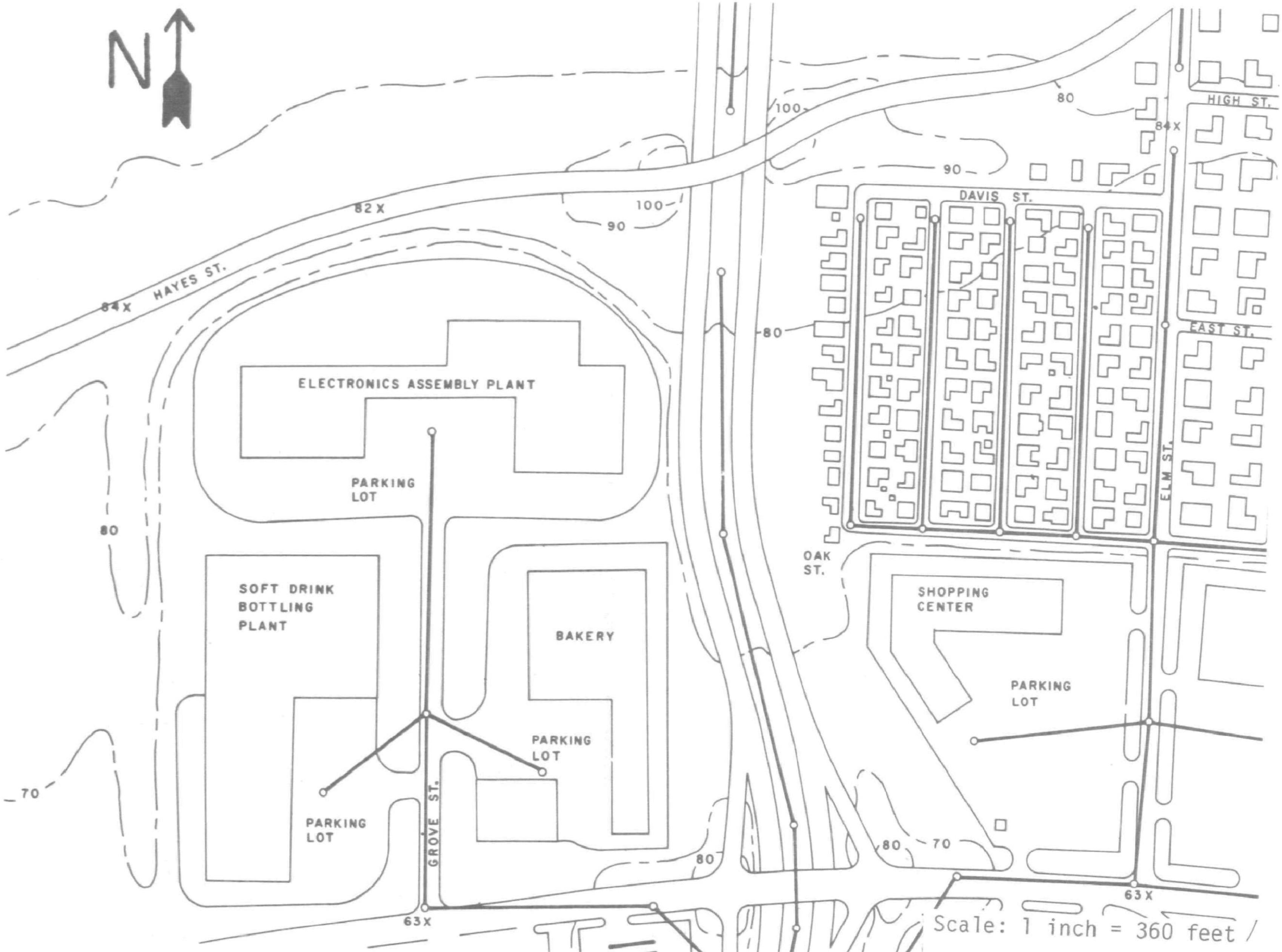


FIGURE 1B. TOPOGRAPHIC MAP OF NORTHWEST QUADRANT OF CASE STUDY BASIN.

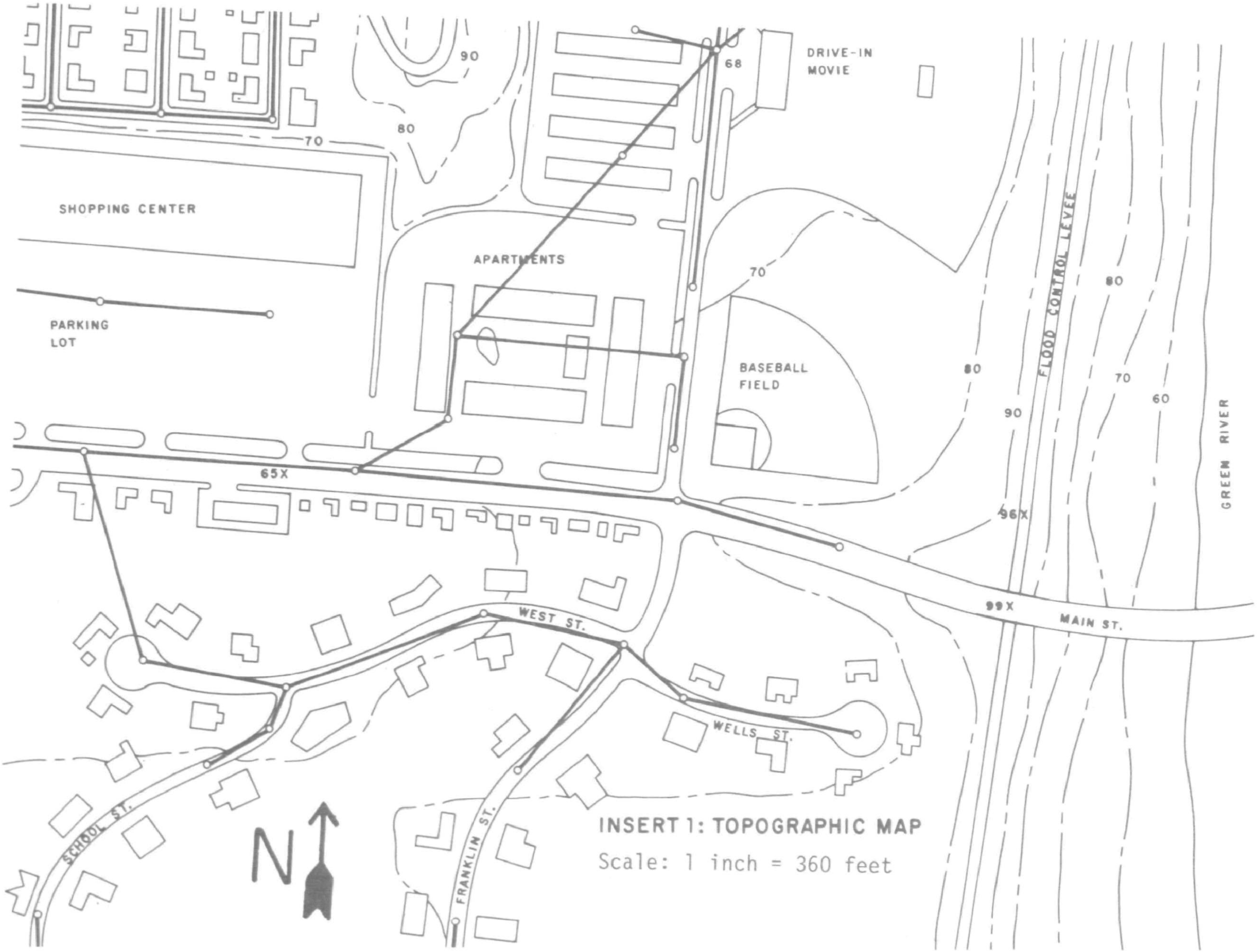


FIGURE 1C. TOPOGRAPHIC MAP OF NORTHEAST QUADRANT OF CASE STUDY BASIN.

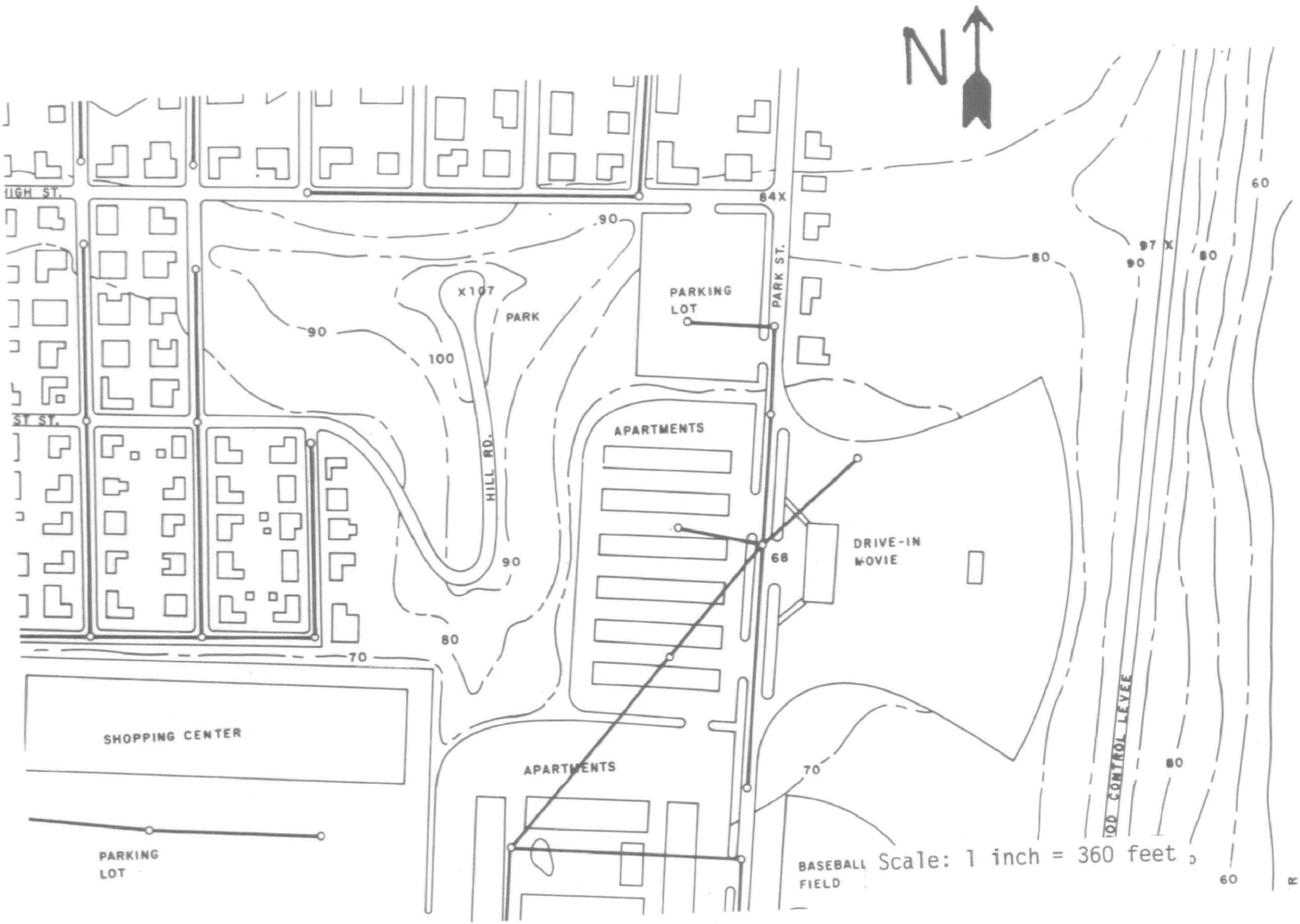


FIGURE 1D. TOPOGRAPHIC MAP OF SOUTHEAST QUADRANT OF CASE STUDY BASIN.

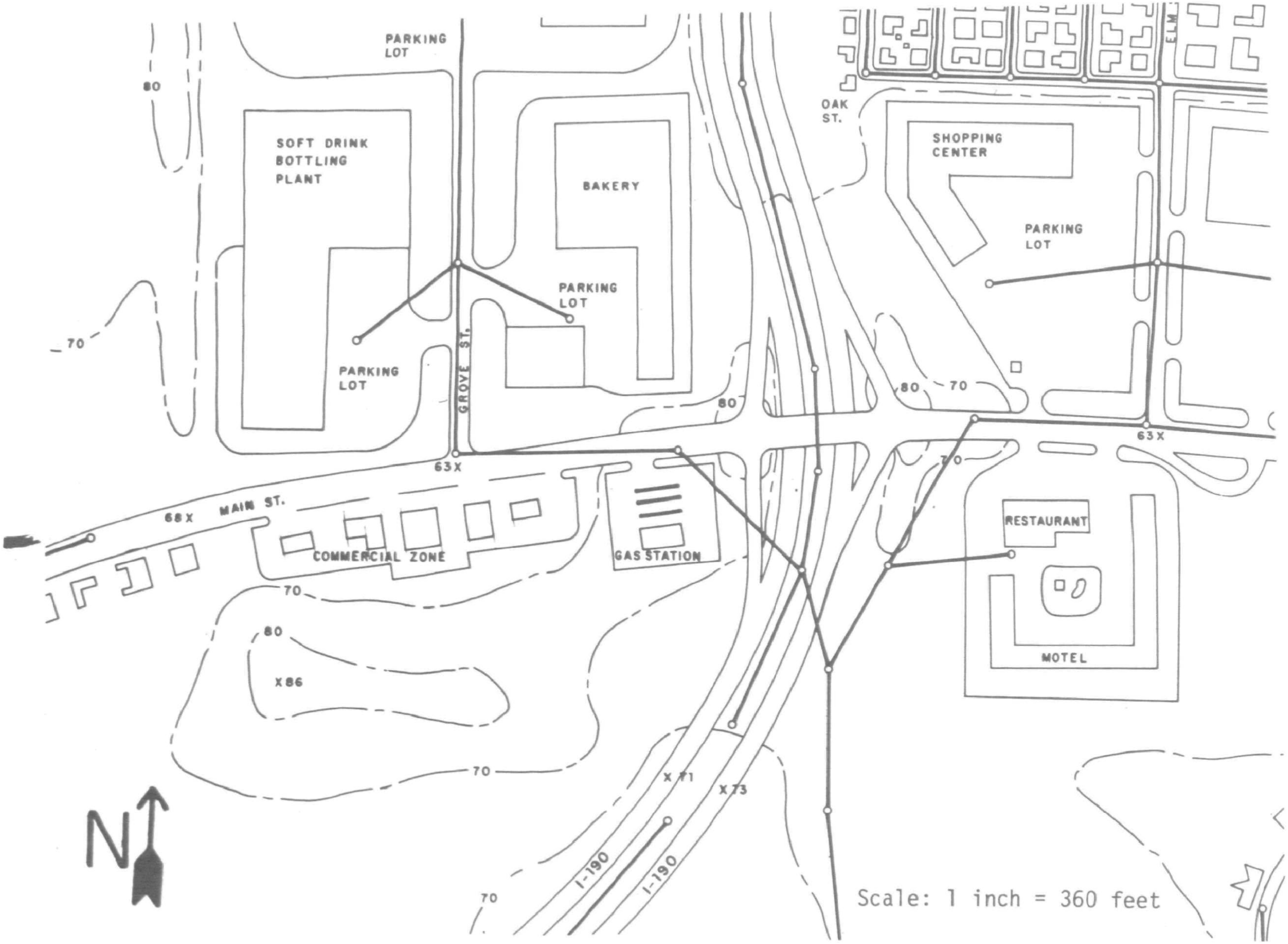


FIGURE 1E. TOPOGRAPHIC MAP OF SOUTHWEST QUADRANT OF CASE STUDY BASIN.

SEWER MAP

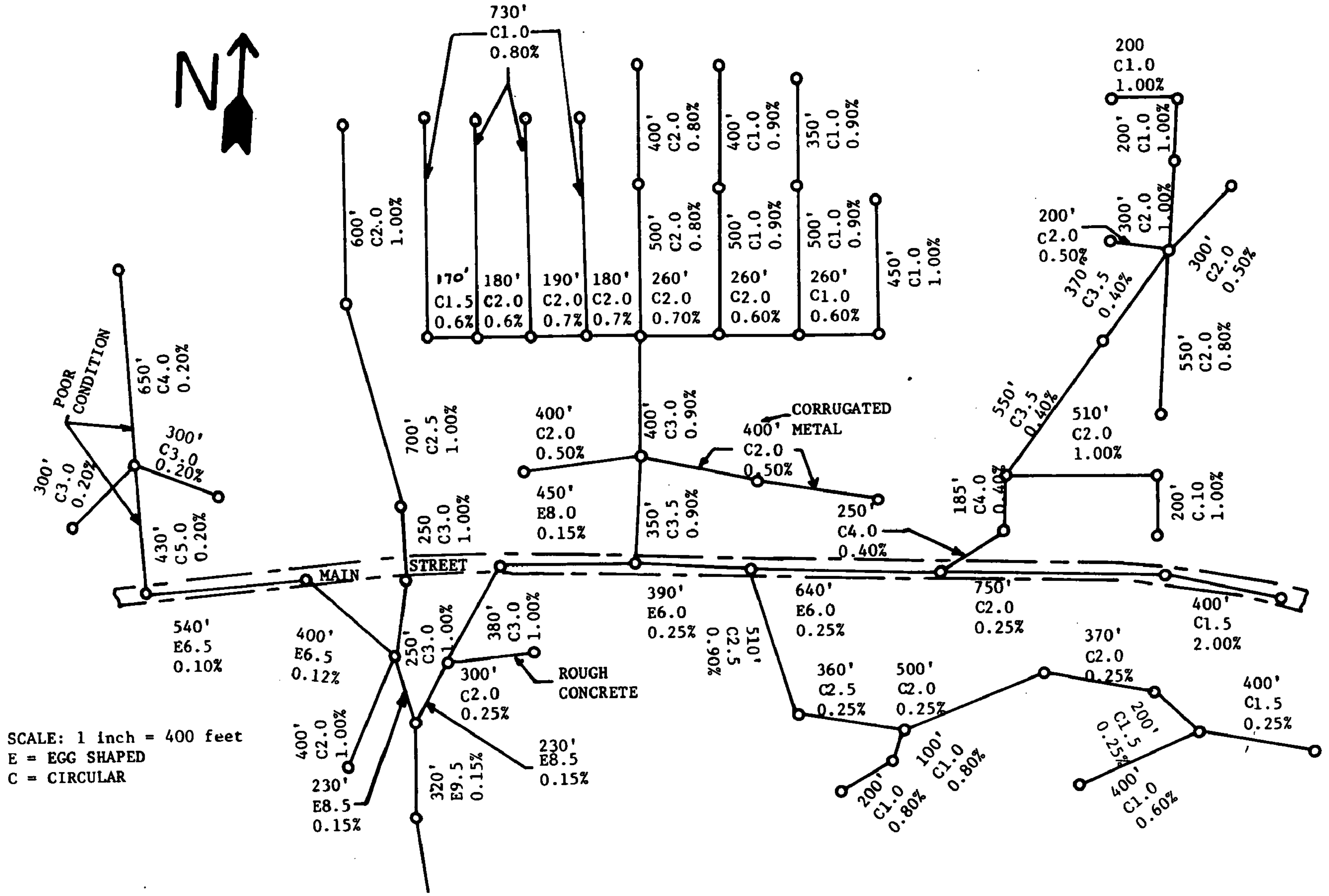


FIGURE 2. SEWER MAP FOR CASE STUDY BASIN.

PREPARATION OF RUNOFF BLOCK DATA

Considerable time and effort are required to gather and prepare the necessary data to run the Storm Water Management Model. The following may be of some assistance in identifying sources of these data. Municipal Department of Public Works (DPW) or Engineering Office files will provide the bulk of the data that is readily available. Most subcatchment data can be derived from up-to-date, large scale (1:200 preferrable) topographic maps. If these maps are not available or are inaccurate, appropriate aerial photos may be used; particularly for the determination of land use and percent impervious area. In very flat areas, sewer maps may be helpful in delineating subcatchments. Land use and zoning maps are also useful.

During data preparation for the Runoff Block, reference must be made to the existing sewer system. Most sewer data can be derived from sewer plates available in the DPW files. Some municipalities may have a comprehensive sewer system map that will assist greatly in coordination between Runoff and Transport Block data preparation.

If available data is insufficient, on-site inspections to check recent changes in land use and local topography, establish surface elevation, or inspect the condition and configuration of the existing sewer system may be necessary.

Preliminary Data Preparation

Preliminary data preparation for the Runoff Block include constructing a composite topographic map for the basin to be modeled, overlaying the major portion of the existing sewer system on the composite map and delineating the outer basin boundary. Topographic maps used should be large scale (1:200 is recommended) and up to date. Individual map sheets can be assembled to yield one topographic map of the basin. Data can be extracted from the sewer system map to overlay the major sewer trunk lines on the topographic map. Special attention should be given to including all of the sewer lines near the periphery of the basin. Finally, the outer basin boundary can be delineated from the composite topographic map, taking into account the sewer system layout. Knowledge of basic map reading techniques is necessary to establish basin boundaries. Additional information on topographic map interpretation can be found in Reference 1. Figure 3 illustrates the result of preliminary data preparation; i.e., the arrangement of the sewer system within the basin on the composite topographic map.

Discretization of Basin Into Subcatchments

Discretization can be defined as the division of the basin into overland flow areas and the sewer system into discrete elements. A decision must be

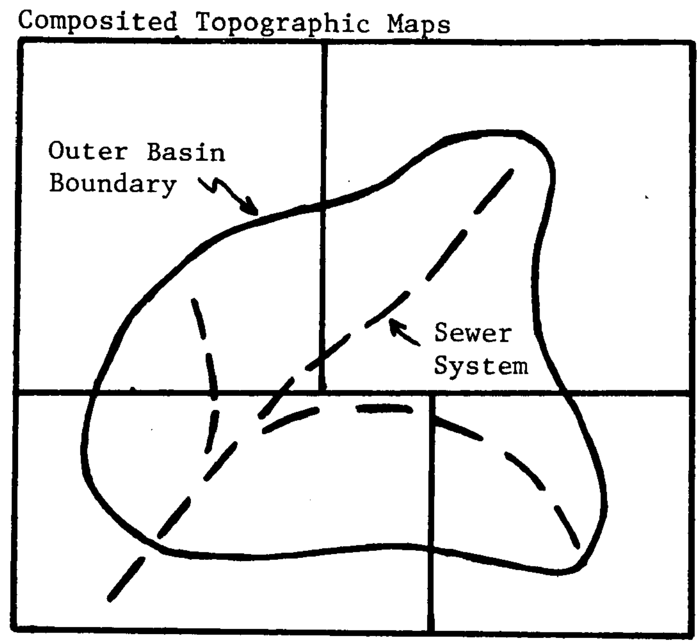


FIGURE 3. PRELIMINARY DATA PREPARATION.

made as to how many subcatchments should be included in the basin. Finer discretization grids (more subcatchments in a given basin) more accurately represent the actual rainfall-runoff process. Because computers have a limited memory capacity, a practical limit has to be placed on the number of subcatchments or sewer elements that can be modeled. With SWMM, a maximum of 200 subcatchments can be modeled in Runoff and 160 sewer elements in Transport. Factors to consider when determining the approximate number of subcatchments include the size of the basin, variations in land use within the basin, the degree of accuracy required (planning study vs. design study), and the method of calibration to be used. Funds available for the study must also be taken into account because data gathering and program execution costs will increase with the number of subcatchments modeled.

The choice of a discretization scheme must be based on engineering judgment. In order to select an appropriately fine discretization, it may be desirable to first apply two or more discretization grids to a small, representative basin. The small basin can then be used to calibrate the model; i.e., the most efficient discretization scheme should be selected and values chosen for the required input parameters to be used in modeling the remainder of the urban area. Model calibration will be discussed further in the Transport Block section.

Subcatchment Data Preparation

Once the outer basin boundary has been delineated and the approximate number of subcatchments fixed, the next step is to divide the basin into subcatchments. Topographic divides will again be used; however, it is now increasingly important to consider carefully the sewer system layout. For example, flow in the sewer system may be in a different direction from that of overland flow. Thus, water will be conducted in the opposite direction from that indicated by the land surface slope. Consideration must also be given to the location of the entry point of flow from the modeled subcatchment to the modeled sewer system; i.e., all of the flow from a large subcatchment should not be routed through a small sewer if this does not actually occur in the field. Placement of entry points may also have some effect on the position of subcatchment boundaries.

Subcatchments should be drawn to represent as closely as possible a homogeneous land use. Irregular subcatchment shapes should be avoided because the SWMM depicts overland flow in subcatchments as if the subcatchment was a rectangular plane of uniform slope and roughness. Unrealistic overland flow conditions can result if subcatchment geometry differs substantially from that assumed by the model.

Subcatchments should be arranged to reflect the actual field conditions as closely as possible. The total runoff from the basin, however, is much more sensitive to accurate outer basin boundary delineation than to the placement of subcatchments within the basin. This is especially true for large basins with a large number of subcatchments. Inaccuracies in subcatchment placement tend to cancel each other out, while errors in outer boundary delineation directly affect the total rainfall volume to be modeled.

Data that must be obtained for each subcatchment include area, slope, characteristic width, percent impervious area and land use. Preparation of each of the elements of data will be discussed. Other data required by the model, but not usually available, are overland flow resistance factors, retention depths and infiltration rates. To compensate for these deficiencies in data, the model has pre-programmed default values that are assumed if no data are supplied. The default values are recommended unless data is available or engineering judgment warrants otherwise.

Area of Subcatchment - Area of subcatchments can be measured with a planimeter. The planimeter readings can be scaled to give area in acres by first measuring a known area drawn to the same scale as the map to be used. The difference in planimeter readings at the start and finish divided by the known area will give a scaling factor of planimeter units/acre, designated as B. All subsequent planimeter results can be divided by this factor to give area in acres. Equations 1 and 2 show the computation and use of the scaling factor, respectively.

$$B = (R_1 - R_2)/A_k \tag{1}$$

where R_1 = planimeter reading at start of trace

R_2 = planimeter reading at end of trace

A_k = known area scaled to map (acres)

B = scaling factor (planimeter units/acre)

$$A = (R_1 - R_2)/B \tag{2}$$

where A = area of subcatchment (acres)

Subcatchment Slope - The subcatchment slope used for model input should be the best estimate of the average slope of the subcatchment. Accuracy of this estimate will be increased if subcatchments do not contain complex slopes; i.e., subcatchments should be constructed so that flow is over ground of fairly uniform slope. In many instances, this may not be practical. A method of calculating an area-weighted-average slope, is illustrated in Figure 4. This method, if used properly, will give a good estimate of the average subcatchment slope for complex cases. The steps involved in

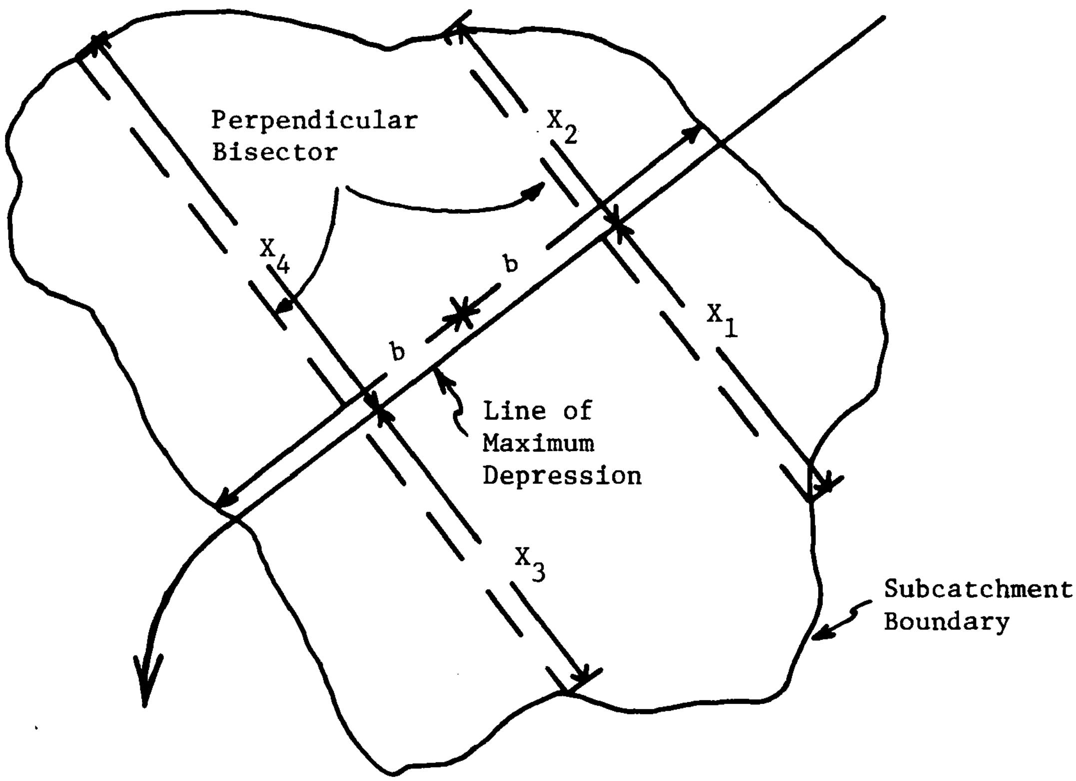


FIGURE 4. DETERMINATION OF WEIGHTED SLOPE, S_W.

calculating the weighted average slope are as follows;

1. Determine the line of maximum depression through the subcatchment (see Figure 2). This line is analogous to the position of a free flowing stream if one existed in the subcatchment.

2. Divide the line of maximum depression into equal increments of length b. The number of increments to be used is based on engineering judgment and depends on the complexity of the subcatchment.

3. Draw perpendicular bisectors through each of the increments chosen. Extend these bisectors to the subcatchment boundary.

4. Measure the distance, X_i, along each bisector from the line of maximum depression to the subcatchment boundary. Measure the difference in elevation, ΔZ_i, between the line of maximum depression and the subcatchment boundary.

5. Compute the weighted slope, S_w, as shown by Equation 3:

$$S_w = \sum_{i=1}^{n} \Delta Z_i / \sum_{i=1}^{n} X_i \tag{3}$$

where n = total number of areas formed.

The derivation of Equation 3 is given below:

$$S_w = \frac{b\ X_1 S_1}{b \sum_{i=1}^{n} X_i} + \frac{b\ X_2 S_2}{b \sum_{i=1}^{n} X_i} + \cdots + \frac{b\ X_n S_n}{b \sum_{i=1}^{n} X_i} \tag{4}$$

$$= \frac{X_1 S_1}{\sum_{i=1}^{n} X_i} + \frac{X_2 S_2}{\sum_{i=1}^{n} X_i} + \cdots + \frac{X_n S_n}{\sum_{i=1}^{n} X_i}$$

$$\Delta Z_i = X_i S_i \tag{5}$$

$$\therefore S_w = \sum_{i=1}^{n} \Delta Z_i / \sum_{i=1}^{n} X_i \tag{3}$$

Characteristic Width of Subcatchment - The characteristic width of a subcatchment is the distance over which overland flow leaves the subcatchment surface and enters the main drainage conduit passing through the subcatchment. This conduit may be a sewer element, a gutter or a natural channel. Figure 5 illustrates the meaning of characteristic width.

The SWMM User's Manual recommends that twice the length of the main drainage conduit be used as an approximation of the characteristic width. This works well if subcatchments are symmetrical about the main drainage conduit as is shown in Figure 5. Some amount of error would be introduced, however, if this approximation is used to model subcatchments in which the drainage system is located to either side of the subcatchment, as shown in Figure 6(a).

Analysis of an idealized 100 percent impervious subcatchment has indicated that the peak runoff from a modeled subcatchment with $W = 2L$ as shown in Figure 6(b), can be as much as 20 percent higher than from a subcatchment modeled with $W = L$, as shown in Figure 6(c). Thus, it is recommended that the characteristic width be taken as L when the drainage conduit is skewed entirely to one side of the subcatchment.

For configurations that are between the two extremes, a relatively simple method has been developed that accurately accounts for the position of the main drainage conduit within the subcatchment (see page 56, Reference 2). The method involves calculating a skewness factor S which is an area-weighted measure of the position of the principle drainage conduit or canal with respect to the centerline axis of the subcatchment. The equation for the skewness factor, S, is

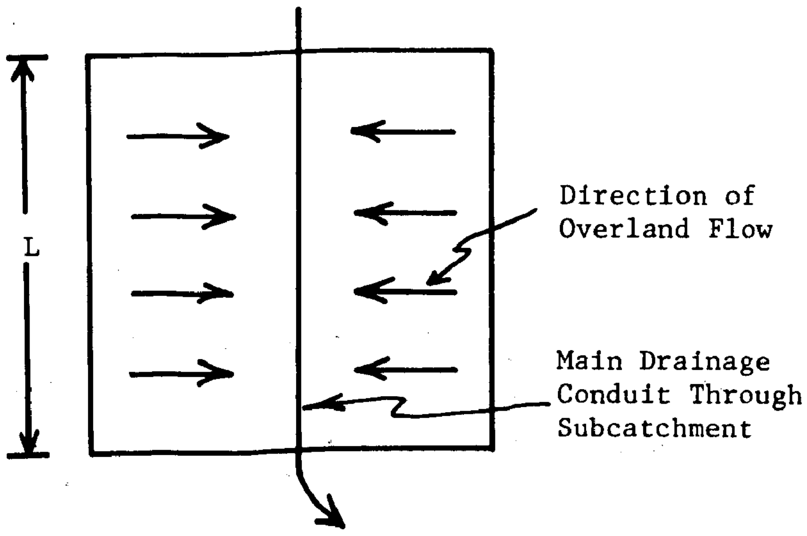
$$S = \left|(A-B)/(A+B)\right| \tag{6}$$

where A and B represent the areas to either side of the drainage conduit as shown in Figure 7. The equation for calculating the corrected characteristic width is given by

$$W = (2-S)L \tag{7}$$

where L is the length of the drainage conduit. The skewness factor as a function of W/L is shown in Figure 8.

Percent Impervious Area - The Runoff Block output is highly sensitive to the percent impervious area modeled and this factor is difficult to estimate accurately because some runoff from impervious areas may flow onto pervious areas, infiltrate and thus not contribute to the storm runoff. Therefore, it is recommended that an initial study of topographic maps and aerial photographs be made to estimate the actual percent impervious area with the understanding that this parameter may need adjustment during the calibration stage. Table 1



L = Total Length of Main Drainage Conduit
W = Characteristic Width = 2L = Total Width of Overland Flow.

FIGURE 5. CHARACTERISTIC WIDTH.

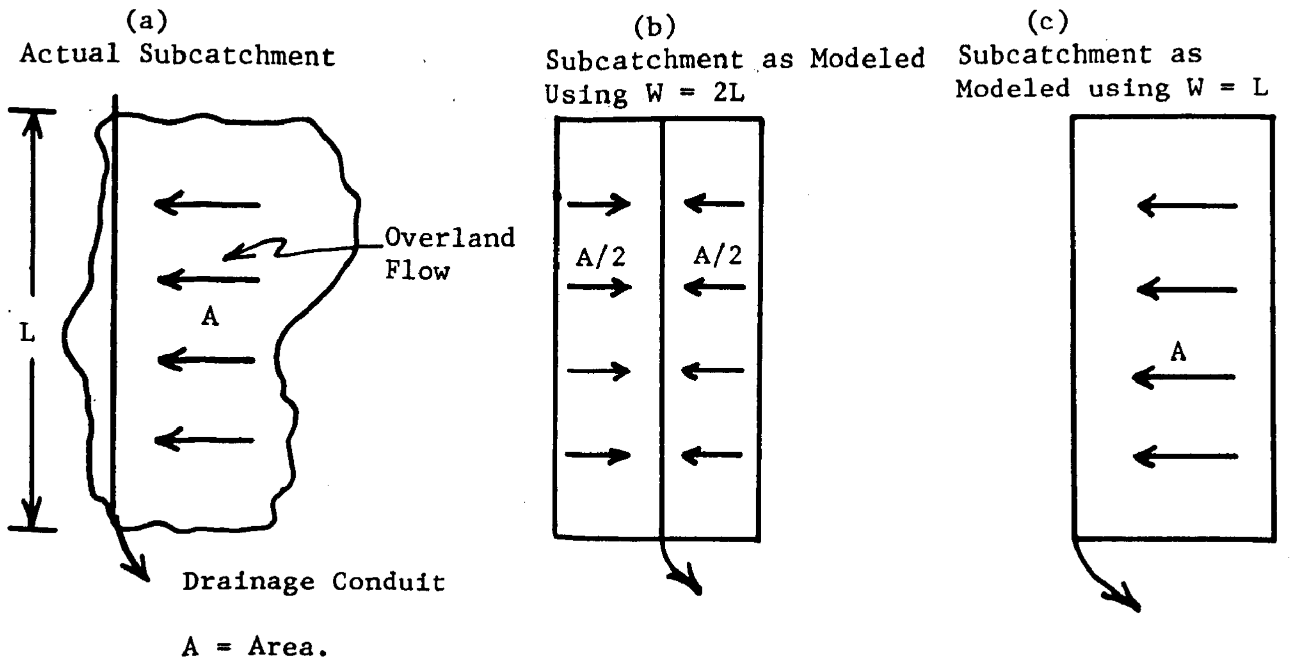


FIGURE 6. SUBCATCHMENT WITH SKEWED DRAINAGE CHANNEL.

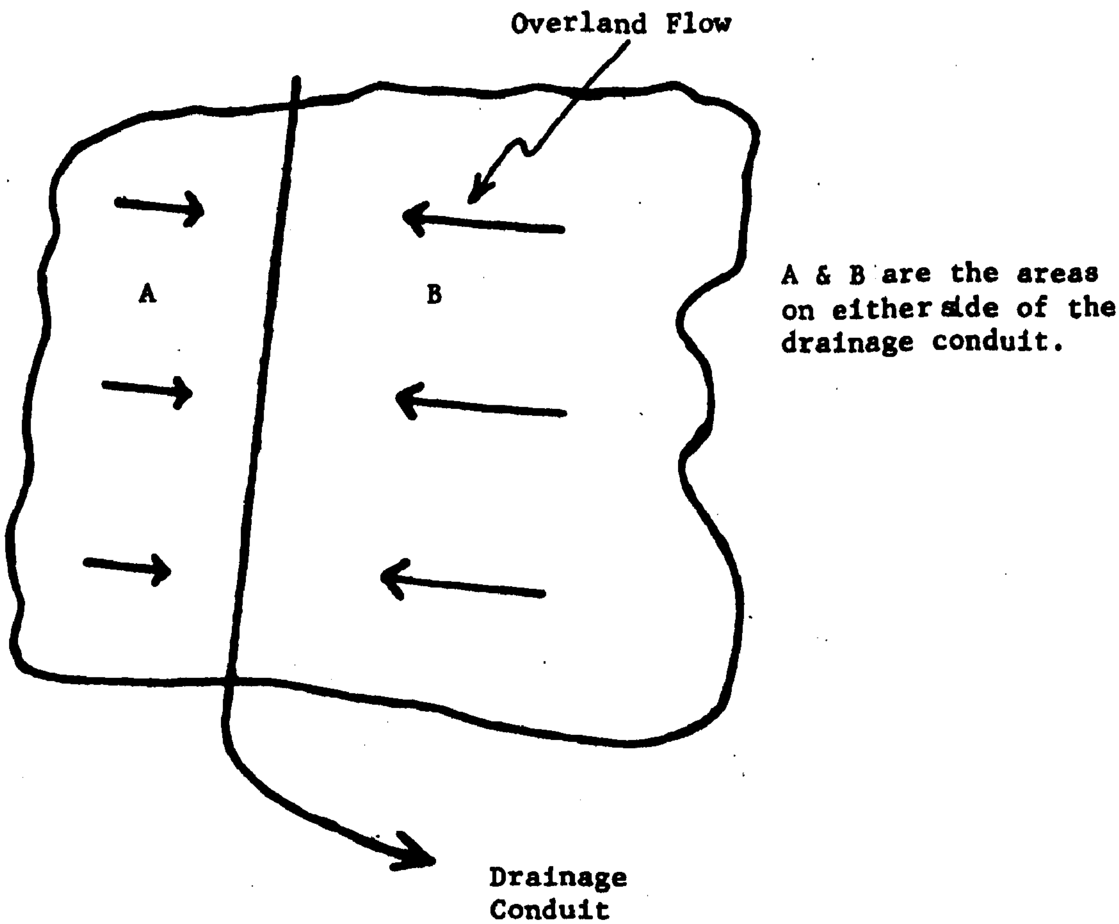


FIGURE 7. CALCULATED CHARACTERISTIC WIDTH.

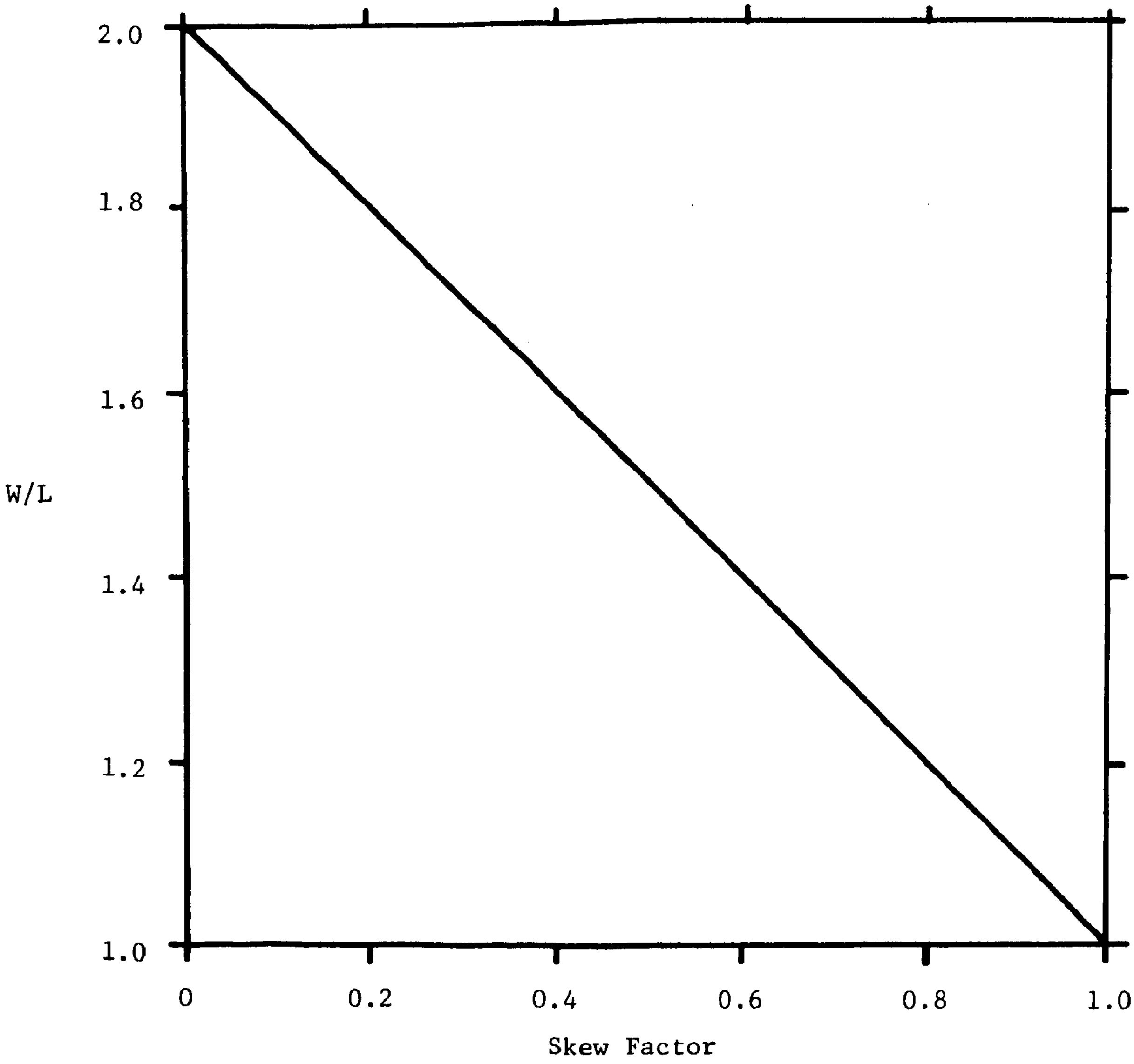


FIGURE 8. W/L VS. SKEW FACTOR.

Table 1. PERCENT IMPERVIOUSNESS AS FOR VARIOUS LAND USES IN THE SAN FRANCISCO AREA (Reference 3)

Type of development	Density in units per acre	Percent impervious	
		Santa Clara County	San Francisco Bay Region
Residential:			
Hill areas	0.5- 2	6	8
Low urbanization	3 - 6	10	15
Medium urbanization	7 -10	20	25
Heavy urbanization (apartments)	11 -20	32	40
Industrial:			
Nonmanufacturing		50	60
Manufacturing		40	50
Reserve		20	25
Commercial		50	60
Transportation		70	75
Public buildings		40	50
Public parks		12	12
Agricultural		4	4
Natural watersheds		2	2

taken from Reference 3 may be used as a guide in estimating impervious area.

Land Use Designation - The model predicts runoff water quality based on one of five land use designations: single family residential, multi-family residential, commercial, industrial or undeveloped land. As much as possible, subcatchments should be arranged so that each represents a single land use. When this is not possible, the prevailing land use should be assigned to the subcatchment. Zoning maps and land use maps will be helpful in determining land use; however, up-to-date checks by area reconnaissance and/or discussions with local officials may be necessary.

Gutters and Pipes - Up to 200 gutters or pipes can be modeled in the Runoff Block. These are in addition to the 160 sewer elements that can be modeled in the Transport Block. Use of gutters and pipes should be considered supplemental to the transport system, either to extend the number of elements modeled beyond 160 or to model conduits such as street gutters or natural channels that cannot be modeled in the Transport Block. The Transport Block should be used in situations where: 1) backwater effects are significant; 2) hydraulic elements other than pipes and gutters (such as pumps or flow dividers) are used, and 3) solids deposition or suspension is important.

To decide which elements to model as gutters and as pipes, the extent of the transport system modeled in the Transport Block must be estimated. Generally, all trunk sewers and most branch sewers three feet in diameter or larger should be modeled in the Transport Block. Smaller branch lines, gutters and natural channels can be modeled in the Runoff Block. Some adjustment of the number of gutters and pipes modeled in the Runoff Block may be desirable after the transport system has been completely descritized. When a choice exists, an element should be modeled in the Transport Block because of its more accurate flow routing procedure.

All of the flow through a modeled gutter or pipe enters at the head end; i.e., there is no differential entry along the length of the element. The same is true in the Transport Block but is less critical because of the larger elements and flows which are modeled.

Data required for gutters and pipes include length, width or diameter, invert slope and Manning's roughness factor. Special data required only for gutters are side slopes and depth when full. Most data will be available on sewer plates for pipes. Data for gutters or natural channels, on the other hand, will have to be gathered in the field. Some surveying may be necessary to ascertain slopes. Typical values of Manning's roughness coefficients are found in Table 5.

Placement of Subcatchment Flow Outlet Point - All of the flow from a subcatchment must enter either a single manhole or a single gutter/pipe. It is important that this point of entry in the modeled system be located approximately where most of the flow would enter the actual system. In other words, even though flows in the actual subcatchment may be entering the sewer system at several manholes, the model must route all of the flow through a single point. Inaccurate location of the entry point will result in errors in the modeled routing time. Placement of the outlet point will also influence which gutters or pipes should be modeled.

Network Arrangement and Numbering - Completion of Runoff Block data preparation provides a system of subcatchments, gutter/pipes and outlet manholes. These outlet manholes are the points through which flows will later be transferred to the sewer system modeled in the Transport Block. A typical runoff block arrangement of numbered manholes and gutter/pipes is shown in Figure 9.

The numbers assigned to subcatchments, gutter/pipes and inlet manholes are the only way the computer has of joining the whole system together; therefore, care must be taken to avoid errors in assigning numbers to the various elements. The following numbering procedure is recommended. Subcatchments should be numbered from one to the total number to be modeled. Because a maximum of 70 manholes can be used to transfer flows to the Transport Block, the numbers 1-70 should be used to identify outlet manholes. Subcatchments and outlet manholes may be assigned the same numbers; however, numbers should not be duplicated between manholes and gutter/pipes. Therefore it is recommended that gutters and pipes be assigned numbers between 201 and 400. This numbering system is illustrated in Figure 9.

As shown in Figure 9, gutter/pipes may be arranged in tree structures; however, all such trees must eventually terminate at an outlet manhole. For example, gutter/pipes 203 and 204 flow into gutter/pipe 202 which in turn flows into manhole 2. The overland flow from subcatchment 2 could either enter the head of gutter/pipe 202 or manhole 2, whichever more accurately represents the physical situation.

Quality Data - Data required for surface quality computations are included in card groups 9-11 of Table 5-7, pp. 77-78, User's Manual (reference 6). The number of dry days prior to a storm can be determined from U.S. Weather Bureau rainfall records or local records if available. Street cleaning frequency, number of street sweeper passes and catchbasin storage volume can be obtained from municipal Department of Public Works records. In the absence of measured BOD concentrations in catchbasins, the recommended value of 100 mg/l should be used.

The SWMM provides a choice of two methods for calculating suspended solids generation from subcatchments. ISS = 0 is an exponential decay function and ISS = 1 is a regression equation. No recommendation is given in the User's Manual as to which method is better. Probably the best approach would be to compare both methods to measured SS data during the model calibration stage and choose the closer approximation.

The erosion portion of the quality subroutine is discussed thoroughly in the User's Manual and will not be included here (Reference 6). Other data required are land use classification (as discussed previously), the total number of catchbasins in each subcatchment (estimated from sewer maps or field surveys) and total length of all gutters within each subcatchment (scaled from the topographic map).

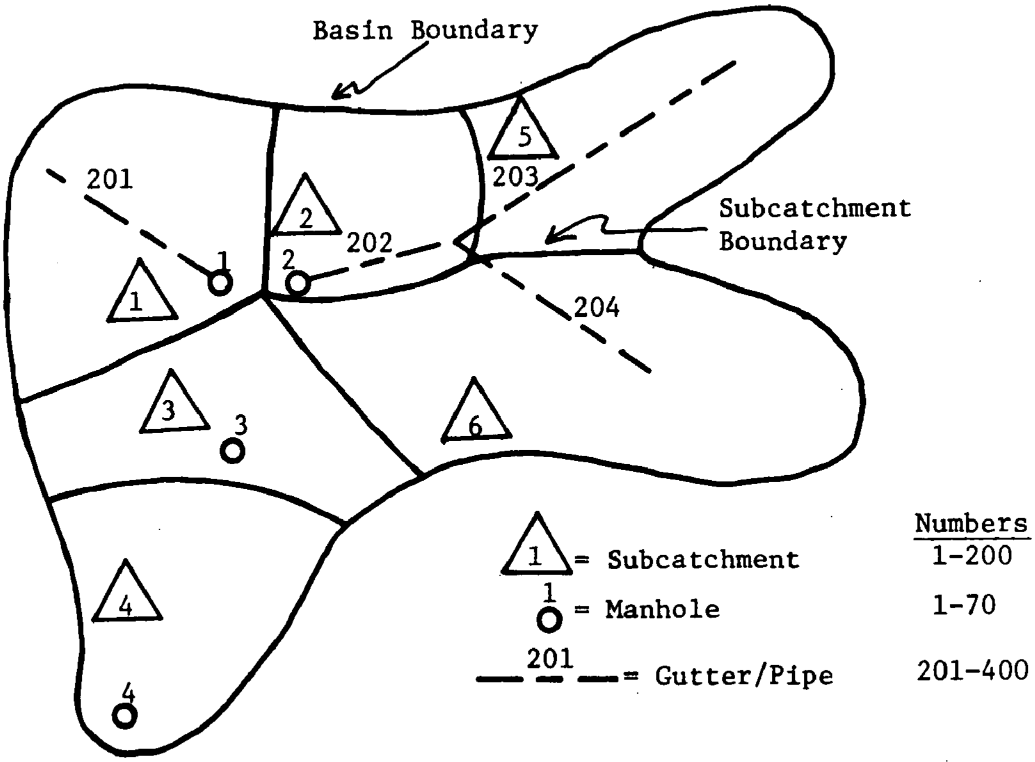


FIGURE 9. TYPICAL RUNOFF NETWORK ARRANGEMENT.

PREPARATION OF CODING FORMS

It is recommended that all subcatchment and gutter/pipe data be tabulated on forms similar to those supplied with these case study materials. This will make transfer of data to the computer coding forms much easier. Standard FORTRAN coding forms should be used to prepare the data as shown in Table 5-7, pp. 74-79 of the User's Manual.

CASE STUDY MATERIALS

General Description

The objectives of the case study are to model an existing sewer system in a typical urban drainage basin and to identify possible surcharging in the system. The initial task is to complete the descritization and data preparation for the Runoff Block. The basin (Figure 1) should be subdivided into ten or less subcatchments for illustrative purposes. Although this is a relatively small basin, some gutters and pipes should be included in the Runoff Block. Data for pipes can be obtained from the sewer map supplied with the Transport Block Case Study materials. Data for gutters can be scaled from the topographic map. Zero side slope for gutters and 12" depth when full are to be assumed. Streets are constructed of smooth asphalt.

Table 2 may be used as a guide to organizing the various tasks. Tables 3 and 4 have been provided for data tabulation. The columns are arranged in the order the data will be transcribed to computer coding forms.

Design Storm

The storm to be used in the case study is a ten year, two hour duration storm. Data for this storm were taken from an actual point frequency analysis of rainfall in Cleveland, Ohio (4). A ten year return period was chosen because of the commercial and industrial land uses within the Case Study basin. Figure 10, also taken from Reference 4, has been used to convert the total volume of rainfall (2.10 inches) into a hyetograph. This hyetograph will be used for program input and is shown in Figure 11.

Additional Data

The following additional data are provided.

1. Number of time steps (NSTEP): 50 should be sufficient.
2. Hour of start of storm (NHR): 12
3. Minutes of start of storm (NMN); 00
4. Integration period (DELT); 5.0 minutes seems to be a good compromise between accuracy and raingauge data gathering capability.
5. Number of hyetographs (NRGAG): 1
6. Number of data points for each hyetograph (NHISTO); 24
7. Time interval between values (THISTO): 5.0 min.
8. Number of dry days (DRYDAY): 14 days
9. Street cleaning frequency (CLEFREQ): 21 days
10. Number of street sweeper passes (NPASS); 2
11. Catchbasin storage volume (CBVOL): 18 ft^3
12. BOD concentration in catchbasins (CBFACT(4)): 100 mg/l
13. Number of catchbasins in subcatchment (BA): Assume 1 catchbasin/acre.

Table 2. RUNOFF BLOCK DATA PREPARATION CHECK LIST

1. Composite Topographic Map
2. Sewer System Overlay
3. Outer Basin Boundary
4. Descritization of Basin into Subcatchments
5. Subcatchment Data Preparation
    a. Area
    b. Slope
    c. Characteristic Width
    d. Percent Impervious Area
    e. Overland Flow Resistance Factors, Retention Storage Depths and Infiltration Rates, when available.
    f. Land Use
    g. Gutters and Pipes
    h. Placement of Subcatchment Flow Outlet Point
    i. Network Arrangement and Numbering
    j. Quality Data in Addition to Land Use.
6. Preparation of Coding Forms

Table 3. SUBCATCHMENT DATA

Quantity						QUALITY		
Subcatchment Number	Gutter or Manhole for Drainage	Width (ft)	Area (Acres)	Percent Impervious	Ground Slope (ft/ft)	Land Use	Number of Catchbasins	Total Length of Gutters

Table 4. GUTTER DATA

Gutter/ Pipe Number	Gutter or Inlet Number for Drainage	1 = Gutter 2 = Pipe	Bottom Width of Gutter or Pipe (ft) Diameter (ft)	Length (ft)	Invert Slope (ft/ft)	Left Side Slope (ft/ft)	Right Side Slope (ft/ft)	Manning's Coefficient	Gutter Depth when Full (in.)

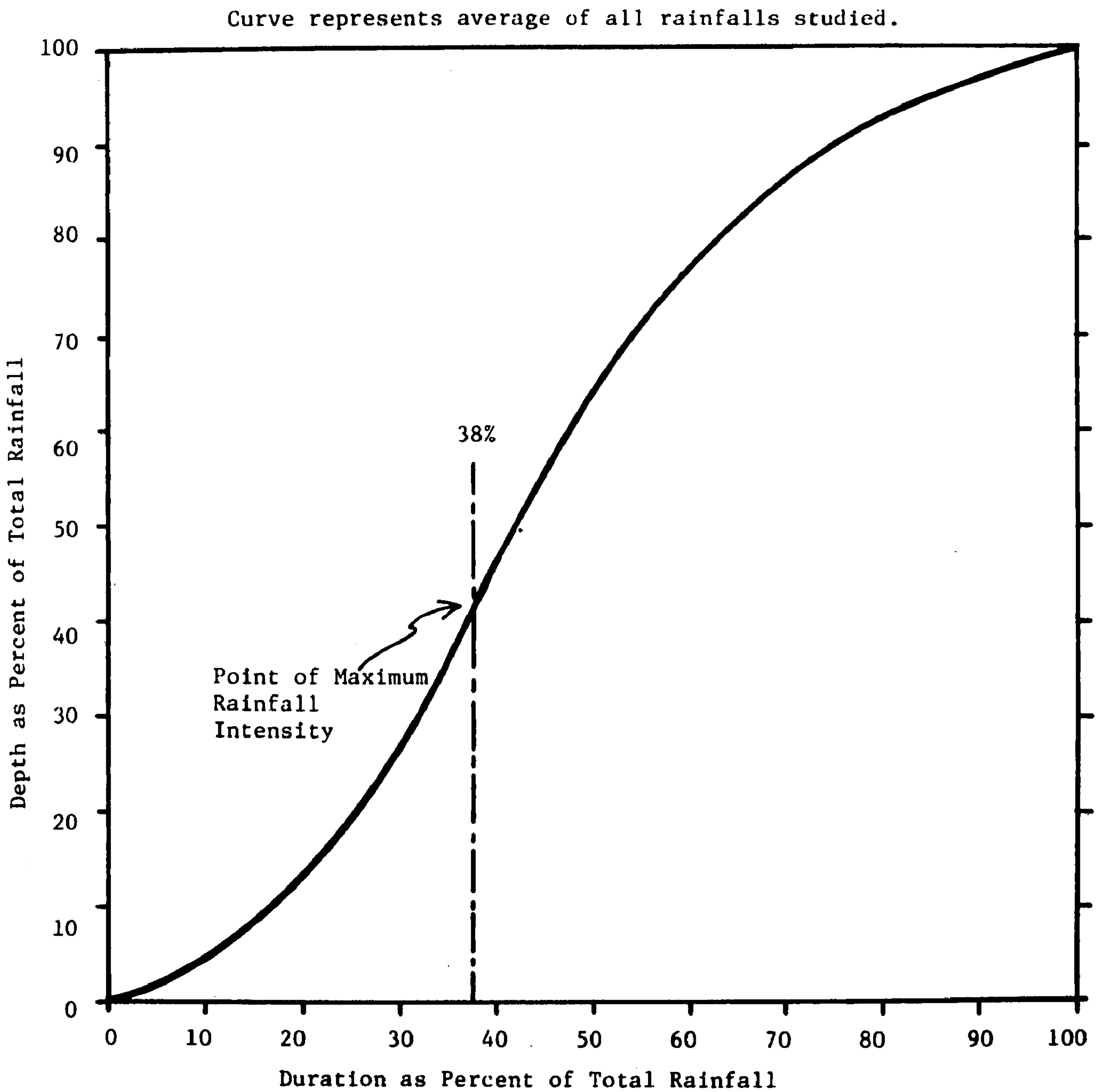


FIGURE 10. CLEVELAND, OHIO, RAINFALL MASS DIAGRAM.

NOTE: Taken from Page 9, Reference 4.

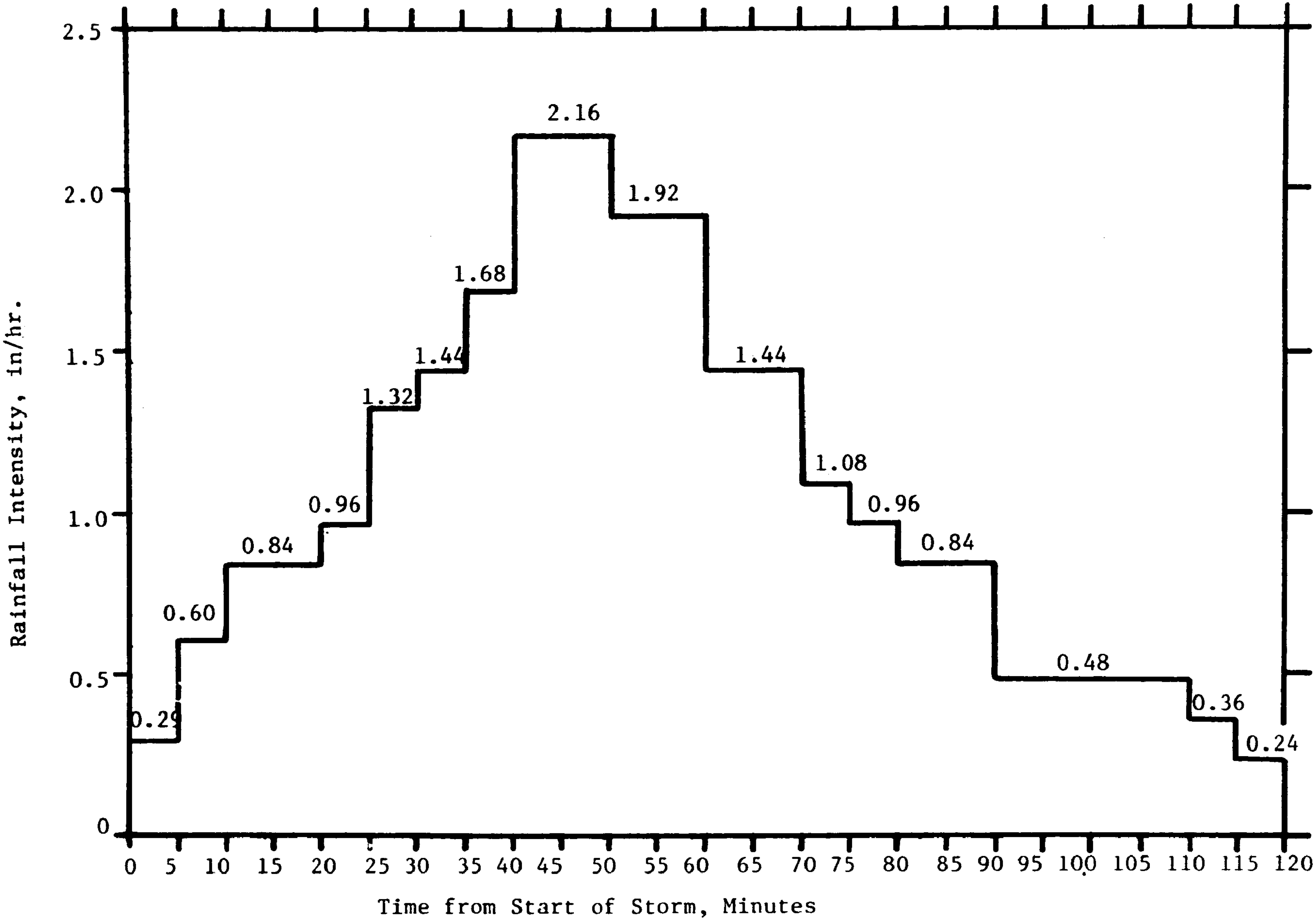


FIGURE 11. RAINFALL HYETOGRAPH.

SAMPLE DATA INPUT AND PROGRAM OUTPUT: RUNOFF BLOCK

The suggested solution to the case study depicts the given basin as ten subcatchments with seven gutters or pipes. All pertinent input data and program output for the Runoff Block Case Study are provided on the following pages. The specific items of information included are:

Basin Map Showing Descritization Grid Used (Figure 12)

Sample Coding Forms (given for Runoff Block only)

Runoff Block Card Data

Runoff Block Program Output

FIGURE 12. BASIN MAP SHOWING DECRITIZATION GRID USED

IBM **FORTRAN Coding Form**

X-7327-6 U/M 050
Printed in U.S.A.

PROGRAM: SWMM   PAGE 1 OF 3

PROGRAMMER: DATE:

FORTRAN STATEMENT

```
RUNOFF

  Card Group 1
  Runoff  Block  Workshop
  Case  Study  Basin,  10 Year - 2 Hour Duration Design Storm

  Card Group 2
    1   60  1200   5.0     1

  Card Group 3
  24  5.0

  Card Group 4
0.29 0.60 0.84 0.84 0.96 1.32 1.44 1.68 2.16 2.16
1.92 1.92 1.44 1.44 1.08 0.96 0.84 0.84 0.48 0.48
0.48 0.48 0.36 0.24

  Card Group 5
          50    13     1   30.0 1100.   .011   0.   0.   .013  12.
          51     7     1   70.0 1800.   .009   0.   0.   .013  12.
          52    14     2    2.5 1230.   .003             .013
          53     9     1   30.0 1500.   .007   0.   0.   .013  12.
          54     3     2    8.0 1200.   .010             .013
          55     5     2    5.0 1080.   .002             .017
```

IBM **FORTRAN Coding Form** GX28-7327-6 U/M 050** Printed in U.S.A.

PROGRAM: SWMM

PROGRAMMER:

DATE:

PUNCHING INSTRUCTIONS: GRAPHIC / PUNCH

PAGE 23

CARD ELECTRO NUMBER

FORTRAN STATEMENT

56	5	1	70.0	1200.	.012	0.	0.	.013		12.

Card Group 6

Blank Card

Card Group 7

1	1	501850.	43.0	25.	.023
1	2	511900.	15.1	20.	.011
1	3	521400.	36.1	15.	.016
1	4	122600.	26.9	75.	.021
1	5	531700.	42.1	40.	.024
1	6	83400.	38.1	95.	.014
1	7	61600.	19.7	40.	.013
1	8	545100.	21.7	40.	.010
1	9	552400.	50.5	75.	.013
1	10	561250.	13.7	40.	.034

Card Group 8

Blank Card

Card Group 9

1	14.	21.	2	18.	100.	1

Card Group 10

*A standard card form, IBM ... 888 ...

**Number of forms per pad may vary slightly

IBM **FORTRAN Coding Form** GX28-7327-6 U/M 050** Printed in U.S.A.

PROGRAM: SWMM | PROGRAMMER: | DATE: | PUNCHING INSTRUCTIONS: GRAPHIC / PUNCH | PAGE 3 OF 3 | CARD ELECTRO NUMBER*

Omit

Card Group 11

1	1	43.0	39.5
2	1	13.1	47.5
3	1	36.1	55.0
4	2	26.9	32.0
5	2	42.1	200.0
6	3	38.1	54.0
7	3	19.7	25.5
8	1	21.7	150.0
9	4	50.6	65.0
10	3	13.7	40.5

Card Group 12

1 1

Card Group 23

8

**Number of forms per pad may vary slightly

RUNOFF BLOCK CARD DATA

```
                                                                                             CARD
                                                                                             GROUP
   0  10  10   9
   1   2   3   4   7
RUNOFF
   RUNOFF  BLOCK  WORKSHOP                                                                    1
   CASE  STUDY  BASIN,  10 YEAR - 2 HOUR DURATION DESIGN STORM                                2
    1    50 1200   5.0     1                                                                  3
   24   5.0                                                                                   4
 0.29 0.60 0.84 0.84 0.96 1.32 1.44 1.68 2.16 2.16
 1.92 1.92 1.44 1.44 1.08 0.96 0.84 0.84 0.48 0.48
 0.48 0.48 0.36 0.24
        50    13     1     30.0    1100.     .011       0.       0.     .013      12.
        51     7     1     70.0    1800.     .009       0.       0.     .013      12.
        52    14     2      2.5    1230.     .003                       .013
        53     9     1     30.0    1500.     .007       0.       0.     .013      12.        5
        54     3     2      3.0    1200.     .010                       .013
        55     5     2      5.0    1080.     .002                       .017
        56     5     1     70.0    1200.     .012       0.       0.     .013      12.

    1     1    501850. 43.0   25. .023
    1     2    511900. 13.1   20. .011
    1     3    524400. 36.1   15. .015
    1     4    122600. 26.9   75. .021
    1     5    531700. 42.1   40. .024
    1     6     83400. 38.1   95. .014                                                        7
    1     7     61600. 19.7   40. .013
    1     8    545100. 21.7   40. .010
    1     9    552430. 50.5   75. .013
    1    10    561350. 13.7   40. .034

          1         14.         21.          2        18.        100.     1                   9
          1     1        43.0       39.5
          2     1        13.1       47.5
          3     1        36.1       55.0
          4     2        26.9       32.0
          5     2        42.1      200.0
          6     3        38.1       54.0                                                      11
          7     3        19.7       25.5
          8     1        21.7      150.0
          9     4        50.5       65.0
         10     3        13.7       40.5
    1     1                                                                                  12
    0                                                                                        13
```

ENVIRONMENTAL PROTECTION AGENCY - STORM WATER MANAGEMENT MODEL *** RELEASE II ***

DEVELOPED BY METCALF + EDDY, INC.

UNIVERSITY OF FLORIDA

WATER RESOURCES ENGINEERS, INC.

SEPTEMBER 1970

UPDATED BY UNIVERSITY OF FLORIDA

FEBRUARY 1975

THIS IS A NEW RELEASE OF THE SWMM. IF ANY PROBLEMS OCCUR IN RUNNING THIS MODEL PLEASE CONTACT WAYNE HUBER OR ALAN PELTZ AT THE UNIVERSITY OF FLORIDA. PHONE 1-904-392-0846

TAPE OR DISK ASSIGNMENTS

JIN(1)	JIN(2)	JIN(3)	JIN(4)	JIN(5)	JIN(6)	JIN(7)	JIN(8)	JIN(9)	JIN(10)
0	10	-0	-0	-0	-0	-0	-0	-0	-0

JOUT(1)	JOUT(2)	JOUT(3)	JOUT(4)	JOUT(5)	JOUT(6)	JOUT(7)	JOUT(8)	JOUT(9)	JOUT(10)
10	9	-0	-0	-0	-0	-0	-0	-0	-0

NSCRAT(1)	NSCRAT(2)	NSCRAT(3)	NSCRAT(4)	NSCRAT(5)
1	2	3	4	7

RUNOFF BLOCK WORKSHOP

CASE STUDY BASIN, 10 YEAR - 2 HOUR DURATION DESIGN STORM

BASIN NUMBER 1

NUMBER OF TIME STEPS 50

INTEGRATION TIME INTERVAL (MINUTES): 5.00

25.0 PERCENT OF IMPERVIOUS AREA HAS ZERO DETENTION DEPTH

FOR 24 RAINFALL STEPS, THE TIME INTERVAL IS 5.00 MINUTES

FOR RAINGAGE NUMBER 1 RAINFALL HISTORY IN INCHES PER HOUR

.29	.60	.84	.84	.95	1.32	1.44	1.68	2.16	2.16
1.92	1.92	1.44	1.44	1.08	.96	.84	.84	.48	.48
.48	.48	.36	.24						

***** G U T T E R A N D P I P E D A T A *****

	GUTTER NUMBER	GUTTER CONNECTION	WIDTH (FT)	LENGTH (FT)	SLOPE (FT/FT)	SIDE SLOPES L	SIDE SLOPES R	MANNING N	OVERFLOW (IN)
1	50	13	30.0	1100.	.011	0.0	0.0	.013	12.00
2	51	7	70.0	1800.	.009	0.0	0.0	.013	12.00
3	* 52	14	2.5	1230.	.003	-0.0	-0.0	.013	10.00
4	53	9	30.0	1500.	.007	0.0	0.0	.013	12.00
5	* 54	3	3.0	1200.	.010	-0.0	-0.0	.013	10.00
6	* 55	5	5.0	1080.	.002	-0.0	-0.0	.017	10.00
7	56	5	70.0	1200.	.012	0.0	0.0	.013	12.00

TOTAL NUMBER OF GUTTERS/PIPES: 7

ASTERISK (*) DENOTES CIRCULAR PIPE, DIAMETER=WIDTH.

* * * * * S U B C A T C H M E N T   D A T A * * * * *

SUBCATCH- MENT NO.		GUTTER OR INLET	WIDTH (FT)	AREA (AC)	PERCENT IMPERV.	SLOPE (FT/FT)	RESISTANCE FACTOR IMPERV.	RESISTANCE FACTOR PERV.	SURFACE STORAGE(IN) IMPERV.	SURFACE STORAGE(IN) PERV.	INFILTRATION RATE(IN/HR) MAXIMUM	INFILTRATION RATE(IN/HR) MINIMUM	DECAY RATE (1/SEC)	GAGE NO.
1	1	50	1850.0	43.	25.0	.023	.013	.250	.062	.184	3.00	.52	.00115	1
2	2	51	1990.0	13.	20.0	.011	.013	.250	.062	.184	3.00	.52	.00115	1
3	3	52	4400.0	36.	15.0	.015	.013	.250	.062	.184	3.00	.52	.00115	1
4	4	12	2600.0	27.	75.0	.021	.013	.250	.062	.184	3.00	.52	.00115	1
5	5	53	1700.0	42.	40.0	.024	.013	.250	.062	.184	3.00	.52	.00115	1
6	6	8	3400.0	38.	95.0	.014	.013	.250	.062	.184	3.00	.52	.00115	1
7	7	6	1600.0	20.	40.0	.013	.013	.250	.062	.184	3.00	.52	.00115	1
8	8	54	5100.0	22.	40.0	.010	.013	.250	.062	.184	3.00	.52	.00115	1
9	9	55	2400.0	51.	75.0	.013	.013	.250	.062	.184	3.00	.52	.00115	1
10	10	56	1350.0	14.	40.0	.034	.013	.250	.062	.184	3.00	.52	.00115	1

TOTAL NUMBER OF SUBCATCHMENTS, 10

TOTAL TRIBUTARY AREA (ACRES), 304.90

.....QUALITY SIMULATION INCLUDED IN THIS RUN.....

INPUT PARAMETERS AS FOLLOWS

NUMBER OF CONSTITUENTS 8

NUMBER OF DRY DAYS 14.0

STREET CLEANING FREQ 21.0 DAYS

PASSES PER CLEANING 2

STD CATCHBASIN VOLUME 18.00 FT3

CATCHBASIN CONTENTS BOD 100.0 MG/L

METHOD FOR CALCULATING SS,
SPECIAL TECHNIQUE.
SAME AS IN ORIGINAL
RELEASE I OF THE SWMM.
ISS = 1

WATERSHED QUALITY DEFINITIONS

SUBAREA NUMBER		LAND USE CLASS.	TOTAL GUTTER LENGTH*10**2 FT.	NUMBER OF CATCHBASINS
1	1	1	39.50	43.00
2	2	1	47.50	13.10
3	3	1	55.00	36.10
4	4	2	32.00	26.90
5	5	2	200.00	42.10
6	6	3	54.00	38.10
7	7	3	25.50	19.70
8	8	1	150.00	21.70
9	9	4	65.00	50.50
10	10	3	40.50	13.70

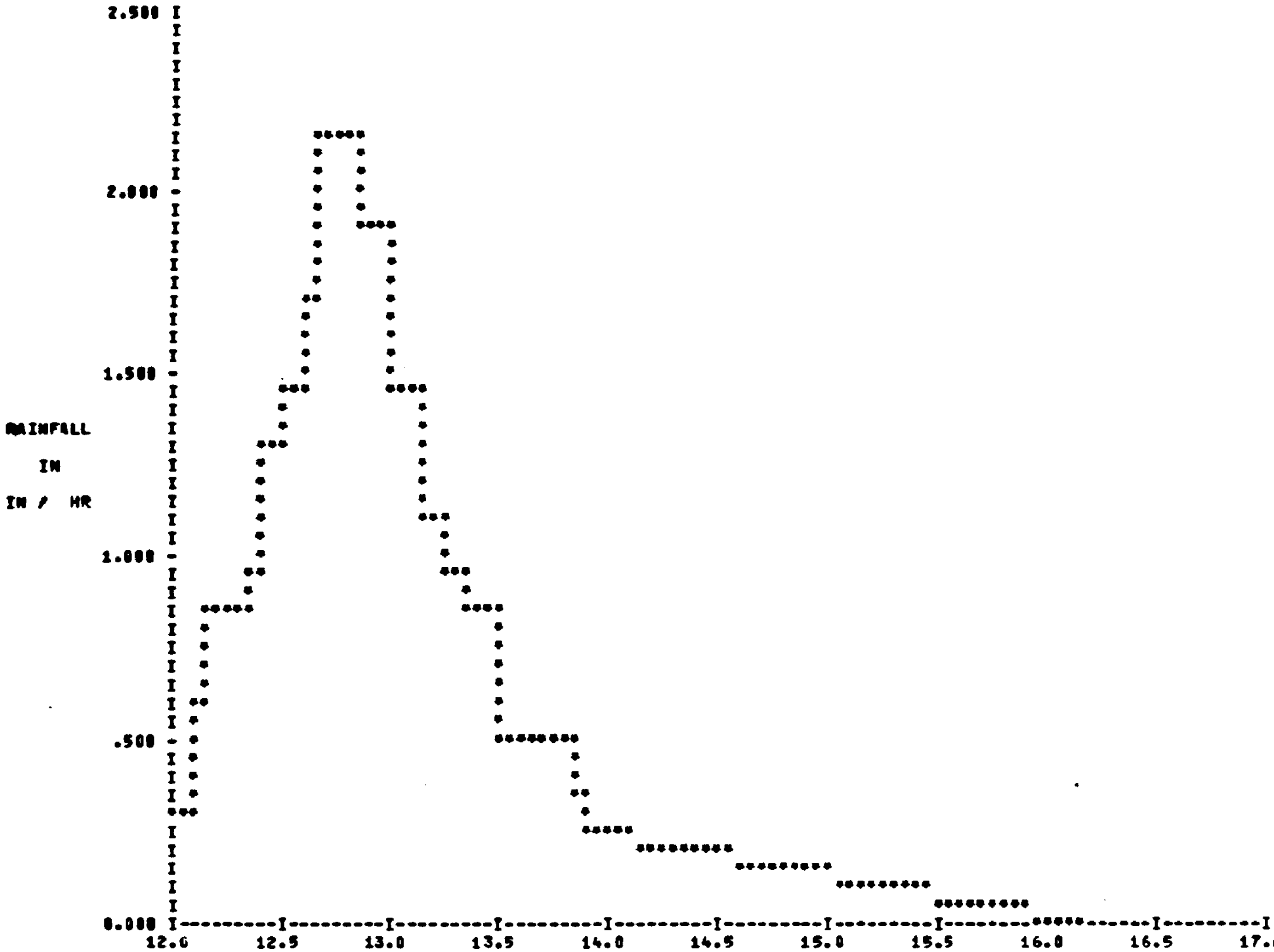
HYDROGRAPHS WILL BE LISTED FOR THE FOLLOWING 1 GUTTERS OR INLETS

8

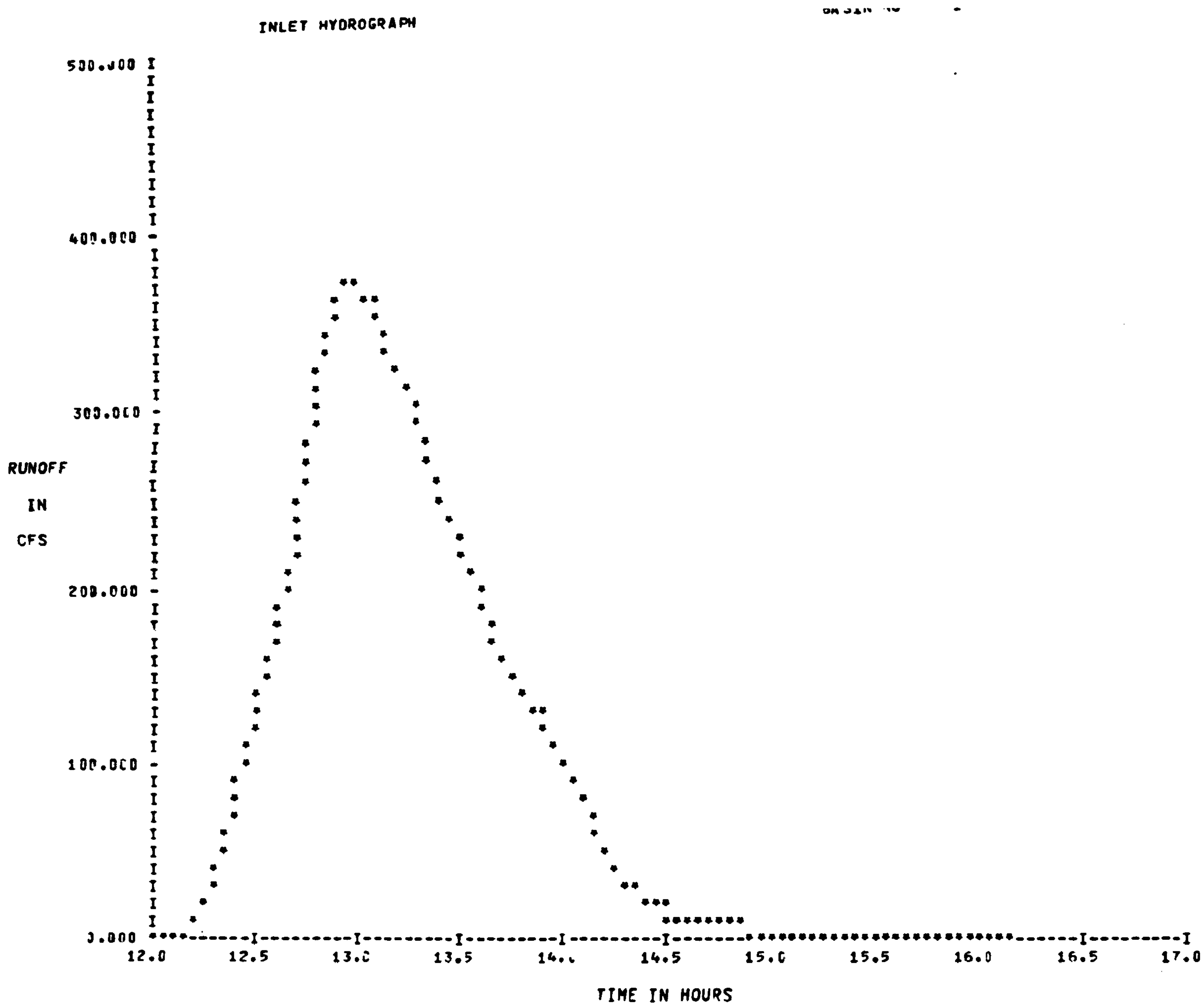
GUTTER	3	SURCHARGED,	SURCHARGE=	1053.	CUFT,FLOW=	24.2 CFS
GUTTER	3	SURCHARGED,	SURCHARGE=	2214.	CUFT,FLOW=	24.2 CFS
GUTTER	3	SURCHARGED,	SURCHARGE=	3419.	CUFT,FLOW=	24.2 CFS
GUTTER	3	SURCHARGED,	SURCHARGE=	4456.	CUFT,FLOW=	24.2 CFS
GUTTER	3	SURCHARGED,	SURCHARGE=	4836.	CUFT,FLOW=	24.2 CFS
GUTTER	3	SURCHARGED,	SURCHARGE=	4712.	CUFT,FLOW=	24.2 CFS
GUTTER	3	SURCHARGED,	SURCHARGE=	4119.	CUFT,FLOW=	24.2 CFS
GUTTER	3	SURCHARGED,	SURCHARGE=	2655.	CUFT,FLOW=	24.2 CFS
GUTTER	3	SURCHARGED,	SURCHARGE=	250.	CUFT,FLOW=	24.2 CFS

TOTAL RAINFALL (CU FT)	2328864.
TOTAL INFILTRATION (CU FT)	820492.
TOTAL GUTTER FLOW AT INLET (CU FT)	1479557.
TOTAL SURFACE STORAGE AT END OF STORM (CU FT)	27460.
ERROR IN CONTINUITY, PERCENTAGE OF RAINFALL,	.05815

RAINFALL HYETOGRAPH BASIN NO 1



RAINGAGE LEGEND 1 = *


INLET HYDROGRAPH
RUNOFF
IN
CFS
500.000
400.000
300.000
200.000
100.000
0.000
12.0
12.5
13.0
13.5
14.0
14.5
15.0
15.5
16.0
16.5
17.0
TIME IN HOURS

RUNOFF BLOCK WORKSHOP
CASE STUDY BASIN, 10 YEAR - 2 HOUR DURATION DESIGN STORM

SUMMARY OF QUANTITY AND QUALITY RESULTS AT LOCATION 8

FLOW IN CFS AND QUALITY IN MG/L (AND COLIF IN MPN/L)

TIME	FLOW	BOD	SUS-S	COLIF	COD	SET-S	NIT	PO4	GREASE
12 5.0	.11	72.74	17.85	.226E+07	242.60	3.72	1.38	.18	1.33
12 10.0	.99	39.89	59.06	.125E+07	125.49	2.09	2.98	.32	.73
12 15.0	5.89	54.40	495.18	.129E+07	90.78	3.13	22.63	2.28	.76
12 20.0	15.57	54.74	871.39	.142E+07	46.56	11.17	39.67	3.99	.83
12 25.0	24.02	39.67	645.11	.146E+07	33.95	32.44	29.71	3.00	.86
12 30.0	33.70	23.35	336.34	.119E+07	27.3[illegible]	57.34	16.00	1.62	.70
12 35.0	43.74	7.38	37.24	.881E+06	20.20	76.30	2.65	.28	.52
12 40.0	52.79	5.56	37.19	.591E+06	13.56	51.23	2.33	.24	.35
12 45.0	65.05	3.91	35.26	.342E+06	7.85	29.66	1.97	.20	.20
12 50.0	77.04	2.89	34.76	.185E+06	4.24	16.00	1.77	.18	.11
12 55.0	77.25	2.38	35.66	.951E+05	2.18	8.24	1.71	.17	.06
13 -.0	74.06	2.02	34.66	.453E+05	1.04	3.93	1.61	.16	.03
13 5.0	66.74	1.84	34.04	.225E+05	.52	1.95	1.56	.16	.01
13 10.0	58.37	1.72	32.98	.117E+05	.27	1.02	1.50	.15	.01
13 15.0	51.24	1.62	31.50	.650E+04	.15	.56	1.42	.14	.00
13 20.0	43.09	1.57	30.96	.399E+04	.09	.35	1.40	.14	.00
13 25.0	37.41	1.48	29.32	.253E+04	.06	.22	1.32	.13	.00
13 30.0	33.08	7.38	147.40	.169E+04	.04	.13	6.63	.66	.00
13 35.0	28.60	9.20	183.93	.125E+04	.03	.08	8.28	.83	.00
13 40.0	22.78	12.03	240.59	.974E+03	.02	.05	10.83	1.08	.00
13 45.0	20.16	12.89	257.76	.726E+03	.02	.03	11.60	1.16	.00
13 50.0	18.89	12.27	245.43	.574E+03	.01	.02	11.04	1.10	.00
13 55.0	17.17	11.88	237.63	.484E+03	.01	.02	10.69	1.07	.00
14 -.0	14.28	12.28	245.56	.429E+03	.01	.02	11.05	1.11	.00
14 5.0	10.07	13.59	271.72	.409E+03	.01	.01	12.23	1.22	.00
14 10.0	6.31	13.57	271.40	.397E+03	.01	.01	12.21	1.22	.00
14 15.0	4.27	10.33	206.64	.354E+03	.01	.00	9.30	.93	.00
14 20.0	3.05	7.47	149.31	.328E+03	.01	.00	6.72	.67	.00
14 25.0	2.27	5.33	106.62	.318E+03	.01	.00	4.80	.48	.00
14 30.0	1.74	3.82	76.45	.298E+03	.01	.00	3.44	.34	.00
14 35.0	1.37	2.77	55.40	.288E+03	.01	.00	2.49	.25	.00
14 40.0	1.10	2.04	40.69	.281E+03	.01	.00	1.83	.18	.00
14 45.0	.90	1.52	30.33	.275E+03	.01	.00	1.36	.14	.00
14 50.0	.75	1.15	22.94	.270E+03	.01	.00	1.03	.10	.00
14 55.0	.63	.88	17.61	.267E+03	.01	.00	.79	.08	.00
15 -.0	.54	.69	13.71	.263E+03	.01	.00	.62	.06	.00
15 5.0	.46	.54	10.82	.260E+03	.01	.00	.49	.05	.00
15 10.0	.40	.43	8.65	.258E+03	.01	.00	.39	.04	.00
15 15.0	.35	.35	7.00	.256E+03	.01	.00	.31	.03	.00
15 20.0	.31	.29	5.73	.254E+03	.01	.00	.26	.03	.00
15 25.0	.27	.24	4.74	.252E+03	.01	.00	.21	.02	.00
15 30.0	.24	.20	3.97	.251E+03	.01	.00	.18	.02	.00
15 35.0	.22	.17	3.36	.250E+03	.01	.00	.15	.02	.00
15 40.0	.20	.14	2.86	.249E+03	.01	.00	.13	.01	.00
15 45.0	.18	.12	2.47	.248E+03	.01	.00	.11	.01	.00
15 50.0	.16	.11	2.14	.247E+03	.01	.00	.10	.01	.00
15 55.0	.15	.09	1.87	.246E+03	.01	.00	.08	.01	.00
16 -.0	.13	.08	1.65	.245E+03	.01	.00	.07	.01	.00
16 5.0	.12	.07	1.46	.244E+03	.01	.00	.07	.01	.00
16 10.0	.11	.07	1.30	.243E+03	.01	.00	.06	.01	.00

PREPARATION OF TRANSPORT BLOCK DATA

Sewer System Discretization

The Transport Block models the actual sewer system as a series of links (conduit elements) and nodes (non-conduit elements). Flow and area data for 13 standard conduit shapes are included in the program and the user has the option of supplying data for up to three additional shapes. Non-conduit elements include manholes, lift-stations, flow dividers, storage elements and backwater elements.

A maximum of 160 sewer elements can be modeled in the Transport Block. Because all conduit elements modeled must be joined by non-conduit elements, the practical limit of the number of conduit elements that can be modeled is 80. In most practical applications, not all of the actual sewer system elements can be modeled. The user has the choice of aggregating sewer elements or using more than one simulation to model the transport system and then aggregating the results using Subroutine COMBINE of SWMM. Unless the system is large or complex, the best approach is to combine elements and reduce the total number modeled to below the permissible maximum.

The approximate extent of the transport system has been established previously by selecting the location of manholes through which flows will be transferred from the Runoff Block to the Transport Block. Discretization of the sewer system involves dividing the system into a series of elements that will adequately protray all significant changes in conduit geometry, roughness and slope, and allow for any flow control structures. Constrictions in the sewer system, rough elements or elements with flat slopes should be modeled as separate elements because these are the locations where surcharging is likely to take place. Conduits having similar characteristics can be modeled as one element, provided that the flow capacity of the aggregate element is approximately equal to the largest capacity among the grouped elements. Flowrate, Q, as given below by the Modified Manning's Equation is most sensitive to changes in area (geometry), less sensitive to roughness and least sensitive to slope.

$$Q = \frac{1.49}{n} A R_h^{2/3} (s_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x})^{1/2}$$

$$= \frac{1.49}{n} \frac{A^{5/3}}{P_w^{2/3}} (s_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x})^{1/2} \qquad (8)$$

where Q = flowrate (cfs)

n = Manning's roughness coefficient

A = cross sectional area of flow (ft^2)

R_h = hydraulic radius = A/P_w (ft)

P_w = wetted perimeter (ft)

s_o = invert slope (ft/ft)

$\frac{\partial y}{\partial x}$ and $\frac{v}{g}\frac{\partial v}{\partial x}$: Terms from the St. Venant Equation added to Manning's Equation to correct for spacially varied flow.

The sensitivity of discharge to these parameters should be taken into account when deciding which elements to combine.

In order to account for all subcatchment runoff, all manholes through which flows will be transferred from the Runoff Block have to be modeled.

Transport Block Numbering System

After selecting the number of model elements for the sewer system, and marking the ends of these elements on the sewer map or sewer plates, elements should be numbered in a manner similar to that used in the Runoff Block. Inlet manholes through which surface runoff will enter the modeled transport system must be assigned the same number as used in the Runoff Block. Those non-conduit elements through which flows can be transferred from the Transport Block to other blocks must be numbered 1-100. Therefore, it is recommended that the numbers 1 to 100 be reserved for all non-conduit elements to enable use of any as a transfer point. Conduit elements could be assigned numbers ranging from 401 to 600 to avoid any confusion with gutters or pipes modeled in the Runoff Block. Figure 13 shows this numbering scheme applied to a typical transport system.

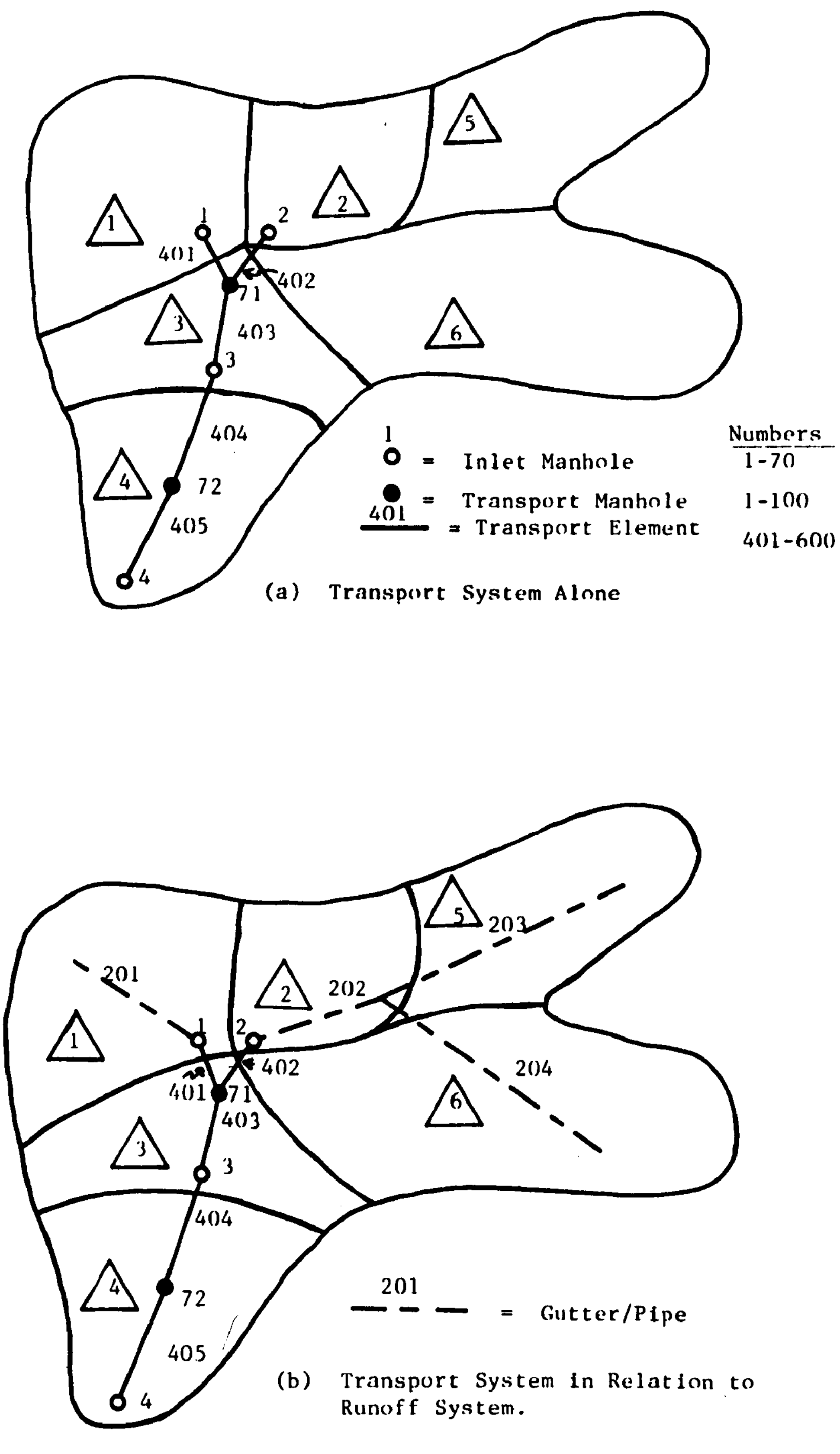


FIGURE 13. TYPICAL TRANSPORT NETWORK ARRANGEMENT

Compilation of Sewer Element Data

Once the sewer system has been divided into elements, the compilation of the required data follows logically. Input data requirements for conduit elements are detailed in Table 6-2 (p. 120) and card group 15 of Table 6-6 (pp. 160-161), User's Manual (Reference 6). Generally these data include geometric dimensions, invert slope and Manning's roughness coefficient. A current sewer master plan, if available, should contain most of the data needed for conduit elements. If not available, individual sewer plates will have to be assembled. In most cases, some data will have to be obtained and/or verified by on-site inspection.

Invert slope is required in ft/100 ft which is the same as percent slope. Manning's roughness coefficients are best estimated from in-system photographs or on-site inspection. Table 5 is a partial listing of accepted Manning's roughness coefficients for various materials. Reference 5 gives a more complete listing on pages 110-113.

Input data required for non-conduit elements are outlined in Table 6-3 (p. 125) of the User's Manual. Additional data are required for storage elements, as outlined in pages 162-166 of the User's Manual. Data sources for non-conduit elements are generally the same as those for conduit elements.

Table 7, provided with the Transport Block case study materials, will be a useful aid in organizing sewer element data for coding form preparation. It is desirable to arrange entries within this table in a reverse tree order (from extremities of system to outfall) to assist in the correct assembly of the sewer network.

Infiltration Model

Modeling of infiltration into the sewer system requires simultaneous records of rainfall, water table and sewer flow data for several weeks. Some municipalities may already have compiled this data as part of infiltration-inflow studies; where unavailable, field studies should be undertaken at the outset of the storm water management study. Descriptions of the infiltration model and the data preparation required are contained on pages 133-142 of the User's Manual. Organization of infiltration data for coding form preparation is given in card groups 31-33, Table 6-6 (pp. 167-168) of the User's Manual.

Dry Weather Flow Model

Modeling of a combined sewer system requires data for dry weather (sewage) flow. Estimates of dry weather flow and pollutant concentration parameters can be obtained from standard engineering handbooks, water usage data or sewage flow data for the basin. Measured data can be taken at the sewage treatment plant or preferrably gathered at several points within the system. Sewage treatment plant records should also contain data on industrial process flows originating within the basin. The U.S. Bureau

Table 5. TYPICAL VALUES OF MANNING'S n

A. Closed Conduits Flowing Partly Full	
1. Cast Iron	
a. Coated	0.013
b. Uncoated	0.014
2. Corrugated Metal	
a. Sub drain	0.019
b. Storm drain	0.024
3. Concrete	
a. Culvert w/bends, connections and some debris	0.013
b. Unfinished concrete	0.014
c. Rubble Masonry	0.025
B. Lined or Built up Channels	
1. Concrete	
a. Trowel finish	0.013
b. Unfinished	0.017
2. Brick in cement mortar	0.015
3. Asphalt	
a. Smooth	0.013
b. Rough	0.016
C. Excavated Channels	
1. Clean earth (straight channel)	0.022
2. Earth with weeds (winding channel)	0.030
D. Natural Streams	
1. Clean and straight	0.030
2. Weedy reaches, deep pools	0.100

of Census tract information will contain data on population distribution, family income and the number and age of dwelling units. Land use can be determined as in the Runoff Block. Subareas modeled in Subroutine FILTH do not have to coincide with runoff subcatchments.

Obviously, the best data available should be used for input into Subroutine FILTH. The additional time and expense of gathering field data within the system may be worthwhile if the first flush effects of combined sewers are to be modeled.

The dry weather flow model is discussed in more detail on pages 142-153 of the User's Manual. Card groups 34-45, Table 6-6 (pp. 168-175) of the User's Manual detail the organization of dry weather flow data for coding form preparation. Note that card groups 42-44 are only used when measured sewage flow data are input. Table 8 will aid in the preparation of subarea data.

PREPARATION OF CODING FORMS

Table 6-6, pp. 154-175 of the User's Manual details the proper organization of input data for coding form preparation. (Coding forms are supplied with the case study materials.) All I Format numbers must be right justified and all numbers must be contained within the field width assigned to them. Careful coding form preparation will greatly reduce input data errors and facilitiate card punching.

At this point, a connectivity diagram of the transport system should be constructed to ensure a correctly numbered sewer system. Using the data from the computer coding forms, the network should be reconstructed by starting at the outfall. All elements should be checked to insure proper connection. Elements at the extreme tree branches should all be inlet manholes. If errors exist, the problem should be located by working backward in the data preparation sequence.

CASE STUDY MATERIALS

General Discussion

A 25-year-old combined sewer system is to be modeled. Infiltration-inflow studies have been conducted and will be included in the modeling.

A sewer system map for the case study basin has been provided (Figure 2). This map gives lengths, dimensions and slopes of sewer elements. All conduits are concrete and in good condition unless otherwise denoted on the sewer map. Minor pipes and house connections are not shown on the sewer map; however, it can be assumed that all buildings are connected to the sewer system. The dimension given for egg-shaped conduits is the height.

Descritize the sewer system and prepare all necessary input data. For illustrative purposes, the number of elements modeled should be limited to 30.

Tables 6, 7, and 8 should be of some assistance in data preparation. Inlet manholes to be modeled in Transport are available from the completed Runoff Block case study. Additional data is supplied for infiltration and dry weather flow. Some execution control parameters are also provided.

Infiltration Data

The following infiltration data are available concerning the case study basin.

Base Dry Weather Infiltration:	80 gal/min
Groundwater Infiltration:	0.0
Rainwater Infiltration:	125 gal/min
Day of Year of Estimate:	280
Peak Residual Moisture:	100 gal/min
Average Distance Between Joints:	7.0 ft
Monthly Degree Days:	Use data for Cleveland, Ohio from Table A-1, User's Manual

Dry Weather Flow Data

The Storm starts on Monday at 12:00 AM. Dry weather flow quantity and quality have been measured at the point of exit from the case study basin. The average flow measured was 0.89 MGD with average concentrations of 360 mg/l BOD_5, 250 mg/l suspended solids and 2.5 x 10^7 MPN/100 ml total coliforms. Daily variations of the data recorded are shown in Figure 14 and hourly variations in Table 9. In addition, industrial waste contributions are indicated in Table 10.

Demographic and land use data for the remainder of the basin are as follows:

1. Average number of people/household: 4.0

2. The two apartment complexes have a total of 480 units with an average occupancy of 2.9 people/unit.

3. The motel has 144 units, each occupied by an average of 2.0 people during the hours of 4 PM-8 AM daily.

4. The shopping center parking lot is designed for 1250 cars and a 50% average usage rate. Hours of operation are 10 AM-10 PM daily. Studies have determined that the average number of people in each car using the lot is 2.

Table 6. TRANSPORT BLOCK DATA PREPARATION CHECK LIST

1. Sewer System Descritization
2. Sewer Element Numbering
3. Compilation of Sewer Element Data
    a. Type
    b. Length
    c. Dimensions
    d. Invert Slope
    e. Manning's Roughness
4. Internal Storage Data Preparation
5. Infiltration Model
    a. Data Gathering
    b. Data Preparation
6. Dry Weather Flow Model
    a. Daily and Hourly Correction Factors
    b. Average Dry Weather Flow Derived from
        (1) Monitoring points within study area
        (2) Sewage treatment plant data
        (3) Water usage data
        (4) Default values
    c. Subarea Designations
    d. Census Tract Data
    e. Garbage Grinder Data
    f. Subarea Data Preparation
7. Coding Form Preparation
8. Connectivity Diagram

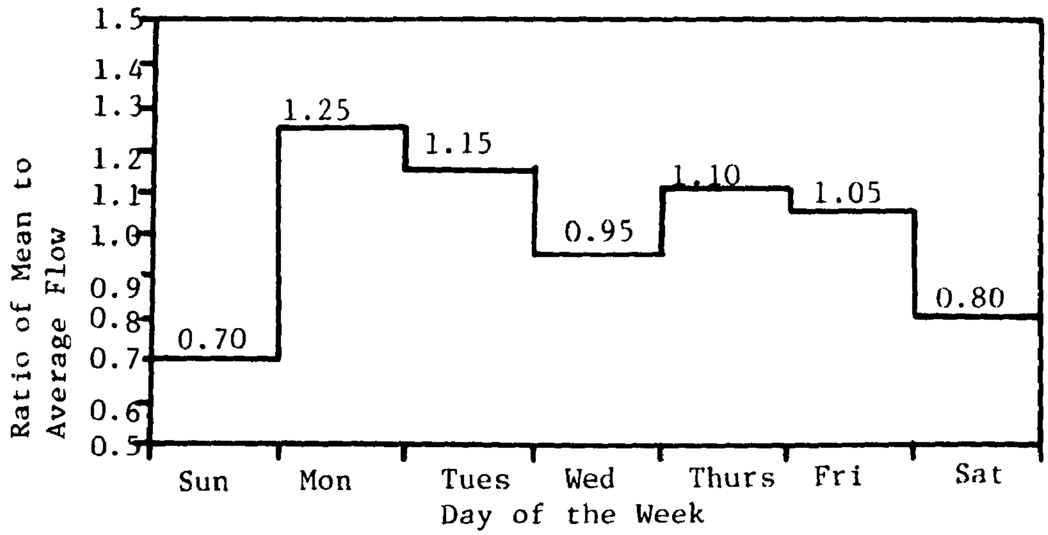
Table 7. SEWER ELEMENT DATA

Element Number	Upstream Element 1	Upstream Element 2	Upstream Element 3	Class	Length (ft)	First Dimension (ft)	Invert Slope (ft/100ft)	Manning's n	Second Dimension (ft)	Number of Barrels	Third Dimension (ft)

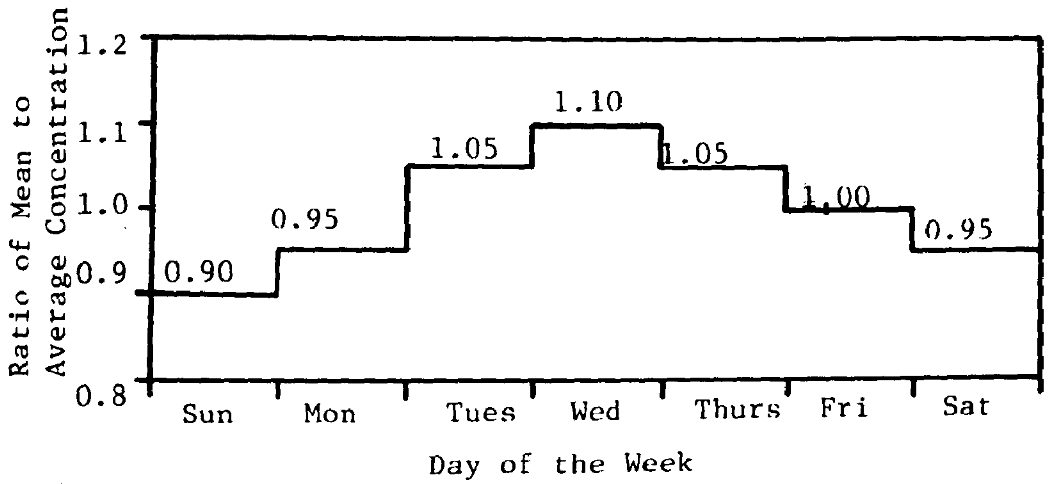
Table 8. DRY WEATHER FLOW SUBAREA DATA

NOTE: To conserve space, variable names are used for headings. Refer to Card Group 45, Table 6-6, pp. 173-175 of the <u>User's Manual</u> for further explanation.

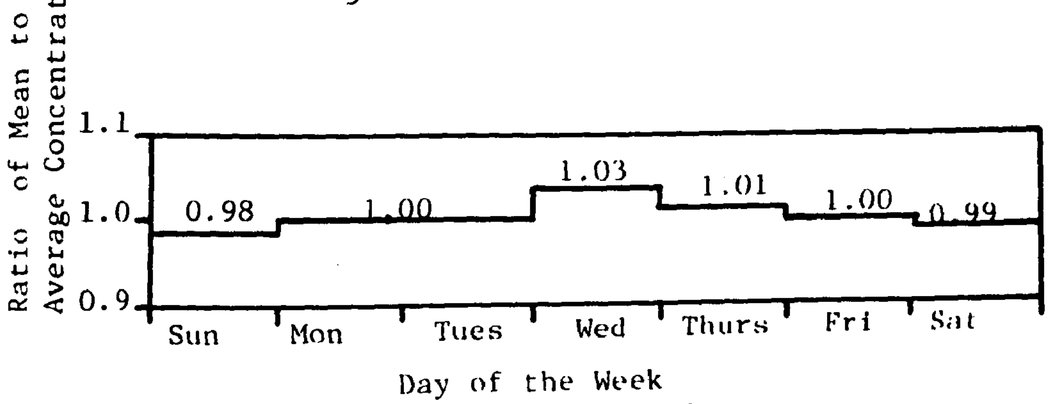
KNUM	INPUT	KLAND	METHOD	KUNIT	WATER	PRICE	SEWAGE	A-SUB	POPDEN	DWLINGS	FAMILY	VALUE	PCGG	SAQPF	SABPF	SASPF	XINCOM	MSUBT



(a) Flow Variation



(b) BOD_5 Variation



(c) Suspended Solids Variation

FIGURE 14, DAILY FLOW AND CONCENTRATION VARIATION.

Table 9. HOURLY FLOW AND CONCENTRATION VARIATION

	Hour Period	Ratio of Hourly Mean to Daily Average Flow or Concentration			
		Flow	BOD_5	SS	COLIF
AM	M-1	0.74	0.85	1.05	1.10
	1-2	0.67	0.71	1.05	0.64
	2-3	0.63	0.60	1.10	0.45
	3-4	0.59	0.41	0.50	0.87
	4-5	0.54	0.46	0.66	0.54
	5-6	0.56	0.49	1.33	0.48
	6-7	0.67	0.72	1.10	1.29
	7-8	0.96	0.87	0.88	1.18
	8-9	1.42	0.77	1.03	1.37
	9-10	1.19	1.57	0.91	1.49
	10-11	1.20	1.02	0.66	1.30
	11-N	1.15	0.87	0.63	1.12
PM	N-1	1.17	0.91	0.94	0.89
	1-2	1.11	0.94	0.94	0.58
	2-3	1.08	1.07	1.05	0.45
	3-4	1.15	1.07	1.05	0.67
	4-5	1.21	1.14	1.16	0.96
	5-6	1.23	0.99	0.94	1.18
	6-7	1.25	1.45	1.33	0.84
	7-8	1.21	1.16	1.22	1.01
	8-9	1.17	1.55	1.44	2.82
	9-10	1.15	1.29	1.10	1.77
	10-11	0.88	0.99	0.88	0.84
	11-M	1.07	1.60	1.05	0.71

Table 10. INDUSTRIAL WASTE CONTRIBUTIONS TO DRY WEATHER FLOW

Industry	Process Waste Characteristics* Average Daily Flow cfs	Average Daily BOD mg/1	Average Daily SS mg/1	Number of Employees	Number of Shifts
Soft Drink Bottling Plant	.340	430	360	200	2
Bakery	.155	480	360	150	3
Electronics Plant	-	-	-	600	2

*Sanitary wastes are not included.

5. Family income, dwelling value and garbage grinder usage for selected areas of the basin are:

   a. School, West, Franklin and Wells Streets
      Family Income: \$20,000/yr
      Average Dwelling Value: \$50,000
      Garbage Grinder Usage: 100%

   b. Apartment Complexes
      Family Income: \$10,000/yr
      Average Dwelling Value: \$9,000
      Garbage Grinder Usage: 50%

   c. Area bounded by Oak, Davis and Elm Streets
      Family Income: \$8,000/yr
      Average Dwelling Value: \$20,000
      Garbage Grinder Usage: 10%

   d. Area bounded by High, East, Elm and Oak Streets
      Family Income: \$11,000/yr
      Average Dwelling Value: \$25,000
      Garbage Grinder Usage: 30%

Execution Control Parameters

Additional data required for job control are given in Table 11. In cases where no data are given, the default value should be used.

CALIBRATION OF MODEL

Calibration of the model is best accomplished utilizing a small, representative test basin within the urban area to be studied. A comparison of predicted with measured results for the test basin (for several storms) will indicate which model parameters can be adjusted so that model output will more closely agree with measured results. Test basin results can also be used to choose the best descritization scheme. Information gained from the calibrated model can then be used in modeling the remainder of the urban area. Verification of the model for the entire urban area should be accomplished at some point (preferably an outfall) other than where the model was calibrated.

Most of the parameters that can be changed for calibration are contained in the Runoff Block. However, to more accurately portray routing delay through the sewer system, it is recommended that calibration data be gathered at a point downstream from several subcatchments after routing flows through the Transport Block.

The model should first be calibrated for flow. Accurate field data concerning flow can be gathered at points within the system. If field data on quality exist, an attempt can be made to calibrate the model for quality also. The state of the art of quality modeling is such that an accurate

TABLE 11. EXECUTION CONTROL PARAMETERS

Card Group	Variable	Value
1	NKLASS	0
	KPRINT	0
12	NDT	50
	NPRINT	0
	NPOLL	3
13	DT	300
	DWDAYS	14
14	NCNTRL	0
	NINFIL	1
	NFILTH	1
	JPRINT	1
	JPLOT	0
	NDESN	1

calibration is difficult to obtain. This discussion will concentrate on flow calibration.

There are no set rules as to which program parameters should be adjusted during flow calibration. A great deal will depend on the individual situation. Generally, measurable data such as area and ground slope should not be varied. Although measurable, a reliable estimate of percent impervious area is difficult to obtain because of possible interactions between pervious and impervious areas. Model output has been found to be highly sensitive to the percent impervious area modeled (2). Therefore, percent impervious area modeled should be considered as an adjustable parameter within limits. Other subcatchment flow parameters that may be adjusted are:

1. Resistance factor for impervious areas

2. Resistance factor for pervious areas

3. Surface storage on impervious areas

4. Surface storage on pervious areas

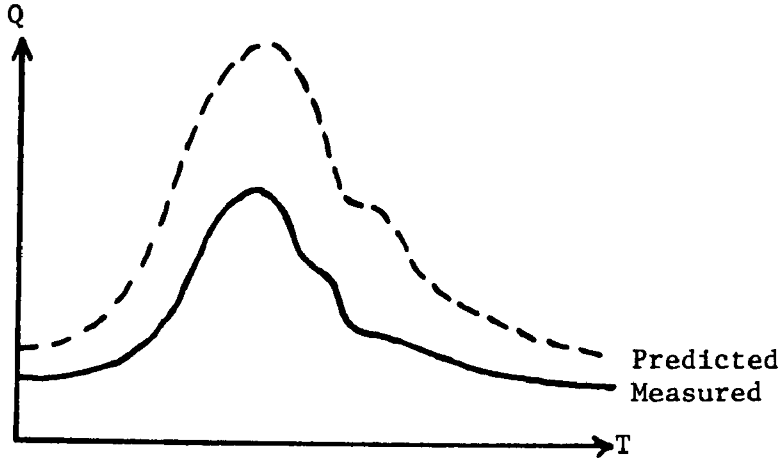
5. Maximum rate of infiltration

6. Minimum rate of infiltration

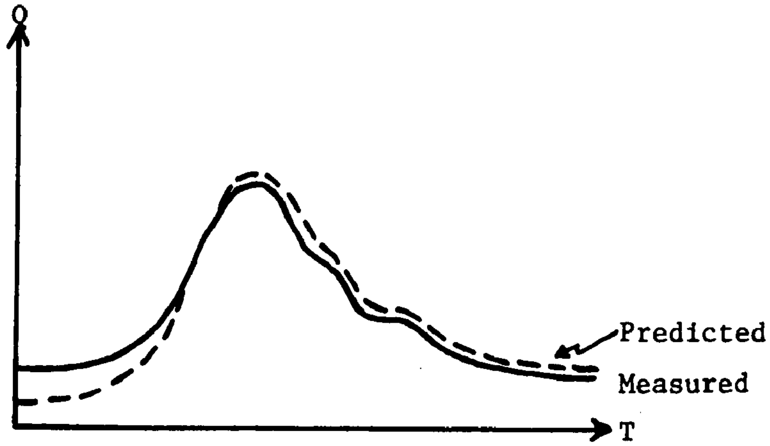
7. Decay rate of infiltration

Changes in the pervious area resistance factor and surface storage depth will have almost no effect on the outflow hydrograph. Likewise, changes in the impervious area resistance factors and surface storage depths have minimal effect on the volume and routing of the flow. Infiltration rates, when properly adjusted, can have significant impact on output (6). Figures 15 (a-d) illustrate the response of the model to several of these parameters. Figure 15 (e) is an example of incorrect routing time which may result from an inaccurately modeled sewer system.

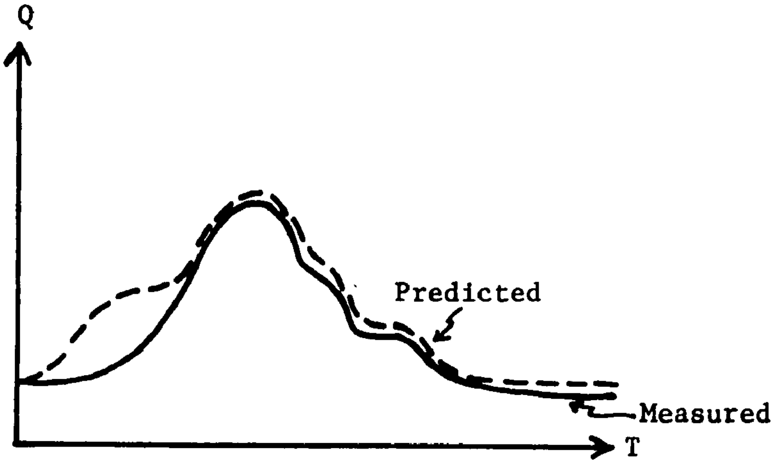
If adjustment of input parameters does not result in a good calibration, the original descritization scheme should be re-evaluated.



(a) Percent Impervious Area Too High

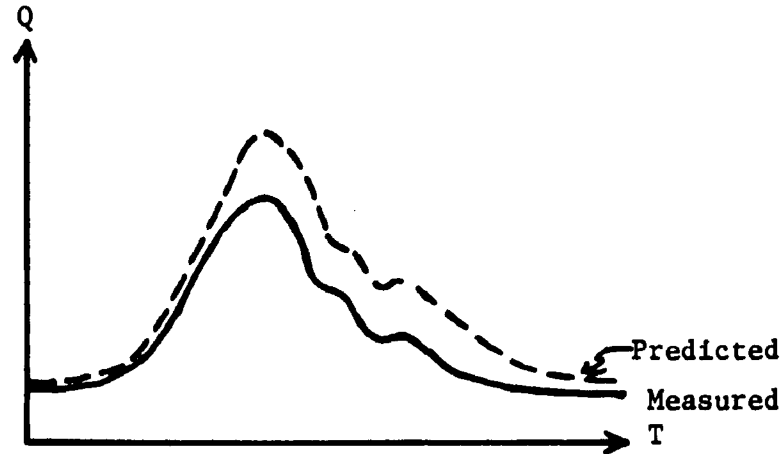


(b) Initial Rate of Infiltration Too High

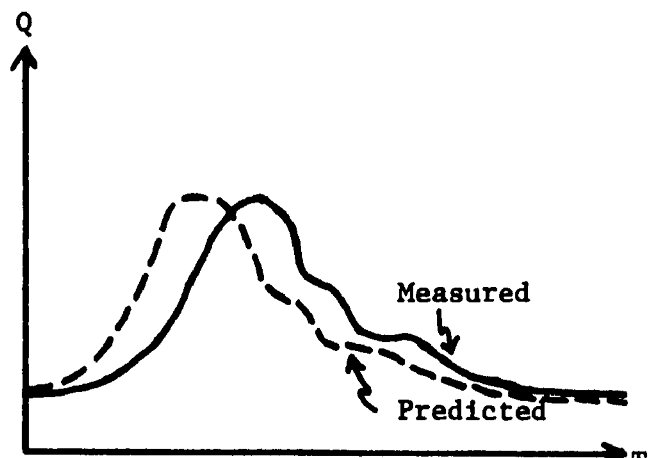


(c) Decay Rate of Infiltration Too High

FIGURE 15. PROGRAM RESPONSE TO INPUT PARAMETERS.



(d) Minimum Rate of Infiltration Too Low



(e) Inadequate Routing Delay Modeled

FIGURE 15, CONTINUED.

SAMPLE DATA INPUT AND PROGRAM OUTPUT: TRANSPORT BLOCK

A total of 27 elements were used to model the case study basin sewer system. Pertinent input data and program output are provided on the following pages. The specific items of information included are:

Schematic Showing Descritized
Sewer System (Figure 16)

Transport Block Card Data

Transport Block Program Output

Note that surcharging occurs at modeled elements 102 and 107. Surcharge conditions at these locations are consistent with the data given in the case study materials and indicate areas of the sewer system that may require modification.

TRANSPORT SEWER MAP

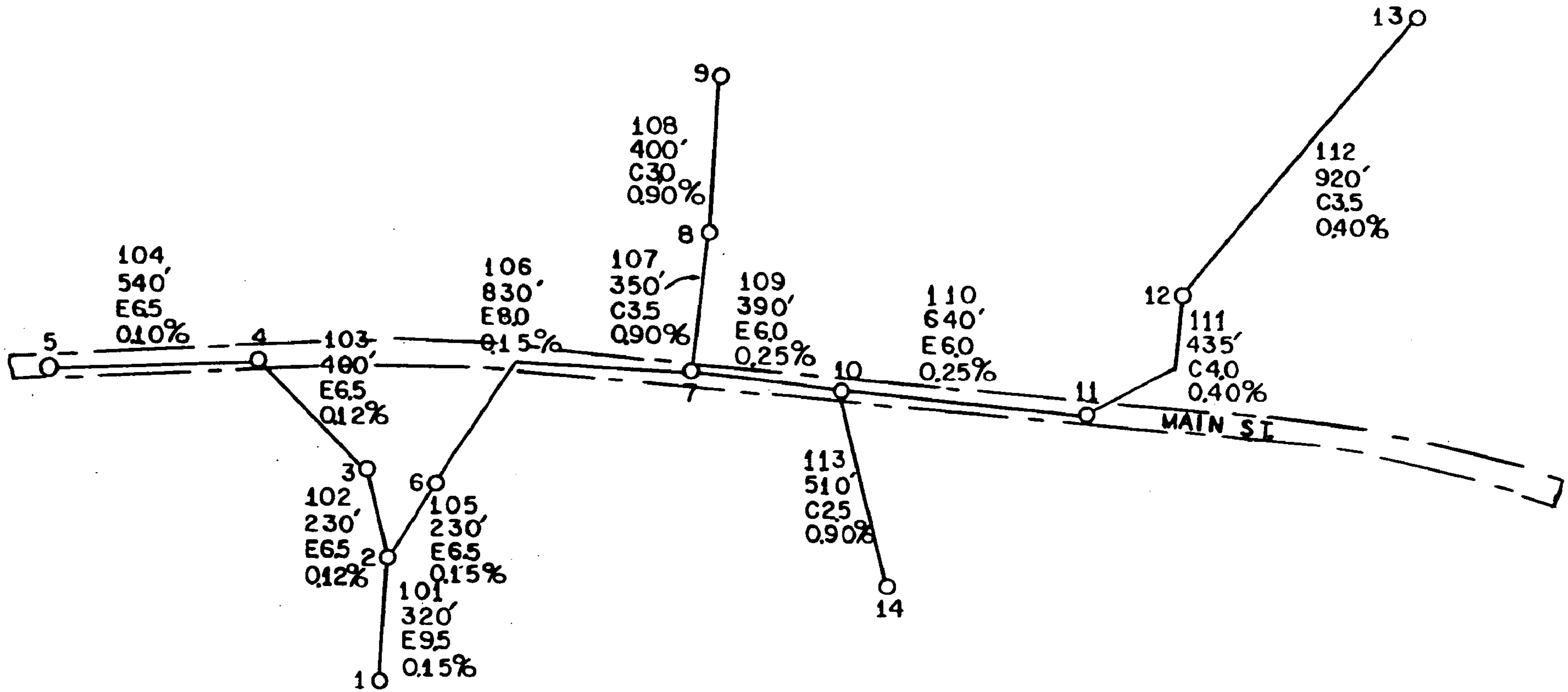


FIGURE 16. SCHEMATIC SHOWING DESCRITIZED SEWER SYSTEM.

TRANSPORT BLOCK CARD DATA

CARD GROUP

```
TRANSPORT
    0     0                                                     1
   TRANSPORT     BLOCK   WORKSHOP                               11
   27    50      9     1     1     1     0     3     4          12
       300.      0.0001         14.                             13
    0     1      1     1     0     0                            14
   1  101    0     0    16
 101    2    0     0     3    320.     9.5      .15     .013
   2  102  105     0    16
 102    3    0     0     3    230.     6.5      .12     .013
   3  103    0     0    16
 103    4    0     0     3    400.     6.5      .12     .013
   4  104    0     0    16
 104    5    0     0     3    540.     6.5      .10     .013
   5    0    0     0    16
 105    6    0     0     3    230.     8.5      .15     .013
   6  106    0     0    16
 106    7    0     0     3    830.     8.0      .15     .013
   7  109  107     0    16
 107    8    0     0     1    350.     3.5      .90     .013
   8  108    0     0    16
 108    9    0     0     1    400.     3.0      .90     .013    15


   9    0    0     0    16
 109   10    0     0     3    390.     6.0      .25     .013
  10  113  110     0    16
 113   14    0     0     1    510.     2.5      .90     .013
  14    0    0     0    16
 110   11    0     0     3    640.     6.0      .25     .013
  11  111    0     0    16
 111   12    0     0     1    435.     4.0      .40     .013
  12  112    0     0    16
 112   13    0     0     1    920.     3.5      .40     .013
  13    0    0     0    16
    1                                                           27
    8                                                           29
    1                                                           30
```

```
    80.      0.    125.                                                                    -31
  280     100.    7.0                                                                      -32
     0     9   60   311   636   995 1101   977   846   510   223    49                     -33
      0.70       1.25       1.15       0.95       1.10       1.05       0.80               -34
      0.99       0.95       1.05       1.10       1.05       1.00       0.95               -35
      0.98       1.00       1.00       1.03       1.01       1.00       0.99               -36
      0.74       0.67       0.63       0.59       0.54       0.56       0.67       0.96
      1.42       1.19       1.20       1.15       1.17       1.11       1.08       1.15    -37
      1.21       1.23       1.25       1.21       1.17       1.15       0.88       1.07
      0.85       0.71       0.60       0.41       0.46       0.49       0.72       0.87
      0.77       1.57       1.02       0.87       0.91       0.94       1.07       1.07    -38
      1.14       0.99       1.45       1.16       1.55       1.29       0.99       1.60
      1.05       1.05       1.10       0.50       0.66       1.33       1.10       0.88
      1.03       0.91       0.66       0.63       0.94       0.94       1.05       1.05    -39
      1.16       0.94       1.33       1.22       1.44       1.10       0.88       1.05
      1.10       0.64       0.45       0.87       0.54       0.48       1.29       1.18
      1.37       1.49       1.30       1.12       0.89       0.58       0.45       0.67    -40
      0.96       1.18       0.84       1.01       2.82       1.77       0.84       0.71
   10     1      0     2    12    00                22.                                    -41
     1.377      360.       250.      2.5E+7                                                -42
   304.9     43.0     55.9     23.4     56.7      0.0     39.2    125.9                    -43
  1 13310 240.   32.       10.7    0.                          0.
  2 1112                    4.0   18.   18.   4.0   20.  10.                       8.
  3 1412                   23.4    5.   28.   4.0   50. 100.                      20.
  4 1222                   17.7   79.  480.   2.9    9.  50.                      10.
  5  912                   19.4   21.  100.   4.0   20.  10.                       8.   -45
  6  912                   15.6   13.   52.   4.0   25.  30.                      11.
  7  832                   30.4    0.                      0.  .082 200. 200.
  8  632                    7.0    0.                      0.  .035 200. 200.
  9  542                   43.0    0.                      0.  .585 408. 336.
 10  532                    7.8    0.                      0.  .020 200. 200.
GRAPH
    9     1     3
GRAPH OF OUTPUT FROM TRANSPORT
           TIME IN HOURS
  FLOW       IN       CFS
   BOD    LBS/MIN
   SS     LBS/MIN
COLIFORM MPN/MIN
ENDPROGRAM
```

TRANSPORT BLOCK WORKSHOP

* * * * * ELEMENT LINKAGES AND COMPUTATION SEQUENCE * * * * *

ELEMENT NO. ZERO IS GIVEN INTERNAL NO. = NO. ELEMENTS + 1 = 28

EXTERNAL ELEMENT NUMBER	INTERNAL ELEMENT NUMBER	TYPE	DESCRIPTION	UPSTREAM ELEMENTS (EXTERNAL NOS.) 1	2	3
1	1	16	MANHOLE	101	0	0
101	2	3	EGG-SHAPED	2	0	0
2	3	16	MANHOLE	102	105	0
102	4	3	EGG-SHAPED	3	0	0
3	5	16	MANHOLE	103	0	0
103	6	3	EGG-SHAPED	4	0	0
4	7	16	MANHOLE	104	0	0
104	8	3	EGG-SHAPED	5	0	0
5	9	16	MANHOLE	0	0	0
105	10	3	EGG-SHAPED	6	0	0
6	11	16	MANHOLE	106	0	0
106	12	3	EGG-SHAPED	7	0	0
7	13	16	MANHOLE	109	107	0
107	14	1	CIRCULAR	8	0	0
8	15	16	MANHOLE	108	0	0
108	16	1	CIRCULAR	9	0	0
9	17	16	MANHOLE	0	0	0
109	18	3	EGG-SHAPED	10	0	0
10	19	16	MANHOLE	113	110	0
113	20	1	CIRCULAR	14	0	0
14	21	16	MANHOLE	0	0	0
110	22	3	EGG-SHAPED	11	0	0
11	23	16	MANHOLE	111	0	0
111	24	1	CIRCULAR	12	0	0
12	25	16	MANHOLE	112	0	0
112	26	1	CIRCULAR	13	0	0
13	27	16	MANHOLE	0	0	0

ORDER OF COMPUTATIONS AT EACH TIME STEP (PROCEEDING DOWNSTREAM)

COMPUTATION SEQUENCE	EXTERNAL NUMBER	INTERNAL NUMBER	INTERNAL UPSTREAM ELEMENT NUMBERS		
1	5	9	28	28	28
2	104	8	9	28	28
3	4	7	8	28	28
4	103	6	7	28	28
5	3	5	6	28	28
6	102	4	5	28	28
7	9	17	28	28	28
8	108	16	17	28	28
9	8	15	16	28	28
10	107	14	15	28	28
11	14	21	28	28	28
12	113	20	21	28	28
13	13	27	28	28	28
14	112	26	27	28	28
15	12	25	26	28	28
16	111	24	25	28	28
17	11	23	24	28	28
18	110	22	23	28	28
19	10	19	20	22	28
20	109	18	19	28	28
21	7	13	18	14	28
22	106	12	13	28	28
23	6	11	12	28	28
24	105	10	11	28	28
25	2	3	4	10	28
26	101	2	3	28	28
27	1	1	2	28	28

RUNOFF BLOCK WORKSHOP

NUMBER OF ELEMENTS= 27
NUMBER OF TIME INT= 50
TIME INTERVAL= 300.0 SECONDS.

ELEMENT PARAMETERS

EXT. ELE. NUM.	TYPE	DESCRIPTION	SLOPE (FT/FT)	DISTANCE (FT)	MANNING ROUGHNESS	GEOM1 (FT)	GEOM2 (FT)	GEOM3 (FT)	NUMBER OF BARRELS	AFULL (SQ.FT)	QFULL (CFS)	QMAX (CFS)	SUPER-CRITICAL FLOW WHEN LESS THAN 95 FULL'
1	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
101	3	EGG-SHAPED	.0015	320.0	.0130	9.5	-0.0	-0.0	1.0	46.073	306.480	326.401	NO
2	15	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
102	3	EGG-SHAPED	.0012	230.0	.0130	6.5	-0.0	-0.0	1.0	21.569	99.644	106.121	NO
3	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
103	3	EGG-SHAPED	.0012	400.0	.0130	6.5	-0.0	-0.0	1.0	21.569	99.644	106.121	NO
4	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
104	3	EGG-SHAPED	.0010	540.0	.0130	6.5	-0.0	-0.0	1.0	21.569	90.963	96.875	NO
5	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
105	3	EGG-SHAPED	.0015	230.0	.0130	8.5	-0.0	-0.0	1.0	36.884	227.819	242.627	NO
6	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
106	3	EGG-SHAPED	.0015	830.0	.0130	8.0	-0.0	-0.0	1.0	32.672	193.811	206.409	NO
7	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
107	1	CIRCULAR	.0090	350.0	.0130	3.5	-0.0	-0.0	1.0	9.621	95.704	103.360	NO
8	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
108	1	CIRCULAR	.0090	400.0	.0130	3.0	-0.0	-0.0	1.0	7.069	63.446	68.522	NO
9	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
109	3	EGG-SHAPED	.0025	390.0	.0130	6.0	-0.0	-0.0	1.0	18.378	116.181	123.732	NO
10	15	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
113	1	CIRCULAR	.0090	510.0	.0130	2.5	-0.0	-0.0	1.0	4.909	39.017	42.138	NO
14	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
110	3	EGG-SHAPED	.0025	640.0	.0130	6.0	-0.0	-0.0	1.0	18.378	116.181	123.732	NO
11	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
111	1	CIRCULAR	.0040	435.0	.0130	4.0	-0.0	-0.0	1.0	12.566	91.393	98.380	NO
12	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	
112	1	CIRCULAR	.0040	920.0	.0130	3.5	-0.0	-0.0	1.0	9.621	63.802	68.907	NO
13	16	MANHOLE	-0.0000	-0.0	-0.0000	0.0	-0.0	-0.0	1.0	0.000	0.000	0.000	

EPSILON = .000100 NO. OF ITERATIONS IN ROUTING ROUTINE = 4

HYDROGRAPHS AND POLLUTOGRAPHS PROVIDED TO SUBSEQUENT PROGRAMS FOR THE FOLLOWING ELEMENTS

1

TOTAL AREA INFILTRATION(IN GPM) DUE TO-

BASE FLOW	GROUND WATER	MELT	RAIN
80.0000	0.0000	65.7940	125.0000

APPORTIONED INFILTRATION

ELEMENT NO.	QINFIL(CFS)	PROP. TOT. INFIL.	INFIL. INPUT AT UPSTREAM ELE. NO.
104	.059	.0971	5
103	.043	.0719	4
102	.025	.0413	3
108	.025	.0412	9
107	.025	.0420	8
113	.026	.0437	14
112	.067	.1104	13
111	.036	.0597	12
110	.064	.1062	11
109	.039	.0647	10
106	.111	.1836	7
105	.033	.0541	6
101	.051	.0841	2

QUANTITY AND QUALITY OF D W F FOR EACH SUBAREA

A1BOD = 2525.17 LBS PER DAY/CFS
A1SS = 1753.59 LBS PER DAY/CFS
A1COLI = 3.83E+10 MPN/DAY PER CAPITA
ADWF = 1.38 CFS

KNUM	INPUT	DWF CFS	+	INFIL CFS	=	DQDWF CFS	KLAND	DWBOD LBS/MIN	DWSS LBS/MIN	TOTPOP PERSONS	BODCONC MG/L	SSCONC MG/L	COLIFORMS MPN/100ML
1	13	.01		.01		.02	3	.02	.01				
2	11	.01		.00		.01	1	.01	.01				
3	14	.02		.01		.02	1	.05	.03				
4	12	.12		.05		.18	2	.25	.17				
5	9	.04		.02		.06	1	.07	.05				
6	9	.02		.01		.03	1	.04	.03				
7	8	.08		.04		.12	3	.13	.09				
8	6	.04		.02		.05	3	.06	.04				
9	5	.59		0.00		.59	4	.89	.74				
10	5	.02		.01		.03	3	.03	.02				
TOTALS		.94		.16		1.10		7.74 LBS	5.95 LBS	2198.	377.	290.	3.13E+06

COMPARISON OF MEASURED AND CALCULATED TOTAL SEWAGE FLOW- ADWF = 1.38 CFS SMTDWF = 1.10 CFS
CORRECTION FACTOR (CF2) OF 1.25 APPLIED TO THE DWF (QUANTITY AND QUALITY) AT EACH INLET

DAILY AND HOURLY CORRECTION FACTORS FOR SEWAGE DATA

DAY	DVDWF	DVBOD	DVSS	DVCOLI
1	.700	.900	.980	
2	1.250	.950	1.000	
3	1.150	1.050	1.000	
4	.950	1.100	1.030	
5	1.100	1.050	1.010	
6	1.050	1.000	1.000	
7	.800	.950	.990	

HOUR	DVDWF	DVBOD	DVSS	DVCOLI
1	.740	.850	1.050	1.100
2	.670	.710	1.050	.640
3	.630	.600	1.100	.450
4	.590	.410	.500	.870
5	.540	.460	.660	.540
6	.560	.490	1.330	.480
7	.670	.720	1.100	1.290
8	.960	.870	.880	1.180
9	1.420	.770	1.030	1.370
10	1.190	1.570	.910	1.490
11	1.200	1.020	.660	1.300
12	1.150	.870	.630	1.120
13	1.170	.910	.940	.890
14	1.110	.940	.940	.580
15	1.080	1.070	1.050	.450
16	1.150	1.070	1.050	.670
17	1.210	1.140	1.160	.960
18	1.230	.990	.940	1.180
19	1.250	1.450	1.330	.840
20	1.210	1.160	1.220	1.010
21	1.170	1.550	1.440	2.820
22	1.150	1.290	1.100	1.770
23	.880	.990	.880	.840
24	1.070	1.600	1.050	.710

ELEMENT FLOWS, AREAS, AND CONCENTRATIONS ARE INITIALIZED TO DRY WEATHER FLOW AND INFILTRATION VALUES.

ELE. NO.	TYPE	FLOW (CFS)	AREA (SQ. FT.)	INIT. VEL. (FPS)	BOD (LBS/CF)	S.S. (LBS/CF)	E COLI. (MPN/ML)	CPOLL NO.4
104	3	1.148	.834	1.3766	.0200	.0125	0.	
103	3	1.192	.807	1.4776	.0193	.0120	0.	
102	3	1.217	.817	1.4893	.0189	.0118	0.	
108	1	.135	.057	2.3606	.0207	.0110	4.05E+09	
107	1	.308	.118	2.6161	.0195	.0103	1.77E+09	
113	1	.056	.027	2.0905	.0200	.0106	1.86E+09	
112	1	.089	.051	1.7441	.0054	.0029	0.	
111	1	.347	.182	1.9065	.0192	.0102	3.60E+09	
110	3	.424	.392	1.0803	.0165	.0087	3.10E+09	
109	3	.519	.421	1.2319	.0156	.0083	2.73E+09	
106	3	.938	.773	1.2138	.0150	.0080	2.10E+09	
105	3	1.033	.851	1.2149	.0150	.0079	1.90E+09	
101	3	2.301	1.327	1.7341	.0167	.0098	8.54E+08	

INITIAL BED OF SOLIDS (LBS) IN SEWER DUE TO
14.0 DAYS OF DRY WEATHER PRIOR TO STORM

ELEMENT NUMBER	SOLIDS IN BOTTOM (LBS)
104	61.52884
103	49.28971
102	48.52239
108	1.06046
107	.51431
113	1.29961
112	1.13011
111	5.73588
110	6.21376
109	6.22549
106	18.26440
105	18.17518
101	25.59151

```
TIME=  3000. SECONDS, TIME STEP=   10. ELEMENT 107 SURCHARGING. SURCHARGE OF    4537.64CU. FT. STORED AT UPSTREAM ELEMENT   8
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.16

TIME=  3300. SECONDS, TIME STEP=   11. ELEMENT 107 SURCHARGING. SURCHARGE OF   10798.91CU. FT. STORED AT UPSTREAM ELEMENT   8
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.38

TIME=  3600. SECONDS, TIME STEP=   12. ELEMENT 102 SURCHARGING. SURCHARGE OF    7412.40CU. FT. STORED AT UPSTREAM ELEMENT   3
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.25

TIME=  3600. SECONDS, TIME STEP=   12. ELEMENT 107 SURCHARGING. SURCHARGE OF   16758.26CU. FT. STORED AT UPSTREAM ELEMENT   8
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.58

TIME=  3900. SECONDS, TIME STEP=   13. ELEMENT 102 SURCHARGING. SURCHARGE OF   15194.23CU. FT. STORED AT UPSTREAM ELEMENT   3
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.51

TIME=  3900. SECONDS, TIME STEP=   13. ELEMENT 107 SURCHARGING. SURCHARGE OF   20547.65CU. FT. STORED AT UPSTREAM ELEMENT   8
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.72

TIME=  4200. SECONDS, TIME STEP=   14. ELEMENT 102 SURCHARGING. SURCHARGE OF   21135.53CU. FT. STORED AT UPSTREAM ELEMENT   3
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.71

TIME=  4200. SECONDS, TIME STEP=   14. ELEMENT 107 SURCHARGING. SURCHARGE OF   21320.72CU. FT. STORED AT UPSTREAM ELEMENT   8
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.74

TIME=  4500. SECONDS, TIME STEP=   15. ELEMENT 102 SURCHARGING. SURCHARGE OF   24668.23CU. FT. STORED AT UPSTREAM ELEMENT   3
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.83

TIME=  4500. SECONDS, TIME STEP=   15. ELEMENT 107 SURCHARGING. SURCHARGE OF   19230.10CU. FT. STORED AT UPSTREAM ELEMENT   8
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.67

TIME=  4800. SECONDS, TIME STEP=   16. ELEMENT 102 SURCHARGING. SURCHARGE OF   24914.64CU. FT. STORED AT UPSTREAM ELEMENT   3
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.83

TIME=  4800. SECONDS, TIME STEP=   16. ELEMENT 107 SURCHARGING. SURCHARGE OF   13850.48CU. FT. STORED AT UPSTREAM ELEMENT   8
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.48

TIME=  5100. SECONDS, TIME STEP=   17. ELEMENT 102 SURCHARGING. SURCHARGE OF   21488.85CU. FT. STORED AT UPSTREAM ELEMENT   3
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.72

TIME=  5100. SECONDS, TIME STEP=   17. ELEMENT 107 SURCHARGING. SURCHARGE OF    5839.34CU. FT. STORED AT UPSTREAM ELEMENT   8
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.20

TIME=  5400. SECONDS, TIME STEP=   18. ELEMENT 102 SURCHARGING. SURCHARGE OF   15081.38CU. FT. STORED AT UPSTREAM ELEMENT   3
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.50

TIME=  5700. SECONDS, TIME STEP=   19. ELEMENT 102 SURCHARGING. SURCHARGE OF    5996.32CU. FT. STORED AT UPSTREAM ELEMENT   3
RATIO OF REQUIRED FLOW TO PRESENT QFULL= 1.20
```

SELECTED INLET HYDROGRAPHS - CFS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
8	.108	.989	5.894	15.573	24.017	33.704	43.742	52.794	65.850	77.043
	77.254	74.061	66.739	58.365	51.235	43.091	37.413	33.878	28.602	22.777
	20.161	18.888	17.172	14.275	10.073	6.311	4.269	3.049	2.268	1.742
	1.373	1.104	.904	.752	.633	.539	.463	.402	.351	.309
	.274	.244	.218	.196	.177	.161	.146	.134	.122	.112

SELECTED INLET POLLUTOGRAPHS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
				*** BOD IN LBS/MIN ***						
8	.030	.148	1.113	3.193	3.569	2.948	1.209	1.100	.964	.835
	.688	.560	.461	.377	.310	.254	.208	.937	.986	1.027
	.974	.868	.764	.657	.513	.321	.165	.085	.045	.025
	.014	.008	.005	.003	.002	.001	.001	.001	.000	.000
	.800	.000	.000	.000	.000	.000	.000	.000	.000	.000
				** SUSPENDED SOLIDS IN LBS/MIN **						
8	.007	.219	10.933	50.831	58.036	42.461	6.102	7.353	8.698	10.032
	10.318	9.615	8.510	7.211	6.046	4.997	4.109	18.705	19.705	20.526
	19.466	17.364	15.284	13.130	10.252	6.416	3.304	1.705	.906	.499
	.285	.168	.103	.065	.042	.028	.019	.013	.009	.007
	.005	.004	.003	.002	.002	.001	.001	.001	.001	.001
				*** COLIFORM IN MPN/MIN ***						
8	3.83E+09	2.12E+09	2.19E+09	2.41E+09	2.47E+09	2.02E+09	1.50E+09	1.00E+09	5.81E+08	3.14E+08
	1.61E+08	7.70E+07	3.82E+07	1.99E+07	1.10E+07	6.77E+06	4.29E+06	2.87E+06	2.13E+06	1.65E+06
	1.23E+06	9.75E+05	8.21E+05	7.28E+05	6.95E+05	6.74E+05	6.01E+05	5.57E+05	5.27E+05	5.06E+05
	4.90E+05	4.77E+05	4.67E+05	4.59E+05	4.53E+05	4.47E+05	4.42E+05	4.38E+05	4.35E+05	4.31E+05
	4.29E+05	4.26E+05	4.24E+05	4.22E+05	4.20E+05	4.19E+05	4.17E+05	4.16E+05	4.15E+05	4.13E+05

SELECTED OUTFLOW HYDROGRAPHS - CFS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
1	2.325	2.666	5.799	20.691	53.438	90.502	131.999	176.596	230.348	278.217
	317.737	321.795	321.008	323.128	314.035	309.435	302.295	290.072	268.192	224.749
	177.825	151.283	135.729	119.535	102.178	80.342	61.303	42.094	31.918	24.620
	19.878	15.343	12.610	10.588	8.898	7.605	6.617	5.871	5.293	4.835
	4.480	4.214	3.950	3.759	3.579	3.437	3.313	3.204	3.114	3.058

SELECTED OUTFLOW POLLUTOGRAPHS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10

*** BOD IN LBS/MIN ***

1	2.315	2.481	3.705	7.788	14.929	20.028	22.168	21.822	20.299	16.653
	12.475	9.064	7.741	7.393	6.878	6.675	6.968	7.538	8.345	8.327
	7.172	7.190	7.042	6.349	5.927	4.969	4.332	3.344	3.005	2.715
	2.499	2.384	2.314	2.283	2.239	2.204	2.192	2.227	2.290	2.343
	2.386	2.419	2.434	2.459	2.468	2.484	2.497	2.508	2.524	2.565

** SUSPENDED SOLIDS IN LBS/MIN **

1	6.592	13.594	18.873	50.237	148.156	251.347	307.375	305.655	276.310	213.797
	142.253	97.917	82.141	75.424	72.365	69.940	75.710	87.694	103.195	109.413
	98.876	97.081	93.067	81.442	71.893	55.048	42.194	26.761	19.608	13.964
	12.806	8.928	6.593	5.688	4.364	3.861	3.240	3.012	2.740	2.635
	2.528	2.453	2.376	2.335	2.283	2.255	2.228	2.209	2.193	2.205

*** COLIFORM IN MPN/MIN ***

1	1.20E+11	1.32E+11	1.93E+11	3.20E+11	3.77E+11	2.50E+11	1.60E+11	1.52E+11	1.48E+11	1.35E+11
	1.24E+11	1.21E+11	1.16E+11	1.07E+11	9.79E+10	9.82E+10	1.04E+11	1.04E+11	9.86E+10	9.03E+10
	9.22E+10	9.30E+10	9.15E+10	9.32E+10	8.87E+10	7.33E+10	5.93E+10	4.93E+10	4.99E+10	5.00E+10
	4.98E+10	4.89E+10	4.89E+10	4.89E+10	4.79E+10	4.71E+10	4.65E+10	4.56E+10	4.40E+10	4.18E+10
	4.03E+10	3.95E+10	3.90E+10	3.90E+10	3.89E+10	3.92E+10	3.96E+10	3.99E+10	4.04E+10	4.16E+10

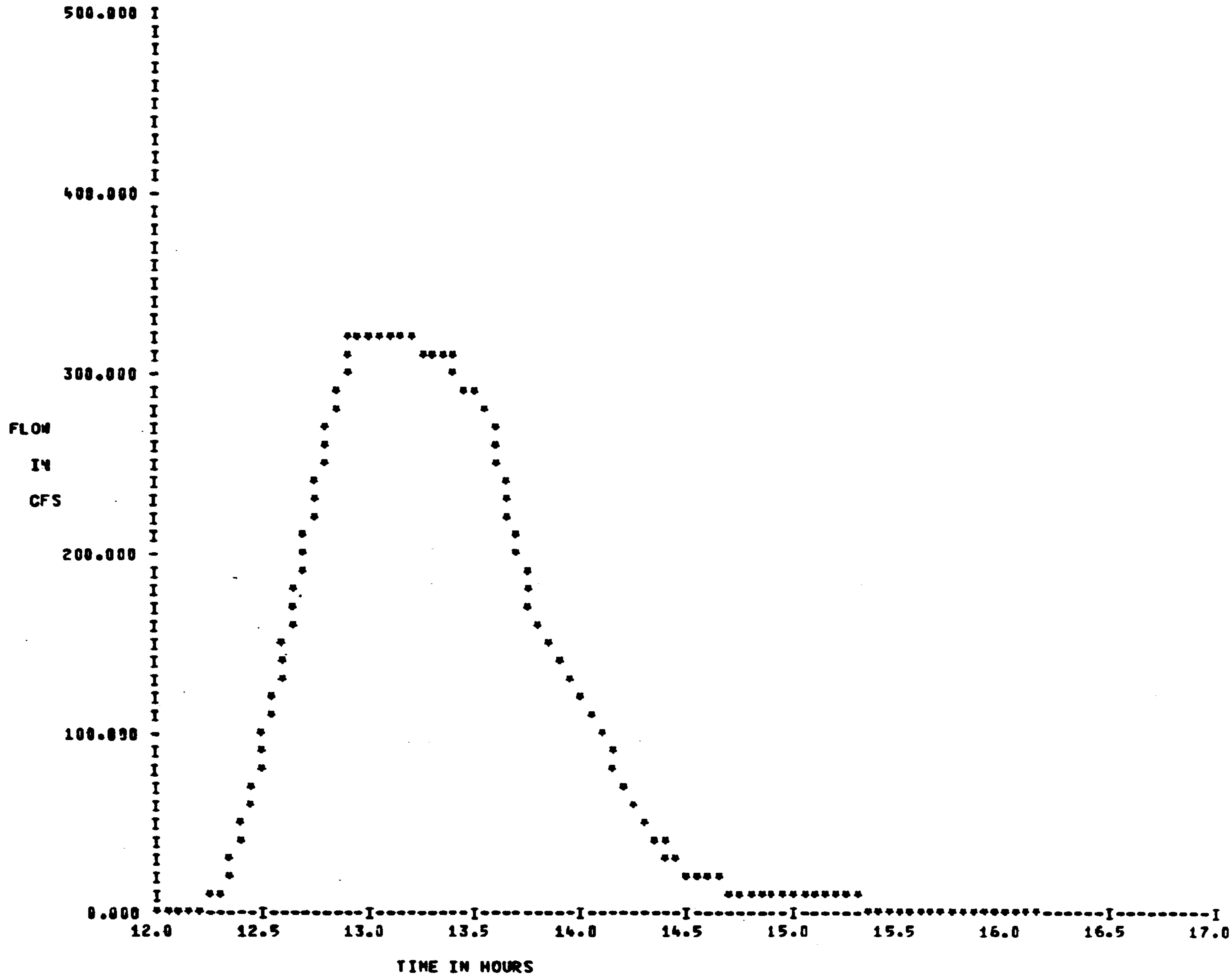
SELECTED OUTFLOW POLLUTOGRAPHS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
*** BOD IN MG/L ****										
1	266.340	248.872	170.914	100.686	74.730	59.198	44.925	33.054	23.573	16.012
	10.503	7.535	6.451	6.120	5.859	5.771	6.166	6.952	8.323	9.911
	10.837	12.713	13.878	14.208	15.516	16.545	18.903	21.252	25.181	29.499
	35.040	41.563	49.086	57.687	67.326	77.523	88.620	101.491	115.710	129.644
	142.441	153.561	164.842	175.019	184.425	193.298	201.585	209.376	216.803	224.406
*** SUSPENDED SOLIDS IN MG/L ***										
1	758.466	1363.841	870.579	649.484	741.638	742.913	622.903	462.994	320.875	205.561
	119.761	81.396	68.449	62.440	61.642	60.462	66.996	80.870	102.929	130.225
	148.201	171.666	183.421	182.254	188.215	183.284	184.115	170.064	164.335	151.728
	168.334	155.656	139.871	143.711	131.190	135.802	130.982	137.260	138.463	145.775
	150.479	155.731	160.879	166.168	170.636	175.530	179.874	184.395	188.360	192.891
***COLIFORM IN MPN/100ML ***										
1	3.02E+06	2.91E+06	1.95E+06	9.07E+05	4.14E+05	1.62E+05	7.10E+04	5.06E+04	3.76E+04	2.86E+04
	2.29E+04	2.20E+04	2.12E+04	1.94E+04	1.83E+04	1.86E+04	2.02E+04	2.10E+04	2.16E+04	2.36E+04
	3.06E+04	3.61E+04	3.96E+04	4.58E+04	5.09E+04	5.35E+04	5.68E+04	6.88E+04	9.18E+04	1.19E+05
	1.51E+05	1.87E+05	2.28E+05	2.71E+05	3.16E+05	3.64E+05	4.12E+05	4.56E+05	4.87E+05	5.08E+05
	5.27E+05	5.50E+05	5.79E+05	6.08E+05	6.39E+05	6.70E+05	7.01E+05	7.31E+05	7.61E+05	7.99E+05

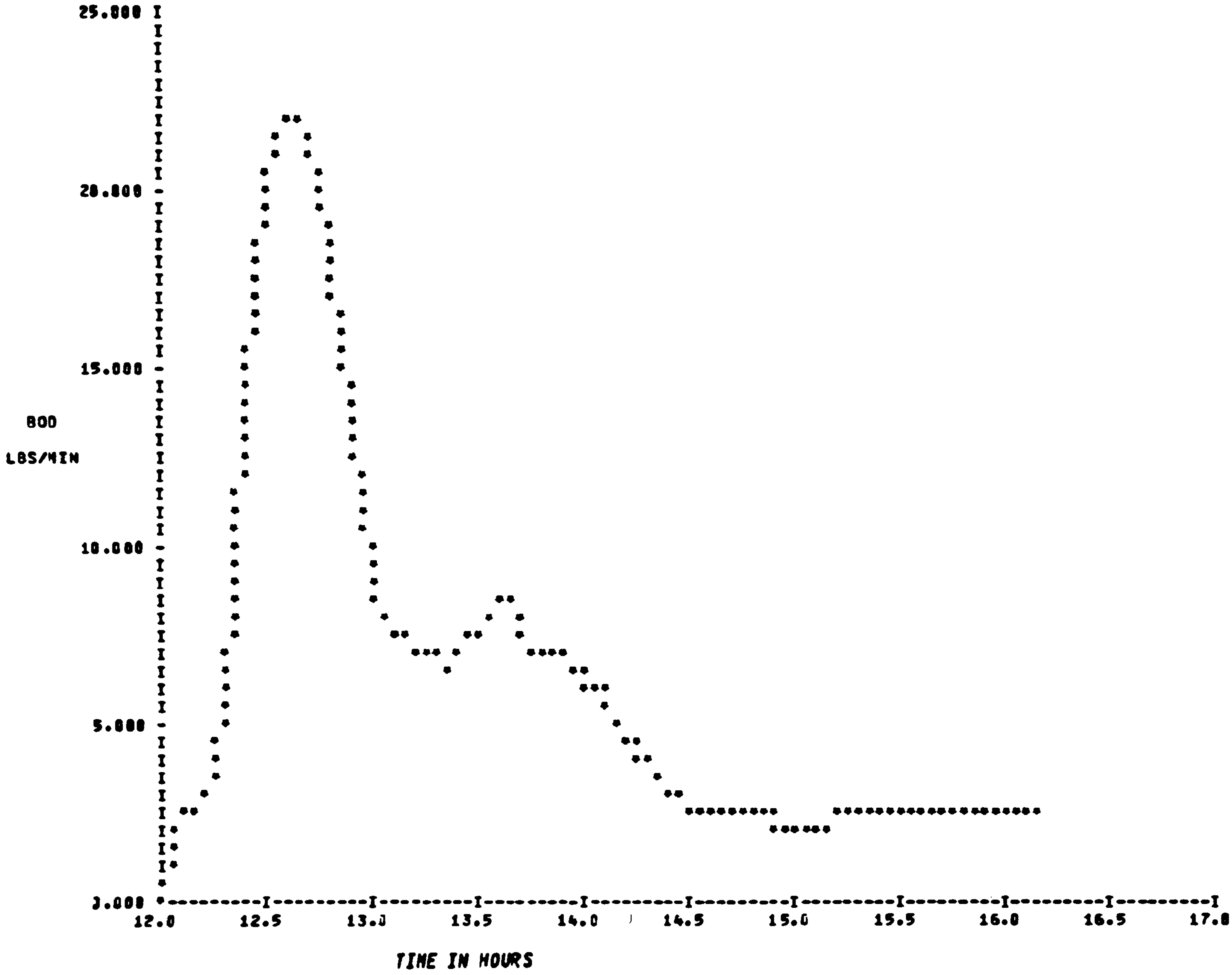
TOTAL POUNDS OF SUSPENDED SOLIDS OUTPUT FROM ELEMENT 1 = 15916.24

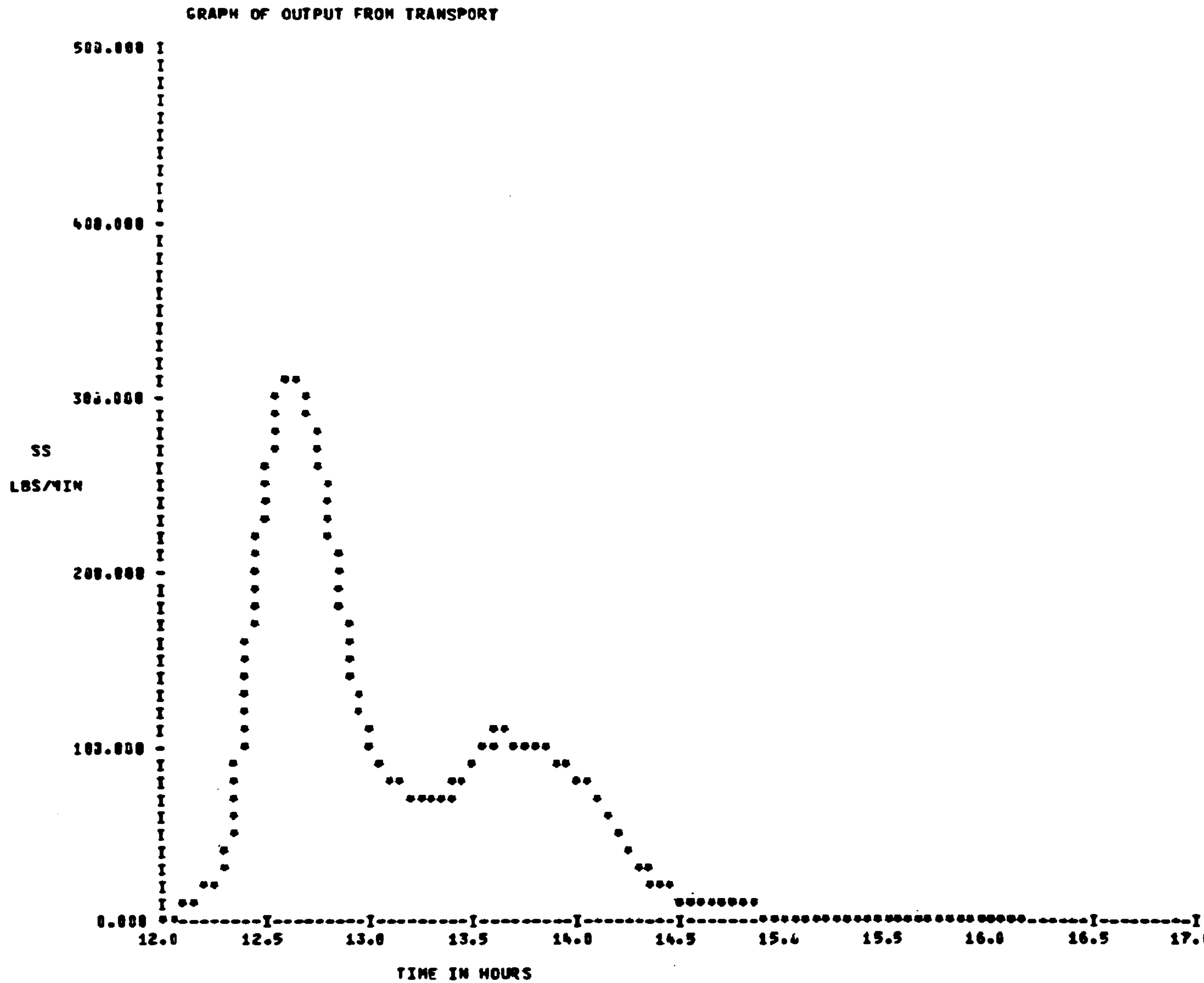
TOTAL POUNDS OF FIVE-DAY BOD OUTPUT FROM ELEMENT 1 = 1566.77

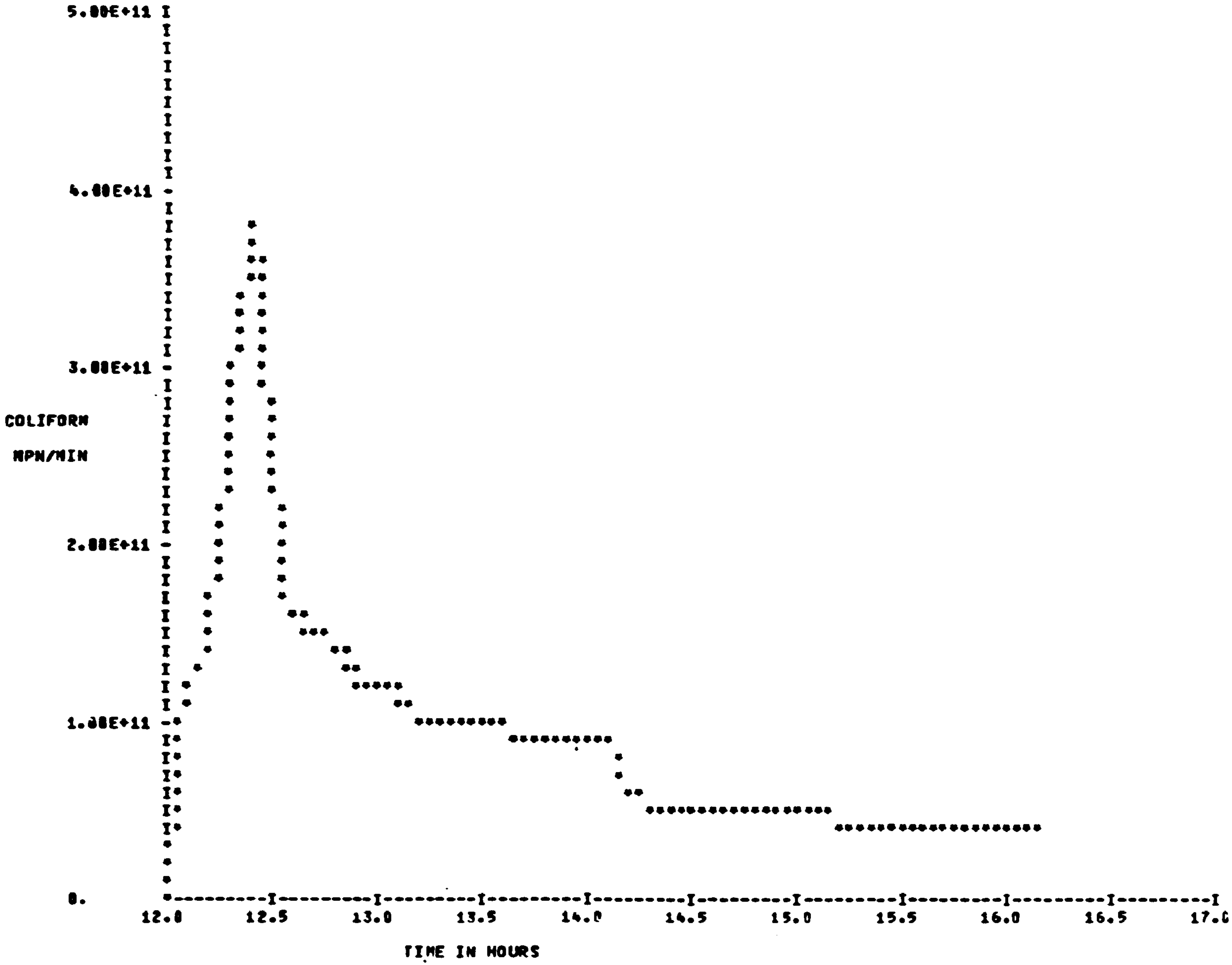
GRAPH OF OUTPUT FROM TRANSPORT



GRAPH OF OUTPUT FROM TRANSPORT




GRAPH OF OUTPUT FROM TRANSPORT
500.000
400.000
300.000
200.000
100.000
0.000
SS
LBS/MIN
12.0
12.5
13.0
13.5
14.0
14.5
15.0
15.5
16.0
16.5
17.0
TIME IN HOURS


5.00E+11
4.00E+11
3.00E+11
2.00E+11
1.00E+11
0.
COLIFORM
MPN/MIN
12.0
12.5
13.0
13.5
14.0
14.5
15.0
15.5
16.0
16.5
17.0
TIME IN HOURS

DISCUSSION OF OPTIONS AVAILABLE

Primary uses of the Storage/Treatment Block include modeling of:

1. existing sewage treatment plants to estimate treatment effectiveness under storm conditions.

2. proposed changes to existing treatment plants that are designed to upgrade treatment of storm flows.

3. proposed plants designed specifically for storm water treatment.

4. storage facilities used in conjunction with any of the above.

Output from the block can be used to compare treatment or cost effectiveness between two or more given storage/treatment schemes. The program is not a primary design tool because no provisions have been made to optimize storage/treatment units. All storage and treatment parameters are either pre-programmed or input by the user, and remain constant throughout the simulation.

Input data requirements for the Storage/Treatment Block are minimal. Data for existing facilities can be extracted from facility plans or operational records. Data for proposed facilities can be taken from design drawings or specifications. In the case of existing treatment facilities, the user may wish to replace programmed treatment parameters with site specific data. Equations used in treatment calculations are discussed in a later section.

Multiple treatment schemes can be simulated during the same computer run. Only the results of the first simulation, however, can be transferred to the Receiving Water or other blocks. Figure 17 provides an example of multiple simulations.

Three types of storage facilities can be modeled by the SWMM. The first is a natural reservoir of irregular geometry. For this type, surface area versus water depth data must be input. The second and third types are covered and uncovered reservoirs of regular geometry. For these, base area, base circumference and side slope are required inputs. Additional data requirements are: the type of basin flow (plug flow or completely mixed), maximum (flooding) reservoir depth, type of basin outflow works (pump, orifice, weir), and storage facility initial conditions (storage volume utilized and outflow rate at time zero).

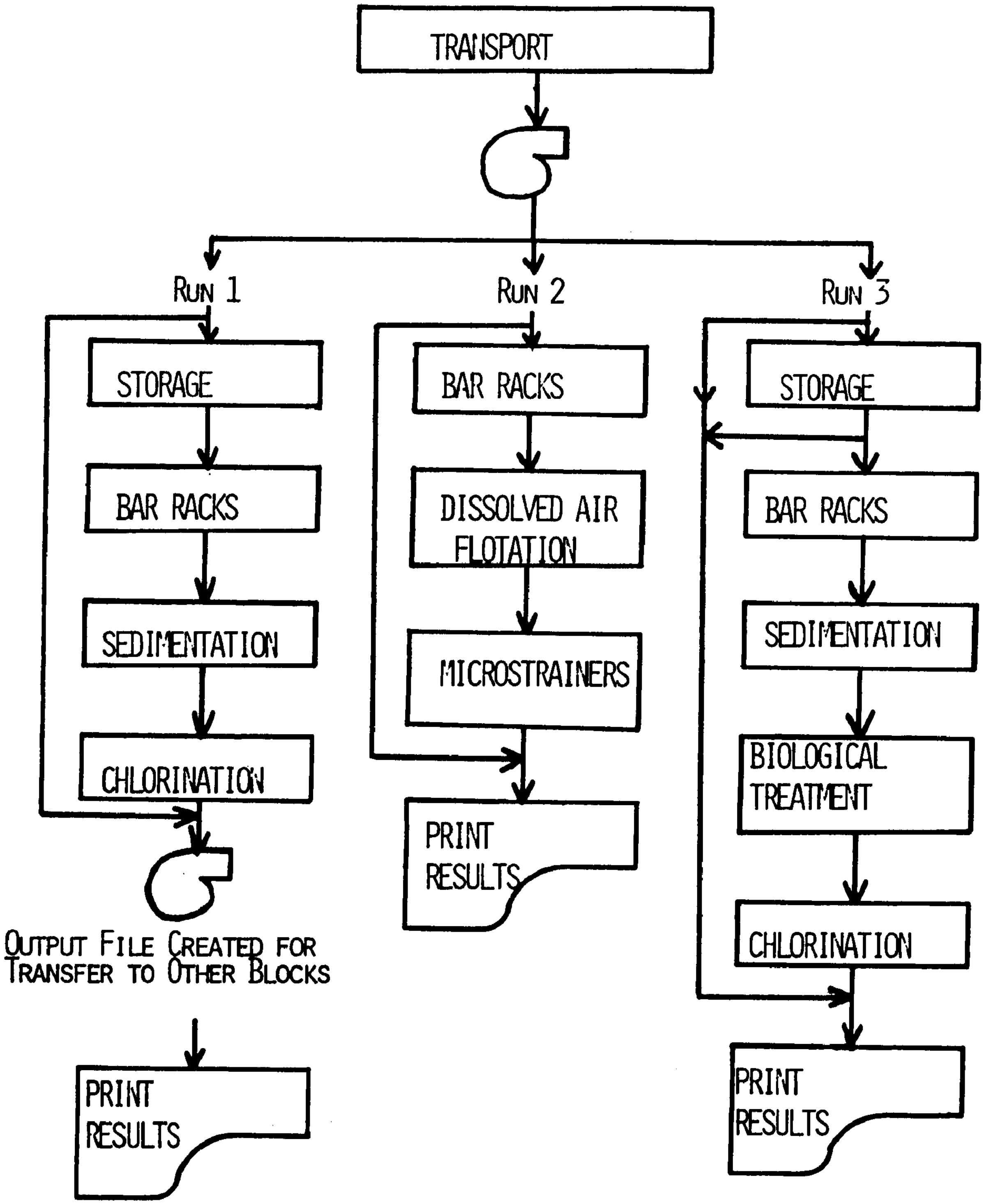


FIGURE 17. EXAMPLE OF MULTIPLE STORAGE/TREATMENT SIMULATIONS

Single or dual rate outlet pumps can be simulated. For dual rate pumps, the second pump starts if the first pump fails to lower the water level in storage. Data required are the outflow pumping rate (QPUMP), the depth of water in storage at pump start-up (DSTART) and the depth of water in storage at pump shut-down (DSTOP). The volume of water contained in storage at DSTOP must be equal to or greater than the volume pumped during one time step. Figure 18 depicts the important datum for outflow pumping of a storage reservoir.

For orifice outlets, the orifice centerline is assumed to be located at zero storage tank depth. The product of the orifice outlet area and the orifice discharge coefficient, C, must be input. Values for C can be found in King's Handbook of Hydraulics, Reference 7 or equivalent. Required inputs for a weir outlet are weir height and length.

SWMM II provides the options of bypassing storage, routing all flow through storage, or routing flow in excess of the treatment capacity through storage. If the Storage/Treatment Block is called, flows must be routed through at least one treatment module. Treatment modules are arranged in seven levels; preliminary treatment, inlet pumping, primary treatment, secondary treatment, effluent screens, outlet pumping and chlorination. Preliminary treatment can include bar racks or swirl concentrators. Primary treatment options available are sedimentation, fine screens, dissolved air flotation or fine screens in conjunction with dissolved air flotation. Secondary treatment can be accomplished by microstrainers, high rate filters or biological treatment. Standard or high rate disinfection can be applied to the treatment plant effluent and high rate disinfection can be applied to treatment plant overflows. Any of the levels of treatment can be bypassed.

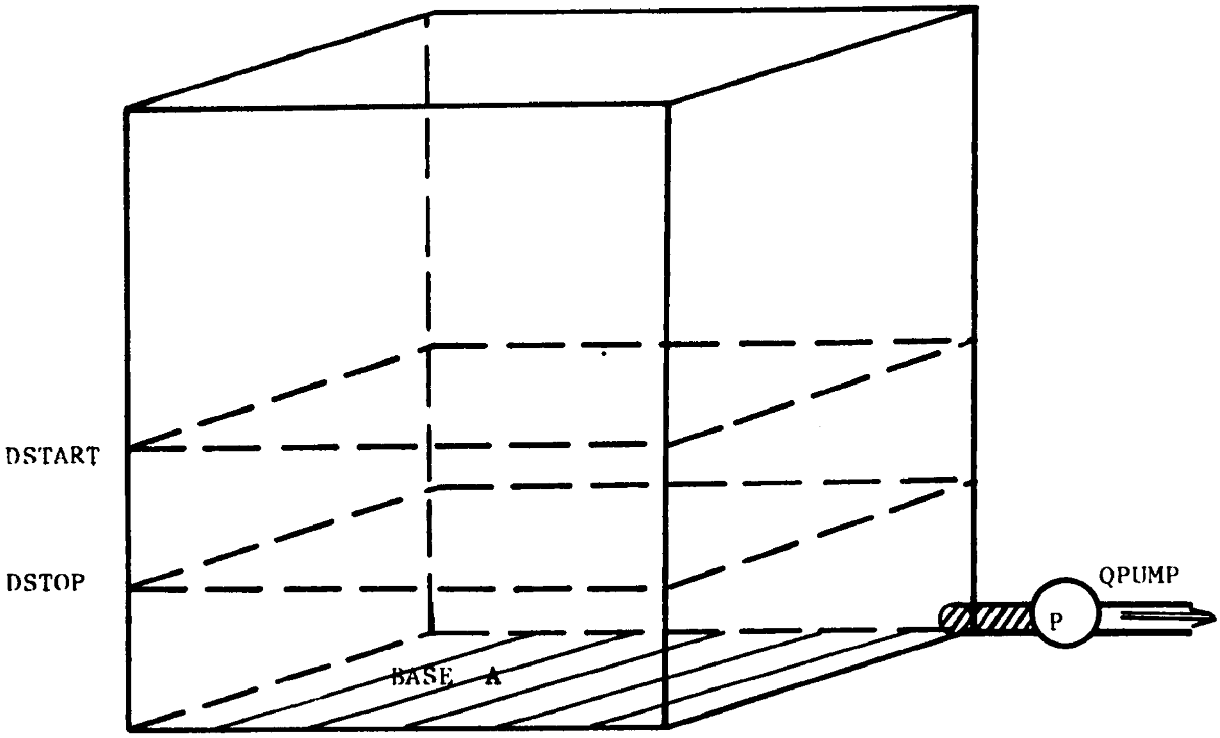
The user can either specify the treatment capacity desired or have the program determine the treatment capacity based on a percentage of the peak flow. Once the treatment capacity has been set, all flows in excess of this capacity are bypassed and re-combined with the treatment facility effluent before the output hydrograph and pollutographs are written on the output file.

Many of the treatment parameters are pre-programmed. However, additional data are required for the following treatment modules: high-rate disinfection of overflows (flow in excess of design flow); swirl concentrator; dissolved air flotation; sedimentation; and high rate filtration.

The Storage/Treatment Block will also provide estimates of the capital and operational costs for the storage and treatment facilities modeled. Figure 7-2 on page 220 of the User's Manual (reference 6) provides a schematic diagram of all the storage/treatment options available.

PREPARATION OF STORAGE TREATMENT FLOW DIAGRAM

The storage/treatment flow diagram is a schematic diagram of the modules to be called, annotated with all of the required user supplied data. The development of such a flow diagram is straightforward for existing storage or



For Rectanuglar Basin

DSTOP*BASEA$\geq$QPUMP*DT

BASEA = Base Area

DT = Time Step, seconds

FIGURE 18. PUMP OUTLET PARAMETERS.

treatment works. In the case of proposed facilities, the model may be useful in testing various alternatives at the preliminary planning stage. Several computer runs may be desirable to compare various treatment configurations, to test the effects of varying treatment parameters or to determine the best combination of storage and treatment.

To simulate proposed facilities, realistic storage and treatment parameters must be input to the model. Required capacity of the storage facility can be estimated from the volume of flow generated by the design storm and the volume of flow treated during the simulation. A mass curve (Ripple diagram) can be used to determine the volume of the storage facility required to prevent any overflow during the simulation period. Figure 19 shows such a diagram constructed for the case study hydrograph.

Treatment parameters can be taken from bench or pilot scale studies already completed, from design specifications or from standard engineering references. References 8 and 9 contain suggested parameters for the types of treatment modeled by the Storage/Treatment Block.

The storage/treatment flow diagram can be used to complete the computer coding forms for the Storage/Treatment Block. Instructions for coding form preparation are contained in Table 7-2, pages 227-237 of the User's Manual.

CASE STUDY MATERIALS

A hypothetical 3000 acre basin was used to generate hydrographs and pollutographs in the Runoff Block. These were then routed through the Transport Block. For the purpose of developing these data, land use within the basin was assumed to be equally divided among the five types modeled by SWMM; i.e., single family residential, multi-family residential, commercial, industrial and open land. Impervious area of subcatchments varied from 10% to 60%, with the average being 35.4%. The design storm used in the Runoff Block Case Study produced the overland runoff. Dry weather flow and infiltration were modeled in the Transport Block. Thirty days of antecedent dry weather and a street cleaning frequency of 40 days were chosen as critical design conditions. The total length of all conduits modeled in Transport was 3 miles. The resulting hydrographs and pollutographs for entry into the Storage/Treatment Block are given in Figure 20. A total volume of 116.85 million gallons (for an average flowrate of 112.18 MGD for 25 hours) is discharged. The total storm runoff contains 84,628 pounds of BOD_5 and 295,655 pounds of suspended solids. (Note the large first flush of pollutants indicated in Figure 20.)

Transfer of flow from the Transport to the Storage/Treatment Block occurs at modeled Element 10 as shown in Figure 21. Flows in excess of the design capacity of 50 MGD are bypassed and then re-combined with the treatment plant effluent prior to discharge into the receiving waters.

A preliminary planning study is now underway to determine the feasibility of replacing the existing treatment plant with a facility de- designed to treat storm flows as well as dry weather flow. The Storage/

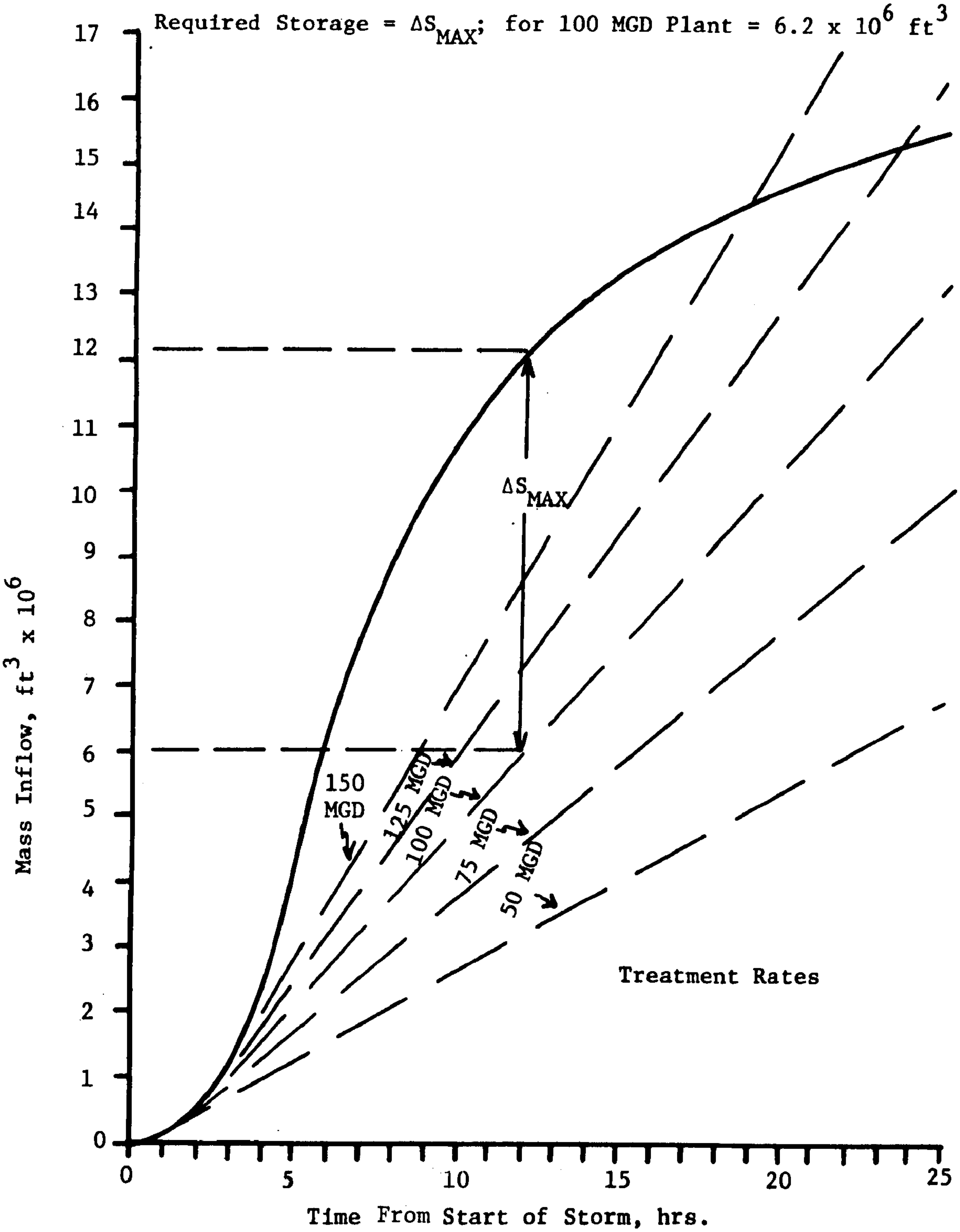
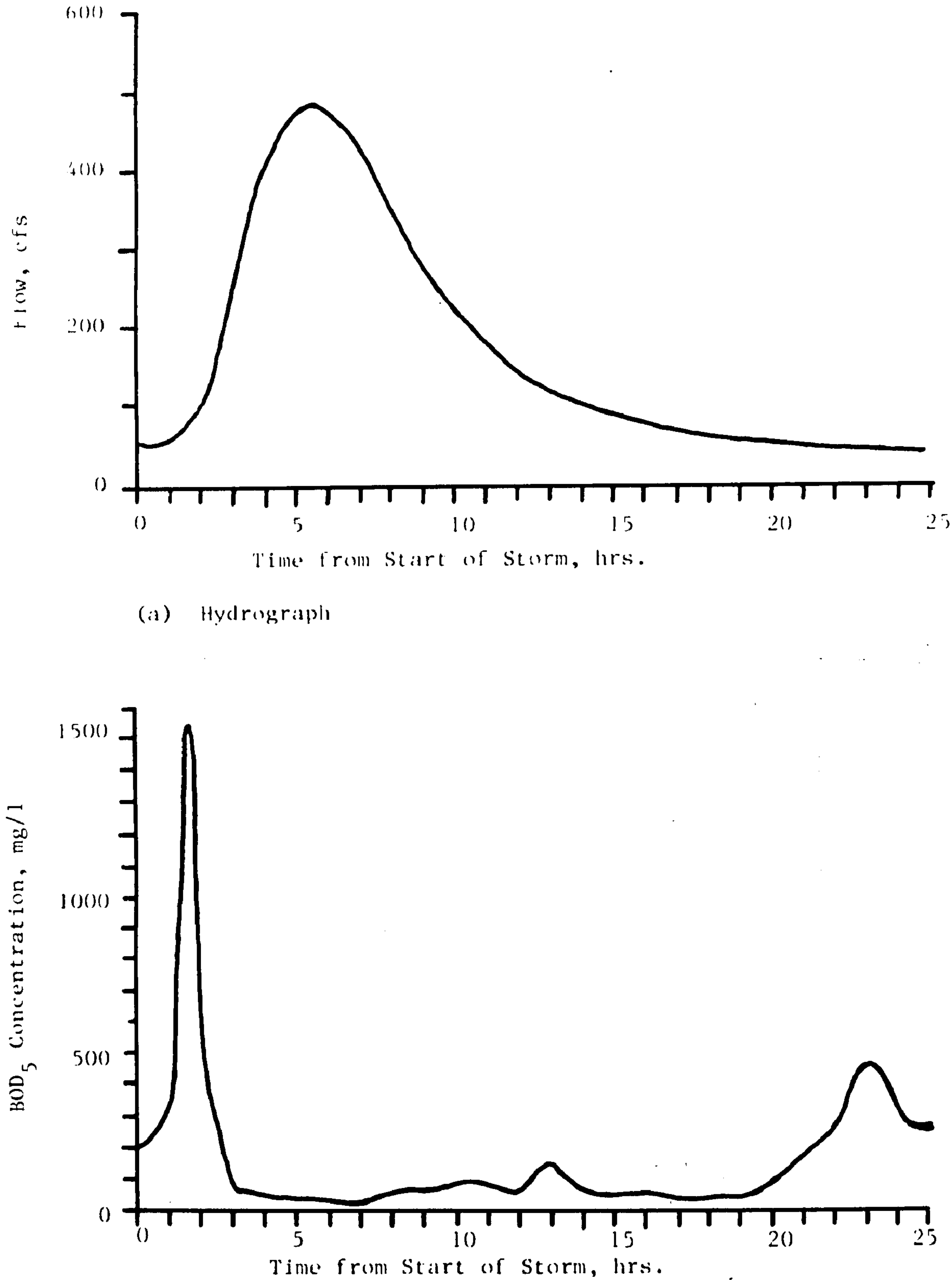


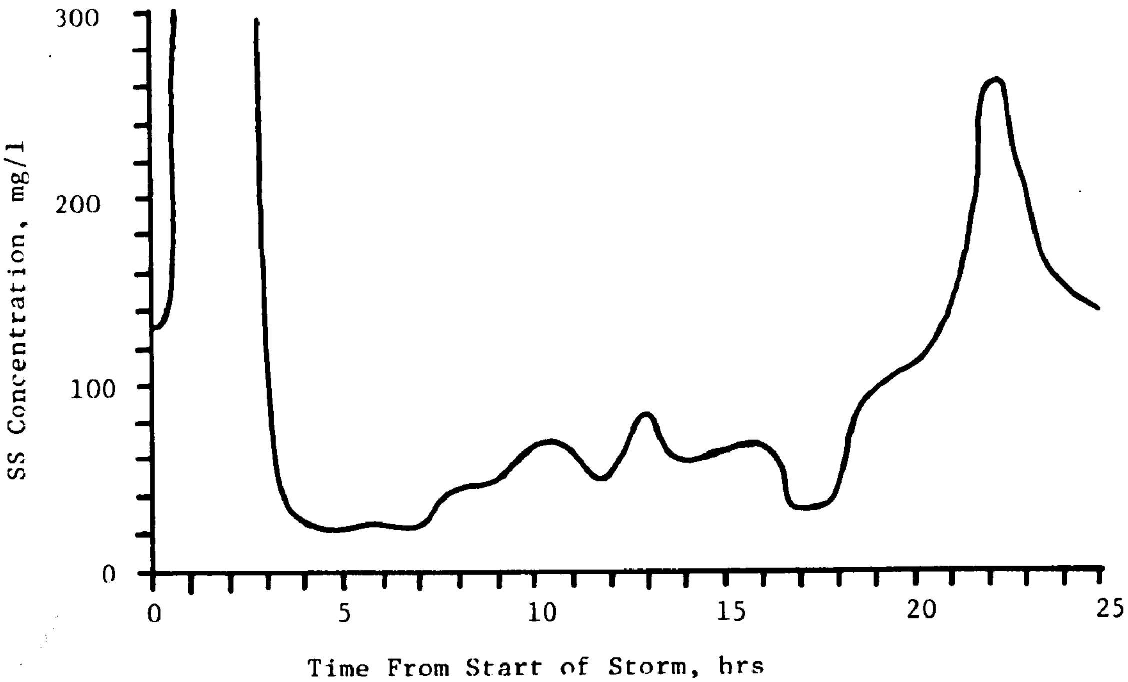
FIGURE 19. MASS STORAGE CURVE.



(a) Hydrograph

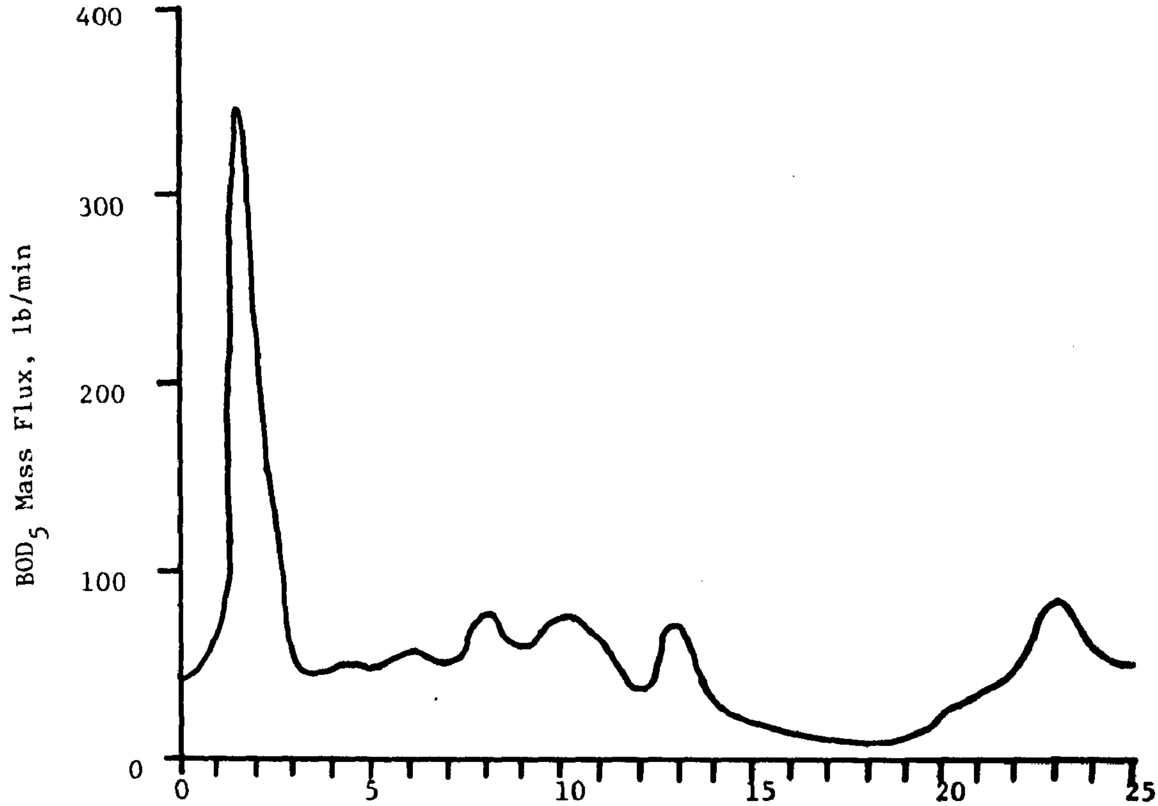
(b) BOD_5 Pollutograph (mg/l)

FIGURE 20. HYDROGRAPH AND POLLUTOGRAPHS FOR 3000 ACRE BASIN.

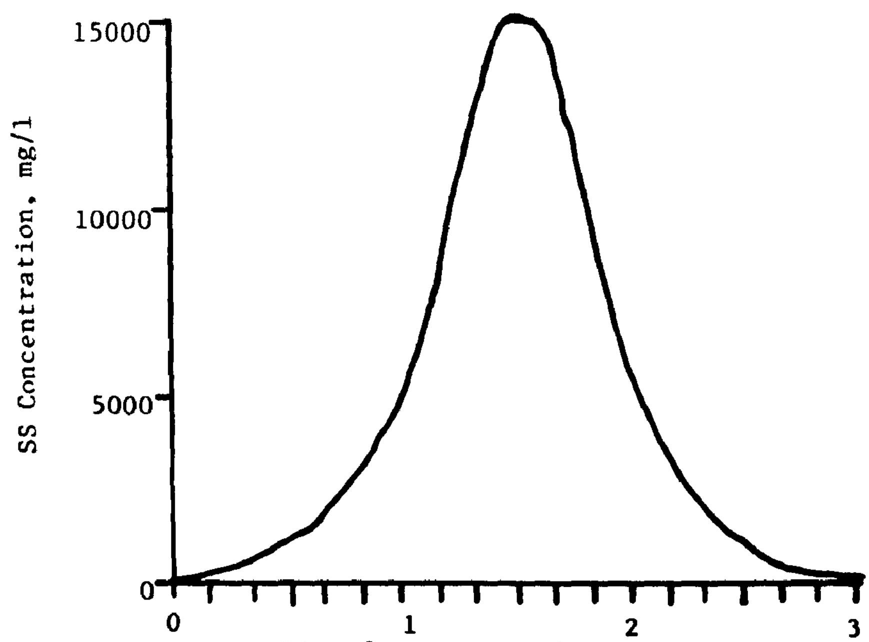


(e) SS Pollutograph for Entire Storm (mg/l)

FIGURE 20., CONTINUED.



(c) BOD_5 Pollutograph (lb/min)



(d) SS Pollutograph Showing First Flush (mg/l)

FIGURE 20., CONTINUED.

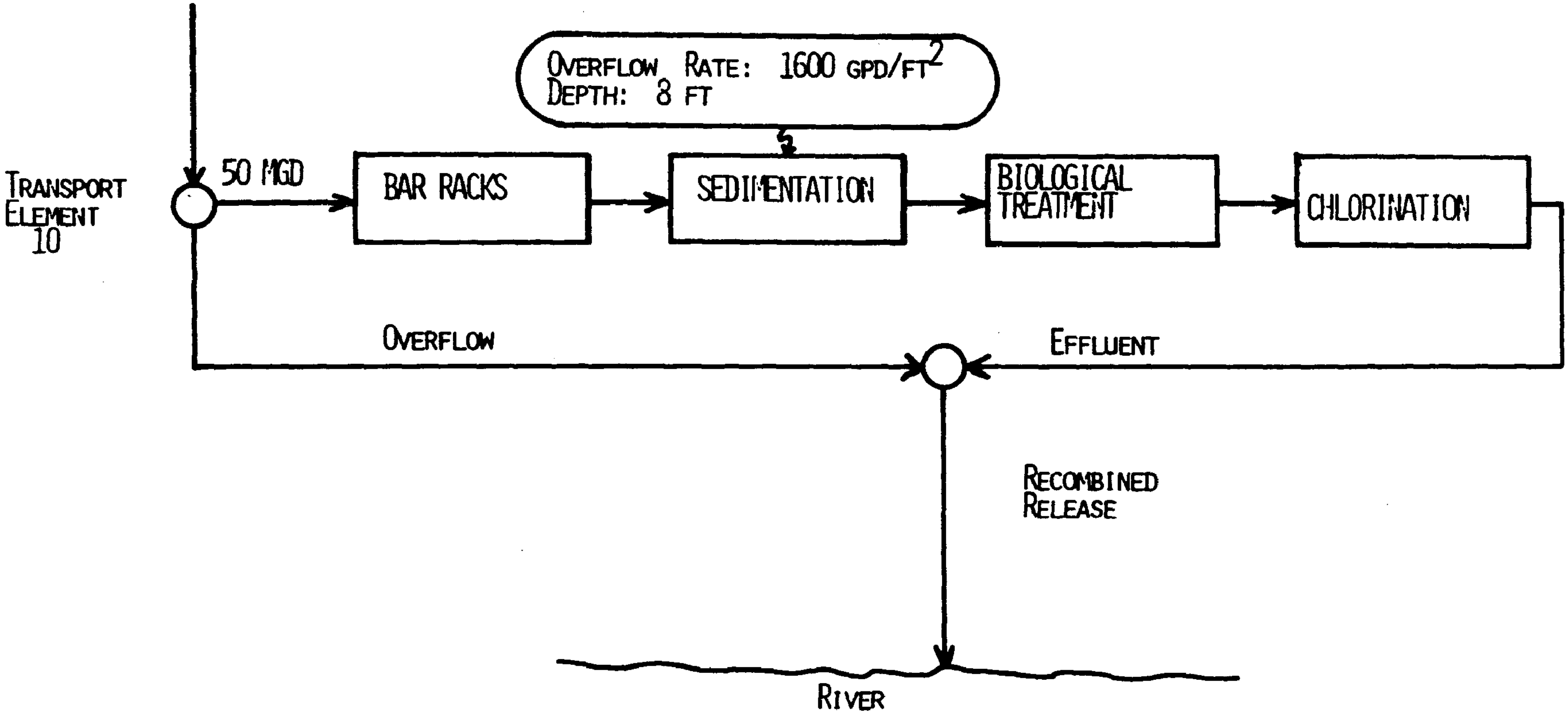


FIGURE 21. EXISTING TREATMENT PLANT CONFIGURATION.

Treatment Block of SWMM will be used to evaluate the treatment effectiveness that will be realized by the installation of a physical-chemical treatment plant, with or without storage. Later, the results of these simulations will be input to the Receiving Water Block to determine the response in receiving water quality to treatment of storm flows.

The assignments for the Storage/Treatment Block workshop are to develop the storage/treatment flow diagram for the physical-chemical treatment plant with storage facilities and to prepare the necessary computer coding forms. Use the following guidelines for design:

1. Design the storage and/or treatment facilities so that no overflow will occur during the storm event. Figure 19 can be used for this purpose.

2. Include chlorination in the treatment string.

3. Use any reasonable combination of treatment modules.

4. Transfer data from the Transport Block to the Storage/Treatment Block through external element number 10.

5. Assume the time of start of simulation to be 12:00 AM.

Typical treatment parameters for the modules contained in the Storage/Treatment Block are presented in Table 12 and may be used if desired.

Default values for cost computations should be used with the exceptions given in Table 13 for excavation and the ENR Cost Index.

PROGRAMMED TREATMENT PROCESS CONSTANTS AND COST GENERATION EQUATIONS

To utilize the Storage/Treatment Block effectively, the user must have an understanding of the logic involved in treatment and cost calculations. This will aid in interpretation of simulation results and will allow the user to change treatment and cost parameters to reflect local conditions. The following synopsis of treatment efficiency calculations is presented here as Table 14. This has been taken from Sections 15 and 16 of Reference 11 and from the SWMM II program listing. Similarly, cost estimation equations are included in Table 15. These have been revised from those given in Reference 11 to reflect capital costs given in Reference 6. Tables 16 and 17 are also extracted from reference 11 and give irreducible maintenance and storm event costs used in the model.

Table 12. TYPICAL TREATMENT PARAMETERS

Dissolved Air Flotation

Design Overflow Rate = 5000 gpd/ft^2

% Flow Recirculation = 15%

Depth of DAF Tank = 8 ft

Primary Sedimentation

Design Overflow Rate = 1600 gpd/ft^2

Depth of Sedimentation Tank = 8 ft

High Rate Filter

Maximum Operating Rate = 20 gpm/ft^2

Maximum Design Head Loss = 10 ft

Maximum Solids Holding Capacity = 3 lb/ft^2

Swirl Concentrator*

Number of Particle Sizes = 5

Concentrator Diameter = 36 ft

Specific Gravity of Particles = 2.65

Particle Size (cm) =	0.02	0.05	0.1	0.15	0.20
Fractional % by weight =	0.10	0.10	0.15	0.25	0.40

*Data taken from Table 2, page 15, Reference 10.

Table 13. EXCAVATION COSTS AND ENR COST INDEX

Dollars/Cubic Yard for Storage Excavation (CPCUYD): $6.00

ENR Cost Index for Year and Location (IENR):

Year	Index Value
1970	1363
1971	1545
1972	1712
1973	1886
1974	2035
1975	2180
1976	2312
1977	2431
1978	2537
1979	2630
1980	2710

Table 14, Synopsis of Treatment Efficiency Calculations

Fine Screens

Hydraulic Loading Rate: 50 gpm/ft^2

SS Removal: 27%

BOD_5 Removal: 22%

Backwash Flow: 0.75% of entering flow

Sedimentation

$$\text{SS Removal} = 0.656 + \frac{\text{SS Conc. } (0.06)}{190} - 0.40 \left(\frac{\text{OVFRA}-300}{2000}\right)$$

OVFRA = overflow rate in gpm/ft^2/day
(assumed to be at least 300 gpd/ft^2)

SS removal efficiency is limited to a range of 0.30 to 0.76

BOD_5 Removal = 0.55 (SS removal) w/o chlorination

= 0.66 (SS removal) with chlorination

Sludge volume is based on a solids content of 5% and a minimum sludge pumping rate of 45 gpm.

Dissolved Air Flotation

$$\text{SSREMV} = 0.656 + \frac{0.06\ (\text{SSINF})}{190} - 0.40 \left(\frac{\text{OVFRA}-1{,}000}{7000}\right)$$

$$+ \text{ICHEM} \left(\frac{20{,}000-\text{OVFRA}}{100{,}000}\right)$$

SSREMV = Suspended Solids removal efficiency

SSINF = Suspended Solids in inflow, mg/l

ICHEM = 0 if no chemicals are added

= 1 if chemicals are added

SS removal efficiency is restricted to not less than 0.20 and not more than 0.82.

Table 14., Continued

$$\text{BODREM} = 0.59 + \frac{0.05\ (\text{BOD Conc})}{100} - 0.36\ \left(\frac{\text{OVFRA}-1000}{7000}\right)$$

$$+ \text{ICHEM}\ (0.02) + \text{ICL2}\ (0.15)$$

ICL2 = 0 if no chlorine is added

= 1 if chlorine is added

BOD_5 removal efficiency is restricted to not less than 0.18 and not more than 0.60

Minimum Overflow Rate: 1000 gpd/ft^2

If chlorination option is chosen:

10 mg/l chlorine will be added if the BOD_5 of the influent is 130 mg/l or less

15 mg/l will be added if the influent BOD_5 is greater than 130 mg/l.

Dissolved Air Flotation with Fine Screens

$$\text{SSREMV} = 0.528 + \frac{0.06\ (\text{SS Conc})}{190} - 0.486\ \left(\frac{\text{OVFRA}-1000}{7000}\right)$$

$$+ \text{ICHEM}\ (1.37)\ \left(\frac{20{,}000-\text{OVFRA}}{100{,}000}\right)$$

$$\text{BODREMV} = 0.475 + \frac{0.05\ (\text{BOD Conc})}{100} - 0.405\ \left(\frac{\text{OVFRA}-1000}{7000}\right)$$

$$+ \text{ICHEM}\ (1.30)\ (0.02) + \text{ICL2}\ (1.30)\ (0.15)$$

SS Removal efficiency is limited to not less than 0.15 and not more than 0.75.

BOD_5 removal efficiency is limited to not less than 0.15 and not more than 0.48.

Table 14, Continued

Microstrainer

Design Based on: 4.3 rpm, application rate of 40 gpm/ft^2 of submerged area, 69% submergence and removal rate of 0.0135 lb of solids/ft^2/revolution

$$\text{SS Removal (mg/l)} = \frac{10{,}000\ (F)}{\text{gpm/ft}^2 \text{ submerged area}}$$

F = area correction factor

= 1.0 for areas up to 400 acres

$$= \sqrt{\frac{400}{\text{area in acres}}} \text{ for areas } >400 \text{ acres}$$

The model restricts flow to 40 gpm/ft^2 of submerged area, regardless of solids concentration. If solids concentration is so great that flow cannot be strained, part is bypassed and the overall performance is computed by the model.

BOD_5 Removed (ppm) = BOD_5 in influent - 10.0, if BOD_5 in is ≥ 27 ppm

$$= \frac{17(BOD_5 \text{ in influent})}{27.0}, \text{ if } BOD_5 \text{ in is } <27 \text{ ppm}$$

Biological Treatment

SSRM = SSIN (0.8)

BDRM = BDIN (0.8)

SSRM = Suspended Solids removed, lb/time step.

SSIN = Suspended Solids inflow, lb/time step

BDRM = BOD_5 removed, lb/time step

Table 14, continued

BDIN = BOD_5 inflow, lb/time step

High Rate Filters

The filter modeled is a tri-media filter consisting of:

36 inches of anthracite, 24 inches of garnet-sand mixture, 3 inches of coarse garnet and 9 inches of gravel.

Design Capacity: not to exceed 20 gpm/ft^2

Maximum Size of Units: 1400 ft^2

Maximum Capacity of Each Unit: 40 MGD or 62 cfs

An even number of units will always be specified.

Maximum Head Loss: not to exceed 10 ft.

Maximum Solids Holding Capacity: 3.0 lb/ft^2

SS removal of Ripened Filter: 80% w/o chemicals

95% with chemicals

Reduction in removal efficiency for fresh filters is modeled.

If chemicals are specified, dosages of 150 mg/l alum and 4 mg/l flocculent aid are used.

Calculations for Head Loss:

$$H_C = (q/q_m)^{1.18} \times (0.40)\ H_m$$

$$H_{CL} = (q/q_m)\ (S/Sqm)\ (0.60\ H_m)$$

$$H_L = H_C + H_{CL}$$

H_C = head loss when filter is clean

H_{CL} = head loss due to clogging

Table 14, continued

H_L = total head loss

q_m = maximum design flow rate

H_m = maximum allowable head loss

S = integrated sum of the solids removed in each time step (lb/ft^2)

Sqm = solids holding capacity of the filter (lb/ft^2)

Effluent Screens

Effluent screens are used for aesthetic reasons only and are assumed not to remove any SS or BOD_5.

Included in the treatment stream for cost computations.

Chlorination

Detention Time: 15 minutes at design flow.

If detention time is less than 15 minutes, chlorination efficiency is adjusted based on the ratio of the detention time to 15 minutes.

Chlorine Demand: estimated at 10% of BOD_5 content measured just ahead of the point of application. Chlorine demand is limited to not less than 6 ppm and not more than 25 ppm. Chlorination results in a BOD_5 reduction of 2 times the chlorine demand, but not more than 50% of the contact tank influent BOD_5 concentration.

$$\text{Coliforms at Point of Application} = \text{Coliforms in influent} \times \frac{\text{SS at point of application}}{\text{SS at influent to treatment units}}$$

Chlorination Efficiency: 0.999

Table 15. TREATMENT COST SUMMARY

Option	Derived Capital Cost Equation, Dollars	Applicable Capacity Range, mgd
Bar Racks Supply:	$1000(11 + \frac{S}{240})n(\frac{ENR}{1000})F$	All
	where n = Number of screens s = Screen capacity (cfs)-120, with a minimum of zero ENR = ENR index for prescribed year (see Volume III) F = Site factor (see Volume III)	
Install:	$10,000\ Q^{-.625}(\frac{ENR}{1034})F$	≤ 100
	$1,780\ Q(\frac{ENR}{1034})F$	> 100
	where Q = Capacity (mgd)	
Inlet Pumping	$25,000\ Q^{0.58}(\frac{ENR}{1314})F$	$\leq$ 20
	$16,000\ Q^{0.73}(\frac{ENR}{1314})F$	$20<Q\leq 100$
	$16,000\ Q(100)^{-0.27}(\frac{ENR}{1314})$	>100
Dissolved Air Flotation	$31,100\ Q\ (\frac{ENR}{1400})F$	All
Fine Screens	$12,000\ Q\ (\frac{ENR}{1314})F$	All
Microstrainers	$10,000\ Q\ (\frac{ENR}{1383})F$	All
High Rate Filters	$85,000\ Q^{0.67}\ (\frac{ENR}{1058})F$	All

Table 15, Continued

Sedimentation In New Tanks	$43{,}000(\frac{700Q}{R})^{0.91} (\frac{ENR}{1000})F$	<100
	$430\ Q\ (\frac{70{,}000}{R})^{0.91} (\frac{ENR}{1000})F$	$\geq$100
	where R = overflow rate (gpd/sq ft)	
In Storage	$\frac{UF}{27} (\frac{ENR}{1314})F$	All
	where U = Construction cost ($/cy) V = Maximum storage during storm (cy)	
Biological Treatment	$55{,}000\ Q(\frac{ENR}{1400})F$	All
Effluent Screens Supply and Install:	$5{,}000\ Q^{0.3} (\frac{ENR}{1314})F$	<100
	$200\ Q\ (\frac{ENR}{1314})F$	$\geq$100
Channel works:	$7{,}000\ Q^{0.625} (\frac{ENR}{1034})F$	<100
	$1{,}246\ Q\ (\frac{ENR}{1034})F$	$\geq$100
Outlet Pumping	(Same as Inlet Pumping)	
Chlorine Contact Tank & Equipment	$18{,}350\ Q^{0.628} (\frac{ENR}{1000})F$	
High Rate Disinfection	$2000\ (\frac{Q + Q_d}{1.546}) (\frac{ENR}{1314})F$	
	Q_d = Design flow (MGD)	

Table 16. IRREDUCIBLE MAINTENANCE COSTS

Treatment Process	Assumed Percentage of Total Capital Investment Per Year (%)
Bar screens	1
Pumping station, influent	2
Dissolved air flotation	2
Fine screens	2
Sedimentation tanks	1
Microstrainers	2
High rate filters	2
Effluent screens	1.5
Pumping station, effluent	2
Chlorine contact tanks	2

Table 17. STORM EVENT COSTS

Treatment Process	Assumed Variable Cost	Assumed Fixed Cost, $/storms
Bar racks	Based on the volume of solids to be disposed	15
Pumping station	Based on 2¢/kwh and assumed efficiencies	15
Dissolved air flotation	0.4¢/1,000 gal	15
Fine screens	0.6¢-1,000 gal	15
Sedimentation tanks	0.4¢/1,000 gal.	15
Microstrainers	0.6¢/1,000 gal.	15
High rate filters	1.0¢/1,000 gal.	15
Effluent screens	Based on the volume of solids to be disposed	15
Pumping station	Based on 2¢/kwh and assumed efficiencies	15
Chlorine contact tanks including chlorinators	Based on cost of chlorine 20¢/lb	15

SAMPLE DATA INPUT AND PROGRAM OUTPUT: STORAGE/TREATMENT BLOCK

The sample solution for the Storage/Treatment Block case study uses a physical-chemical treatment scheme in conjunction with storage facilities to completely contain and treat the stormwater flow resulting from the 10 year-2 hour duration design storm. A treatment rate of 125 MGD was chosen for this run. Based on this treatment rate, the mass flow diagram (Figure 19) indicated a required storage capacity of 5.0 x 10^6 gallons. Applying a 50% safety factor to allow for future development within the basin, a storage volume of 7.5 x 10^6 gallons was selected.

Descriptions of the treatment modules used and the storage facility are contained in the sample program output. Figure 22 compares the treatment effectiveness of the chosen scheme with an identical scheme without storage facilities and with the conventional treatment plant shown in Figure 17. Sample data input and sample program output are also provided.

STORAGE BLOCK CARD DATA

```
     9   10
     1    2     3     4     7
 STORAGE
              10
               1
       3      12    21    35    43    51    61    72
               2          1           1           1
         193.0
       1       3     6
               1           1
            30.
    250000.       2000.         0.
       193.0       1.2          .5
            0.      0.
            6.
         1600.      8.          0
           20.      1          10.        3.
     12       10
            7.     25         1975        1.
          1363   1545         1712      1886      2035      2180      2312      2431
          2537   2630         2710
        20000.    .02          .20      1.25       .03
 ENDPROGRAM
```

CARD GROUP

1
2
3
4
5
8
10
12
15
16
17
20
21
23
24
25
26

```
STORAGE  BLOCK CALLED


ENTRY MADE TO STORAGE/TREATMENT MODEL

STORAGE/TREATMENT MODEL UPDATED BY UNIVERSITY OF FLORIDA FEBRUARY 1975



 TEST CASE, 3000 ACRE BASIN
OUTPUT FROM EXTERNAL STORAGE/TREATMENT MODELS


INPUT DATA-SET OUTFALLS AT THE FOLLOWING ELEMENT NUMBERS:
10

INPUT TO STORAGE/TREATMENT MODEL SUPPLIED FROM EXTERNAL ELEMENT NUMBER  10


NUMBER OF RUNS             =        1

TIME-STEP SIZE             =    10.00 MIN.

NO. TIME-STEPS MODELED     =      150

TRIBUTARY AREA             =  3000.00 ACRES

NO. TRANSP. MOD. OUTFALLS  =        1

NO. OF POLLUTANTS          =        3

TIME ZERO                  =  43200.0 SEC
```

```
-------  RUN NO. 1  -------

INPUT DATA FOR TREATMENT PACKAGE FOLLOWS



                           CHARACTERISTICS OF THE TREATMENT PACKAGE ARE

                                LEVEL     MODE          PROCESS

                                  0        03         STORAGE ROUTED
                                  1        12           BAR RACKS
                                  2        21                                 (BYPASS)
                                  3        35         SEDIMENTATION
                                  4        43        HIGHRATE FILTERS
                                  5        51                                 (BYPASS)
                                  6        61                                 (BYPASS)
                                  7        72          CONTACT TANK
                                                                                  IPRINT = 2,  ICOST = 1,  IRANGE = 1,   ITABLE = 1

     SPECIFIED TREATMENT CAPACITY USED.

               DESIGN FLOWRATE =    193.00 CFS.
               TREATMENT SYSTEM INCLUDES MODULE UNITS
                    DESIGN FLOW IS THEREFORE INCREASED TO NEXT LARGEST MODULE SIZE.
                    ADJUSTED DESIGN FLOWRATE =     193.38 CFS.,  =    125.00 MGD.
                    (KMOD = 11)
                    CHARACTERISTICS OF STORAGE UNIT ARE
                         OUTLET TYPE  =  6
                         STORAGE MODE =  1
                         STORAGE TYPE =  3

                    IPOL = 1, PRINT CONTROL (ISPRIN) = 1

               MAN-MADE RESERVOIR, WITH MAX. DEPTH =  30.00 FT.,  AND CHARACTERISTICS
                         BASE AREA =  250000. SQ.FT.,        BASE CIRCUMF. =      2000. FT.,      COT(SIDESLOPE) =   0.00000
                    RESERVOIR OUTFLOW BY FIXED-RATE PUMPING
                    PUMPING RATE = 193.00 CFS, PUMPING START DEPTH =  1.20 FT, PUMPING STOP DEPTH =    .50 FT
                    DEPTH(FT) STOR(CU.FT)     DEPTH(FT) STOR(CU.FT)     DEPTH(FT) STOR(CU.FT)     DEPTH(FT) STOR(CU.FT)
                       0.00          0.          3.00     750000.          6.00   1500000.          9.00   2250000.
                      12.00   3000000.         15.00   3750000.         18.00   4500000.         21.00   5250000.
                      24.00   6000000.         27.00   6750000.         30.00   7500000.
                    STORAGE BETWEEN PUMP START AND STOP LEVELS =        1.51 TIMES (QPUMP*DT)
                    ASSUMED UNIT COST (EXCAVATION, LINING, ETC.) = 6.00 $/CU.YD.

               PRELIMINARY TREATMENT BY MECHANICALLY CLEANED BAR RACKS  (LEVEL 1)
                    NUMBER OF SCREENS    =       2
                    CAPACITY PER SCREEN =      96.69 CFS
                    SUBMERGED AREA       =     32.23 SQ.FT. (PERPENDICULAR TO THE FLOW)
                    FACE AREA OF BARS    =     45.12 SQ.FT.

               INFLOW BY GRAVITY (NO PUMPING)  (LEVEL 2)

               TREATMENT BY SEDIMENTATION (LEVEL 3) - (NO ASSOCIATED STORAGE)
                    DESIGN OVERFLOW RATE =   1600.00 GPD/SQ.FT.  (1600 SUGGESTED)
                    SED TANK DEPTH        =      8.00 FEET      (8 FEET SUGGESTED)
                    NUMBER OF SED TANKS  =         8
                    SURFACE AREA          =   9765.63 SQ.FT./TANK
                    NO CHLORINE ADDED
```

```
TREATMENT BY HIGH RATE FILTERS (LEVEL 4)
     NUMBER OF UNITS              =          4
     FILTER AREA PER UNIT         =    1085.32 SQ.FT
     MAX. OPERATING RATE          =      20.00 GPM/SQ.FT.
     SOLIDS REMOVAL EFFICIENCY    =      95.00 PERCENT
     BOD REMOVAL EFFICIENCY       =      80.00 PERCENT
     MAX. DESIGN HEAD LOSS        =      10.00 FT.
     MAX. SOLIDS HOLDING CAP.     =       3.00 LB/SQ.FT. (AT MAX H AND Q)
     CHEMICALS WILL BE ADDED

NO EFFLUENT SCREENS  (LEVEL 5)

OUTFLOW BY GRAVITY (NO PUMPING)  (LEVEL 6)

TREATMENT BY CHLORINE CONTACT TANK (LEVEL 7)
     NUMBER OF DOSING UNITS =          4
     DOSING RATE PER UNIT   =    8000.00 LB/DAY
     MAXIMUM DEMAND RATE    =   26077.10 LB/DAY
     VOLUME OF CONTACT TANK =    174038. CU.FT, AT 15 MIN. DETENTION TIME



PERFORMANCE PER TIME STEP
         NOTE-  NO BOD OR SS ARE REMOVED IN LEVELS 2, 5, & 6, REGARDLESS OF THE OPTIONS SELECTED
                NO SS REMOVALS IN LEVEL 7 (CHLORINE CONTACT TANK)
                LEVEL 1 & 5 REMOVALS (AT BAR RACKS AND EFFLUENT SCREENS) ARE REPORTED IN SUMMARY ONLY
                ***** IN HIGH RATE FILTER REMOVAL INDICATES MATERIAL IN FILTER BED



         NOTE:  TIME HISTORY OF FLOW RATES AND CONCENTRATIONS ARE PRINTED AT EACH TIME STEP
```

PERFORMANCE PER TIME STEP FOR BOD

			TREATMENT OPERATION									STORAGE OPERATION						OVERFLOW	
TIME	TOTAL INFLOWS		INFLOWS			BOD				OUTFLOWS		INFLOWS		STORAGE		OUTFLOWS		BYPASS	
	FLOW	BOD	FLOW	BOD		LEVEL-1	LEVEL-3	LEVEL-4	LEVEL-7	FLOW	BOD	FLOW	BOD	STORAGE	DEPTH	FLOW	BOD	FLOW	BOD
HR:MIN	CFS	MG/L	CFS	MG/L		MG/L	MG/L	MG/L	MG/L	CFS	MG/L	CFS	MG/L	CF	FT	CFS	MG/L	CFS	MG/L
12:10	58.57	199.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	58.6	199.	.18E+05	.1	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
12:20	58.14	196.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	58.1	196.	.53E+05	.2	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
12:30	58.26	190.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	58.3	190.	.88E+05	.4	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
12:40	58.33	193.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	58.3	192.	.12E+06	.5	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
12:50	55.91	215.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	55.9	215.	.16E+06	.6	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
13: 0	53.27	381.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	53.3	380.	.19E+06	.8	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
13:10	52.50	880.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	52.5	878.	.22E+06	.9	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
13:20	54.20	1419.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	54.2	1416.	.25E+06	1.0	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
13:30	59.75	1552.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	59.8	1549.	.29E+06	1.1	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
13:40	69.22	1336.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	69.2	1333.	.33E+06	1.3	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
13:50	82.86	918.	193.0	111.	IN	111.0	110.56	82.49	49.50	192.8	37.	82.9	916.	.26E+06	1.0	193.0	202.	0.0	0.
					OUT	110.6	82.49	49.50	37.50										
					REMOVED	.43	31545.4	238.3	12.0										
14: 0	99.74	550.	193.0	202.	IN	201.9	201.37	151.99	74.57	190.6	60.	99.7	549.	.19E+06	.8	193.0	940.	0.0	0.
					OUT	201.4	151.99	74.57	59.65										
					REMOVED	.57	4138.9	552.8	14.9										
14:10	120.83	363.	193.0	940.	IN	940.0	938.36	747.25	149.45	180.7	119.	120.8	363.	.15E+06	.6	193.0	478.	0.0	0.
					OUT	938.4	747.25	149.45	119.56										
					REMOVED	1.65	3731.2	4045.6	29.9										
14:20	144.48	240.	193.0	478.	IN	477.8	476.79	371.54	74.31	148.4	59.	144.5	24[illegible].	.11E+06	.4	193.0	634.	0.0	0.
					OUT	476.8	371.54	74.31	59.45										
					REMOVED	.97	2793.6	532.1	14.9										
14:30	169.96	150.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	170.0	150.	.20E+06	.8	0.0	0.	0.0	0.

					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
14:40	197.97	94.	0.0	0.	IN	0.0	0.00	0.00	0.00	0.0	0.	198.0	94.	.31E+06	1.3	0.0	0.	0.0	0.
					OUT	0.0	0.00	0.00	0.00										
					REMOVED	0.00	0.0	0.0	0.0										
14:50	228.74	66.	193.0	58.	IN	58.4	58.07	43.54	13.06	155.6	1.	228.7	66.	.33E+06	1.3	193.0	351.	0.0	0.
					OUT	58.1	43.54	13.06	1.06										
					REMOVED	.36	2568.1	594.4	12.0										
15: 0	260.47	52.	193.0	351.	IN	350.7	349.94	269.78	80.93	150.4	65.	260.5	52.	.36E+06	1.4	193.0	113.	0.0	0.
					OUT	349.9	269.78	80.93	64.75										
					REMOVED	.79	2691.6	980.6	16.2										
15:10	291.38	46.	193.0	113.	IN	112.8	112.37	84.44	25.33	155.2	13.	291.4	45.	.41E+06	1.6	193.0	70.	0.6	0.
					OUT	112.4	84.44	25.33	13.33										
					REMOVED	.44	3574.8	1101.7	12.0										
15:20	320.45	43.	193.0	70.	IN	70.2	69.78	52.19	15.66	192.4	4.	320.4	43.	.47E+06	1.9	193.0	56.	0.0	0.
					OUT	69.8	52.19	15.66	3.66										
					REMOVED	.37	5499.8	263.3	12.0										
15:30	347.35	39.	193.0	56.	IN	56.4	56.09	41.88	8.38	192.7	4.	347.4	39.	.56E+06	2.2	193.0	45.	0.0	0.
					OUT	56.1	41.88	8.38	4.19										
					REMOVED	.35	9146.5	241.8	4.2										
15:40	371.32	36.	193.0	45.	IN	45.5	45.12	33.66	6.73	156.6	3.	371.3	36.	.66E+06	2.6	193.0	40.	0.0	0.
					OUT	45.1	33.66	6.73	3.37										
					REMOVED	.34	15286.8	58.4	3.4										
15:50	392.35	34.	193.0	40.	IN	39.8	39.58	29.46	8.84	156.6	4.	392.4	34.	.77E+06	3.1	193.0	37.	0.0	0.
					OUT	39.5	29.46	8.84	4.42										
					REMOVED	.33	19364.7	94.4	4.4										
16: 0	410.26	33.	193.0	37.	IN	37.3	36.96	27.56	8.27	156.6	4.	410.3	33.	.96E+06	3.6	193.0	35.	0.0	0.
					OUT	37.0	27.56	8.27	4.13										
					REMOVED	.32	18118.1	135.7	4.1										
16:10	425.25	32.	193.0	35.	IN	35.2	34.83	25.98	7.79	156.6	4.	425.2	32.	.10E+07	4.1	193.0	32.	0.0	0.
					OUT	34.8	25.98	7.79	3.90										
					REMOVED	.32	17076.5	174.5	3.9										
16:20	448.47	32.	193.0	32.	IN	32.2	31.86	23.76	7.13	192.9	4.	448.5	32.	.12E+07	4.7	193.0	30.	0.0	0.
					OUT	31.9	23.76	7.13	3.56										
					REMOVED	.32	15617.7	120.2	3.6										
16:30	448.01	32.	193.0	30.	IN	30.5	30.16	22.49	4.50	192.9	2.	448.0	32.	.13E+07	5.3	193.0	28.	0.0	0.
					OUT	30.2	22.49	4.50	2.25										
					REMOVED	.31	14783.5	130.0	2.2										
16:40	469.46	31.	193.0	28.	IN	28.1	27.79	20.73	4.15	192.9	2.	469.5	31.	.15E+07	6.0	193.0	27.	0.0	0.
					OUT	27.8	20.73	4.15	2.07										
					REMOVED	.31	13626.1	119.8	2.1										
16:50	461.79	30.	193.0	27.	IN	27.0	26.64	19.87	3.97	192.9	2.	461.8	30.	.17E+07	6.6	193.0	26.	0.0	0.
					OUT	26.6	19.87	3.97	1.99										
					REMOVED	.31	13060.8	114.9	2.0										
17: 0	476.89	30.	193.0	26.	IN	25.6	25.30	18.87	3.77	192.9	2.	476.9	30.	.18E+07	7.3	193.0	25.	0.0	0.
					OUT	25.3	18.87	3.77	1.89										
					REMOVED	.31	12404.0	109.1	1.9										
17:10	469.52	31.	193.0	25.	IN	24.7	24.36	18.17	3.63	192.9	2.	469.5	31.	.20E+07	8.0	193.0	24.	0.0	0.

SUMMARY OF TREATMENT EFFECTIVENESS

TOTALS	FLOW (M.G.)	BOD (LB)	SS (LB)	COLIF (MPN)
INPUT	115.202	51178.1	149615.8	1.44E+16
OVERFLOW (BYPASS)	0.000	0.0	0.0	0.
TREATED	115.202	51178.1	149615.8	1.44E+16
REMOVED	1.512	46355.3	142041.7	1.44E+16
RELEASED	113.690	4822.8	7574.1	3.70E+11

REMOVALS	FLOW(M.G.)	BOD (LB)	SS (LB)	
LEVEL 1	.005	259.1	5181.9	= BAR RACKS
LEVEL 3 (TOTAL)	.205	12941.3	66742.9	= SEDIMENTATION
LEVEL 4	1.302	29739.2	70116.9	= HIGHRATE FILTERS
LEVEL 5	0.000	0.0	0.0	= NO EFFL. SCREENS
LEVEL 7	0.000	3415.7	0.0	= CONTACT TANK

TRASH-
BAR RACKS 690.924 CU.FT (AT 50 LB/CU.FT.)
EFFLUENT SCREENS 0.000 CU.FT (AT 50 LB/CU.FT.)

REMOVAL PERCENTAGES	FLOW (VOL)	BOD (LB)	SS (LB)	COLIF (MPN)
OF OVERALL INPUTS	1.31	90.58	94.94	100.00
OF TREATED FRACTIONS	1.31	90.58	94.94	100.00

CONSUMPTIONS (LB)	CHLORINE	POLYMERS	
LEVEL 3	0.0	0.0	= SEDIMENTATION
LEVEL 4	0.0	3839.1	= HIGHRATE FILTERS
LEVEL 7	5784.2	0.0	= CONTACT TANK
TOTAL	5784.2	3839.1	

REPRESENTATIVE VARIATION OF TREATMENT PERFORMANCE WITH TIME (OVERALL).

TIME	12/10	14/30	16/50	19/10	21/30	23/50	2/10	4/30	6/50	9/10	11/30
WATER											
AV. FLOW (CFS)	0.00	0.00	192.95	192.95	192.95	192.95	192.95	192.95	192.95	192.95	192.95
BOD											
ARRIVING (MG/L)	0.00	0.00	26.95	19.29	28.72	18.64	33.26	37.99	54.74	33.83	75.38
RELEASED (MG/L)	0.00	0.00	1.98	1.41	1.52	1.37	2.45	2.81	4.05	2.50	5.59
REDUCTION (PCT)	0.00	100.00	92.63	92.66	92.65	92.66	92.62	92.61	92.59	92.62	92.58
S. SOLIDS											
ARRIVING (MG/L)	0.00	0.00	15.71	8.16	8.32	6.89	11.82	14.98	18.18	17.18	34.73
RELEASED (MG/L)	0.00	0.00	.28	.07	.08	.04	.17	.26	.34	.32	.79
REDUCTION (PCT)	0.00	100.00	98.23	99.09	99.06	99.42	98.54	98.28	98.11	98.16	97.73
COLIFORMS											
ARR (MPN/100ML)	0.	0.	8.28E+05	1.00E+06	1.48E+06	1.89E+06	1.66E+06	3.55E+06	6.22E+06	2.64E+06	8.96E+06
REL (MPN/100ML)	0.	0.	1.46E+01	9.10E+00	1.40E+01	1.09E+01	2.42E+01	6.10E+01	1.17E+02	4.87E+01	2.03E+02
REDUCTION (PCT)	0.00	0.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

SUMMARY OF BYPASS FLOW AND POLLUTANTS

OVERFLOWS OCCURRED DURING 0 TIME STEPS

FLOW (CU. FT)	BOD (LB)	SS (LB)	COLIF (MPN)
0.	0.0	0.0	0.

SUMMARY OF FLOWS - MAXIMA, AVERAGES, AND MINIMA

	ARRIVING	OVERFLOW	TO TREATMENT	LEVEL 3 REMOVAL	LEVEL 3 OUTFLOW	LEVEL 4 REMOVAL	LEVEL 4 OUTFLOW	LEVEL 7 REMOVAL	LEVEL 7 OUTFLOW	RECOMBINED RELEASE
FLOW RATES (M.G.D.)										
MAXIMUM	124.758	0.000	124.758	7.970	124.687	23.438	124.687	0.000	124.687	124.687
AVERAGE	110.618	0.000	110.618	.197	110.416	1.250	109.166	0.000	109.166	109.166
MINIMUM	0.000	0.000	0.000	.065	116.782	0.000	95.906	0.000	95.906	0.000
BOD CONCENTRATIONS (MG/L)										
MAXIMUM	940.0	0.0	940.0	134847.2	747.3	4045.6	149.5	29.9	119.6	119.4
AVERAGE	47.3	0.0	47.3	15036.4	35.5	189.8	8.0	3.3	4.7	4.7
MINIMUM	0.0	0.0	0.0	2568.1	13.3	56.4	2.7	1.3	1.1	0.0
SUSPENDED SOLIDS CONCENTRATIONS (MG/L)										
MAXIMUM	6937.6	0.0	6937.6	50043.3	3975.4	25558.3	335.7	0.0	335.7	335.9
AVERAGE	138.2	0.0	138.2	10480.9	74.6	674.4	8.0	0.0	8.0	8.0
MINIMUM	0.0	0.0	0.0	1126.6	.7	4.7	.0	0.0	.0	0.0
COLIFORM CONCENTRATIONS (MPN/100ML)										
MAXIMUM	2.26E+07	0.	2.26E+07						2.15E+03	2.15E+03
AVERAGE	2.94E+06	0.	2.94E+06						7.78E+01	7.78E+01
MINIMUM	0.	0.	0.						0.	0.

INLET HYDROGRAPH - CFS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
10	58.571	58.139	58.255	58.325	55.911	53.267	52.499	54.202	59.750	69.216
	82.858	99.745	120.827	144.480	169.962	197.966	228.741	260.468	291.377	320.447
	347.353	371.320	392.354	410.258	425.250	448.474	448.009	469.460	461.794	476.888
	469.522	482.776	469.500	480.412	474.569	478.338	478.853	475.484	469.429	460.784
	450.549	438.709	426.104	412.755	398.857	384.922	371.053	357.375	343.528	330.430
	317.758	305.083	293.160	281.686	270.657	260.362	250.436	240.975	231.973	223.319
	215.154	207.553	200.259	193.317	186.740	180.543	173.577	168.798	163.802	158.902
	154.090	149.477	145.829	141.068	136.903	133.191	129.715	126.433	122.169	120.034
	117.518	114.969	112.474	110.006	107.422	105.220	103.053	100.953	98.944	96.984
	94.992	93.201	91.504	89.857	88.242	86.685	85.042	83.636	82.237	80.857
	79.506	78.184	76.762	75.595	74.404	73.198	71.976	70.812	69.723	68.547
	67.362	66.215	65.104	64.026	63.288	62.157	61.054	60.033	59.075	58.165
	58.384	56.604	55.413	54.405	53.510	52.719	53.299	51.677	50.545	49.559
	48.979	48.380	47.357	47.198	46.960	46.655	46.320	45.942	45.546	45.139
	44.708	44.370	44.093	43.870	43.784	43.777	43.782	43.808	43.855	43.923

INLET POLLUTOGRAPHS

BOD IN LB/MIN

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
10	43.606	42.686	41.388	41.974	45.011	75.797	172.662	287.599	346.691	345.699
	284.327	205.108	164.135	129.834	95.360	69.756	56.684	50.641	49.658	51.408
	51.086	50.303	50.423	50.138	51.278	53.593	52.968	53.667	52.415	53.779
	54.692	59.849	59.098	60.009	58.929	59.926	56.780	52.945	52.005	52.649
	52.412	52.437	60.929	75.535	79.602	77.785	78.037	78.115	72.701	62.965
	59.454	60.564	60.538	60.417	65.884	75.621	79.011	78.055	78.115	78.175
	74.243	66.685	63.386	63.939	64.032	63.966	57.618	44.087	36.563	37.384
	37.708	37.497	46.427	64.622	74.413	74.134	73.752	73.905	63.592	40.721
	26.654	27.175	27.839	27.526	25.986	22.642	20.751	20.805	20.866	20.838
	19.855	17.763	16.551	16.531	16.576	16.562	15.222	12.350	10.639	10.559
	10.613	10.599	10.705	10.889	10.904	10.879	10.882	10.884	11.107	11.618
	11.980	12.037	12.029	12.031	13.716	17.628	20.587	21.223	21.189	21.197
	23.432	29.267	34.966	36.703	37.001	36.967	37.641	40.494	45.938	48.418
	49.109	48.991	55.534	70.391	80.145	82.400	82.617	82.660	77.513	65.504
	56.761	54.621	54.286	54.228	52.563	48.401	45.218	44.449	44.345	44.322

SUSPENDED SOLIDS IN LB/MIN

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
10	29.486	38.719	132.515	335.895	502.515	774.605	1487.078	2474.570	3339.552	3863.462
	3637.960	2857.935	2227.587	1608.771	967.019	475.211	226.881	117.677	67.550	51.045
	44.952	42.171	41.139	40.118	40.654	42.173	41.567	42.023	40.996	42.034
	43.334	48.368	47.911	48.467	47.647	48.430	44.900	40.133	38.982	39.762
	39.490	39.512	45.207	55.064	57.870	56.562	56.786	56.841	54.932	51.463
	50.066	50.460	50.486	50.419	52.655	56.599	57.928	57.540	57.579	57.597
	53.691	46.090	42.716	43.239	43.326	43.259	39.344	30.850	25.938	26.360
	26.575	26.450	29.167	34.980	38.524	38.574	38.415	38.471	35.776	29.254
	24.408	24.295	24.555	24.449	23.982	22.901	22.147	22.116	22.144	22.132
	22.109	21.998	21.831	21.796	21.808	21.806	19.053	13.096	9.477	9.220
	9.312	9.283	9.735	10.688	11.191	11.219	11.211	11.216	13.716	19.273
	22.920	23.465	23.432	23.450	23.176	22.808	23.099	23.305	23.265	23.263
	23.306	24.022	25.967	26.716	26.892	26.849	29.125	35.395	42.820	45.998
	46.947	46.979	45.450	41.104	36.443	34.880	34.536	34.460	32.831	29.043
	26.180	25.392	25.236	25.205	24.900	24.044	23.315	23.104	23.065	23.057

TREATED OUTFLOW HYDROGRAPH - CFS

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	192.820	190.606	180.662	148.366	0.000	0.000	155.625	150.360	155.192	192.369
	192.691	156.589	156.634	156.634	156.634	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891	192.891
	192.891	192.891	192.882	192.833	192.841	0.000	0.000	0.000	0.000	0.000

TREATED OUTFLOW POLLUTOGRAPHS

BOD IN LB/MIN

EXTERNAL ELEMENT NUMBER	TIME STEP 1	2	3	4	5	6	7	8	9	10
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	27.028	42.506	80.748	32.971	0.000	0.000	.617	36.393	7.735	2.630
	3.017	1.970	2.588	2.421	2.282	2.570	1.622	1.495	1.433	1.361
	1.310	1.250	1.221	1.198	1.176	1.173	1.153	1.115	1.095	1.084
	1.072	1.058	1.021	1.013	1.025	1.025	1.024	1.029	1.025	1.100
	1.112	1.105	1.095	1.093	1.110	1.110	1.098	1.083	1.081	1.102
	1.101	1.070	1.037	1.036	.968	.968	.964	.960	.962	.987
	.987	.998	1.003	1.011	1.028	1.032	1.231	1.231	1.500	1.577
	1.663	1.719	1.728	1.738	1.772	1.810	1.846	1.881	1.842	1.819
	1.671	1.635	1.606	1.644	1.704	1.758	1.789	1.843	2.026	2.216
	2.582	2.702	2.763	2.829	2.916	3.020	2.981	2.817	2.745	2.811
	2.922	3.026	2.925	2.403	1.971	2.047	2.134	2.514	3.798	.863
	1.158	1.151	3.442	2.003	2.136	2.103	1.802	1.808	1.841	1.623
	1.615	1.554	1.168	1.162	1.240	1.300	1.404	1.579	1.978	3.064
	4.034	3.479	6.466	12.574	26.004	0.000	0.000	0.000	0.000	0.000

SUMMARY OF TREATMENT COSTS

ASSUMED FUTURE ENGINEERING NEWS RECORD INDICES
CONSTRUCTION - 20 CITY AVERAGE

YEAR	ENR INDEX
1970	1363
1971	1545
1972	1712
1973	1886
1974	2035
1975	2180
1976	2312
1977	2431
1978	2537
1979	2630
1980	2710

COST PARAMETERS . .
INTEREST RATE = 7.00 PERCENT
AMORTIZATION PERIOD = 25 YEARS
CAP. RECOVERY FACTOR = .0858
YEAR OF SIMULATION = 1975
SITE LOCATION FACTOR = 1.0000

UNIT COSTS . .
LAND = 20000.00 $/ACRE
POWER = .020 $/KWH
CHLORINE = .200 $/LB
POLYMERS = 1.250 $/LB
ALUM = .03 $/LB

		CAPITAL COSTS		ANNUAL COSTS			STORM EVENT COSTS		
TREATMENT	LEVEL	INSTAL	LAND	INSTAL	LAND	MIN MAINT	CHLORINE	CHEM	OTHER
BAR RACKS	1	517061.	2409.	44369.	169.	5171.	0.	0.	442.
NO INLET PUMPING	2	0.	0.	0.	0.	0.	0.	0.	0.
SEDIMENTATION	3	0.	229484.	0.	16064.	0.	0.	0.	576.
HIGHRATE FILTERS	4	4449585.	7968.	381821.	558.	88992.	0.	9118.	1165.
NO EFFL. SCREENS	5	0.	0.	0.	0.	0.	0.	0.	0.
NO OUTLET PUMPS	6	0.	0.	0.	0.	0.	0.	0.	0.
CONTACT TANK	7	829759.	14697.	71202.	1029.	16595.	1157.	0.	15.
SUBTOTAL		$ 5796404.	$ 254559.	$ 497392.	$ 17819.	$ 110757.	$ 1157.	$ 9118.	$ 2198.
TOTAL		$ 6050963.		$ 625969.			$ 12472.		
TOTAL PER TRIB ACRE		$ 2017.		$ 209.			$ 4.		

TOTAL LAND REQUIREMENT = 7.96 ACRES.

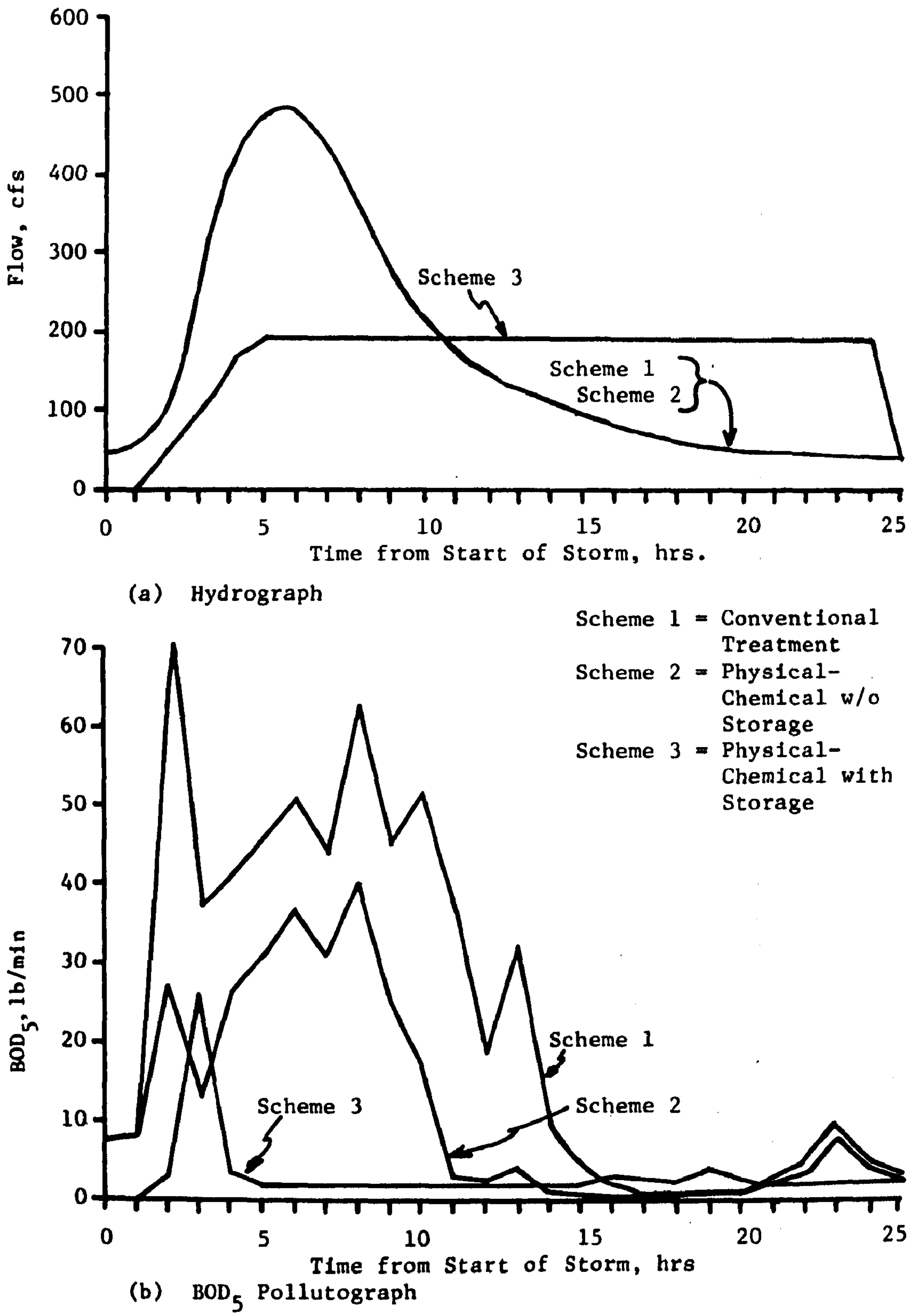
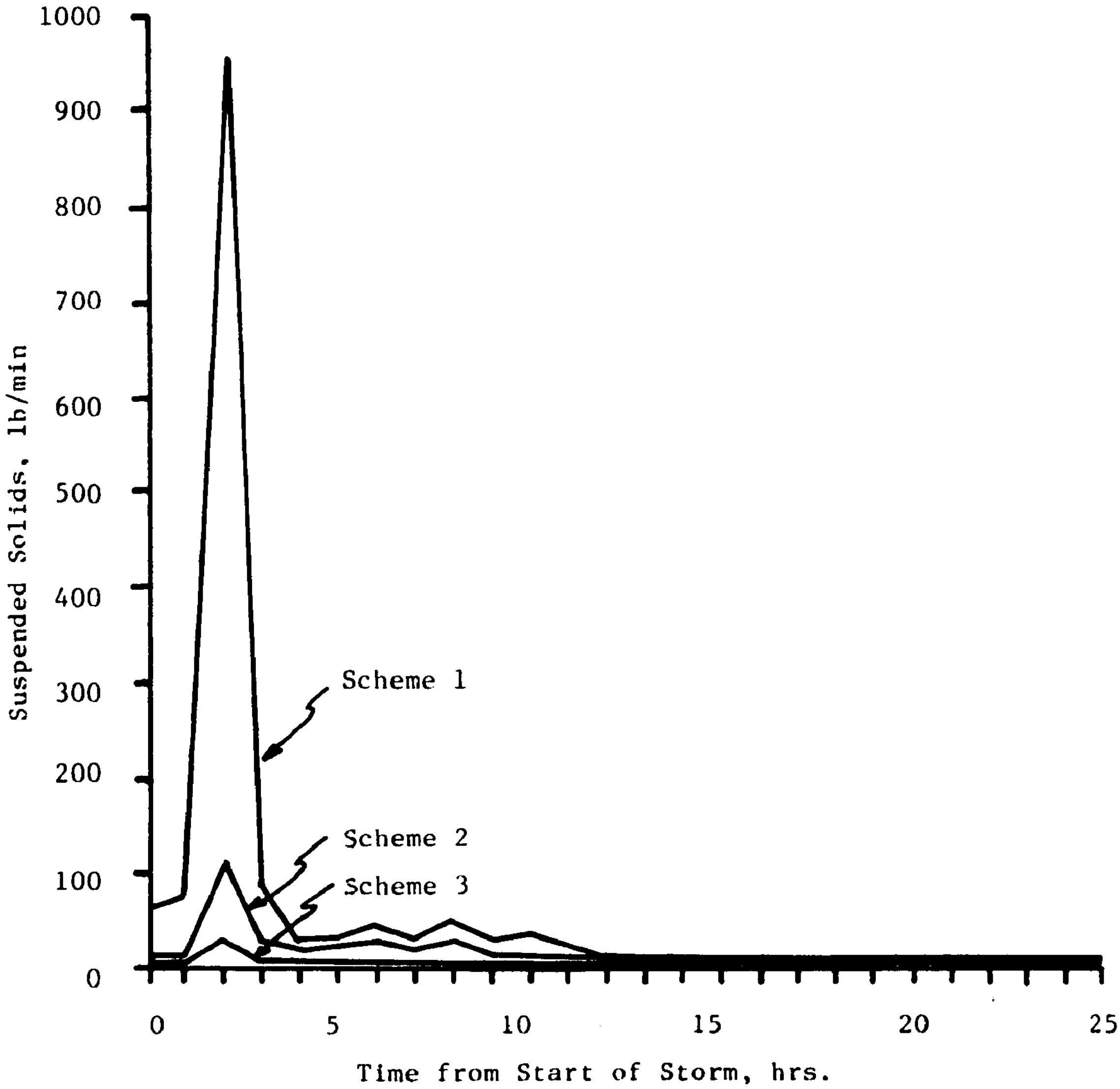


FIGURE 22. OUTFLOW HYDROGRAPH AND POLLUTOGRAPHS FROM STORAGE/TREATMENT BLOCK



(c) Suspended Solids Pollutograph

FIGURE 22, CONTINUED

PREPARATION OF RECEIVING WATER BLOCK DATA

Data Gathering

Prior to application of the Receiving Water model, data for physical and meteorological characteristics as well as data to define boundary and initial conditions are required. Most data are either available or measurable; however, the complete description of the system may require considerable field work.

Physical data include tidal variations (for estuarine systems), water surface elevations and areas, water depths and roughness coefficients. Tidal variation data can be acquired from the National Oceanographic and Atmospheric Administration. Water surface elevations (referenced to datum) may be available at some points in the field where stage recorders are installed. Otherwise, the stage can be computed by that portion of Receive which models the hydrodynamics of the receiving water body. Surface areas and water depths can be determined from U.S. Coast and Geodetic Survey nautical charts or U.S. Geological Survey quadrangle maps. Manning's roughness coefficients can be estimated using standard tables as given in Reference 5. Meteorological data required include rainfall, evaporation, and wind velocity and direction. These data, or information on additional sources of data, should be available from the U. S. Weather Bureau.

A required boundary condition is the mode of downstream flow control; options available are tidal exchange, dam (weir) or specified outflow. For tidal conditions, a tidal exchange ratio i.e. the ratio of the returning outflow from prior ebb tide to the total inflowing flood tide must be supplied. The tidal exchange ratio can be estimated from dye tracer or pollutant analysis. For weir boundary conditions, the weir factor, W_f, power law for the weir, X, and height of the weir, H, referenced to the datum must be supplied. The SWMM then uses Equation 9 to calculate flow over a weir.

$$Q = W_f H^X \qquad (9)$$

where Q = Flow, cfs

W_f = Weir Discharge Coefficient (C_d) x Weir Width (ft)

X = Power Law for Weir

For a rectangular suppressed weir, values of 3.33 for C_d and 1.5 for X are generally accepted. Hydrograph data must be supplied to the model for specified outflow boundary conditions. Data to generate the hydrograph can be taken from stage-discharge rating curves based on the type of downstream boundary.

Optional boundary conditions are constant inflows to, or outflows from, any of the modeled nodes (junctions). Either quantity or quality constituents can be introduced or removed from the system in this manner. This option is mainly used to introduce flows at the upper end of the water body to be modeled; quantity and quality constituents must be measured in the field.

At the beginning of simulation, the initial conditions required for input include velocity of channel flow, pollutant concentrations and dissolved oxygen saturation level (at the boundary nodes), reaeration coefficient, and first order decay coefficient for non-conservative pollutants. Velocity of channel flow can be derived from flow/area data for stream segments. For estuaries or lakes, velocity data can either be taken from a previous computer simulation or set equal to arbitrary values and allowed to equilibrate during the initial day of simulation. Pollutant concentrations at the boundary node must be measured. Saturation concentrations for dissolved oxygen can be found from tables based on water temperature and chloride concentrations, (Page 480 of Reference 12 contains such a table). Reaeration coefficients and first order decay coefficients can be estimated from laboratory tests conducted on the receiving water or from values found in the literature for waters of similar characteristics. In addition to the concentrations at the boundary nodes, an optional initial condition is the concentration of pollutants at nodes to be modeled. Because these parameters would be impractical to measure in the field, they can either be estimated from previous computer simulations or set arbitrarily.

Discretization of the Receiving Water

The model simulates the receiving body of water as a system of links (channels) and nodes (junctions) organized in a linear, triangular or higher order polygon configuration. The SWMM is designed to accept up to 100 junctions and 225 channels. Figure 23 shows a representative receiving water divided into linear, triangular, quadrilateral and pentagonal elements; here, circled numbers refer to channels and uncircled to junctions.

Choice of the discretization scheme to be used is based on engineering judgments and will generally depend on the size and complexity of the receiving water system. The maximum permissible time step depends on the values of channel length and depth as given by Equation 10.

$$\Delta t \leq 0.75 \ (L/\sqrt{gd})_{MIN} \tag{10}$$

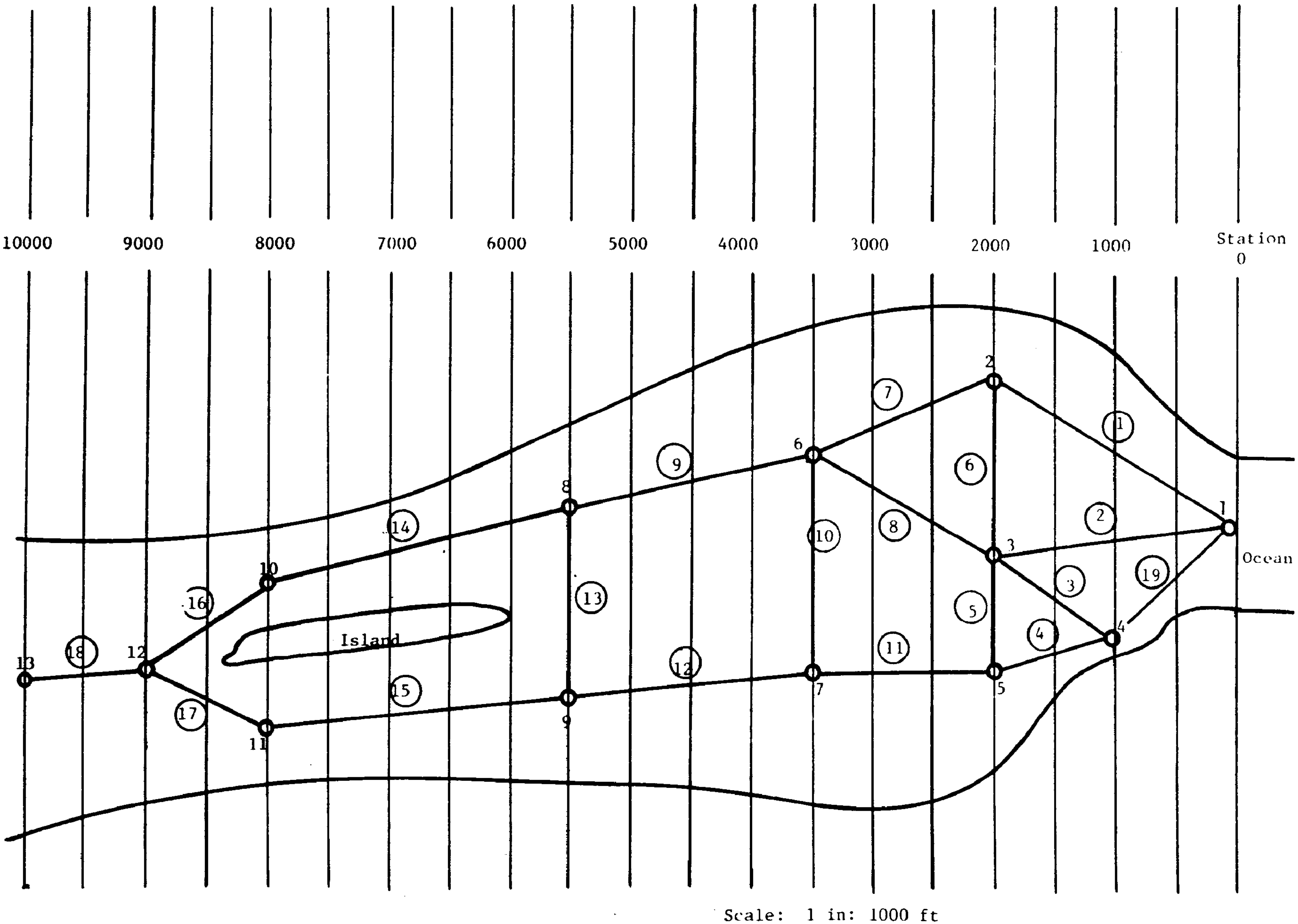


FIGURE 23. REPRESENTATIVE RECEIVING WATER.

where Δt = time step length, sec., usually between 30 and 300 sec.

L = length of channel, ft

d = average depth of channel, ft

Channels must be of sufficient length to provide a realistic time step size. The grid system should be arranged so that it portrays all significant changes in receiving water geometry and profile.

Junction and Channel Data Preparation

Required data preparation involves describing the physical system and boundary and initial conditions. Boundary and initial conditions have been discussed under data gathering. Physical system data can be divided into junction and channel data.

Junction data include surface elevation, surface area and depth. The surface elevation at the junction at the start of the simulation is referenced to the standard datum chosen. For estuarine systems the datum is usually taken as mean low low water. For bodies of water where surface elevation is governed by a downstream weir, the elevation of the top of the weir may provide a convenient datum. Any datum may be used as long as all measurements are referenced to that elevation.

Surface area attributable to each junction can be found by constructing the Thiessen polygon surrounding the junction and planimetering the area. Polygons are constructed by locating perpendicular bisectors through the lines joining that junction and all adjacent junctions. Each of these perpendicular bisectors represents the locus of points equidistant from two junctions. The closed figure formed by these bisectors defines the area belonging to each junction. A typical polygon thus formed is designated as a-b-c-d-e-f-a in Figure 24. In Figure 24, perpendicular bisectors have to be drawn through some lines that are not being modeled as channels; for example, line b-c of Figure 24 is the perpendicular bisector of the line drawn between junctions 7 and 8.

Depth is the distance measured from the junction surface to the bottom of the water body. According to this convention, depth measurements are positive in the downward direction.

Figure 24 shows the Thiessen polygon network constructed for the receiving water shown previously in Figure 23. Table 18 gives a tabulation of the junction data for this sytem. Figure 24 and Table 18 will be used for the case study.

Channel data include length, average width and average depth. These are in addition to Manning's roughness coefficient and initial channel velocity which were discussed previously. The length of the channel is the distance along the channel centerline between the junctions on either end. The channel axis need not be a straight line. The average width is taken

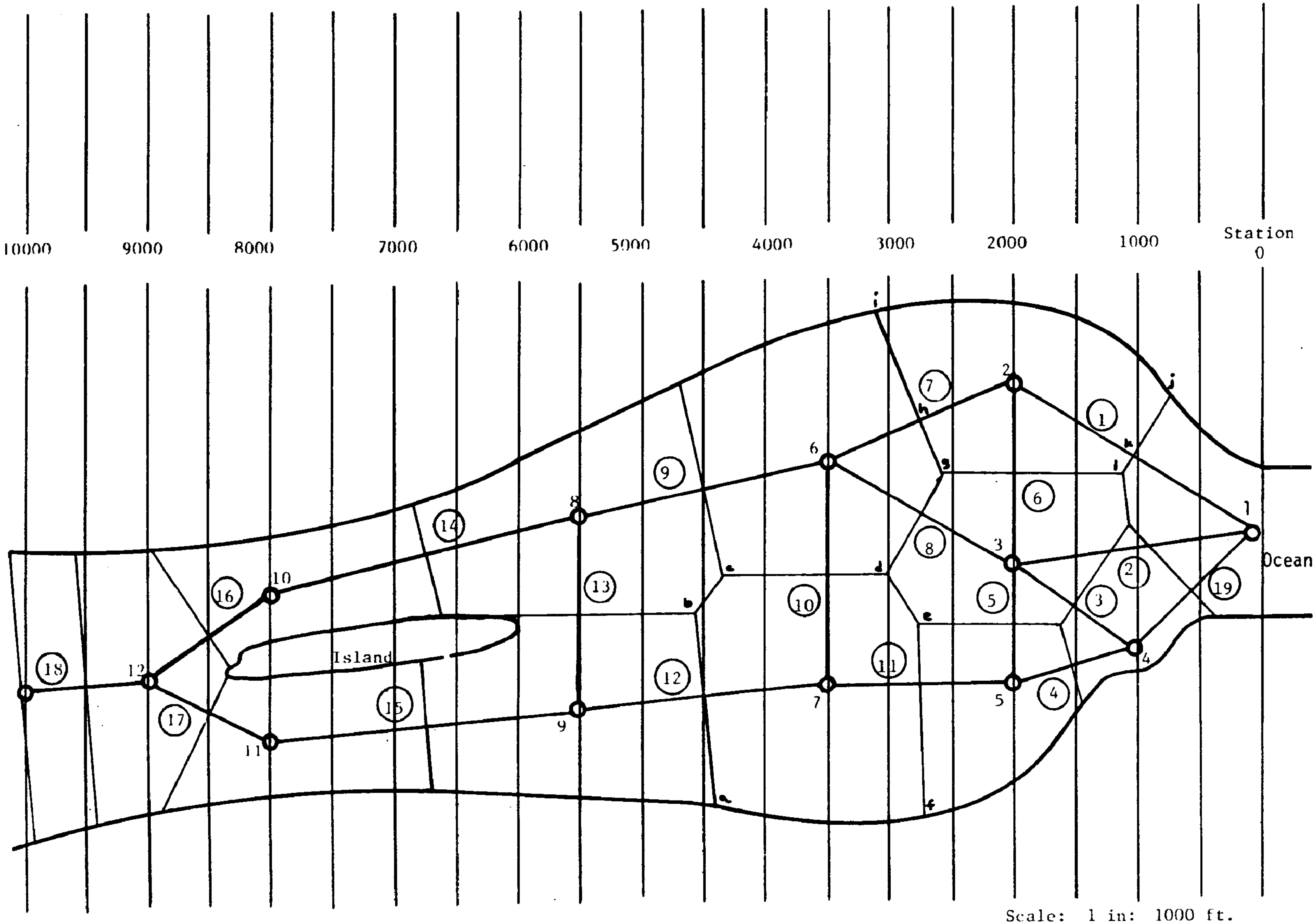


FIGURE 24. SAMPLE RECEIVING WATER SHOWING THIESSEN POLYGON CONSTRUCTION.

Table 18. JUNCTION DATA

Junction Number	Surface Elevation (ft)	Surface Area ($10^6 ft^2$)	Constant Junction Flow In (cfs)	Constant Junction Flow Out (cfs)	Junction Depth (ft)
1	0	1.38			15.0
2	0	2.59			13.0
3	0	2.04			13.0
4	0	0.99			14.0
5	0	1.58			13.0
6	0	4.32			11.5
7	0	3.55			11.5
8	0	3.46			9.5
9	0	3.39			10.4
10	0	1.85			7.0
11	0	2.09			10.2
12	0	2.01			10.1
13	0	1.25	1000		10.0

as the length of the perpendicular bisector through the channel. Thus, in Figure 24 distance a-b would be the average width of channel 12 and distance d-g the average width of channel 8. Assuming a uniform slope between junctions, the average depth of a channel is the average of the junction depths on either end. Table 19 gives the channel data derived for the system shown in Figure 24 and will be used in the case study.

For triangular networks, the program has the capability of generating junction areas and channel lengths and widths for the interior of acute triangles. Required data are the coordinates of each junction used in the computations. Since the triangulation subroutine computes data only for the interior of triangles, the area assigned to junction 2 of Figure would be area 2-k-l-g-h-2 and the width of channel 1 would be distance l-k. The user must add area 2-h-i-j-k-2 to junction 2 in card group 15. Similarly, an additional channel must be modeled between junctions 1 and 2 to account for the channel defined by width k-j. Therefore, unless an extensive network of triangular elements is to be modeled, it is recommended that geometric data be determined using the Thiessen polygon method.

Having described the receiving water system, the model can be run using the output of the Runoff, Transport or Storage/Treatment Block (or a combination of two or more) as the input to the Receiving Water Block. Several optional data inputs may be used to allow the user to more accurately model the receiving water system provided that such data are available.

Optional Data Input

Options available in the quantity portion of the Receiving Water Block include multiple downstream boundary conditions; spatially variable rainfall; stormwater input from cards; parallel channels between the same two junctions; Manning's roughness coefficient computed at each time step; and plots of stage versus time for selected junctions. Quality options include the ability to re-start the quality simulation; variable oxygen saturation coefficients; and variable reaeration coefficients. Each of these options will be described briefly. Table 20 has been prepared to aid the user in determining which card groups are to be completed for a standard run and for the various options available. A more detailed set of data preparation instructions are contained on pages 273-288 of the User's Manual (Reference 6).

Multiple downstream boundary conditions can also be simulated for systems influenced tidally, systems controlled by weirs, or a combination of these. For example, multiple boundary conditions could be used to represent more than one outlet to the sea in the case of a tidal system, or a dam with spillways at two different elevations in the case of a weir controlled system. An estuarine system with relief works to control the water level would represent a combination of a tidal and weir controlled system.

Another optional input is spatially variable rainfall where two or more rainguages are used to gather rainfall data at different locations in the system. However, the User's Manual states that unless the receiving waters are shallow or are characterized by low flows, rainfall is likely to have

TABLE 19. CHANNEL DATA

Channel Number	Lower Junction	Upper Junction	Length of Channel (ft)	Average Width (ft)	Average Depth (ft)	Manning's n	Initial Velocity
1	1	2	2325	750	14.0	0.025	0
2	1	3	2000	1250	14.0	0.025	0
3	3	4	1225	1000	13.5	0.025	0
4	4	5	1025	650	13.5	0.025	0
5	3	5	950	1150	13.0	0.025	0
6	2	3	1475	1450	13.0	0.025	0
7	2	6	1675	1400	12.3	0.025	0
8	3	6	1725	950	12.3	0.025	0
9	6	8	2100	1575	10.5	0.025	0
10	6	7	1800	1350	11.5	0.025	0
11	5	7	1525	1600	12.3	0.025	0
12	7	9	2050	1550	11.0	0.025	0
13	8	9	1550	1500	10.0	0.025	0
14	8	10	2600	950	8.3	0.025	0
15	9	11	2550	1075	10.3	0.025	0
16	10	12	1225	1125	8.5	0.025	0
17	11	12	1125	1250	10.1	0.025	0
18	12	13	1000	2250	10.0	0.025	0
19	1	4	1250	500	14.5	0.025	0

Table 20. RECEIVE DATA PREPARATION REQUIREMENTS

I. Standard Run

Card Groups	Comments
1,2,3	
4	ISWCH(1) = 0, 1 or 2
5,6,7,8,9	
11,12	if modeling tidal system
13	downstream weir boundary
15,16,17,18,19,20,21,30,31,33,34, 35,36,37	

II. Multiple Boundary Conditions

Card Groups	Comments
4	ISWCH(1) = 3
10	
11,12	tidal system
13	wier boundary

III. Spatially Variable Rainfall

Card Groups	Comments
4	ISWCH(3) = 1
23,24,25,27	

IV. Stormwater Input from Cards

Card Group	Comments
5	NJSW
26	if spatially varied rainfall is input.
28	if spatially varied rainfall is not input.
33	NJSW

Table 20, continued.

IV. Stormwater Input from Cards, Continued

Card Group	Comments
38,39	

V. Parallel Channels

Card Group	Comments
4	ISWCH(5) = 1
5	NCGT

VI. Triangles used to Compute Geometric Data

Card Group	Comments
4	ISWCH(4) = 1
	ISWCH (6)
17	NTEMP(3) = third junction making up triangle. Also: new values for variables ALEN, WIDTH, RAD

VII. Manning's Roughness Coefficient Computed at Each Time Step

Card Group	Comments
4	ISWCH(7) = 1
14	

VIII. Quality Restart Option

Card Group	Comments
31	For first run ISWCH(3) = 1 for subsequent restarts ISWCH(1) = 1

Table 20, continued

IX. Variable Oxygen Saturation Coefficients

Card Group	Comments
31	ISWCH(7) = 1 or 2
34	TEMP & ICL required if ISWCH(7) = 2
35	CSAT = boundary DO concentration
36	STT Required

X. Variable Oxygen Reaeration Coefficients

Card Group	Comments
31	ISWCH(8) = 1 or 2
36	ATT required.

little effect on the simulation. Thus, data on spatial variations in rainfall may not be needed.

Card input can be used for storm flows and other variable inflows to,or outflows from, the receiving waters. Parallel channels can be used to model a deeper channel within a marshy area. Manning's roughness coefficients can be calculated at each time step; however,the user must supply the n vs. depth data from which the coefficient will be calculated. The stage versus time relationship for up to 50 junctions can also be plotted.

The re-start option allows quality calculations to continue from the point at which the program stopped on a prior run. Quantity calculations from the previous run are used over again. Use of the re-start option permits a longer quality simulation.

Finally, variable oxygen saturation coefficients and reaeration coefficients are optional inputs which would probably be used only for estuarine systems. Oxygen saturation is computed based on water temperature and chlorides concentration. Reaeration coefficients are calculated based on the O'Connor-Dobbins formula as given in Reference 13.

CASE STUDY MATERIALS AND PREPARATION OF CODING FORMS

The receiving water shown in Figure 23 accepts the effluent from the sewage treatment plant modeled in the Storage/Treatment Block. The outfall is at junction 10 of the receiving water which corresponds to Element 10 of the Transport Block.

Total area of the receiving water to be modeled is 700 acres (1.09 sq. This is a tidal system with a constant inflow of 1000 cfs at junction 13. Pollutant concentrations of the inflow include 1.0 mg/l BOD_5, 10.0 mg/l SS and 100 MPN/100 ml total coliforms. Depth and area data for each of the junctions are given in Table 18. Channel data are contained in Table 19.

A tidal stage relationship is specified for junction 1. Tidal stage is given for four times during the initial day of simulation. The SWMM will develop tidal elevations for the remainder of the simulation period. Tidal stages as referenced to mean low low water for day one are:

Time (hrs)	Stage (ft)
0.3	-1.1
5.8	.7
11.6	-2.0
18.7	2.4

The assignment for the Receiving Water Block Workshop is to collate the data and prepare computer coding forms as outlined in Table 8-1, pages 289-305, User's Manual. Table 20 may be of assistance.

The following additional data can be used where required:

Initial Time for Start (TZERO): 12

Evaporation (EVAP): 3.0 in/month

Wind Velocity: 0 mph

Wind Direction: 0

Quality Constituents Modeled: BOD_5, SS, Coliforms

Water Temperature: 14°C

Oxygen Saturation Concentration: 9.0 mg/l

Reaeration Coefficient: 0.1 day^{-1}

First Order Decay Exponent: 0.2 day^{-1}

DO Concentration of Inflow at Junction 13: 8.0 mg/l

Datum Chosen: Mean low low water

Tidal Exchange Ratio: 0.80.

SAMPLE DATA INPUT AND PROGRAM OUTPUT: RECEIVING WATER BLOCK

Data contained in Tables 18 and 19 and on pages 220 and 221 were used as input for the Receiving Water Block. An additional assumption made was that the initial concentrations of constituents at all nodes would be the same as the inflow to Node 13. The hydrograph and pollutographs shown in Figure 20 (Input to Storage/Treatment Block) were used as dynamic input into the Receiving Water Block to show the effects of raw combined sewage on the receiving water. A standard run, as defined in Table 20, was performed.

The input data and sample program output are provided. Figure 25 shows the variation of dissolved oxygen concentration with time, for selected points within the receiving water. Dispersion of the storm water pollutants away from the simulated outfall can be seen. In this case, pollutants have not been completely dispersed and the system has not returned to its pre-storm state during the five days simulated. Therefore, it may be desirable to use the restart option and increase the length of quality simulation. The significant quality oscillations shown in Figure 25 are the result of the tidal variation and the high ocean exchange ratio for the system.

RECEIVING WATER BLOCK CARD DATA

```
                                                                                                   CARD
                                                                                                   GROUP
   9  10
   1   2    3    4    7
RECEIVIN                                                                                             1
QUANTITYQUALITY                                                                                      2
RECEIVING WATER BLOCK WORKSHOP
CASE STUDY                                                                                           3
DESIGN STORM
10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES
    1                                                                                                4
    5  24.    1.  40.  12.    13    18   10    3.   0.    0.     1    0    12     1                 5
         .45     1450.         .84     1460.      1.14     1470.      1.56     1480.
        2.16     1490.        1.92     1500.      1.44     1510.      1.02     1520.                 6
         .84     1530.         .48     1540.       .48     1560.       .30     1570.
           1          2          3          4          5          6          7          8
           9         10         11         12         13                                           7
       1   2      1   3      1   4      4   5      3   5      2   3      2   6      3   6
       6   8      6   7      5   7      7   9      8   9      8  10      9  11     10  12          8
      11  12     12  13
           1          2          3          4          5          6          7         19
          12         13                                                                            9
    1     4   50                                                                                  11
   .3        -1.1        5.8        .7         11.6      -2.0        18.7       2.4                12
    1    0.       1.38           15.
    2    0.       2.59           13.
    3    0.       2.04           13.
    4    0.       0.99           14.
    5    0.       1.58           13.
    6    0.       4.32           11.5
    7    0.       3.55           11.5                                                             15
    8    0.       3.46            9.5
    9    0.       3.39           10.4
   10    0.       1.85            7.
   11    0.       2.09           10.15
   12    0.       2.01           10.05
   13    0.       1.251000.      10.
99999                                                                                             16
```

1	1	2	2325.	750.	14.	.025	
2	1	3	2000.	1250.	14.	.025	
3	3	4	1225.	1000.	13.5	.025	
4	4	5	1025.	650.	13.5	.025	
5	3	5	950.	1150.	13.	.025	
6	2	3	1475.	1450.	13.	.025	
7	2	6	1575.	1400.	12.3	.025	
8	3	6	1725.	950.	12.3	.025	
9	6	8	2100.	1575.	10.5	.025	
10	6	7	1800.	1350.	11.5	.025	17
11	5	7	1525.	1600.	12.3	.025	
12	7	9	2050.	1550.	11.0	.025	
13	8	9	1550.	1500.	10.0	.025	
14	8	10	2600.	950.	8.25	.025	
15	9	11	2550.	1075.	10.25	.025	
16	10	12	1225.	1125.	8.53	.025	
17	11	12	1125.	1250.	10.1	.025	
18	12	13	1000.	2250.	10.0	.025	
19	1	4	1250.	500.	14.5	.025	
99999							18

```
   WORKSHOP CASE STUDY, RECEIVING WATER BLOCK                                19
   TIME FROM START OF STORM, HRS                                             20
 WATER   LEVEL  AT OCEAN                                                     21
ENDQUANT                                                                     30
                   1     1                                                   31
        1     6   30                                                         33
    5   3     1  .80   14.          3                                        34
    1         0. 9.00    .1    .2       BOD                                  35


    1         1.                    8.         8.
    2         1.                    8.         8.
    3         1.                    8.         8.
    4         1.                    8.         8.
    5         1.                    8.         8.
    6         1.                    8.         8.                           36
    7         1.                    8.         8.
    8         1.                    8.         8.
    9         1.                    8.         8.
   10         1.                    8.         8.
   11         1.                    8.         8.
   12         1.                    8.         8.
   13         1.     10766.         8.         8.
99999                                                                        37
    1         0. 9.00    .1             SUSPENDED SOLIDS                     35
    1        10.
    2        10.
    3        10.
    4        10.
    5        10.
    6        10.
    7        10.                                                             36
    8        10.
    9        10.
   10        10.
   11        10.
   12        10.
   13        10.    107660.
99999                                                                        37
    1         0. 9.00    .1             COLIFORMS                            35
    1       100.
    2       100.
    3       100.
    4       100.
    5       100.
    6       100.
    7       100.                                                             36
    8       100.
    9       100.
   10       100.
   11       100.
   12       100.
   13       100.    3.39E+9
99999                                                                        37
ENDPROGRAM
```

RECEIVING WATER BLOCK WORKSHOP
CASE STUDY

E.P.A.
RECEIVING WATER HYDRODYNAMICS

DAYS SIMULATED 5

WATER QUALITY CYCLES PER DAY 24

INTEGRATION CYCLES PER WATER QUALITY CYCLE 90

LENGTH OF INTEGRATION STEP IS 40. SECONDS

INITIAL TIME 12.00 HOURS

EVAPORATION RATE, 3.0 INCHES PER MONTH

WIND VELOCITY, 0. MPH WIND DIRECTION, 0. DEGREES FROM NORTH

ESTURIAL SYSTEM

WRITE CYCLE STARTS AT THE 1 TIME CYCLE

RAIN IN INCHES PER HOUR, AND TIME IN MINUTES, MEASURED FROM START OF STORM

	IN./HR.	MINUTES	IN./HR.	MINUTES	IN./HR.	MINUTES	IN./HR.	MINUTES	IN./HR.	MINUTES
1 TO 5	.450	1450.000	.840	1460.000	1.140	1470.000	1.560	1480.000	2.160	1490.000
6 TO 10	1.920	1500.000	1.440	1510.000	1.020	1520.000	.840	1530.000	.480	1540.000
11 TO 15	.480	1560.000	.300	1570.000	0.000	0.000	0.000	0.000	0.000	0.000
16 TO 20	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

PRINTED OUTPUT AT THE FOLLOWING 13 JUNCTIONS

1 2 3 4 5 6 7 8 9 10 11 12 13

PRINTED OUTPUT FOR THE FOLLOWING 18 CHANNELS

1002	1003	1004	4005	3005	2003	2006	3006
6008	6007	5007	7009	8009	8010	9011	10012
11012	12013						

KO IS 1 NUMBER OF TERMS IS 4 MAXIMUM NUMBER OF ITERATIONS IS 50 TIDE CHECK SWITCH IS-0

COEFFICIENTS FOR TIDAL INPUT WAVE AT JUNCTION 1

A1	A2	A3	A4	A5	A6	A7	PERIOD(HRS)
.008	-.808	-.109	.019	.531	-1.567	-.076	24.00

WHERE THE WAVEFORM IS GIVEN BY

H(J) = A1 + A2.SIN(WT) + A3.SIN(2WT) + A4.SIN(3WT) + A5.COS(WT) + A6.COS(2WT) + A7.COS(3WT)

* * * * * C H A N N E L D A T A * * * * *

CHANNEL NUMBER	LENGTH (FT)	WIDTH (FT)	AREA (SQ FT)	MANNING COEF.	VELOCITY (FPS)	HYD RADIUS (FT)	JUNCTIONS AT ENDS		MAX TIME STEP (SEC)	EXCEEDED BY STEP OF 40
1	2325.	750.00	10500.	.025	-0.00	14.0	1	2	108.	
2	2000.	1250.00	17500.	.025	-0.00	14.0	1	3	93.	
3	1225.	1000.00	13500.	.025	-0.00	13.5	3	4	58.	
4	1025.	650.00	8775.	.025	-0.00	13.5	4	5	48.	
5	950.	1150.00	14950.	.025	-0.00	13.0	3	5	46.	
6	1475.	1450.00	18850.	.025	-0.00	13.0	2	3	71.	
7	1675.	1400.00	17220.	.025	-0.00	12.3	2	6	83.	
8	1725.	950.00	11685.	.025	-0.00	12.3	3	6	85.	
9	2100.	1575.00	16538.	.025	-0.00	10.5	6	8	112.	
10	1800.	1350.00	15525.	.025	-0.00	11.5	6	7	92.	
11	1525.	1600.00	19680.	.025	-0.00	12.3	5	7	75.	
12	2050.	1550.00	17050.	.025	-0.00	11.0	7	9	107.	
13	1550.	1500.00	15000.	.025	-0.00	10.0	8	9	84.	
14	2600.	950.00	7838.	.025	-0.00	8.3	8	10	155.	
15	2550.	1075.00	11019.	.025	-0.00	10.3	9	11	137.	
16	1225.	1125.00	9596.	.025	-0.00	8.5	10	12	72.	
17	1125.	1250.00	12625.	.025	-0.00	10.1	11	12	61.	
18	1000.	2250.00	22500.	.025	-0.00	10.0	12	13	54.	
19	1250.	500.00	7250.	.025	-0.00	14.5	1	4	57.	

* * * * * J U N C T I O N D A T A * * * * *

JUNCTION NUMBER	INITIAL HEAD (FT)	DEPTH (FT)	SURFACE AREA (10**6 SQ FT)	INPUT (CFS)	OUTPUT (CFS)	CHANNELS ENTERING JUNCTION												COORDINATES X	Y
1	0.00	15.0	1.38	-0.	-0.	1	2	19	0	0	0	0	0	0	0	0	0	-0.0	-0.0
2	0.00	13.0	2.59	-0.	-0.	6	7	1	0	0	0	0	0	0	0	0	0	-0.0	-0.0
3	0.00	13.0	2.04	-0.	-0.	3	5	8	2	6	0	0	0	0	0	0	0	-0.0	-0.0
4	0.00	14.0	.99	-0.	-0.	4	3	19	0	0	0	0	0	0	0	0	0	-0.0	-0.0
5	0.00	13.0	1.58	-0.	-0.	11	4	5	0	0	0	0	0	0	0	0	0	-0.0	-0.0
6	0.00	11.5	4.32	-0.	-0.	9	10	7	8	0	0	0	0	0	0	0	0	-0.0	-0.0
7	0.00	11.5	3.55	-0.	-0.	12	10	11	0	0	0	0	0	0	0	0	0	-0.0	-0.0
8	0.00	9.5	3.46	-0.	-0.	13	14	9	0	0	0	0	0	0	0	0	0	-0.0	-0.0
9	0.00	10.4	3.39	-0.	-0.	15	12	13	0	0	0	0	0	0	0	0	0	-0.0	-0.0
10	0.00	7.0	1.85	-0.	-0.	16	14	0	0	0	0	0	0	0	0	0	0	-0.0	-0.0
11	0.00	10.2	2.09	-0.	-0.	17	15	0	0	0	0	0	0	0	0	0	0	-0.0	-0.0
12	0.00	10.1	2.01	-0.	-0.	18	16	17	0	0	0	0	0	0	0	0	0	-0.0	-0.0
13	0.00	10.0	1.25	1000.	-0.	18	0	0	0	0	0	0	0	0	0	0	0	-0.0	-0.0

TOTAL AREA FOR THE SYSTEM 30.50 * 10**6 SQ FT OR 1.09 SQ MILES

DESIGN STORM 10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES

RECEIVING WATER BLOCK WORKSHOP
CASE STUDY

E.P.A.
RECEIVING WATER HYDRODYNAMICS

DESIGN STORM

10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES

DAY IS 2

* T I M E   H I S T O R Y   O F   S T A G E *

HOUR	JUNCTION 1 HEAD(FEET)	JUNCTION 2 HEAD(FEET)	JUNCTION 3 HEAD(FEET)	JUNCTION 4 HEAD(FEET)	JUNCTION 5 HEAD(FEET)	JUNCTION 6 HEAD(FEET)
12.00	-1.9537	-1.9547	-1.9545	-1.9543	-1.9549	-1.9555
13.00	-1.6147	-1.7167	-1.7067	-1.6944	-1.7307	-1.7660
14.00	-.9143	-.9123	-.9119	-.9115	-.9125	-.9128
15.00	.0282	.0983	.0923	.0846	.1065	.1277
16.00	1.0257	1.0349	1.0345	1.0339	1.0354	1.0371
17.00	1.8616	1.8390	1.8415	1.8447	1.8354	1.8263
18.00	2.3421	2.3148	2.3175	2.3210	2.3108	2.3010
19.00	2.3527	2.3304	2.3323	2.3349	2.3275	2.3203
20.00	1.8973	1.8845	1.8853	1.8865	1.8833	1.8801
21.00	1.1045	1.1027	1.1023	1.1020	1.1031	1.1040
22.00	.1944	.1968	.1961	.1953	.1979	.2002
23.00	-.5868	-.5904	-.5904	-.5904	-.5903	-.5903
24.00	-1.0429	-1.0485	-1.0481	-1.0476	-1.0490	-1.0504
25.00	-1.0870	-1.0851	-1.0852	-1.0853	-1.0850	-1.0845
26.00	-.7641	-.7624	-.7622	-.7621	-.7625	-.7626
27.00	-.2290	-.2351	-.2342	-.2330	-.2364	-.2396
28.00	.3093	.3140	.3138	.3136	.3142	.3147
29.00	.6560	.6655	.6648	.6638	.6666	.6693
30.00	.6879	.6917	.6912	.6907	.6923	.6937
31.00	.3823	.3788	.3788	.3789	.3786	.3782
32.00	-.1856	-.1930	-.1899	-.1898	-.1901	-.1904
33.00	-.8703	-.8720	-.8722	-.8724	-.8716	-.8710
34.00	-1.4960	-1.4990	-1.4990	-1.4990	-1.4989	-1.4989
35.00	-1.8973	-1.9007	-1.9005	-1.9003	-1.9009	-1.9015
36.00	-1.9537	-1.9543	-1.9541	-1.9540	-1.9544	-1.9548

RECEIVING WATER BLOCK WORKSHOP
CASE STUDY

E.P.A.
RECEIVING WATER HYDRODYNAMICS

DESIGN STORM

10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES

DAY IS 2

* * * * * * * * * * * * * * * T I M E H I S T O R Y O F F L O W A N D V E L O C I T Y * * * * * * * * * * * * *

HOUR	CHANNEL 1 2 FLOW (CFS)	VEL. (FPS)	CHANNEL 1 3 FLOW (CFS)	VEL. (FPS)	CHANNEL 1 4 FLOW (CFS)	VEL. (FPS)	CHANNEL 4 5 FLOW (CFS)	VEL. (FPS)	CHANNEL 3 5 FLOW (CFS)	VEL. (FPS)	CHANNEL 2 3 FLOW (CFS)	VEL. (FPS)
12.00	25.	.00	16.	.00	88.	.01	72.	.01	-61.	-.00	39.	.00
13.00	3014.	.34	5288.	.36	3122.	.51	2098.	.29	2386.	.19	-722.	-.05
14.00	-2209.	-.22	-3881.	-.24	-2338.	-.34	-1533.	-.20	-1866.	-.13	575.	.03
15.00	2531.	.23	4442.	.24	2344.	.31	1365.	.15	1916.	.12	-729.	-.04
16.00	4051.	.36	7122.	.38	3821.	.49	2330.	.25	3173.	.20	-1151.	-.06
17.00	2381.	.20	4193.	.22	2106.	.26	1225.	.13	1885.	.11	-720.	-.03
18.00	395.	.04	705.	.04	113.	.02	-43.	-.00	347.	.02	-198.	-.01
19.00	-1274.	-.10	-2227.	-.11	-1519.	-.18	-1051.	-.10	-935.	-.05	255.	.01
20.00	-2451.	-.20	-4300.	-.22	-2623.	-.32	-1709.	-.17	-1842.	-.11	589.	.03
21.00	-2840.	-.25	-4994.	-.27	-2919.	-.38	-1850.	-.19	-2155.	-.13	724.	.04
22.00	-2363.	-.22	-4167.	-.24	-2364.	-.32	-1459.	-.16	-1807.	-.12	631.	.03
23.00	-1610.	-.16	-2851.	-.17	-1566.	-.23	-943.	-.11	-1257.	-.09	453.	.03
24.00	-979.	-.10	-1744.	-.11	-920.	-.14	-557.	-.07	-810.	-.06	289.	.02
25.00	-442.	-.05	-801.	-.05	-377.	-.06	-230.	-.03	-415.	-.03	145.	.01
26.00	786.	.08	1351.	.08	865.	.13	536.	.06	508.	.04	-183.	-.01
27.00	804.	.08	1386.	.08	862.	.12	521.	.06	517.	.04	-188.	-.01
28.00	318.	.03	533.	.03	341.	.05	184.	.02	140.	.01	-61.	-.00
29.00	91.	.01	137.	.01	101.	.01	56.	.01	-1.	-.00	2.	.00
30.00	-358.	-.03	-648.	-.04	-356.	-.05	-202.	-.02	-309.	-.02	126.	.01
31.00	-1120.	-.10	-1983.	-.11	-1118.	-.15	-665.	-.07	-876.	-.06	333.	.02
32.00	-1766.	-.17	-3119.	-.18	-1753.	-.25	-1055.	-.12	-1374.	-.09	512.	.03
33.00	-1812.	-.18	-3203.	-.20	-1782.	-.26	-1063.	-.13	-1417.	-.10	532.	.03
34.00	-1401.	-.15	-2484.	-.16	-1355.	-.21	-795.	-.10	-1114.	-.08	428.	.03
35.00	-833.	-.09	-1488.	-.10	-780.	-.12	-451.	-.06	-701.	-.05	274.	.02
36.00	13.	.00	-5.	-.00	79.	.01	69.	.01	-74.	-.01	45.	.00

DESIGN STORM

10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES

RECEIVING WATER BLOCK WORKSHOP
CASE STUDY

E.P.A.
DYNAMIC STORM WATER QUALITY

MAXIMUM JUNCTION NUMBER 13

MAXIMUM CHANNEL NUMBER 19

NUMBER OF QUALITY CYCLES PER DAY 24

NUMBER OF DAYS 5

NUMBER OF CONSTITUENTS 3

** COLIFORMS MUST BE CONSTITUENT NUMBER 3

LENGTH OF QUALITY INTEGRATION STEP (SECONDS) 3600.

PRINT INTERVAL, 1 DAYS

EXCHANGE REQUIREMENT AT OCEAN = RATIO OF ESTUARINE FLOW RETURNED TO AMOUNT THAT FLOWS OUT, PER TIDAL CYCLE .80

AVERAGE WATER TEMPERATURE (DEGREES CENTIGRADE) 14.00

THERE ARE -0 STORMWATER INPUT JUNCTIONS

QUALITY CYCLE CONCENTRATIONS, PRINTOUT STARTS IN TIME CYCLE 1 ,

PRINTED EVERY 6 HOUR(S), FOR A TOTAL OF 180 HOURS.

* * * * * SWITCH SETTINGS * * * * *

SWITCH NUMBER	1	2	3	4	5	6	7	8	9	10
SWITCH SETTING	-0	-0	-0	1	1	-0	-0	-0	-0	-0

CONSTITUENT NUMBER 1 BOD

SINK CONCENTRATION, MGL (OR MPN/L) 0.00

OXYGEN SATURATION (MGL) 9.00

REAERATION COEFFICIENT (1/DAY) .100

DECAY COEFFICIENT (1/DAY) .20

DISSOLVED OXYGEN FOR THIS CONSTITUENT IS CONSTITUENT 4

INITIAL CONCENTRATIONS,MGL (OR MPN/L), BY JUNCTION

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.100E+01	.100E+01	.100E+01	.100E+01	.100E+01	.100E+01	.100E+01	.100E+01	.100E+01	.100E+01
11 TO 13	.100E+01	.100E+01	.100E+01							

MASS LOADINGS, THOUSANDS OF LBS/DAY (OR OF MPN/MIN), BY JUNCTION

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
11 TO 13	0.	0.	.100E+02							

INITIAL DISSOLVED OXYGEN CONCENTRATIONS (MGL), BY JUNCTION

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01
11 TO 13	.800E+01	.800E+01	.800E+01							

DISSOLVED OXYGEN CONCENTRATION OF INFLOW (MGL), JUNCTION

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01	.800E+01
11 TO 13	.800E+01	.800E+01	.800E+01							

CONSTITUENT NUMBER 2 SUSPENDED SOLIDS

SINK CONCENTRATION, MGL (OR MPN/L) 0.00

INITIAL CONCENTRATIONS,MGL (OR MPN/L), BY JUNCTION

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.100E+02	.100E+02	.100E+02	.100E+02	.100E+02	.100E+02	.100E+02	.100E+02	.100E+02	.100E+02
11 TO 13	.100E+02	.100E+02	.100E+02							

MASS LOADINGS, THOUSANDS OF LBS/DAY (OR OF MPN/MIN), BY JUNCTION

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
11 TO 13	0.	0.	.100E+03							

TEST CASE, 3000 ACRE BASIN
FOR STORAGE/TREATMENT BLOCK INPUT

DATA TRANSMITTED FROM INPUT FILE

NUMBER OF STEPS	=	150
NUMBER OF INPUT POINTS	=	1
NUMBER OF CONSTITUENTS	=	3
TIME INCREMENT	=	600. SECS
INITIAL TIME	=	12.00 HRS
TOTAL AREA	=	3000.00 ACREA

LOADINGS FROM DATA FILE

TIME (HR.)	JUNCTION	BOD (LBS/MIN)	SS (LBS/MIN)	CLF (MPN/MIN)
12.17	10	43.606	29.486	.131E+14
12.33	10	43.606	29.486	.131E+14

LOADINGS FROM DATA FILE

TIME (HR.)	JUNCTION	BOD (LBS/MIN)	SS (LBS/MIN)	CLF (MPN/MIN)
12.50	10	41.388	132.515	.449E+13
12.67	10	41.974	335.895	.120E+13
12.83	10	45.011	502.515	.725E+12
13.00	10	75.797	774.605	.150E+13
13.17	10	172.662	1487.078	.766E+13
13.33	10	287.599	2474.570	.287E+14
13.50	10	346.691	3339.552	.313E+14
13.67	10	345.699	3863.462	.294E+14
13.83	10	284.327	3637.960	.201E+14
14.00	10	205.108	2857.935	.141E+14
14.17	10	164.135	2227.587	.117E+14
14.33	10	129.834	1608.771	.870E+13
14.50	10	95.360	957.019	.689E+13
14.67	10	69.756	475.211	.705E+13
14.83	10	56.684	226.881	.703E+13
15.00	10	50.641	117.677	.685E+13
15.17	10	49.658	67.550	.636E+13
15.33	10	51.408	51.045	.524E+13
15.50	10	51.086	44.952	.498E+13
15.67	10	50.303	42.171	.517E+13

DESIGN STORM

10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES

JUNCTION CONCENTRATIONS, MGL (OR MPN/L), DURING TIME CYCLE (DAY) 2, QUALITY CYCLE 6, TIME (SEC) 64800.

CONSTITUENT NUMBER 1 BOD

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.55	.66	.62	.55	.72	.81	.93	1.53	1.25	26.98
11 TO 13	1.74	2.02	1.94							

CONSTITUENT NUMBER 2 SUSPENDED SOLIDS

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	6.35	8.20	7.61	6.52	9.07	10.19	11.72	17.11	15.24	211.83
11 TO 13	19.70	21.72	19.99							

CONSTITUENT NUMBER 3 COLIFORMS

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	635.27	819.69	760.83	652.12	907.17	1018.88	1171.86	102074.27	1680.52	6044500.49
11 TO 13	12595.53	42088.59	1996.34							

CONSTITUENT NUMBER 4 BOD (DO)

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	8.33	8.14	8.20	8.31	8.06	7.98	7.91	7.83	7.85	4.86
11 TO 13	7.84	7.87	7.96							

LOADINGS FROM DATA FILE

TIME (HR.)	JUNCTION	BOD (LBS/MIN)	SS (LBS/MIN)	CLF (MPN/MIN)
18.33	10	52.945	40.133	.149E+14
18.50	10	52.005	38.982	.152E+14
18.67	10	52.649	39.762	.149E+14
18.83	10	52.412	39.490	.150E+14
19.00	10	52.437	39.512	.150E+14
19.17	10	68.929	45.207	.136E+14
19.33	10	75.535	55.064	.112E+14
19.50	10	79.602	57.870	.105E+14
19.67	10	77.705	56.562	.109E+14
19.83	10	78.037	56.786	.108E+14
20.00	10	78.115	56.841	.108E+14

DESIGN STORM — 10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES

RECEIVING WATER BLOCK WORKSHOP
CASE STUDY

E.P.A.
DYNAMIC STORM WATER QUALITY

AVERAGE JUNCTION CONCENTRATIONS DURING TIDAL OR TIME CYCLE 2 , CONSTITUENT NUMBER 1 BOD

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.778E+00	.199E+01	.128E+01	.758E+00	.940E+00	.420E+01	.116E+01	.948E+01	.150E+01	.221E+02
11 TO 13	.179E+01	.191E+01	.194E+01							

MAXIMUMS

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.178E+01	.578E+01	.296E+01	.111E+01	.120E+01	.970E+01	.143E+01	.151E+02	.167E+01	.270E+02
11 TO 13	.186E+01	.285E+01	.195E+01							

MINIMUMS

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.524E+00	.662E+00	.622E+00	.549E+00	.723E+00	.807E+00	.933E+00	.106E+01	.125E+01	.160E+01
11 TO 13	.169E+01	.184E+01	.194E+01							

DESIGN STORM — 10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES

RECEIVING WATER BLOCK WORKSHOP
CASE STUDY

E.P.A.
DYNAMIC STORM WATER QUALITY

AVERAGE JUNCTION CONCENTRATIONS DURING TIDAL OR TIME CYCLE 2 , CONSTITUENT NUMBER 4 BOD (DO)

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.817E+01	.771E+01	.791E+01	.808E+01	.792E+01	.715E+01	.784E+01	.612E+01	.781E+01	.500E+01
11 TO 13	.784E+01	.789E+01	.796E+01							

MAXIMUMS

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.837E+01	.814E+01	.820E+01	.831E+01	.806E+01	.798E+01	.791E+01	.788E+01	.786E+01	.770E+01
11 TO 13	.787E+01	.791E+01	.796E+01							

MINIMUMS

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.771E+01	.650E+01	.731E+01	.786E+01	.780E+01	.547E+01	.776E+01	.452E+01	.777E+01	.383E+01
11 TO 13	.782E+01	.787E+01	.795E+01							

LOADINGS FROM DATA FILE

TIME (HR.) JUNCTION BOD (LBS/MIN) SS (LBS/MIN) CLF (MPN/MIN)

DESIGN STORM | 10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES

RECEIVING WATER BLOCK WORKSHOP
CASE STUDY

E.P.A.
DYNAMIC STORM WATER QUALITY

AVERAGE JUNCTION CONCENTRATIONS DURING TIDAL OR TIME CYCLE 2 , CONSTITUENT NUMBER 2 SUSPENDED SOLIDS

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.884E+01	.181E+02	.133E+02	.953E+01	.120E+02	.316E+02	.146E+02	.560E+02	.179E+02	.114E+03
11 TO 13	.201E+02	.206E+02	.200E+02							
					MAXIMUMS					
JUNCTION										
1 TO 10	.173E+02	.406E+02	.246E+02	.139E+02	.158E+02	.560E+02	.182E+02	.840E+02	.199E+02	.270E+03
11 TO 13	.208E+02	.219E+02	.200E+02							
					MINIMUMS					
JUNCTION										
1 TO 10	.600E+01	.820E+01	.761E+01	.652E+01	.907E+01	.102E+02	.117E+02	.130E+02	.151E+02	.179E+02
11 TO 13	.190E+02	.197E+02	.200E+02							

DESIGN STORM | 10 YEAR, 2 HOUR DURATION STORM, TOTAL RAINFALL=2.05 INCHES

RECEIVING WATER BLOCK WORKSHOP
CASE STUDY

E.P.A.
DYNAMIC STORM WATER QUALITY

AVERAGE JUNCTION CONCENTRATIONS DURING TIDAL OR TIME CYCLE 2 , CONSTITUENT NUMBER 3 COLIFORMS

JUNCTION	1	2	3	4	5	6	7	8	9	10
1 TO 10	.559E+05	.419E+06	.183E+06	.168E+05	.114E+05	.119E+07	.692E+04	.340E+07	.105E+05	.826E+07
11 TO 13	.134E+05	.164E+05	.200E+04							
					MAXIMUMS					
JUNCTION										
1 TO 10	.389E+06	.281E+07	.839E+06	.775E+05	.353E+05	.390E+07	.143E+05	.709E+07	.175E+05	.122E+08
11 TO 13	.252E+05	.467E+05	.200E+04							
					MINIMUMS					
JUNCTION										
1 TO 10	.600E+03	.820E+03	.761E+03	.652E+03	.907E+03	.102E+04	.117E+04	.130E+04	.151E+04	.179E+04
11 TO 13	.189E+04	.197E+04	.200E+04							

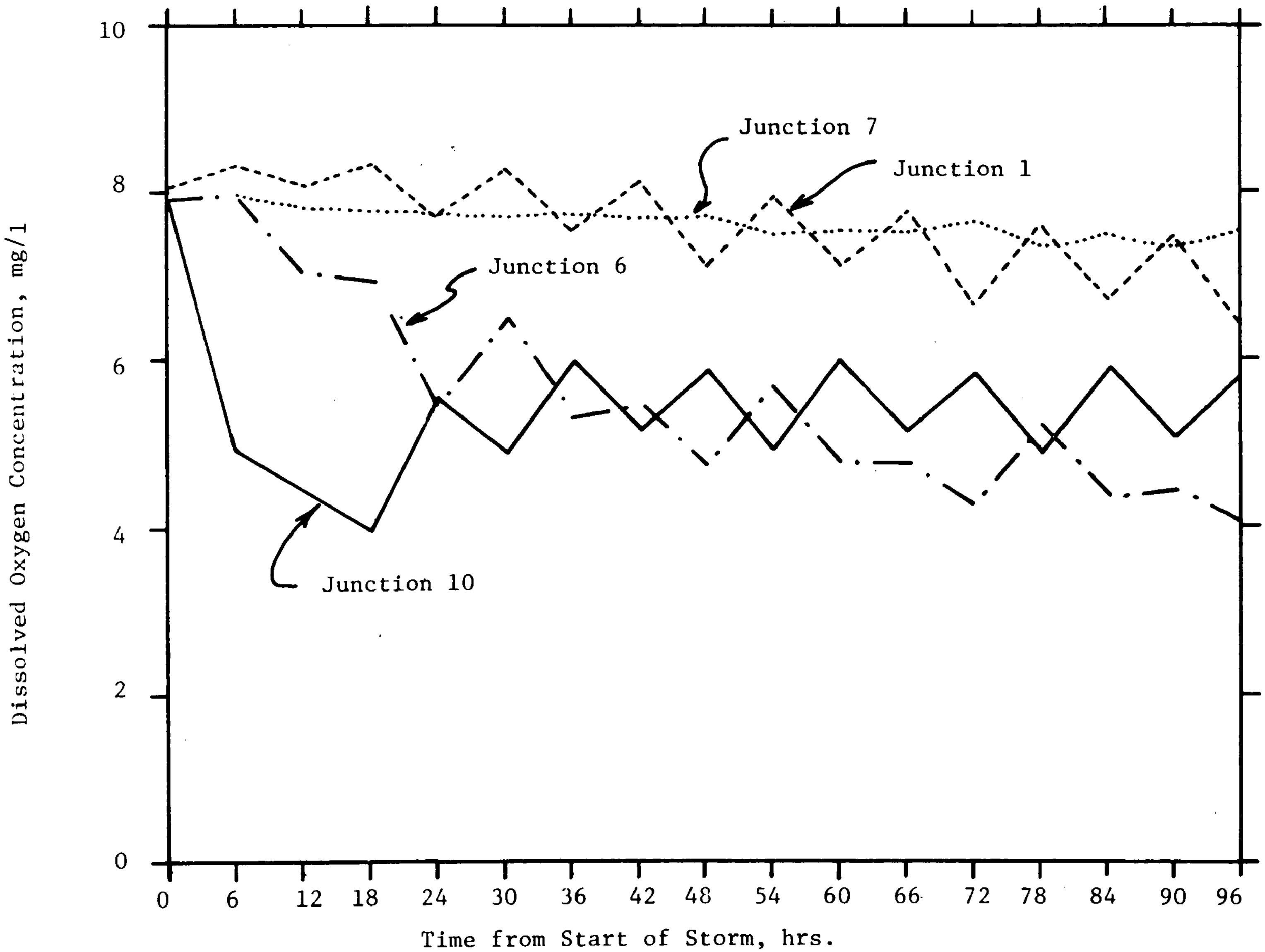


FIGURE 25. DISSOLVED OXYGEN VARIATION WITH TIME FOR SELECTED NODES.

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Part I. CRITERIA FOR SELECTION OF STORMWATER MANAGEMENT MODELS - PLANNING VS. DESIGN

by John A. Lager, P.E.*

Selection of stormwater management models for specific tasks should not be left to chance. Today there are both choices and opportunities for those who will look beyond the problems and focus upon the results.

- Planning without commitment is next to valueless.
- Commitment without objectives is simply foolish.
- Excellence without communication is wasted.

These situations, unfortunately, are not rare in the past and present application of stormwater management technology.

In this lecture we will build up a selection process which, I believe, will increase the visibility of the action decisions necessary to successful performance. The process is intended to be helpful to the model purveyor-developer as well as the responsible manager-solicitor. The evaluation procedure provides a vehicle for both project initiation and project updating and reassessment.

The presentation, Part I, starts with a discussion on defining the study objectives. With rare exception this is the most difficult task of all. Next, the given data base

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will be identified. Then the array of model capabilities and requirements will be presented, leading to an identification of necessary assumptions for application. Finally, the selection process itself is concluded with emphasis on results and fit into the continuing program.

In Part II, a simplified modeling approach will be described with emphasis on the application strategy and potentials for management decision making.

DEFINING THE STUDY OBJECTIVES

Study objectives should be specific. Why? To facilitate communication, focus of effort, and commitment. For example, if the study objective is simply to

> comply with the requirements of PL 92-500

it is unlikely that any two people in the chain of authority will have the same idea in mind except by default. On the other hand, if the objective were amplified to

> assess the relative impact of the non-point discharges of urban runoff from within the city limits of Boomtown to other controllable sources on Segment V of the High Flow River with respect to organic loadings, given that loadings from Segment VI to V will not change from today's level. It is intended that loadings be assessed for two seasonal occurrences...

the chance of common understanding would be greatly improved, and the specifics of model requirements and applicability would be well underway to identification.

The following, then, are the types of information that require definition in the model selection phase.

Improve What, How Much, and Where

Suppose a city is investigating the use of polymers to reduce pipeline friction and thereby increase interception of flows for treatment at the dry-weather plant, as opposed to constructing a satellite overflow treatment facility. Models operating off the same input data (hydrology, catchment characteristics, pipe networks, etc.) would offer an excellent tool for weighing the two alternatives. Simplified models could be used to estimate the expected number of occurrences annually before and after improvements, and detailed dynamic models could identify design criteria for specific critical events.

Other representative questions that should be considered in setting the study objectives include the following:

1. Is flooding the major consideration? If so, then the use of a dynamic backwater model is essential. Why? Because flooding is the result of a system's inability to react.to a specific storm event in the required time interval (i.e., bottlenecks).

2. Is loading on the receiving stream the control? If so, is the nature of the impact short term (indicating the need for dynamic modeling) or long term (simple models--i.e., Streeter-Phelps--may suffice).

3. Is there a repetitive, defined occurrence that is to be abated? For example, are overflow occurrences to be reduced to keep bathing beaches open for the maximum number of days in season? Note

that in bacterial contamination the occurrence or nonoccurrence of an overflow is more important than its duration.

4. What are the keys to the system's performance today? A major study objective may be to deter mine what makes the system respond the way it does. That is, what and where are the controls and how do they react?

5. Are there available facilities that are not being put to maximum use or to effective use? These may dictate some trial criteria for abatement schemes.

6. Are the funds available on the basis of the ability to sell the program? This again is a question of communications and timing.

7. What is the balance between the known (given) data and the assumptions? How will this change over time? An exquisite model running on a series of assumptions may yield no better results than a simple model run on the same base data.

8. What are the potential impacts of new technology? Recent work for the National Commission on Water Quality [1] has shown the optimal storage-treatment balance to be quite cost sensitive within the relative range of costs experienced in EPA's R&D program (Figure 1). Thus, technological changes which decrease the cost of one with respect to the other may have significant impact on program selection.

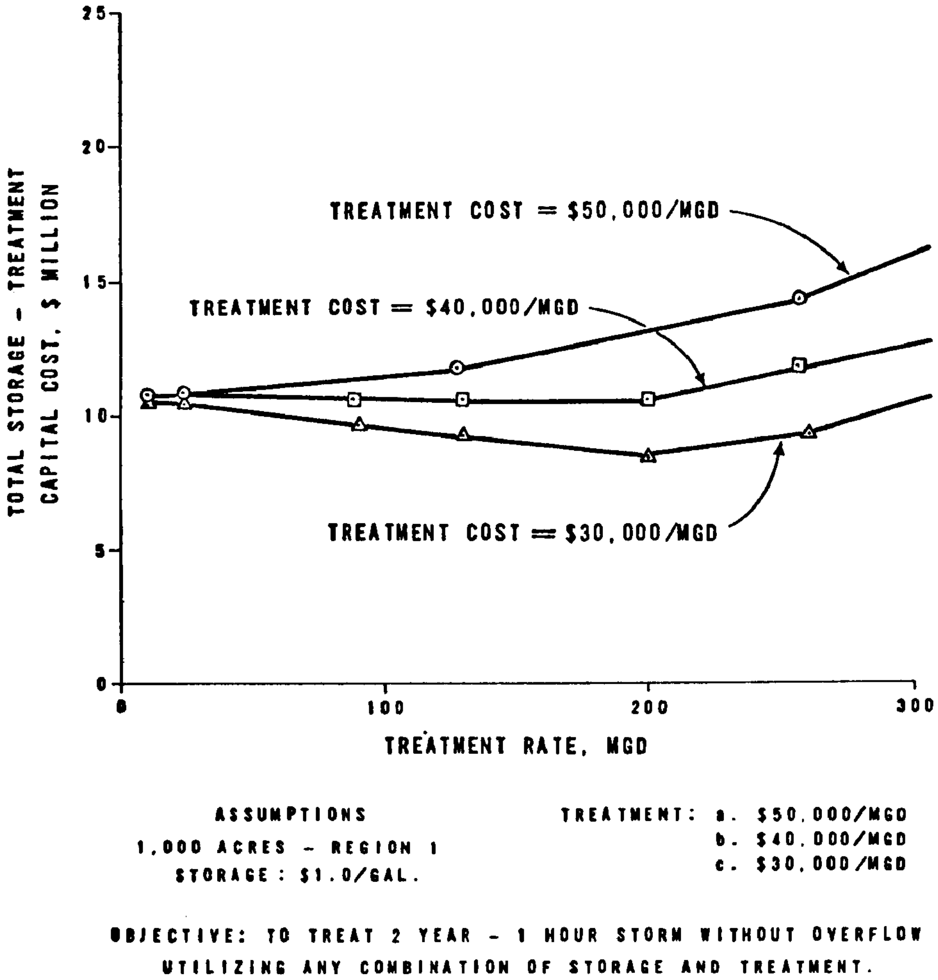

FIGURE 1. COST SENSITIVITY

Required Sensitivity of Results

If the ranges of given variables are known, then the model selected must be sufficiently sensitive to display output differences within such ranges. Also, the modeling requirements might be quite different if the model's purpose is to locate points on the system where monitoring information would be most beneficial, as opposed to additionally projecting expected flow and concentration ranges at such locations.

Who is the User or Reviewer

In order to be implemented, the results or operations of the model must be made known to, and approved by, some person or persons. What are his requirements and needs? Arriving at the correct solution is not enough if you lose the client in the process. The output must be recognizable and usable in terms that the client can easily relate to and work with, for after all, he has to sell it.

Boundaries of Investigation

What are the boundaries of the study area(s)? Who is responsible for defining boundary transfers? Which sources will be used? Which time spans are to be investigated--historical, present, future? And which time increments--annual, daily, hourly, etc.?

Timing and Budget Constraints

What are the milestone dates of the project, and what information must be deliverable by these dates? Are the results to take precedent over the effort? Where are the risk areas--methods of approach, contingency plans? Will the budget resources support the study objectives?

Personnel and Hardware Constraints

Who is going to operate the program and on which machine? Are they compatible with the model requirements? Is the program to be used or further developed following the initial project? By whom and where? Are direct interfaces with other current modeling/data management operations essential or desirable? Are "canned" subroutines or machine-specific notations used in the programming? Is the documentation current, complete, and available? Is the model available?

IDENTIFY GIVEN DATA BASE

Models require input data upon which to operate, whether the data are real or fabricated. Also, the quantity and quality of the data base is likely to change over the course of the work. To select model programs effectively requires that the available data base strengths, weaknesses, and dynamics be identified beforehand.

Rather than selecting a model and then seeing if you can fill its data requirements, it is preferable to analyze your available data and then choose the model that can use these data most effectively to achieve the study objectives.

Statics

As used in this lecture, static data are the fixed data bases available in the initial stages of the project. Such data might include the plans and specifications of the system, historical rainfall records, census tract enumerations, planning and zoning documents, aerial photographs, treatment plant records, files on trouble calls and system maintenance, prior studies and reports, monitoring records, and state-of-the-art literature.

Generally, the most prized data represent cause-effect relationships. Examples might include changes in plant inflow characteristics between dry- and wet-weather periods; measured rainfall-runoff-quality relationships; intensive stream surveys correlated with development, treatment performance, and ideally, storm occurences; and documentation as to the time, location, and duration of overflows.

Personally, I believe that the most effective nationwide pollution abatement step that can be taken by regulatory authorities today is to require the monitoring and reporting of each and every overflow occurrence. The reason is that the real water quality problem solution does not lie in the culmination of a grand cure-all program, but rather in the early implementation of steps moving toward that goal.

However, very important data describe the system itself. Are as-built drawings up-to-date? Is the data available in such a form that it can be taken off readily? Are drainage and census boundaries shown? Are the topography and structures superimposed on the maps? How does the system function? Controls?

With respect to rain gages, what is the quality of the network? Where are the records kept? In what form? What periods are covered? Are they discontinued in the winter? Have any data been computerized?

Dynamics

These are the data that will become available during and subsequent to the study. To what extent can this effort be modified or redirected to suit the needs of a specific model? At what milestone points can "midpoint" corrections be made both in the modeling and data gathering programs?

As a note of caution in setting a value on the dynamic data: one storm does not represent a season, and one season does not represent a historical record; however, the ability or inability of a quantity of flow to get from point A to point B may be both significant and repetitive.

A major advantage of dynamic modeling is that it may identify the key nodes that control or directly reflect system efficiency. Concentrating dynamic data gathering at these nodes may be far more cost effective than, say, a shotgun or intuitive approach.

As a final thought, the data requirements for a specific dynamic model may be so exhaustive as to require several months to collect, check, and apply. Can a simplified model be used to fill this time gap and improve the prospects of success of the more comprehensive program?

IDENTIFY THE ARRAY OF MODEL CAPABILITIES

Mathematical model usage in stormwater management is basically a creation of the last decade, and application experience is even more limited (SWMM was first marketed in 1971). Significantly, the more sophisticated models are in a continuous state of refinement and modification as new uses are attempted or requirements are identified. Documentation, availability, consultation advice, and record of use are equally variable and changing.

Why Model at All

Models follow operating rules without exception or interpretation and can process mountains of data with relative ease. The developer sets the rules (and in most cases options), and the user furnishes the data and is left with the results. That is the package, plain and simple.

In deciding whether or not to model, the potential user should assess this process closely. What are the data he is prepared to give or invest in? What are the operating rules that he is willing to live by? Where are the mountains he wishes to overcome? What will he do with the results? The faults are as obvious as the potential benefits--weak or incorrect rule(s), erroneous or inadequate data, poorly selected or defined mountains, unusable or indeterminant results (i.e., what is the message?).

Where to Find Information

A comprehensive evaluation of stormwater management models is contained in the study, "Evaluation of Mathematical Models for Engineering Assessment, Control, Planning, and Design of Storm and Combined Sewerage Systems," prepared by Battelle Memorial Institute for EPA [1]. The objectives and initial findings of this study were presented by the project director at last year's seminar [2]. The complete report covers model reviews, model selection criteria, detailed descriptions of selected models, test data and results, cost comparisons, and a source listing.

Selected summaries from this and other recent comparative evaluations [3, 4] are shown in Tables 1, 2, and 3 along with the identified references. The models are separable by function (planning, design, operation) and by degree of precision (simple, intermediate, and complex). The purpose in presenting the three similar listings is to illustrate the perspectives of these experienced users.

What (Engineering-Hydraulic) Processes are Being Simulated

The user should screen candidate model documentation to identify what processes or devices are being simulated and linked and with what degree of sophistication. Does the model focus suit the need focus? For example, the Hydrocomp Model is a multigeneration descendent of the Stanford Watershed Model. The original model dealt largely with

	CATCHMENT HYDROLOGY							SEWER HYDRAULICS								WASTEWATER QUALITY									MISCELLANEOUS				
	MULTIPLE CATCHMENT INFLOWS	DRY-WEATHER FLOW	INPUT OF SEVERAL HYETOGRAPHS	SNOWMELT	RUNOFF FROM IMPERVIOUS AREAS	RUNOFF FROM PERVIOUS AREAS	WATER BALANCE BETWEEN STORMS	FLOW ROUTING IN SEWERS	UPSTREAM AND DOWNSTREAM FLOW CONTROL	SURCHARGING AND PRESSURE FLOW	DIVERSIONS	PUMPING STATIONS	STORAGE	PRINTS STAGE	PRINTS VELOCITIES	DRY-WEATHER QUALITY	STORMWATER QUALITY	QUALITY ROUTING	SEDIMENTATION AND SCOUR	QUALITY REACTIONS	WASTEWATER TREATMENT	QUALITY BALANCE BETWEEN STORMS	RECEIVING WATER FLOW SIMULATION	RECEIVING WATER QUALITY SIMULATION	CONTINUOUS SIMULATION	CAN CHOOSE TIME INTERVAL	DESIGN COMPUTATIONS	REAL-TIME CONTROL	COMPUTER PROGRAM AVAILABLE
BATTELLE NORTHWEST	●	●	●		●	●		●			●		●	●	●	●	●	●			●					●	●	●	●
BRITISH ROAD RESEARCH LABORATORY	●	●	●		●			●															●			●			●
CHICAGO FLOW SIMULATION	●	●	●	●	●	●	●	●					●	●									●		●	●			●
CHICAGO HYDRO-GRAPH METHOD	●	●	●		●	●	●	●															●			●	●		●
COLORADO STATE UNIVERSITY	●	●			●			●	●					●	●														●
CORPS OF ENGINEERS				●	●	●	●				●		●				●			●		●			●				●
DORSCH CONSULT	●	●	●		●	●		●	●		●		●	●	●								●			●			
ENVIRONMENTAL PROTECTION AGENCY	●	●	●		●	●		●	●	●	●	●	●			●	●	●	●	●	●		●	●		●	●		●
HYDROCOMP	●	●	●	●	●	●	●	●	●		●		●	●	●	●	●	●		●		●	●	●	●	●		●	
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	●	●	●		●	●		●						●	●								●			●	●		
MINNEAPOLIS-ST. PAUL	●	●	●		●	●		●			●															●		●	●
SEATTLE	●	●	●		●	●	●	●	●	●	●	●	●	●												●		●	●
SOGREAH	●	●			●	●		●	●	●	●	●	●	●	●			●					●	●		●			
UNIVERSITY OF CINCINNATI	●				●	●		●															●			●			●
UNIVERSITY OF ILLINOIS	●	●		●	●	●		●	●		●	●	●	●	●											●	●		●
UNIVERSITY OF MASSACHUSETTS	●	●			●		●	●	●					●	●														●
WATER RESOURCES ENGINEERS	●	●	●		●	●		●	●	●	●	●	●	●	●	●	●	●		●			●	●		●			
WILSEY AND HAM	●				●	●		●																		●	●		

Table 1. MODEL CHARACTERISTICS BY BRANDSTETTER [2]

COMPARISON OF URBAN RUNOFF MODELS, MARCH 1974

Model	Surface Routing	Sewer Routing	Quality Routing	Degree of Sophistication of Surface Flow Routing	Degree of Sophistication of Sewer Flow Routing	Accurate Modeling of Surcharging	Flexibility of Modeling of Sewer Components	Explicit Modeling of In-System Storage	Treatment Modeling	Receiving Model Available	Degree of Calibration/ Verification Required	Simulation Period	Availability	Documentation	Data Requirements
Rational Method	Peak Flows Only	Peak Flows Only	No	Low	Low	No	Low	No	NA	No	Usually not verified	Individual storms	Non-proprietary	Good	Low
Chicago	Yes	No	No	Moderate	NA	NA	NA	NA	NA	No	Moderate	Individual storms	Non-proprietary	Fair	Moderate
Unit Hydrograph	Yes	In combination with surface	No	Low	Low	No	Low	No	NA	No	High	Individual storms	Non-proprietary	Fair	Moderate
Unit Pulse	Yes	In combination with surface	No	Low-Moderate	Low-Moderate	No	Low	No	NA	No	High	Individual storms	Non-proprietary	Fair	Moderate
STORM	Yes	In combination with surface	Yes	Low	Low	No	Low	No	Yes	No	Low	Long term	Non-proprietary	Good	Moderate
RRL	Yes	Yes	No	Moderate	Low-Moderate	No	Low	No	NA	No	Moderate	Individual storms	Non-proprietary	Good	Moderate
MIT	Yes	No	No	High	NA	NA	NA	NA	NA	No	Moderate	Individual storms	Proprietary	Fair	Moderate
Battelle	Yes	Yes	Yes	Low	Moderate	No	Moderate	Yes	No	No	Moderate	Individual storms	Non-proprietary	Poor	Moderate
EPA-SWMM	Yes	Yes	Yes	High	Moderate	No	High	Yes	Yes	Yes	Moderate	Individual storms	Non-proprietary	Good	Extensive
WRE-SWMM	Yes	Yes	Yes	High	High	Yes	High	No	Yes	Yes	Moderate	Individual storms	Proprietary	Poor	Extensive
Cincinnati (UCUR)	Yes	Yes	Yes	High	Low	No	Low	No	No	No	Moderate	Individual	Non-proprietary	Fair	Extensive
Dorsch (HVM)	Yes	Yes	No	High	High	Yes	High	?	NA	Yes	Moderate	Individual storms	Proprietary	Poor	Extensive
SOGREAH	Yes	Yes	?	High	High	Yes	High	?	?	Yes	Moderate	Individual storms or separate	Proprietary	Poor	Extensive
Hydrocomp	Yes	Yes	Yes	Moderate	Moderate	No	Low	No	No	Yes	High	Individual storms Individual storms or long term	Proprietary	Fair	Extensive
Illinois (ISS)	Yes	Yes	No	Moderate	High	No	Low	No	NA	No	Moderate	Individual storms	Non-proprietary	Good	Extensive

TABLE 2. MODEL CHARACTERISTICS BY HUBER [3].

UTILITY OF URBAN RUNOFF MODELS FOR VARIOUS STAGES OF
STORMWATER MANAGEMENT PLANNING

	STORM	MITCAT	HYDROCOMP	EPA SWMM	SF-WRE
Large-Scale Planning (Alternative Screening)	Excellent	Good	Good	Poor	Poor
Intermediate Scale Planning	Poor	Excellent	Excellent	Excellent*	Excellent*
Detailed Planning/Analysis (No Significant Backwater)	Poor	Excellent	Fair	Excellent	Excellent
Detailed Planning/Analysis (Complex Drainage Networks)	Poor	Poor	Poor	Fair	Excellent
Runoff Quality Simulation	Yes	No	Yes	Yes	Yes
Flow Computation	Rational Formula	Kinematic Wave	Kinematic Wave	Kinematic Wave + Manning Equ.with Dynamic Continuity	Kinematic Wave + Complete Dynamic Flow Equations

*The Runoff Model is best suited for these applications

TABLE 3. MODEL CHARACTERISTICS BY ROESNER [4].

unimproved watersheds to determine yields, streamflows, and flood characteristics. Thus, its strengths may be presumed to lie in representation of the natural hydrologic cycle. On the other hand, the WRE-SWMM transport model modifications were developed specifically for complex urban pipe networks where flow reversals, backwaters, and special diversions are not uncommon. The differentiation is obvious.

Getting from rainfall to runoff quantities may be as simple a process as applying a direct percentage (perhaps not a bad assumption when the percent imperviousness is high), or it may be a sophisticated accounting of depression storage, infiltration rates, and surface and channel flows. Or it may be an even more sophisticated accounting of soil types and moisture content, vegetation, ground slopes, air temperature, wind direction, etc.

In developing the treatment process simulation for SWMM, a major consideration was not only to develop the operating rules for efficiencies when operating within prescribed limits, but more importantly, to account for the bypassed flows or process inoperation when the unit was subjected to flows outside the prescribed limits. Such unnormalized conditions should not be overlooked in assessing capabilities.

Examples

Metcalf & Eddy has applied stormwater management modeling techniques to suit many varied client requirements. In the following examples, selection between planning and design are emphasized.

Case 1 - Planning - In 1970 the City of Chicago wanted a modeling capability to support its studies in developing a master plan to abate pollution from combined sewer overflows and flooding. A primary consideration was that under the largest storms of record there must be no backflow into Lake Michigan. Thus, the requirement was event specific. The great magnitude of the problem--650 overflows to 75 miles of waterways--made it economically imperative that an in-house capability of performing the modeling runs be developed. Finally, abatement schemes were quality sensitive. SWMM was selected and successfully applied. Surcharging was not a handicap.

Case 2 - Planning - In 1972 the District of Columbia was committed to a massive upgrading of treatment at its dry-weather facility to tertiary levels never approached in such a scale. Concern was raised that the improvements at the Blue Plains facility would be negated by raw discharges through combined sewer overflows every time it rained. An answer was needed in 90 days as to where the cost-benefits divided between plant upgrading and wet-weather control from the basis of in-stream quality. A combination of simplified models (to determine the approximate frequency, location, and magnitudes of loadings) and dynamic models (SWMM - Receiving Water Block) was used.

Case 3 - Design - In 1972-1973 the City of Saginaw was proceeding into initial implementation of its combined sewer overflow abatement program under a State of Michigan mandate to

> Complete construction of facilities necessary to abate pollution caused by said overflows and place same in continuous operation on or before June 1, 1977.

The 1972-1973 task is summarized in the following sentence taken directly from the report: "The basic plan, being well established and approved, leaves remaining the problem of determining the final location, size, and type of facilities." The Runoff, Transport, and Storage Blocks of SWMM were used to test several alternatives incorporating various sizes of storage and treatment facilities for the design storm conditions. A quasi-steady state model was used to relate impacts on the river system. The solution accounted for both in-system and off-line storage as well as dual treatment functions. The first facility is under construction.

<u>Case 4 - Design</u> - The City of Hartford was interested in developing a modeling capability within its staff and to improve upon the Rational method for design of necessary additions to the urban drainage system. City staff members were trained in the application of the Runoff and Transport Blocks of SWMM, and then they proceeded to check completed, off-the-shelf designs of major drains scheduled for near-term construction. By accounting for the <u>dynamics</u> of flow routing, it was frequently found that one, two, or more pipe diameter increments could be cut from the traditionally designed systems while still handling the design storm.

IDENTIFY NECESSARY ASSUMPTIONS

Each model has a specific set of data requirements. Comparing these with the previously identified given data base will identify the proportions of the assumed data base or the dependency of the results on <u>default</u> values. If a large array of default values is to be used in every case, can they be sidestepped altogether by going to a simpler model? That is, what is the chance of your answer being locked in before you start?

Statics

As with the given data, some assumptions will necessarily remain fixed over the course of the project, while others may change significantly through contributions from other programs, sensitivity analyses, or verification runs. Setting down and differentiating between the two may identify key areas for supplemental R&D work or literature searches.

Typical static assumptions may include friction factors, infiltration ratios, retention storage, daily flow cycles, pre-storm conditions, diffusion coefficients, decay and replenishment rates, boundary exchanges, accounting for nonmodeling discharges, etc.

Dynamics

Dynamic assumptions may include the computational time steps, the "design" event (may be approached through trial and error executions), treatment-cost efficiencies, imperviousness and directly contributing impervious areas, as well as any of static assumptions where special effort is to be taken. The point is to make a realistic, pre-application appraisal of what types of sensitivity analyses are likely to be performed and whether or not they are likely to be converging.

SELECT FOR RESULTS

The success or failure of a modeling attempt can be judged only by the buyer. Did he get the information he needed when he needed it? For the price he expected to pay for it? If an initial attempt was a failure, where did it go wrong--objectives, data base, model capabilities, assumptions? If it was successful, where does the credit lie?

Decision Output Data

As introduced earlier, the display, listing, or packaging of the output data may be as important to the decision maker as the data itself. Are the output values to be accepted because they display eight significant figures or because the cause and effect relationships are visible, reasonable, and capable of verification? If the output is voluminous, where are the checks and balances? How is it displayed? If part of the computation was made outside the game rules, how is it signaled? Corrected? If a trace back into the simulation is required, can one obtain intermediate results at a specific location in the system? At a specific point in the computations? How about restarts to avoid repetitious computations?

A point, perhaps understressed heretofore, is the value of accuracy in the results. A dynamic model may give the impression of a very true result for a clean, circular conduit where, in fact, 50 percent of the line is in a state of collapse, subsided, or filled with debris. How valuable is representing the response of a new system when you are stuck with the one you have? Further, if the transfer of quality data from Transport or Storage to Receive is to be accomplished in hourly blocks, is it necessary that it be computed at 5-minute intervals? The answer, of course, goes back to the study objectives as to what is to be improved, how much, and where.

Building Block in Continuing Program

One of the advantages of modeling is that almost as much value might be obtained from a technically failed run as from a successful one. Why? Because the effort of system analysis that goes into preparing for and executing a run forces the issue into areas that have traditionally fallen into neglect as lost items within a city budget. In selecting a model, the user should assess its impact on his broader continuing program. If the present objectives are met, what is the next step? Will models again be a requirement? Should the experience and data base be built up now in preparation?

Does It Fit

In summing up all of the above selection criteria, it is seen that there are both choices and opportunities in modeling urban stormwater runoff. Finding the model of best fit is best approached by a systematic appraisal, such as the one outlined herein, with emphasis on the preselection definition of study objectives.

Planning Vs. Design

In the author's experience there are basically two levels of approach available: the simplified and the detailed. The advantage is that, when understood, they can and should be entirely complementary. The advantage of the simplified models is their ability to process long periods of record and broad areal coverage at low cost. The advantage of the detailed models is their ability to make a comprehensive analysis of singular events and systems with a corresponding increase in accuracy when supported by a viable data base.

In planning studies the simplified models offer a flexible screening device to identify consequential storm events and potentially attractive alternatives. The detailed models permit the necessary follow-up and technical evaluation between competing plans.

In design, the roles may be somewhat reversed with the detailed models fixing the component dimensions and the simplified models testing the decision against the historical record. These capabilities and interfaces will be described further in Part II: A SIMPLIFIED STORMWATER MANAGEMENT MODEL FOR PLANNING.

PART II. A SIMPLIFIED STORMWATER MANAGEMENT MODEL FOR PLANNING

By

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What are simplified stormwater management models? Why are they needed? How are they applied? What is their value? These questions will be addressed in this lecture through the context of a modeling strategy previously applied and now undergoing critical refinement and documentation.

In the author's opinion, the simplest model that will get the job done is usually the best. The intent here is not to sell a particular model but rather to set forth the approach methodology. If a simplified model has to be disseminated in a preassembled package, then it has already lost a significant fraction of its utility. The user should have the flexibility to assemble model components and detail to build on his data strengths and towards his study objectives.

PURPOSE AND AVAILABILITY

As brought out earlier, the target of the simplified models is to break down data into a form meaningful to the user. In so doing, precision is sacrificed for breadth of coverage. Further, because of the low cost of simplified models, both for setup and execution, multiple assumptions can be tested with relative ease and over a short period of time.

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Functional Location in Master Program

The key sequential steps in a typical urban stormwater management master program are shown in Figure 2. While simplified models are principally an early planning screening device, they can be used effectively throughout a program's development, design, and implementation, including--optimistically--real time control.

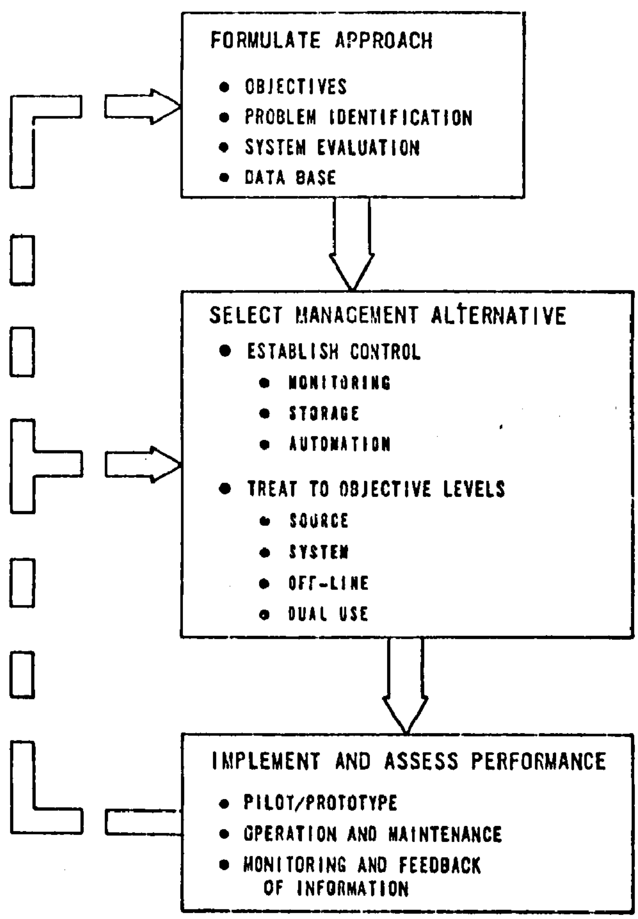


FIGURE 2. URBAN STORMWATER MANAGEMENT STRATEGY [5]

Subsystems

The subsystems to be described in this presentation are (1) rainfall characterization, (2) storage-treatment balance, and (3) discharge-receiving water response.

Rainfall Characterization - Rainfall characterization is basically a sorting and ranking of historical rainfall data from local gages. It requires a simple but precise storm event definition to permit disassociation of chronological sequence for statistical analysis. Official Weather Bureau recordings of hourly data are a practical data source.

Storage-Treatment Balance - Storage-treatment balance models take historic rainfall data in chronological sequence as input and log quantities, times, and durations of overflows for selected storage-treatment combinations.

Discharge-Receiving Water Response - Discharge-receiving water response models apply significant quality characteristics to the discharges and predict their behavior in the receiving stream. The latter model can be very complex, particularly in an estuarine environment.

Primary Goals

Simplified math models or their equivalent analyses are needed for three reasons:

1. To introduce time and probability to stormwater analyses
2. To promote total system consciousness on the part of the user or reviewer
3. To establish size-effectiveness relationships

Just as time and probability analyses are important in sizing water supply impoundments and safe yields, they are--or should be--equally important in determining the effective use of stormwater facilities. Since total capture is not a necessary goal, as opposed to flood control works, there is greater latitude in facility sizing and staged implementation. The trick is determining the relative merits of alternatives, a task for which modeling is ideally suited.

Alternatives should not be limited to stormwater abatement measures alone and may readily encompass dry-weather plant upgrading and separate as well as combined discharges.

Source

Documentation and description of the modeling techniques described herein is being prepared by Metcalf & Eddy under a demonstration project sponsored by the Environmental Protection Agency through a grant to the Rochester Pure Waters District [6]. It is noteworthy that the work is being performed concurrently and as an integrated part of a much larger program involving extensive overflow monitoring and characterization, dynamic simulations, pilot plant construction and operation, and outline of a total program for collection, transmission, and treatment of combined sewage under a central control and management system. The simplified modeling report is expected to be published under EPA's Environmental Protection Technology Series in June 1976.

A similar, directly available model, STORM [7], will be described in Dr. Roesner's lecture this afternoon. For comparison, the present program has approximately 300 statements versus 4,000 statements in STORM and 14,000 in SWMM.

APPLICATION STRATEGY

The successful application strategy of simplified models is one of forced recognition of, and adjustment to, system realities. In this section the component subsystems are described, and in the following section they are assembled into a working program.

Data Base

The first activities must be directed at answering the question: What do we have and how does it work? The process should inlcude not only the identification and compilation of data, but also the development of a functional representation of the system. A first attempt of such a drawing for the Rochester system is shown in Figure 3. An important requirement is that the points of dynamic data generation be shown as well as the relationships between what may prove to be test elements, including maximum capacities when available.

Rainfall Characterization

Data for rainfall characterization are most readily obtained from the U.S. Weather Bureau records either on tape files or through published daily and hourly summaries. Tapes are issued through:

U.S. Department of Commerce
National Climatic Center
NOAA Environmental Data Service
Federal Building
Ashville, N.C. 28801
Tel. (704) 258-2850

Data are available on two record files: Deck 488-USWB HOURLY PRECIPITATION and Deck 345-WBAN SUMMARY OF DAY. Most first order stations are covered. They range in number from 1 in

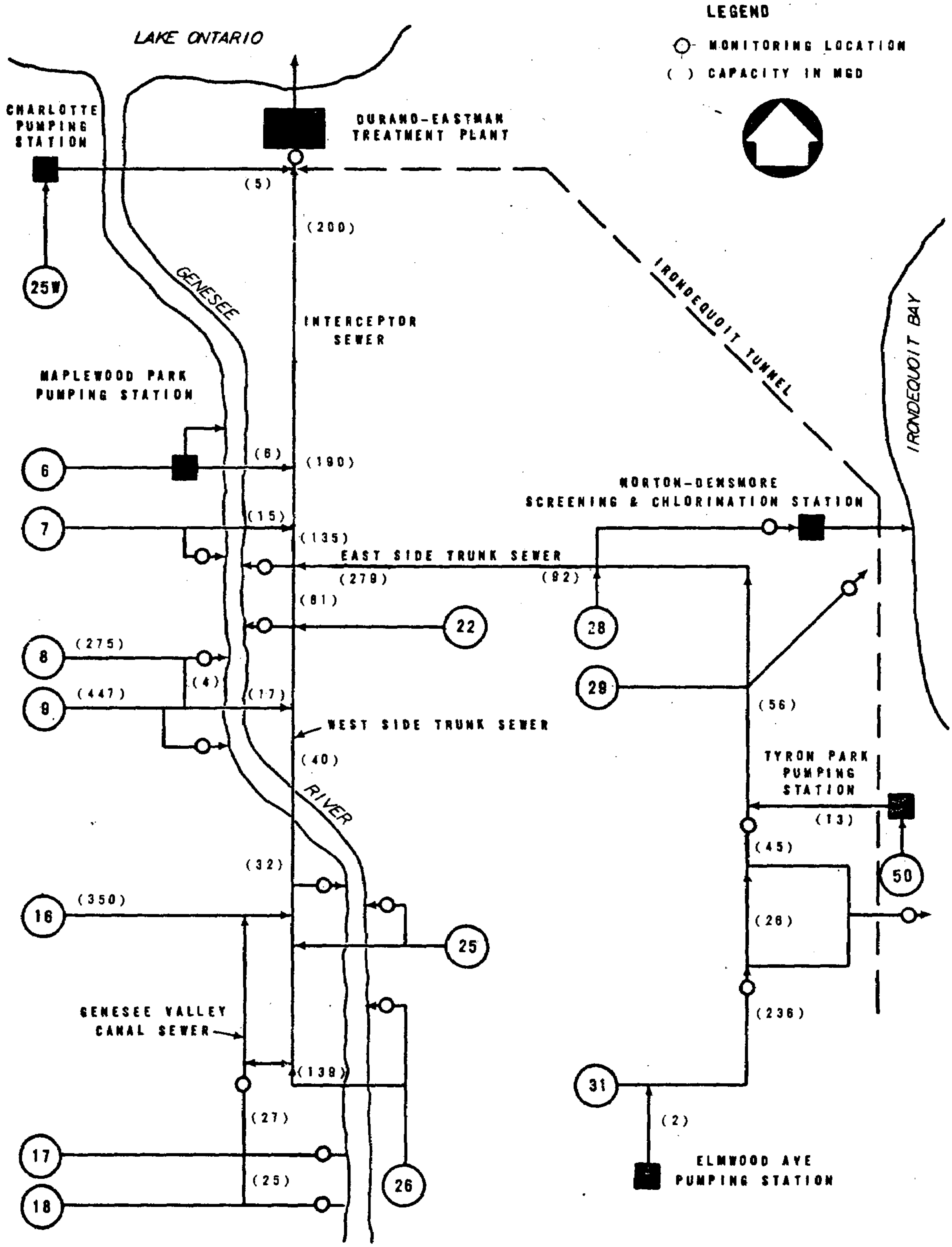


FIGURE 3. FUNCTIONAL ELEMENTS OF THE ROCHESTER SEWERAGE SYSTEM

Delaware to 19 in Texas. The period of record is generally from August 1949 to the current date with some gaps. Tapes are furnished 9 track-800 BPI, unless otherwise specified, and are forwarded air parcel post. Recent experience with these tapes has been excellent. The two tape files for Rochester were ordered by telephone on March 11, shipped on March 20, and received on March 26, for a combined cost of $140.

Sorting the data requires a precise storm event definition. The one Metcalf & Eddy has standardized upon for the present defines a storm as starting with the first measurable rainfall after a minimum of 6 hours with no rainfall and ending when a gap in measured rainfall (precipitation) of at least 6 hours is first encountered. Trace rainfall amounts are disregarded. The 6-hour gap was selected to ensure relative independence between events. For each event in the historical record, the following are noted and punched on data cards or filed on disk: date, starting hour, duration, total rainfall, maximum hourly rainfall and the hour in which it occurred, elapsed days since the previous storm, and occurrences of excessive precipitation and snow. The program routine is shown in Figure 4.

The data are then sorted and ranked in arrays according to items of interest, such as total rainfall, duration, maximum hourly rainfall, and summer storms. Selected results are reflected in Figures 5, 6, and 7. Other advantages of the arrayed data are that current storms can be directly classified in historical perspective, and probabilities of multiple rare occurrences within a given time span can be determined by inspection or simple calculation. For Rochester, sorting and ranking 3,000 storms by one parameter took 0.65 minute of execution time and 0.12 minute of CPU time on an IBM 360.

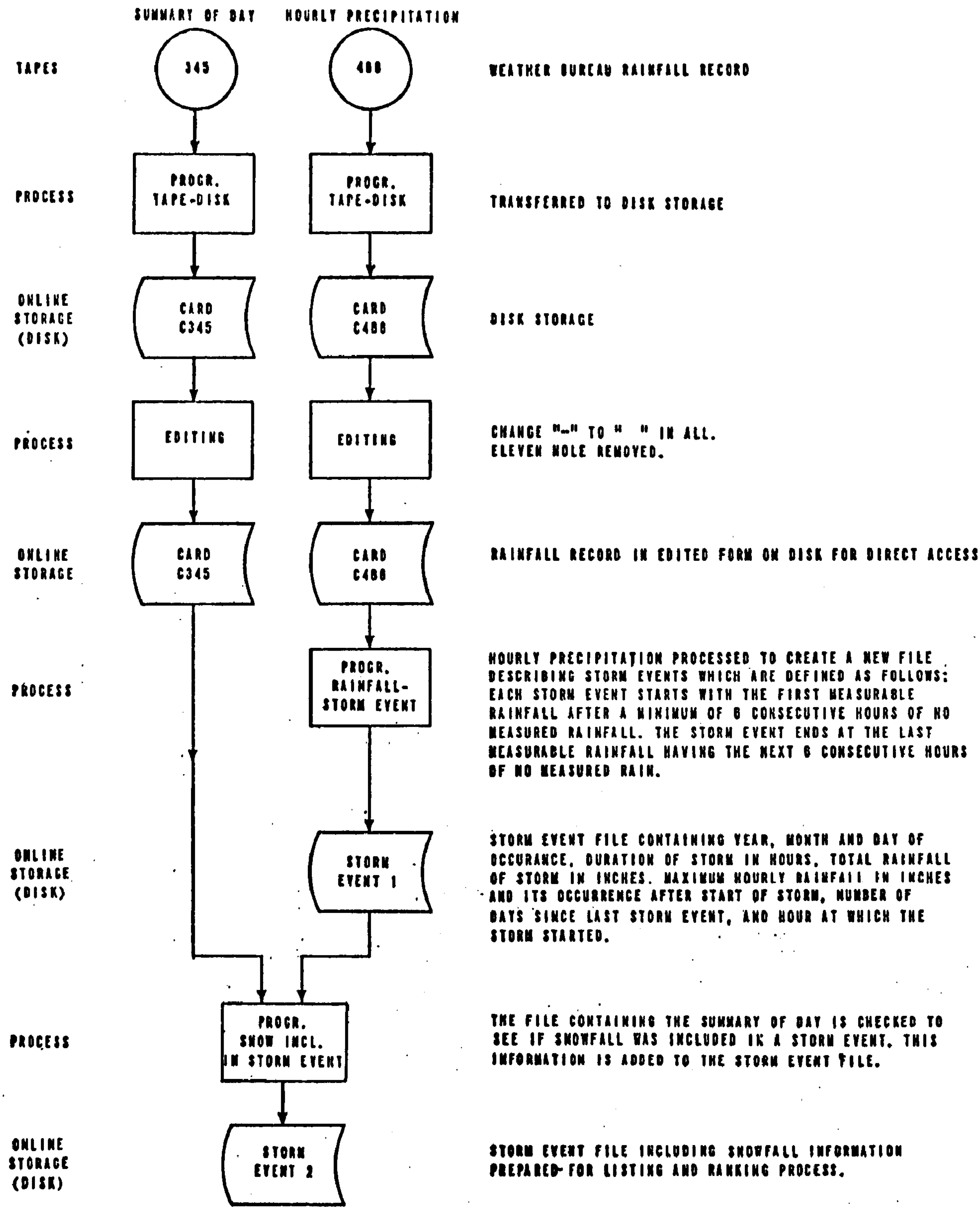


FIGURE 4. TAPED RAINFALL TRANSFER AND EDIT ROUTINE

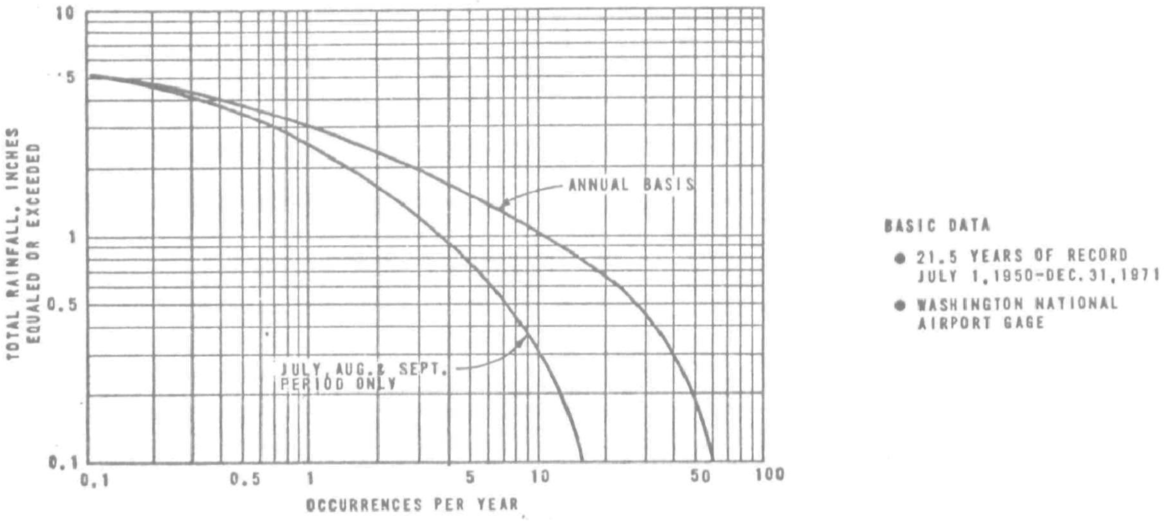


FIGURE 5. STORM MAGNITUDE VS. FREQUENCY OF OCCURRENCE

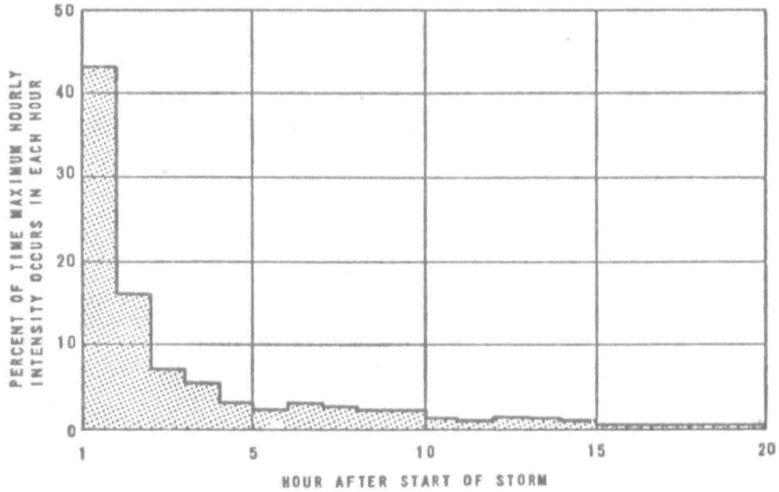


FIGURE 6. HOUR DURING STORM HAVING MAXIMUM RAINFALL

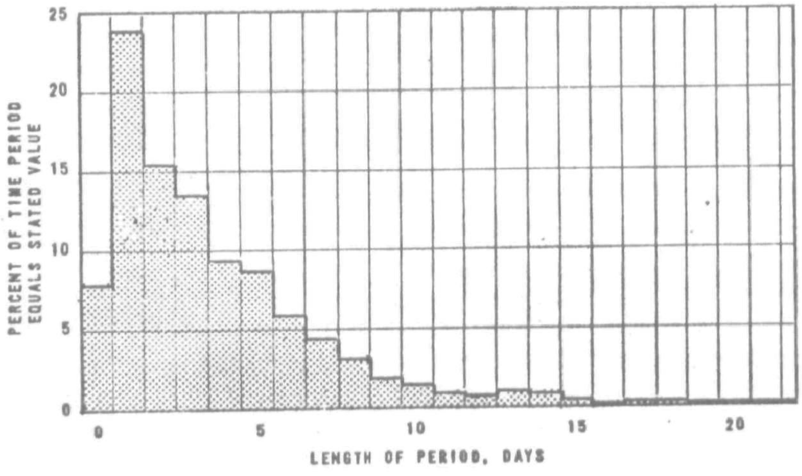


FIGURE 7. DRY PERIOD BETWEEN STORM EVENTS

Storage-Treatment Balance

The storage-treatment balance concept is shown in Figure 8. In the simulation, historical rainfall is read in chronological order, converted by a factor (coefficient of runoff) into runoff, and stored in a specified volume which is emptying at a specified rate. When the runoff exceeds the combined storage-treatment rate, an overflow occurs. The program flow-chart is shown in Figure 9. Initial runs are executed on a daily basis for the full period of record and over the range of the viable alternatives. Simulation of 20 years for one alternative takes less than one minute at a cost of approximately $20.

For periods of greatest interest, those producing overflows, repeat runs may be cycled on an hourly basis to develop a hydrograph and refine the storage-treatment simulation.

Input variables at the present time are simply the land area, runoff coefficient, storage capacity, and treatment rate. Sample output is shown in Table 4. A significant operating rule under reevaluation controls the starting of the treatment facility. As programmed, the treatment deduction from storage does not start until the cycle following the actual runoff, unless it happens to be running from a previous storm. When run on a daily cycle, this is intended to cover the uncertainty as to when during the day the storm actually started. When run on an hourly cycle, this delay represents a logical startup requirement.

The program is used by varying the storage capacity and treatment rates, and noting the changes in overflow occurrences and durations. An example is shown in Figure 10

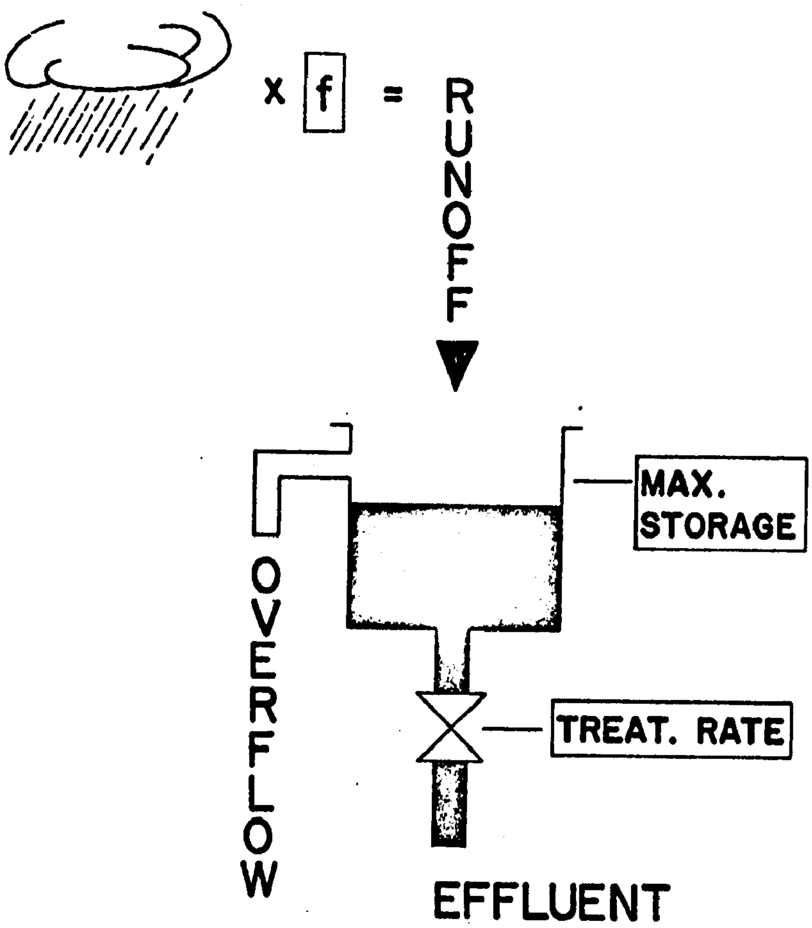


FIGURE 8. STORAGE-TREATMENT BALANCE CONCEPT.

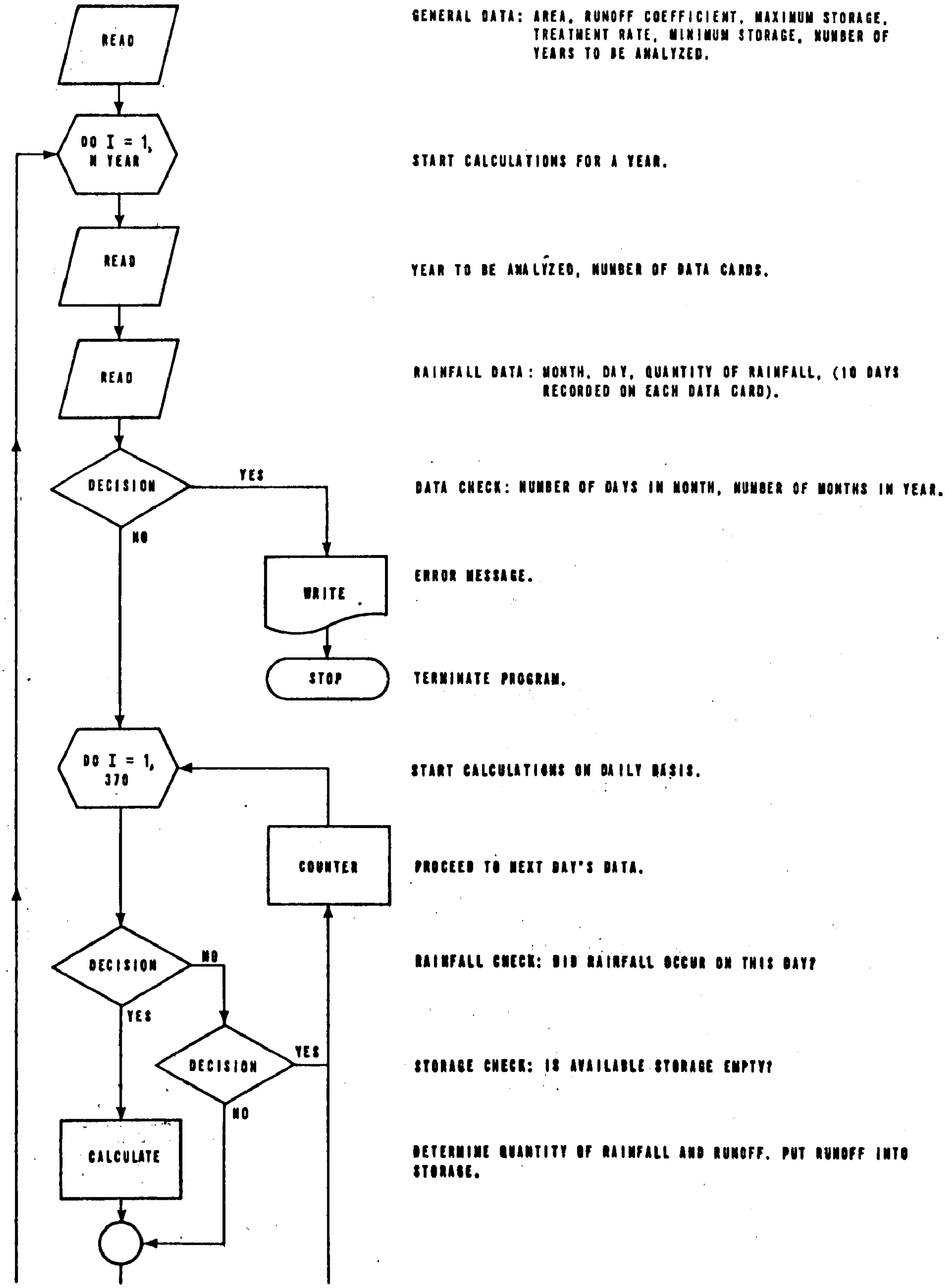
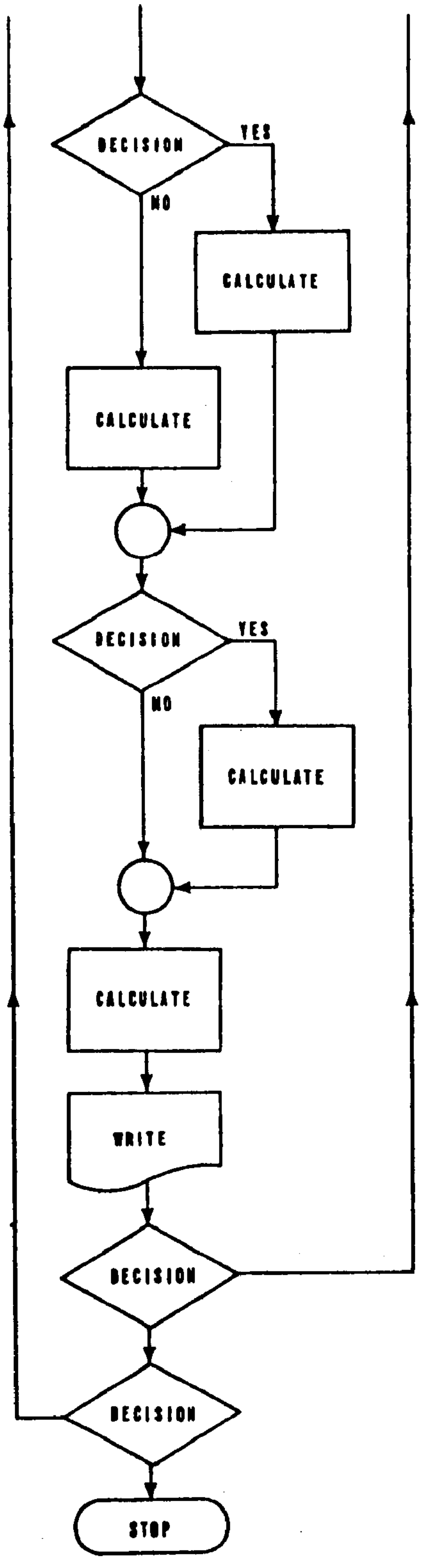


FIGURE 9. STORAGE-TREATMENT ROUTINE



IS TREATMENT OPERATIONAL FOR THIS CYCLE?

REDUCE STORAGE AT RATE OF TREATMENT.

START TREATMENT FOR NEXT CYCLE.

IS MAXIMUM STORAGE EXCEEDED?

CALCULATE QUANTITY OF WATER IN EXCESS OF MAXIMUM STORAGE AND DECLARE IT TO BE OVERFLOW.

CALCULATE: TOTAL RAINFALL FOR MONTH, QUANTITY OF RUNOFF, OVERFLOW TREATED, STORED AND DAYS OF RAIN AND OVERFLOW TO DATE FOR YEAR.

LIST: DATE (MONTH AND DAY), RAINFALL, RUNOFF, STORAGE TREATMENT, AND OVERFLOW FOR DAY OF OCCURRENCE AND SUM OF QUANTITIES TO DATE OF RUNOFF, OVERFLOW, STORAGE TREATMENT, DAYS OF RAIN AND OVERFLOW.

HAS DATE FOR EACH DAY OF YEAR BEEN READ?

HAS EACH YEAR OF RECORD BEEN ANALYZED?

TERMINATE PROGRAM.

FIGURE 9. (CONTINUED)

YEAR 1971				OCCURRING ON THE DATE				ACCUMULATED FROM START OF THE YEAR					
MONTH	DAY	RAIN IN	RAIN DAYS	RUNOFF MG	STORAGE MG	OVERFLOW MG	TREATED MG	T.RUNOFF MG	T.OVERFLOW MG	OVERFL DAYS	T. TREAT MG	TREAT DAYS	MAX STORAGE MG
8	1	0.07	66	19.59	19.59	0.00	0.00	6964.54	0.00	0	7096.08	28.38	495.27
8	2	0.00	66	0.00	0.00	0.00	19.59	6964.54	0.00	0	7115.66	28.46	495.27
8	3	1.62	67	453.30	453.30	0.00	0.00	7417.84	0.00	0	7115.66	28.46	495.27
8	4	0.67	68	187.48	187.48	0.00	250.00	7605.31	0.00	0	7365.66	29.46	495.27
8	5	0.48	69	134.31	134.31	0.00	250.00	7739.62	0.00	0	7615.66	30.46	495.27
8	6	0.00	69	0.00	0.00	0.00	250.00	7739.62	0.00	0	7865.66	31.46	495.27
8	7	0.00	69	0.00	0.00	0.00	25.09	7739.62	0.00	0	7890.75	31.56	495.27
8	8	0.00	69	0.00	0.00	0.00	0.00	7739.62	0.00	0	7890.75	31.56	495.27
8	9	0.00	69	0.00	0.00	0.00	0.00	7739.69	0.00	0	7890.75	31.56	495.27
8	10	0.00	69	0.00	0.00	0.00	0.00	7739.62	0.00	0	7890.75	31.56	495.27
8	11	0.32	70	89.50	89.54	0.00	0.00	7829.16	0.00	0	7890.75	31.56	495.27
8	12	0.00	70	0.00	0.00	0.00	89.54	7829.16	0.00	0	7980.29	31.92	495.27
8	13	0.00	70	0.00	0.00	0.00	0.00	6829.16	0.00	0	7980.29	31.92	495.27
8	14	0.00	70	0.00	0.00	0.00	0.00	7829.16	0.00	0	7980.29	31.92	495.27
8	15	0.00	70	0.00	0.00	0.00	0.00	7829.16	0.00	0	7980.29	31.92	495.27
8	16	0.00	70	0.00	0.00	0.00	0.00	7829.16	0.00	0	7980.29	31.92	495.27
8	17	0.00	70	0.00	0.00	0.00	0.00	7829.16	0.00	0	7980.29	31.92	495.27
8	18	0.00	70	0.00	0.00	0.00	0.00	7829.16	0.00	0	7980.29	31.92	495.27
8	19	0.17	71	47.57	47.57	0.00	0.00	7876.73	0.00	0	7980.29	31.92	495.27
8	20	0.00	71	0.00	0.00	0.00	47.57	7876.73	0.00	0	8027.86	32.11	495.27
8	21	0.00	71	0.00	0.00	0.00	0.00	7876.73	0.00	0	8027.86	32.11	495.27
8	22	0.00	71	0.00	0.00	0.00	0.00	7876.73	0.00	0	8027.86	32.11	495.27
8	23	0.00	71	0.00	0.00	0.00	0.00	7876.73	0.00	0	8027.86	32.11	495.27
8	24	0.00	71	0.00	0.00	0.00	0.00	7876.73	0.00	0	8027.86	32.11	495.27
8	25	0.00	71	0.00	0.00	0.00	0.00	7876.73	0.00	0	8027.86	32.11	495.27
8	26	0.00	72	16.79	16.79	0.00	0.00	7893.51	0.00	0	8027.86	32.11	495.27
8	27	3.79	73	1060.50	590.00	237.29	250.00	8954.01	237.29	1	8277.86	33.11	590.00
8	28	0.00	73	0.00	340.00	0.00	250.00	8954.01	237.29	1	8527.86	34.11	590.00
8	29	0.00	73	0.00	90.00	0.00	250.00	8954.01	237.29	1	8777.86	35.11	590.00
8	30	0.00	73	0.00	0.00	0.00	90.00	8954.01	237.29	1	8867.85	35.47	590.00
8	31	0.00	73	0.00	0.00	0.00	0.00	8954.01	237.29	1	8867.85	35.47	590.00

TOTAL RAIN 7.18

TABLE 4. SAMPLE OUTPUT (DAILY CYCLE)

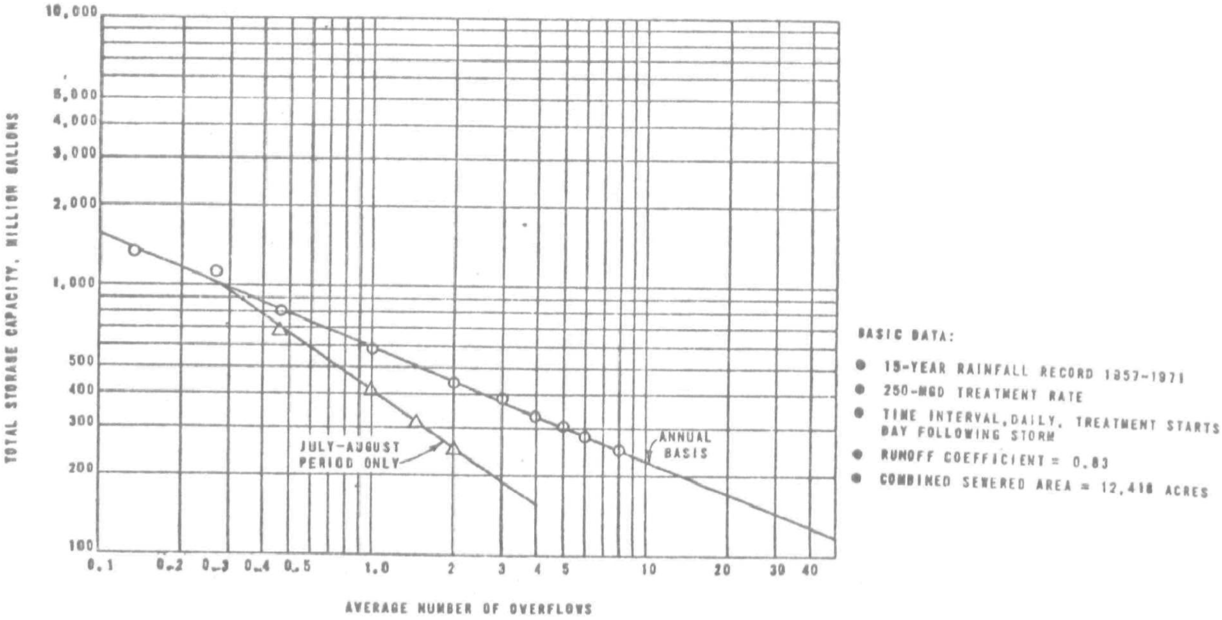


FIGURE 10. ANNUAL OVERFLOW OCCURRENCES VERSUS STORAGE CAPACITY.

comparing varied storage volumes to a constant treatment rate and noting the change in overflow occurrences.

Overflows, Quantity and Quality

The quantity and frequency of overflows is determined by the storage-treatment balance computation. Assigning quality characteristics can follow one of two approaches: (1) the approach can follow a dust and dirt buildup and removal formulation as in SWMM and STORM, or (2) it can directly compute concentrations through some simple relationships or regression techniques. The latter, more direct approach is preferred when supported by at least a minimum of measured real data.

In the District of Columbia study introduced in Part I, significant overflows were computed on an hourly basis. Quality characteristics were added using the following assumed relationships:

> Suspended solids (SS) concentrations in milligrams per liter were computed from the following expression for both combined and separate discharges:
>
> $$SS = 400 \times f_1 \times f_2 \times f_3$$
>
> where f_1 is a function of days since the last storm and the time from the start of overflow (matrix range 0.3 to 2.6)
>
> f_2 is a function of rainfall intensity (0.5 to 1.5)
>
> f_3 is a function of catchment population density (0.5 to 1.0)
>
> A new concentration was computed for each time increment (0.5 to 1.0 hours)

BOD_5 was computed from the suspended solids concentrations as follows:

For separate storm drains

$$BOD_5\ (storm) = .10 \times SS \qquad [SS \lesssim 300]$$

$$BOD_5\ (storm) = 30 + (SS-300) \times .08 \qquad [SS > 300]$$

For combined overflows

$$BOD_5\ (comb) = \alpha D + (1 - \alpha) \times BOD_5\ (storm)$$

where α is the proportion of combined flow attributed to average dry-weather flow

D is the average BOD_5 of dry-weather flow

Where needed, total nitrogen and total phosphorus were approximated by the following expressions:

$$N = .10\ BOD_5$$

$$P = .033\ BOD_5$$

Typical utilization of these data are shown in Table 5 where loadings from various discharges are compared for an intense summer storm. The cost-benefit priorities, based on this one parameter, are also shown. Similar comparisons were made for a major long storm, average summer quarter, and annual bases.

Table 5. COMPARATIVE DISCHARGES FROM INTENSE SUMMER STORM

	BOD_5, 1,000 lb	Priority
Runoff from existing separate sewer areas		
Untreated	125	3
Combined overflows		
Untreated	174	
If separated	164	
With storage-treatment	9	2
STP discharged (daily)		
Existing	134	
Upgraded	13	1

Note: Two-thirds of the existing area has separate sewers.

In Rochester, since field data are being collected concurrently, new relationships are being tested with emphasis on overflow traits at specific locations. Early comparisons are shown in Figure 11, which represents schematic plans of the system with superimposed ranking of land use and population densities, runoff coefficients, and constituent constraints. The intent is not only to identify trends but also to screen out potentially faulty data. While the present scatter may appear rather meaningless, the utility of the scheme is expected to increase as the data base grows.

The purpose of the analysis is, of course, to rank the potential pollution sources so as to guide emphasis in a staged corrective program or operational strategy.

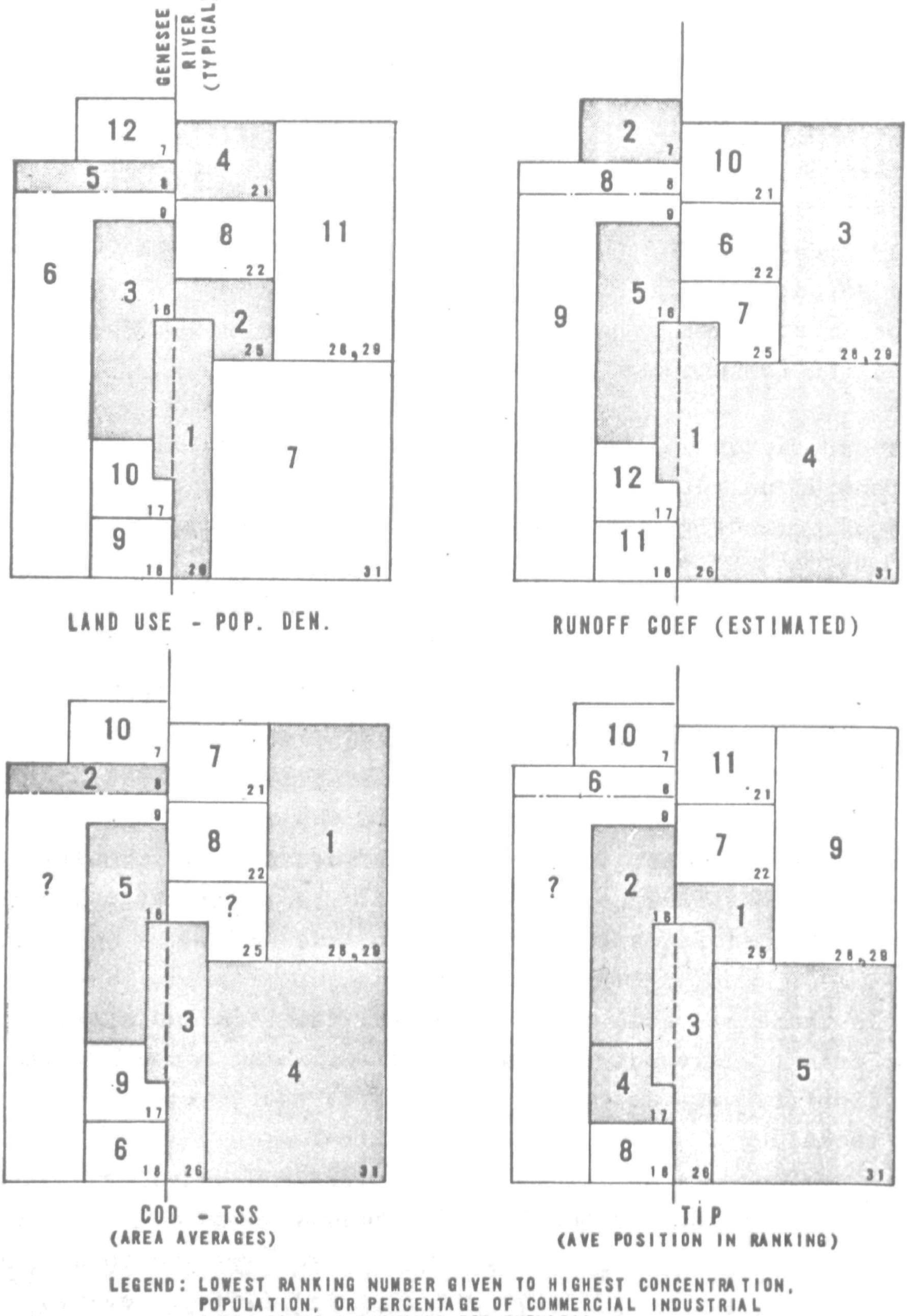
Receiving Water Response

Receiving water response is perhaps the highest ranked study objective, yet it is also the least defined in terms of present day real time data. In the absence of these data, it is our practice to use the best available simulation model that has some record of use in the area. This permits testing wet-weather impacts on a technical level compatible with previous analyses for treatment plant discharges, salt water intrusions, etc.

While there are some misgivings with this limited approach (we frankly have not come upon a simplifying breakthrough), credibility requires and benefits from the similarities in techniques.

ROCHESTER AND D.C. DEMONSTRATION PROJECTS

The final test of a model or approach to modeling is using it. The application of simplified models to solve engineering



LEGEND: LOWEST RANKING NUMBER GIVEN TO HIGHEST CONCENTRATION, POPULATION, OR PERCENTAGE OF COMMERCIAL INDUSTRIAL AREA. RANKS 1-5 SHADED FOR EMPHASIS.

FIGURE 11. OVERFLOW RANKING TRENDS

problems is not new; indeed, several cities, such as Chicago, have rather distinguished programs. It is unfortunate, however, that their use is so little publicized.

Application of simplified models is an art. In fact, the simpler the model, the greater the reliance on the judgment of the user, and perhaps conversely, the greater the danger of misapplication.

Tying It All Together

A simplified PERT worksheet is shown in Figure 12. This worksheet, developed for the Rochester project, illustrates the tying of the various programs together while focusing on the result. Noteworthy are transfers of information between tasks, shown by dashed lines.

Again, contributions of the individual tasks are dependent on timing and level of effort, forcing decisions at the designated transfer points.

Interfaces with SWMM

SWMM is a logical backup model for attempts at simplified approaches. That is, it answer the question--If I had more data or a more sophisticated approach, how would my results be affected? Similarly, if the record of a large number of SWMM runs exists for a particular area, the output may be used as a basis for a simplified approach to broaden the areal and time coverage.

Rochester - In Rochester, SWMM Runoff and Transport are being calibrated for each of the 13 combined sewer overflow points. Once this is accomplished, the 13 overflows are to act as nodes of an entire system analysis for specific events

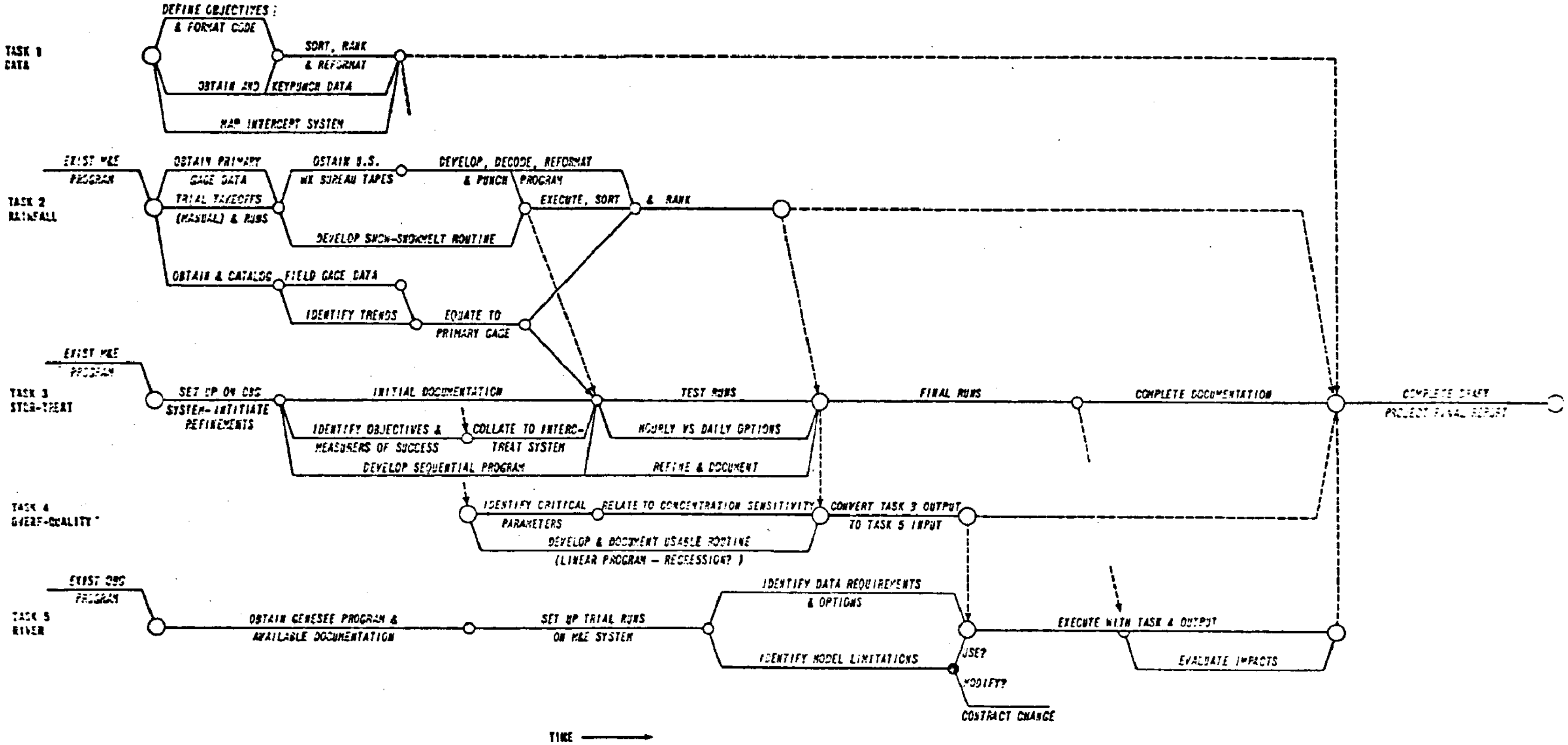


FIGURE 12. SIMPLIFIED APPROACH ACTIVITY DIAGRAM

centering around the dry-weather treatment plant and the interceptor system.

The simplified model application is directed at the same objective but in less detail and in the context of the total historical record. SWMM results are to be used to refine the runoff coefficient estimates in the simplified approach. Also, in the Rochester program, an extension in simplified modeling capabilities is proposed, wherein each of the 13 areas will be run in a cascading loop with the interconnecting interceptor capacities determining the treatment rates. This will give a spacial aspect to overflows and storage devices not previously included in this level of analysis.

District of Columbia - In the District of Columbia simulation, the simplified model results for a storm occurring once in 2 years were applied to a dynamic (SWMM) receiving water model of the Potomac estuary. The receiving water model was fed data believed characteristic of summer quarter tidal cycles and headwater and other background inflows. Figure 13 represents the calculated effect of upgrading the regional dry-weather flow treatment efficiency from 90 to 98 percent in terms of BOD_5. Figure 14 represents the impact of the 2-year storm on the upgraded system 3 and 5 days after the storm occurrence. Surprisingly, reducing the storm BOD_5 loading by one-third before discharge for this major event raised the minimum dissolved oxygen in the river by only 0.5 mg/l, from 2.6 to 3.1 mg/l. From these runs, the time it took the river system to recover from the shock impact of a storm was also computed. In the selected example, dissolved oxygen recovery was essentially complete in 10 days.

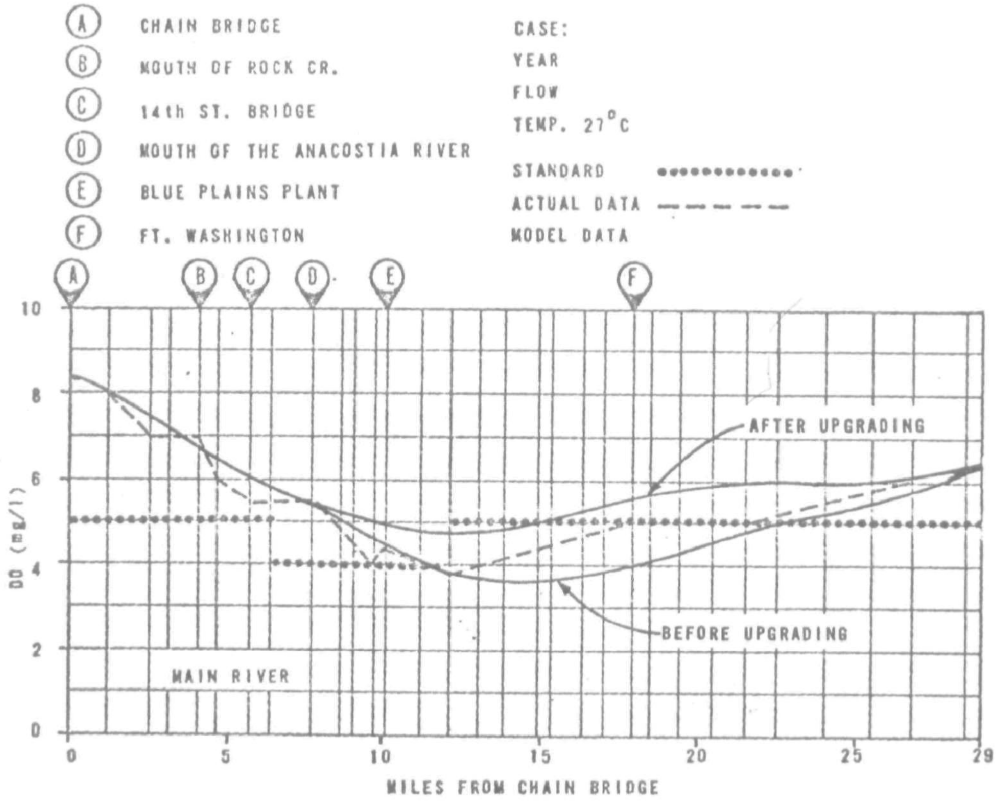


FIGURE 13. IMPACT OF UPGRADING REGIONAL PLANT

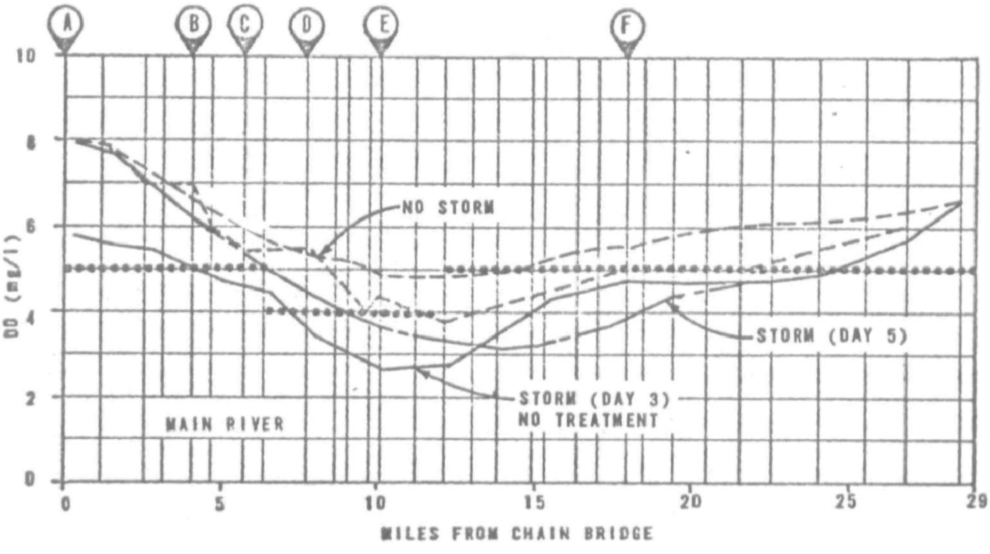


FIGURE 14. IMPACT OF 2-YEAR STORM WITH UPGRADED PLANT

Present Status

As this paper is being prepared, the Rochester project is approximately 25 percent complete. Overflow data from 12 storms and 12 locations (64 discrete data sets because few storms are sampled at all locations) have been collected and analyzed. The historical rainfall has been received, categorized, and ranked. The storage-treatment model is operational on the client's system, and the client's receiving water model has been tested on our system. Snowmelt and sequential routines are under development.

The District of Columbia reconnaissance-level study was concluded with the development of a feasibility study plan, estimated to be capable of capturing and treating approxi- 98 percent of the average annual combined sewer overflows and limiting the average number of overflow occurrences to less than one per year. Implementation of the plan is estimated to be capable of reducing the total long-term pollution loadings on the river system from all District sources by 25 to 35 percent, as opposed to an estimated 7 percent reduction for a program of total sewer separation.

Potentials for Management Decisions

These demonstration projects have shown simplified model approaches to be a valuable tool to decision makers. Benefit potentials lie in the following areas:

- Data reduction and classification
- Relating cause to effect
- Preliminary screening
- Testing alternative approaches to a common baseline
- Sensitivity analyses

- Assessment of data worth, tracking ability, or tendency (i.e., Does it fit the concept?), and basis for mid-course correction
- Setting project priorities
- Predicting outcomes of specific actions in short response times (i.e., real time controls)
- Communication and credibility
- Simplicity and low cost

As an example, suppose a manager is weighing two alternatives, one of which is treatment dominant and the other is storage dominant. He knows that, once functioning, the treatment system will consistently improve performance as the storm progresses and as its operating run lengthens. He also knows that the first hour's operation for each storm is likely to be spotty. Having characterization data, such as shown in Figure 6, would indicate the relative significance of the first hour. In this case, the first hour is by far the most significant; thus, the storage dominant option would be favored. Another interpretation might be that the minimum required storage should at least equal the maximum first hour runoff (adjusted for in-system travel time and equalization).

Distinguishing Between Model Weaknesses and Frills

Model weaknesses are limitations in logic or simulation that lead to inadequate or misrepresented results. Frills are the accessories to the basic logic and simulation that have little basis for use or consequence in results. The danger in frills is not so much that they create inefficiency in data gathering and application but rather that they might mask the true weaknesses by symbolizing a precision that does not exist or an operation which is unnecessary.

SUMMARY

Part I

Selection of stormwater management models should follow a systematic preassessment process:

- Define the study objectives
- Identify the given data base
- Identify the array of model capabilities
- Identify necessary assumptions
- Select for results

Of these tasks, defining the study objectives is both the most difficult and potentially rewarding. The given data base and assumptions include both static criteria, fixed in the initial stages of the project, and dynamic criteria, which change during and following the course of the work. Differentiating between these categories will assist the model selection.

Models follow operating rules without exception or interpretation and can process mountains of data with relative ease. The developer sets the rules and the user furnishes the data and is left with the results. There are several models to choose from. These models are separable by function (planning, design, operation) and by degree of precision (simple, intermediate, and complex).

Simplified models are capable of processing long periods of record and broad areal coverage at low cost. Detailed models reflect the dynamics of a system, and emphasis is placed on the comprehensive analysis of singular events and component systems. The advantage of these two levels of approach is that, when understood, they can and should be entirely complementary.

Only the buyer can make final judgment as to the success or failure of a modeling attempt.

Part II

Simplified modeling approaches should be used by decision makers, the basic reason being that the technique permits a direct assessment of the logic of an approach and the weighted credibility of each of the data inputs without unnecessary masking by detail.

The potentials are not limited to the planning stage, but extend through the design, and into the implementation (control) and feedback stage. Subsystem capabilities include:

- Rainfall characterization
- Storage-treatment balance
- Discharge-receiving water response

Interfaces with the more sophisticated dynamic models have been successfully demonstrated. However, a principal advantage lies in keeping the models simple, flexible, and unencumbered by unnecessary frills. Because of their simplicity and broad perspective, they have a strong potential in communicating ideas and projecting long-term outcomes of rather explicit actions.

Acknowledgment

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A STORAGE, TREATMENT, OVERFLOW AND RUNOFF MODEL FOR METROPOLITAN MASTERPLANNING

By

Larry A. Roesner, Ph.D.*

CONCEPT OF "STORM"

The quantity or urban runoff has traditionally been estimated by using a design storm through frequency-duration-intensity curves or some other statistical means based on rainfall records. Such approaches normally neglect the spacing between storms and the capacity of the urban system to deal with some types of storms better than others.

Often, through natural and artificial storage mechanisms, intense short-duration storms may be completely contained within storage so that no untreated stormwater overflows to receiving waters. Alternately, a series of closely spaced, moderately sized storms may tax the system to the point that excess water must be released untreated. Consider, for example, Figure 1 which shows the response of two different systems to the same rainfall trace. System A, which has a relatively high treatment rate and a small storage capacity, will overflow during the high intensity, short duration storm. However, it will completely contain the second storm of moderate intensity and longer duration. System B, on the other hand, which has a low treatment rate and a large storage capacity, completely contains the first storm. Notice that it would also contain the second storm if the system were analyzed independently of the antecedent storm. However, in this case the spacing of the storms is such that the system analysis must include both rainstorms as a single event to accurately describe the system's response to the rainfall trace illustrated in the figure.

A storm cannot be defined by itself, but must be defined taking into account the response characteristics of the urban stormwater system. It is for this reason that an approach was developed that would not only recognize the properties of rainfall duration and intensity, but would also consider storm spacing and the capacity of the urban stormwater system.

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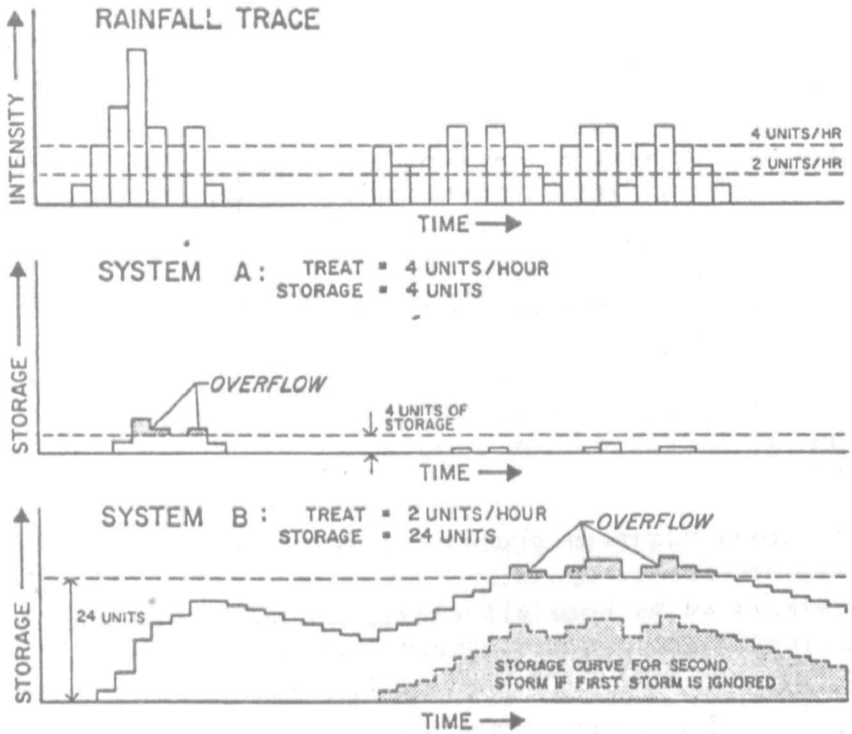


Figure 1. System Response Examples.

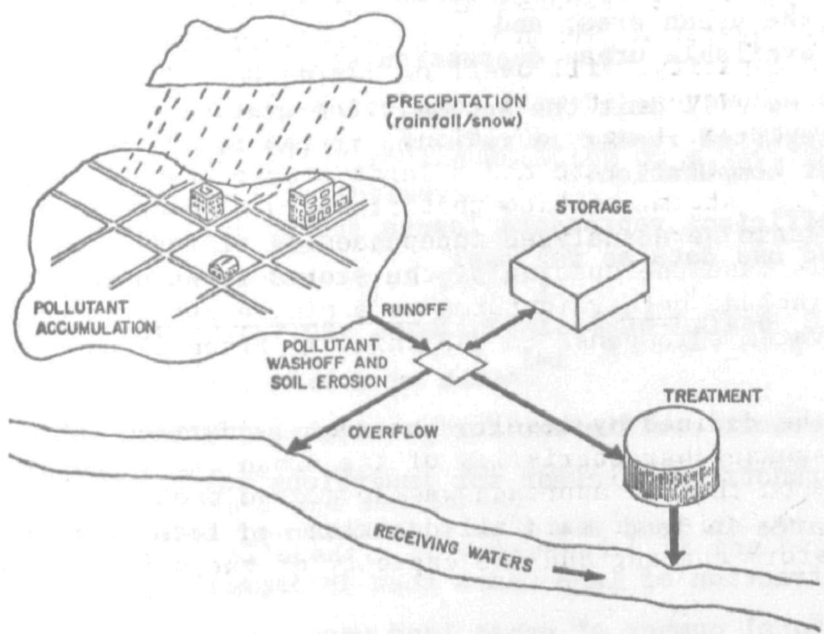


Figure 2. Conceptualized View of Urban System Used in "STORM".

Figure 2 shows, pictorially, the interrelationship of the eight stormwater elements considered in this approach for estimating stormwater runoff quality and quantity. In this approach, rainfall washes dust and dirt and the associated pollutants off the watershed to the storage-treatment facilities so that as much stormwater runoff as possible can be treated prior to its release. Runoff exceeding the capacity of the treatment plant is stored for treatment later. When the storage facilities become inadequate to contain the runoff the untreated excess is wasted through overflow directly into the receiving waters.

For a given precipitation record, the quantity, quality, and number of overflows will vary as the treatment rate, storage capacity, and land use is changed. Land surface erosion is a function of land use, soil types, ground slope, rainfall/snowmelt energy and erosion control practices. A typical method of investigation is to alter the treatment, storage, and land use and note the resulting response of the system. A group of alternatives can then be selected from among those meeting the overflow quantity and quality objectives.

COMPUTATION OF THE QUANTITY OF RUNOFF

Runoff is calculated on an hourly basis as a function of rainfall plus snowmelt using the following expression:

$$R = C(P - f) \tag{1}$$

where

R = urban area runoff in inches per hour;
C = composite runoff coefficient dependent on urban land use;
P = rainfall plus snowmelt in inches per hour over the urban area; and
f = available urban depression storage in inches per hour.

For simplicity we will omit the snowmelt computation in our discussion here. The interested reader is referred to the User's Manual [1] for details of that computation.

The runoff coefficient represents losses due to infiltration. It is computed from land use data as follows:

$$C = C_p + (C_I - C_p) \sum_{i=1}^{L} X_i F_i \tag{2}$$

where

C_p = runoff coefficient for pervious surfaces;
C_I = runoff coefficient for impervious surfaces;
X_i = area in land use i as a fraction of total watershed area;
F_i = fraction of land use i that is impervious; and
L = total number of urban land uses.

Before the runoff coefficient is applied, depression storage losses must be satisfied. Depression storage represents the capacity of the watershed to retain water in ditches, depressions and on foliage. The amount of depression storage at any particular time is a function of past rainfall plus snowmelt and evapotranspiration rates. The function is computed continuously using the following expression, where f is in inches:

$$f = f_o + N_D\, k, \text{ for } f \leq D \qquad (3)$$

where

f_o = available depression storage, in inches, after previous rainfall;

N_D = number of dry days since previous rainfall;

k = recession factor, in inches/day, representing the recovery (evapotranspiration) of depression storage in inches; and

D = maximum available depression storage in inches.

Figures 3a and 3b show graphically the hourly precipitation (P), depression storage (f), precipitation excess (P-f) and the resulting runoff (R). Figures 3b and 3c show how the runoff is distributed between treatment storage and overflow for a system with a treatment rate of 0.02 inches/hour and a storage capacity of 0.16 inches.

ESTIMATION OF THE RATE OF POLLUTANT BUILDUP ON URBAN WATERSHEDS

The estimates of pollutant accumulation rates on the urban watershed, and the rate of washoff during a storm event follow closely the methods used in SWMM(2). The rate of dust and dirt accumulation, DD_L for a given land use L is expressed as:

$$DD_L = dd_L \times (G_L/100) \times A_L \qquad (4)$$

where

DD_L = rate of dust and dirt accumulation on a watershed of land use L in lbs/day;

dd_L = rate of dust and dirt accumulation on watershed L in lbs/day/100 feet of gutter;

G_L = feet of gutter per acre in watershed L; and

A_L = area of watershed L in acres.

The rate factor dd_L should be supplied by the user for his area. Default values, (APWA factors; see Reference 2) are incorporated into STORM and can be used if no better data are available.

The initial quality of a pollutant p on watershed L at the beginning of a storm is then computed as:

$$P_p = (F_p \times DD_L \times N_D) + P_{po} \qquad (5)$$

where

P_p = total pounds of pollutant p on the watershed L at the beginning of a storm;

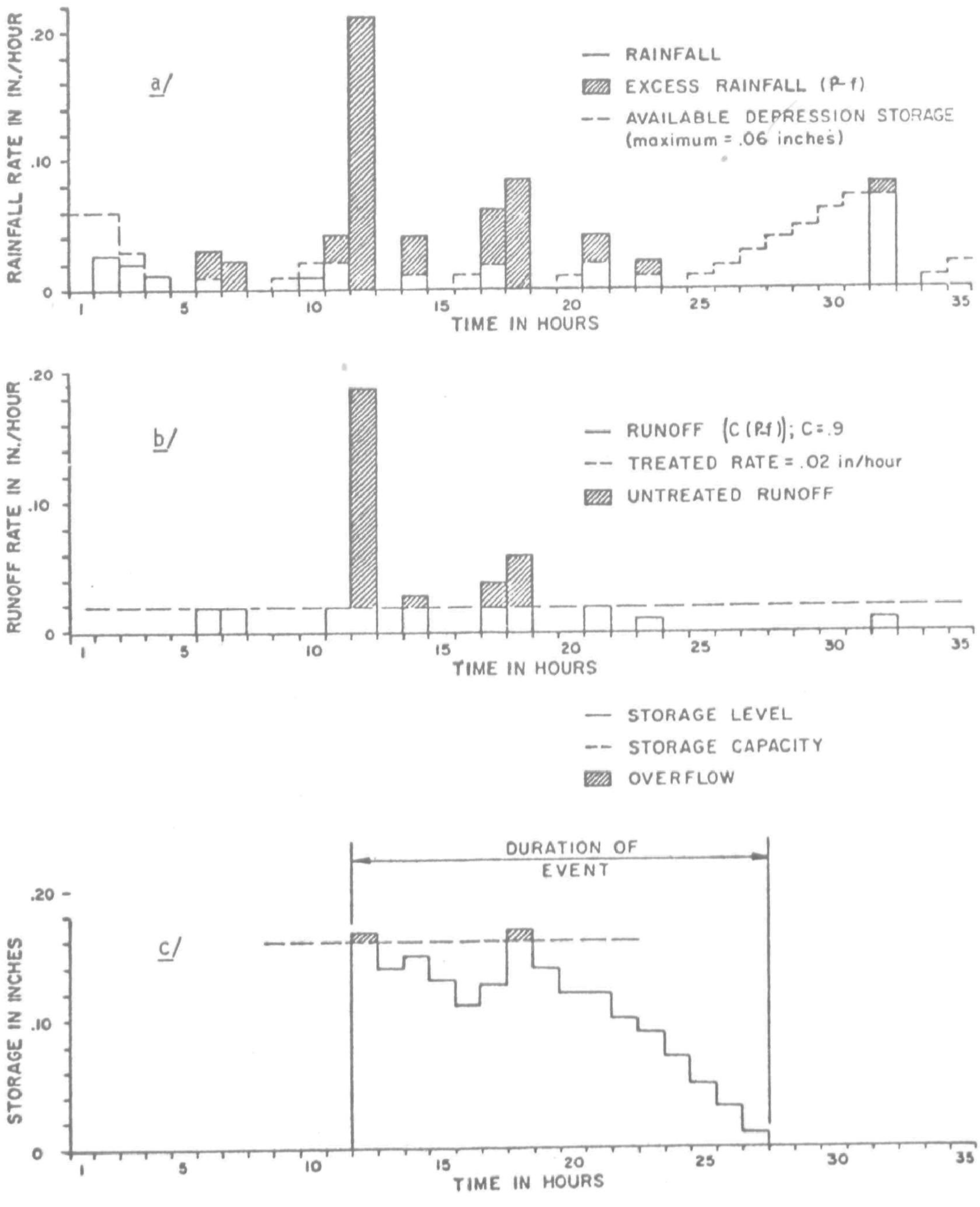


Figure 3. Time Histories of Rainfall, Runoff, and Storage.

F_p = pounds of pollutant p per pound of dust and dirt;

N_D = number of dry days since the last storm, and

P_{po} = total pounds of pollutant remaining on watershed L at the end of the last storm.

In practice, P_p is usually limited to the amount that would be accumulated in a 90-day dry period. The reason for this is that the efficacy of extrapolating daily buildup rates beyond this point (which was arbitrarily selected) is uncertain. Moreover, if equation (5) is used repetitively over long periods of time, positive errors could tend to accumulate in P_{po} resulting in overly large values of P_p.

DETERMINATION OF URBAN RUNOFF POLLUTION LOADS

To compute the amount of pollutant washed off the watershed during a storm, it is assumed that the amount of pollutant removed at any time t is proportional to the amount remaining:

$$\frac{dP_p}{dt} = -KP_p \tag{6}$$

We stated earlier that the runoff rate Q also affects rate of pollutant removal, therefore K must be functionally dependent upon Q. However, given two identical watersheds except for their area size, for the same rainfall rate r on both watersheds a higher runoff rate would occur from the larger watershed. This area effect can be eliminated by dividing the runoff Q by the impervious area of the watershed. The impervious area is used because only a negligible amount of the runoff comes from the pervious area. Since cfs per acre are equivalent to inches per hour, we can say that K is functionally dependent on the runoff rate R from the impervious area, where R is in inches per hour. Finally, assuming that K is directly proportional to R and that a uniform rainfall of ½ inch per hour would wash away 90 percent of the pollutant in one hour (a somewhat arbitrary assumption), we can say that K = 4.6R. Making this substitution into equation (6) and integrating over a time interval Δt (during which R is held constant) gives:

$$P_p\,(t + \Delta t) = P_p\,(t)e^{-4.6R\Delta t} \tag{7}$$

Equation (7) is the basic form of the overland flow quality model developed by Metcalf & Eddy, Inc., as part of the EPA Stormwater Management Model [3]. Although it is simplistic and contains many assumptions, it is the best overland flow water quality predictor or simulation model that presently exists. Moreover, experience with that model (See Reference 4 and Volume II of Reference 3) has shown it to give fairly good results. The rate of removal of mass from the watershed M_p is simply $[P(t)-P(t+\Delta t)]/\Delta t$, which can be expressed as:

$$M_p = P(t) \times (1 - e^{-4.6R\Delta t})/\Delta t \tag{8}$$

The variation of M_p with time for the associated hydrograph is plotted in the lower graph of Figure 4. A plot of M_p versus t is termed a <u>pollutograph</u>, one of the most informative methods for expressing the pollutant load carried by urban runoff. To determine the concentration of a pollutant in the runoff as a function of time, one simply divides the pollutograph value M_p by Q (with appropriate conversion factors).

Equation (8) must be modified, however, because not all of the dust and dirt on the watershed is available for inclusion in the runoff at a given time t. Thus pollutants which are tied to the dust and dirt are not all available either. The Storm Water Management Model study [3] found that for suspended solids the <u>available</u> fraction at any time was:

$$A_{sus} = 0.057 + 1.4R^{1.1} \tag{9}$$

For settleable solids it has been <u>assumed</u> that the availability factor is

$$A_{set} = 0.028 + 1.0R^{1.8} \tag{10}$$

With regard to BOD, nitrogen and phosphate, recall that the APWA data [2] described the <u>dissolved</u> fraction, which is independent of the amount of solids available for runoff. In the Storm Water Management Model study it was found that the BOD associated with the suspended solids was about 10 percent of the suspended solids load. We have further <u>assumed</u> that the BOD tied to the settleable solids is two percent of the settleable solids. For nitrogen and phosphate, we have assumed that BOD, N, and PO4 are associated with suspended and settleable solids in the same proportion as they are in the dissolved state.

Thus, correcting equation (8) for available suspended and settleable solids and adding the BOD, N and PO4 found in the solids, we get the following set of equations which are used in STORM:

Suspended Solids

$$M_{sus}(t) = A_{sus}\, P_{sus}\,(t) \times EXPT \tag{11}$$

where

$$A_{sus} = 0.057 + 1.4R^{1.1}$$

$$EXPT = (1-e^{-4.6R\Delta t})/\Delta t, \text{ with } \Delta t = 1 \text{ hour}$$

Settleable Solids

$$M_{set}(t) = A_{set}\, P_{set}\,(t) \times EXPT \tag{12}$$

where

$$A_{set} = 0.028 + R^{1.8}$$

BOD

$$M_{bod}(t) = P_{bod}(t) \times EXPT + 0.10\, A_{sus} + 0.02\, A_{set} \tag{13}$$

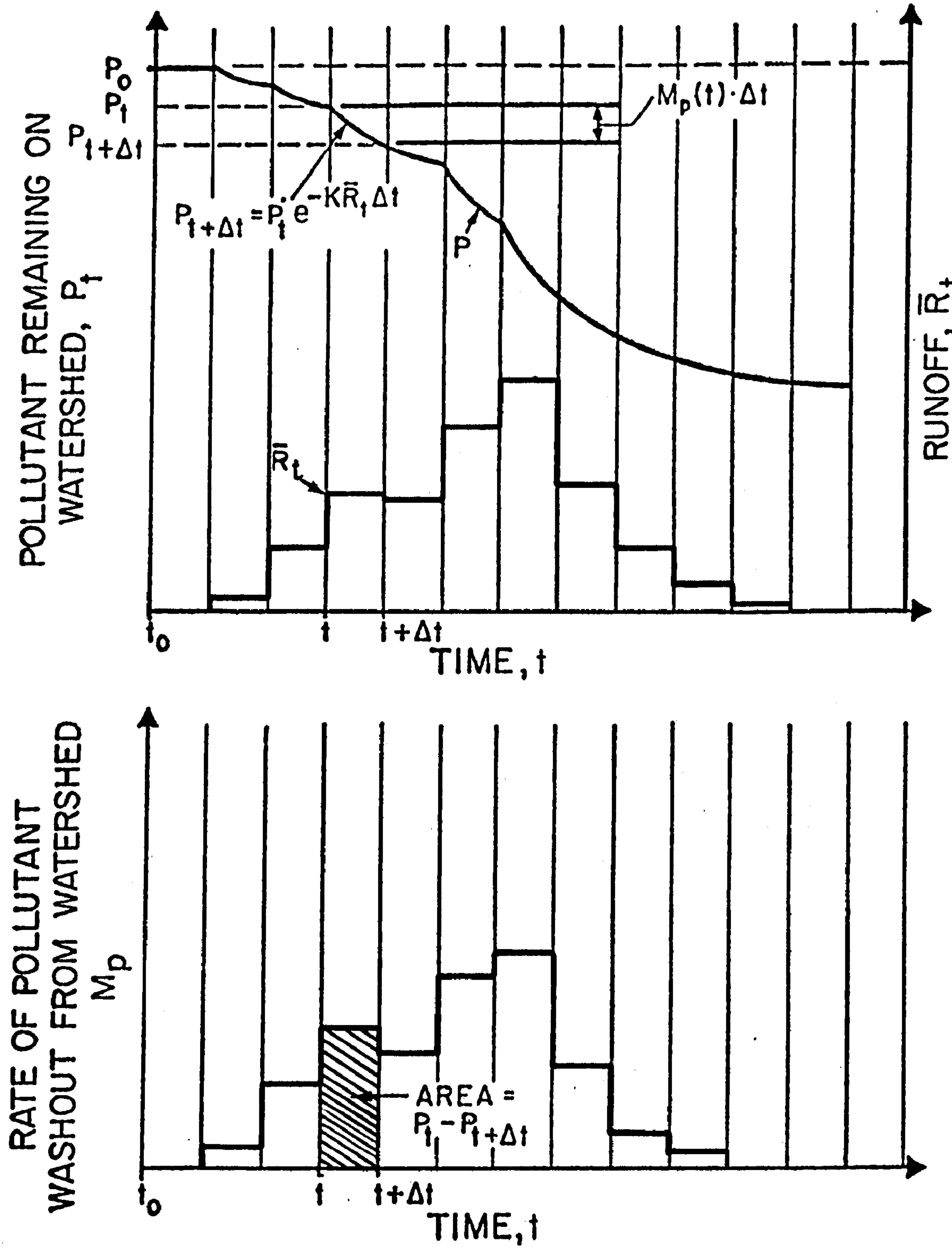


Figure 4. Development of Pollutograph (M_p vs. t) from Time History of P_t.

Nitrogen

$$M_{nit}(t) = P_{nit}(t) \times EXPT + .045\ A_{sus} + .01\ A_{set} \tag{14}$$

PO4

$$M_{PO3}(t) = P_{PO4}(t) \times EXPT + .0045\ A_{sus} + .001\ A_{set} \tag{15}$$

COMPUTATION OF TREATMENT, STORAGE AND OVERFLOW

Computations of treatment, storage and overflow proceed in an hourly step-by-step method throughout a period of rainfall/snowmelt record. For every hour in which runoff occurs the treatment facilities are utilized to treat as much runoff as possible. When the runoff rate exceeds the treatment rate, storage is utilized to contain the runoff. When runoff is less than the treatment rate, the excess treatment rate is utilized to diminish the storage level. If the storage capacity is exceeded, all excess runoff overflows into the receiving waters and does not pass through the storage facility. This overflow is lost from the system and cannot be treated later. While the storm runoff is in storage its age is increasing. Various methods of aging are used including average, first-in: last-out, first-in: first-out, or others depending on the physical conditions encountered.

The computation of storage and the interplays among rainfall/snowmelt, storage and treatment represent a simplistic approach for dividing a rainfall record into unique events such that the event is defined in terms of the urban system. For example, whether two "storms" are considered as two isolated occurrences or as one large storm is entirely dependent upon how the system will react to them. If the system has not recovered from the first when the second arrives, the two definitely will interact and hence must be considered together. "Events" are defined as beginning when storage is required and continues until the storage reservoir is emptied. All the rainfall occurring within this period is regarded as part of the same event. If precipitation produces runoff that does not exceed the treatment rate, the runoff will pass through the treatment process but will not register as an event. From the standpoint of the urban stormwater system, such precipitation is inconsequential and hence is not part of an "event" even if it should occur immediately preceding an obvious event.

The runoff coming into the storage/treatment system is given by equation (1). The quantity of system overflows are computed using:

$$Q_o = R - Q_T - Q_S \tag{16a}$$

$$Q_T = \text{minimum of } (R + Q_{s_{t-1}}, T) \tag{16b}$$

$$Q_s = \text{minimum of } (R - Q_T, S) \tag{16c}$$

where

Q_o = watershed runoff overflow, in inches;

Q_T = watershed runoff treated, in inches;

Q_s = watershed runoff stored, in inches;

$Q_{s_{t-1}}$ = watershed storage remaining in previous hour, inches;

R = watershed runoff as calculated using equation (1), inches;

T = treatment rate in watershed inches/hour; and

S = storage capacity in basin, inches.

The quality of system overflows are computed as follows for each pollutant for each hour:

$$M_{po} = M_p(Q_o/R) \tag{18}$$

$$M_{pT/s} = M_p - M_{po} \tag{19}$$

where

M_{po} = total pounds of pollutant overflowing from system;

M_p = total pounds of pollutant p coming into the system; and

$M_{pT/s}$ = total pounds of pollutant p going to storage/treatment.

The program does not model the treatment process but it does compute the quantity of water treated. It is assumed that the pollutants will be reduced to an acceptable level before the storm water is released. The age of pollutant in storage is computed as previously mentioned.

INPUT AND OUTPUT FOR "STORM"

The basic input data required by STORM and the basic output data generated by the program are illustrated in Figure 5. This section defines the input data requirements more specifically and contains details on STORM output.

INPUT DATA REQUIREMENTS

Hydrogeometric Data

The first step in setting up data for the simulation model is to define the boundaries of the basin which is to be investigated, specifically that area which drains to some specific point of interest such as a receiving water. The size of the area is a computation variable but it should be limited to less than 10 square miles so that travel time in the system can be neglected.

Once the drainage basin boundaries are set the following information is required:

1. Size of the total area of the basin
2. Percent of the total area in each of the following land use groups:

INPUT

I. LAND USE INFORMATION

Residential
Multiple Family
Commercial
Industrial
Park

A. AREA
B. % IMPERV.
C. RUNOFF COEFF.
D. FT. OF GUTTER/ACRE

II. HYDROLOGIC RECORD OF HOURLY RAINFALL

ANALYSIS

I. RUNOFF

$R = C(I - D)$

where:

$$C = \frac{\Sigma C_i A_i}{\Sigma A_i}$$

II. QUALITY

$Mp = P(1 - e^{-KR})$

A. SUSP. SOLIDS
B. SETT. SOLIDS
C. BOD
D. NITROGEN
E. PHOSPHORUS

III. STORAGE

$\Delta S = R - T$

OUTPUT

I. STATISTICS BY EVENT

A. RAINFALL
B. STORAGE
C. OVERFLOW
D. TREATMENT
E. QUALITY

II. AVERAGE STATISTICS FOR ALL EVENTS

A. EVENTS/YR.
B. AVERAGES OF A-E ABOVE FOR ALL EVENTS
C. OVERFLOWS/YR. AVERAGES OF A-E ABOVE FOR ALL OVERFLOW EVENTS

Figure 5. Input-Output Elements.

a. Single Family Residential
b. Multiple Family Residential
c. Commercial
d. Industrial
e. Open or Park

3. Average percent imperviousness of each land use group
4. Feet of gutter per acre for each land use group
5. A runoff coefficient for impervious areas (the usual range is 0.8 to 0.9)
6. A runoff coefficient for pervious areas (the usual range is 0.1 to 0.3)
7. The depression storage available on the impervious areas (usually 0.05 to 0.1 inches).

Determination of the percent of area under the various land uses can be a tedious task. However, most jurisdictions have this information already available in one form or another, or they have maps of sufficient scale that the various land uses can be identified and their areas calculated.

Determination of the average percent imperviousness of each land use group must be done carefully, for this is the most sensitive parameter affecting the amount of storm runoff which comes off a watershed. In particular, impervious surface areas which drain to pervious areas should be excluded from the impervious fraction because runoff from these surfaces will probably be held on the pervious surfaces to which they drain. In this regard, special attention should be given to whether the house gutter drains to pervious land or to the sewer system or gutter. Also, in residential areas where there are parking strips, the sidewalks will most likely drain to the pervious area on either side of the walk.

Estimation of the number of feet of gutter per acre is best done from a plat. But if none is available a reasonable estimate can be based on the average size of the block by taking the perimeter of the block times the number of blocks per acre from a street map.

Hydrologic Data

A record of hourly rainfall is required. The rainfall record may be as long or as short as desired but should be of sufficient length to assure that all storms of interest are included in the record. Ten to thirty years of record is desirable. A long raingage record exists for most cities. Where such information is lacking, however, standard hydrologic procedures for areal translation of rainfall records will have to be applied.

Quality Data

The quality data required for the simulation model consists of:

1. The daily rate of dust and dirt accumulation in pounds per 100 feet of gutter for each of the land use areas:

   a. Single Family Residential
   b. Multiple Family Residential
   c. Commercial
   d. Industrial
   e. Open or Park

2. The pounds of each of the following pollutants per 100 pounds of dust and dirt for each land use category:

   a. Suspended solids
   b. Settleable solids
   c. Soluble BOD
   d. Soluble N
   e. Soluble PO4

3. The interval in days between street sweepings for each land use category

4. Street sweeping efficiency (usual range is .6 to .9).

Because this data is difficult to obtain, default values are provided in the computer program as follows:

1. Daily Rate of Dust and Dirt Accumulation*

Land Use	Amount of D/D by Land Use, lb./day/100 ft. of Gutter
Single Family Residential	0.7
Multiple Family Residential	2.3
Commercial	3.3
Industrial	4.6
Open or Park**	1.5

2. Pounds of Pollutant in Dust and Dirt*

Land Use	Lbs. of Pollutant/100 lbs. of D/D				
	Sus. Solids**	Sett. Solids**	BOD	N	PO4
Single Family Residential	11.1	1.1	0.5	0.048	0.005
Multiple Family Residential	8.0	0.8	0.36	0.061	0.005
Commercial	17.0	1.7	0.77	0.041	0.007
Industrial**	6.7	0.7	0.3	0.043	0.003
Open or Park**	11.1	1.1	0.5	0.048	0.005

3. Street Sweeping Interval 90 days

4. Street Sweeping Efficiency 0.7

*Data is taken from APWA Chicago Study (See Reference 2).
**Estimated values from other sources.

OUTPUT FROM "STORM"

The computer program produces four output reports:

1. Quantity Analysis,
2. Quality Analysis,
3. Pollutograph Analysis, and
4. Land Surface Erosion Analysis.

For the quantity and quality analyses, STORM generates statistics by event plus the average statistics for all events. A complete list of the output statistics from the quantity and quality analyses are contained in Table 1.

Tables 2 through 7 are examples of STORM output from the Quantity Analysis, Quality Analysis and Pollutograph Analysis. For details on the output from the Land Surface Erosion Analysis the reader is referred to the program manual [1].

PLANNING APPLICATIONS

City of San Francisco

The City of San Francisco funded the original development of the Quantity Analysis portion of STORM. The purpose of the program was to create a tool that would enable the City to evaluate the effectiveness of various combinations of treatment rate and storage capacity with respect to their ability to reduce combined sewer system overflows. Results from STORM were used as a guide in the initial sizing of facilities for the City's Master Plan [5]. The following paragraphs are abstracted from the Master Plan Report. Additional details are described in References 12 and 13.

The rainfall record used for the analysis was a 62 year U.S. Weather Service record of hourly values of rainfall measured at the Federal Office Building in the City. A runoff coefficient of 0.65 was assumed for the analysis. Table 7 shows the various combinations of treatment rate and storage capacity that were examined, the resulting events per year, overflows per year and the average quantity of overflow per year. These data are displayed graphically in Figures 6 and 7, which show the relationship between given combinations of storage and treatment with overflow frequencies and with overflow volumes, respectively.

Application of STORM to the Federal Office Building record with 0 storage and a treatment rate of 0.02 inches per hour provided the baseline or existing condition data. From this computation it was determined that approximately one-third of the runoff is presently treated and discharged by the three water pollution control plants and that the other two-thirds, or about 6.0 billion gallons of runoff per year, overflows without treatment. This volume of overflow occurs during an aggregate average of 206 hours per year. On the average, there are 46 days in the year during which 82 overflows occur.

The storage needed to contain all overflows from the greatest recorded storm utilizing the existing treatment rates would be 240 million cubic feet. This storage volume is then the upper limit of an all-storage

TABLE 1
"STORM" Output

I. STATISTICS BY EVENTS

A. RAINFALL

1. DURATION OF RAINFALL EVENT
2. HOURS OF RAIN
3. TOTAL RAINFALL

B. STORAGE

1. TIME SINCE LAST EVENT
2. DURATION OF STORAGE
3. TIME TO EMPTY
4. MAXIMUM STORAGE USED

C. OVERFLOW

1. TIME OVERFLOW STARTS
2. DURATION OF OVERFLOW
3. QUANTITY OF OVERFLOW
4. OVERFLOW IN FIRST THREE HOURS

D. TREATMENT

1. DURATION OF TREATMENT
2. QUANTITY TREATED

E. QUALITY (susp. solids, sett. solids, BOD, nitrogen, phosphorous)

1. MASS EMISSION IN RUNOFF
2. MASS EMISSION OF OVERFLOW
3. MASS EMISSION DURING FIRST THREE HOURS OF OVERFLOW

II. AVERAGE STATISTICS (A-E ABOVE)

A. FOR ALL EVENTS
B. FOR ALL OVERFLOW EVENTS
C. EVENTS / YR
D. OVERFLOWS / YR

TABLE 2

Example of Quantity Statistics by Event

PAGE 1 SELBY STREET WATERSHED SAN FRANCISCO, CA.

QUANTITY ANALYSIS

TREATMENT RATE = .0100 IN/HR, 37.8 CFS, 24.400 MGD FED BLDG, SAN FRANCISCO, CA.

STORAGE CAPACITY= .1032 INCHES, 31.2 AC-FT, 10.168 MG SELBY STREET

EVENT	---D A T E---				HRS NO	-R A I N F A L L-			RUNOFF	HRS TO	--STORAGE--		----O V E R F L O W----					---TREATMENT----		--AGE OF STORAGE---				
	YEAR	MO	DY	HR	STORAG	DURTN	HRS	QUANTY	INCHES	EMPTY	DURTN	MAX	NO	ST	DUR	WASTE	INITL	HRS	QANTY	AGE1	AGE2	AGE3	AGE4	AGE5
****1	*********			*****2 *3	*****4	****5	***6	*****7	****7A	*****8	****9	***10	*11	*12	*13	***14	***15	***16	***17	**18	**19	**20	**21	**22
1	64	10	28	16	3422	33	17	1.73	.84	4	37	.10	1	3	9	.47	.07	37	.37	4.9	11.5	35.5	5.4	8.4
2	64	11	1	2	45	3	2	.16	.08	6	9	.05		3	NO OVERFLOW			9	.08	2.1	5.5	7.5	1.8	1.8
3	64	11	2	3	16	6	6	.49	.23	10	16	.10	2	2	5	.06	.06	16	.16	6.1	13.5	14.5	5.2	5.3
4	64	11	8	11	136	26	16	.88	.42	8	34	.10	3	2	5	.08	.05	34	.33	5.0	13.5	32.5	6.5	10.3
5	64	11	9	24	3	20	10	.97	.46	4	24	.10	4	2	4	.22	.20	24	.23	4.3	10.1	22.1	5.6	7.4
6	64	11	11	8	8	1	1	.03	.02	1	2	.00		1	NO OVERFLOW			8	.04	.5	.5	.5	.5	.5
7	64	11	11	12	2	11	6	.36	.10	7	18	.06		3	NO OVERFLOW			19	.18	3.1	7.5	16.5	3.1	4.3
8	64	11	12	7	1	3	3	.11	.04	2	5	.01		2	NO OVERFLOW			5	.04	1.3	3.5	3.5	1.6	1.6
9	64	11	12	18	198	5	3	.14	.07	2	7	.03		1	NO OVERFLOW			8	.07	1.5	3.5	5.5	1.9	2.4
10	64	11	21	20	19	5	4	.24	.10	6	11	.06		3	NO OVERFLOW			11	.10	3.4	7.5	8.5	2.8	3.0

DEFINITIONS OF QUANTITY COLUMN HEADINGS

----

1 EVENT = SEQUENCING NUMBER.
2 DATE = DATE THIS EVENT BEGAN.
3 HR = NUMBER OF HOURS PAST MIDNIGHT THIS EVENT BEGAN.
4 HRS NO STORAG = NUMBER OF HOURS SINCE END OF LAST EVENT, EXCLUDING SUMMER (MORE THAN, 960 HOURS).

5 DURTN = DURATION OF STORM FROM FIRST HOUR OF RAIN TO LAST HOUR OF RAIN.
6 HRS = NUMBER OF HOURS IN WHICH RAINFALL OCCURRED DURING EVENT.
7 QUANTY = AMOUNT OF RAINFALL DURING THE EVENT IN INCHES.
7A RUNOFF INCHES = RUNOFF DURING EVENT IN INCHES.
8 HRS TO EMPTY = NUMBER OF HOURS FROM LAST RAINFALL TO END OF EVENT.
9 DURTN = TOTAL NUMBER OF HOURS STORAGE WAS UTILIZED. IE, LENGTH OF THE EVENT.
10 MAX = MAXIMUM AMOUNT OF STORAGE UTILIZED, IN INCHES.

11 NO = OVERFLOW EVENT SEQUENCING NUMBER.
12 ST = NUMBER OF HOURS ELAPSED BEFORE OVERFLOW STARTED. OR, IF NO OVERFLOW, HOUR OF MAXIMUM STORAGE.
13 DUR = NUMBER OF HOURS IN WHICH OVERFLOW OCCURED.
14 WASTE = QUANTITY OF WATER RELEASED UNTREATED, IN INCHES.
15 INITL = QUANTITY OF WATER RELEASED UNTREATED DURING THE FIRST 3 HOURS OF OVERFLOW.

16 HRS = NUMBER OF HOURS WATER WAS TREATED DURING THE PRESENT EVENT AND SINCE THE PREVIOUS EVENT.
17 QANTY = QUANTITY OF WATER TREATED DURING THE EVENT AND SINCE THE PREVIOUS EVENT.
18 AGE1 = AVERAGE AGE (HOURS) OF TREATED RUNOFF.
19 AGE2 = MAXIMUM AGE (HOURS) OF STORAGE ON FIRST IN, FIRST OUT BASIS.
20 AGE3 = MAXIMUM AGE (HOURS) OF STORAGE ON FIRST IN, LAST OUT BASIS.
21 AGE4 = QUANTITY WEIGHTED AVERAGE AGE (HRS) OF STORAGE ON FIRST IN, FIRST OUT BASIS.
22 AGE5 = QUANTITY WEIGHTED AVERAGE AGE (HRS) OG STORAGE ON FIRST IN, LAST OUT BASIS.

TABLE 3
Example of Average Quantity Statistics for all Events

```
PAGE    1                             SELBY STREET WATERSHED   SAN FRANCISCO, CA.
                                                         QUANTITY ANALYSIS
TREATMENT RATE  =  .0100 IN/HR,     37.8 CFS,     24.400 MGD                                         FED BLDG, SAN FRANCISCO, CA,
STORAGE CAPACITY=  .1000 INCHES,    31.2 AC-FT,   10.168 MG                                             SELBY STREET

EVENT ---D A T E--- HRS NO -R A I N F A L L- RUNOFF HRS TO --STORAGE-- ----O V E R F L O W---- ---TREATMENT---- --AGE OF STORAGE---
      YEAR MO DY HR STORAG DURTN  HRS QUANTY INCHES  EMPTY DURTN   MAX  NO  ST DUR WASTE INITL  HRS  QANTY AGE1 AGE2 AGE3 AGE4 AGE5
*****1 **********2 *3 *****4 ****5 ***6 *****7 ****7A *****8 ****9 ***10 *11 *12 *13 ***14 ***15 ***16 ***17 **18 **19 **20 **21 **22


AVE OF  168 EVENTS   101.7**10.9  7.6     .47     .22     3.8  14.8   .06     2.7*                    15.5    .15  3.1  7.7 12.8  3.5  4.3
AVE OF   69 OVRFLW EVENTS   19.8 13.1     .92     .45     5.7  25.5   .03*    5.3 5.4    .20    .10  26.0    .25  5.2 13.4 23.5  6.1  7.7

* NON-OVERFLOW EVENTS ONLY.
**EXCLUDING 10 DRY PERIODS


          AVERAGE ANNUAL STATISTICS FOR  5 YEARS OF RECORD FOR THE PERIOD BEGINNING 641027 AND ENDING 681228
          -----------------------------------------------------------------------------------------------


          NUMBER OF EVENTS =      33.6

          NUMBER OF OVERFLOWS =  13.8

                                     INCHES
                                     ------

          TOTAL PRECIPITATION ON WATERSHED   17.95

          TOTAL RUNOFF FROM WATERSHED         7.60     FRACTION OF RAINFALL = .42

          OVERFLOW TO RECEIVING WATER         2.70     FRACTION OF RAINFALL = .15,  OF RUNOFF = .36

          INITIAL OVERFLOW TO RECIEVING WATER 1.42     FRACTION OF RAINFALL = .08,  OF RUNOFF = .19
```

TABLE 4
Example of Quality Statistics by Events

PAGE 1 SELBY STREET WATERSHED SAN FRANCISCO, CA.
QUALITY ANALYSIS

TREATMENT RATE = .0100 IN/HR, 37.8 CFS, 24.400 MGD
STORAGE CAPACITY= .1000 INCHES, 31.2 AC-FT, 10.168 MG

FED BLDG, SAN FRANCISCO, CA.
SELBY STREET

EVENT	DATE EVENT BEGAN YR	MO	DY	HR	RAIN FALL INCHS	STORM RUNOFF INCHS QANTY	STORM RUNOFF TOTAL POUNDS SUSP	SETL	BOD	N	PO4	STORAGE OVERFLOW SEQ NO	INCHS QANTY	TOTAL POUNDS SUSP	SETL	BOD	N	PO4	FIRST 3 HOURS OVERFLOW INCH Q	TOTAL POUNDS SUSP	SETL	BOD	N	PO4
****1	*******2			*3	****4	****5	*****6	****7	****8	****9	**10	*11	***12	****13	***14	***15	***16	**17	**18	***19	**20	**21	**22	**23
1	64	10	28	16	1.73	.84	58378	2150	8134	2849	284	1	.47	27072	1291	3462	1440	144	.07	4248	129	922	264	26
2	64	11	1	2	.16	.08	1741	112	267	97	10		NO OVERFLOW											
3	64	11	2	3	.49	.23	5491	402	702	293	29	2	.06	1317	100	165	70	7	.06	1260	94	157	67	7
4	64	11	8	11	.88	.42	9780	669	1513	548	55	3	.08	1739	125	230	94	9	.05	1073	75	154	59	6
5	64	11	9	24	.97	.46	9153	888	1007	474	47	4	.22	4598	458	490	237	24	.20	4362	429	465	225	22
6	64	11	11	8	.03	.02	104	16	15	6	1		NO OVERFLOW											
7	64	11	11	12	.36	.18	1666	204	206	89	9		NO OVERFLOW											
8	64	11	12	7	.11	.04	310	45	46	17	2		NO OVERFLOW											
9	64	11	12	10	.14	.07	557	73	81	31	3		NO OVERFLOW											
10	64	11	21	20	.24	.10	2360	155	663	162	16		NO OVERFLOW											

TITLE DESCRIPTIONS

1. EVENT = Sequence number
2. DATE = Date this event began
3. HR = Number of hours past midnight this event began
4. RAINFALL = Total quantity of rainfall for this event in inches
5. QANTY = Total quantity of runoff for this event in inches
6. SUSP = Suspended solids
7. SETL = Settleable solids
8. BOD = BOD
9. N = Total nitrogen
10. PO4 = Orthophosphate

(6–10: Total lbs in runoff for this event)

11. NO = Overflow event sequencing number
12. QANTY = Total quantity of overflow for this event in inches
13. SUS =
14. SET =
15. BOD =
16. NIT =
17. PO4 =

(13–17: Total lbs. in overflow for this event)

18. Q = Quantity of overflow during first three hours in inches
19. SUSP =
20. SETL =
21. BOD =
22. N =
23. PO4 =

(19–23: Total lbs. in overflow during first three hours)

TABLE 5
Example of Average Quality Statistics for all Events

PAGE 1

SELBY STREET WATERSHED SAN FRANCISCO, CA.
QUALITY ANALYSIS

TREATMENT RATE = .0100 IN/HR, 37.8 CFS, 24.400 MGD
STORAGE CAPACITY= .1000 INCHES, 31.2 AC-FT, 10.168 MG

FED BLDG, SAN FRANCISCO, CA.
SELBY STREET

EVENT	DATE EVENT BEGAN YR MO DY HR	RAIN FALL INCHS	STORM RUNOFF INCHS QANTY	STORM RUNOFF TOTAL POUNDS SUSP	SETL	BOD	N	PO4	STORAGE SEQ NO	OVERFLOW INCHS QANTY	OVERFLOW TOTAL POUNDS SUSP	SETL	BOD	N	PO4	FIRST 3 HOURS OVERFLOW INCH	TOTAL POUNDS SUSP	SETL	BOD	N	PO4
****1	*******2 *3	****4	****5	*****6	****7	****8	****9	**10	*11	***12	****13	***14	***15	***16	**17	**18	***19	**20	**21	**22	**23
AVE OF	168 EVNTS	.47	.22	9110	688	1408	511	51													
AVE OF	69 OVRFLS	.92	.45	18692	1443	2681	1028	103		.20	8663	732	1058	458	46	.10	5950	461	759	318	32

AVERAGE ANNUAL STATISTICS FOR 5 YEARS OF RECORD FOR THE PERIOD BEGINNING 641027 AND ENDING 681228

	SUSP	SETL	BOD	N	PO4
TOTAL POUNDS WASHOFF FROM WATERSHED	308683	23362	48134	17365	1723
TOTAL POUNDS OVERFLOW TO RECEIVING WATER	119551	10098	14595	6325	632
CONCENTRATION OF POLLUTANTS IN OVERFLOW TO RECEIVING WATER (MG/L)	62.12	4.40	6.36	2.76	.28
FRACTION OF TOTAL WASHOFF OVERFLOWING TO RECEIVING WATER	.39	.43	.30	.36	.37
FRACTION OF TOTAL WASHOFF INITIALLY OVERFLOWING TO RECEIVING WATER	.27	.27	.22	.25	.25

TABLE 6
Example of Hourly Quality Statistics

PAGE 1 SELBY STREET WATERSHED SAN FRANCISCO, CA.
POLLUTOGRAPH FOR EVENT

TREATMENT RATE = .0100 IN/HR, 37.8 CFS, 24.400 MGD
STORAGE CAPACITY= .1000 INCHES, 31.2 AC-FT, 10.168 MG

FED BLDG, SAN FRANCISCO, CA.
SELBY STREET

EVENT	YR	MO	DY	HR	T(0)	RAINFALL	RUNOFF	CFS-OFF	**RUNOFF POLLUTANT LOAD, IN LBS/HR** SUSP	SETL	BOD	N	PO4	** AVE CONCENTRATION, IN MG/L ****** SUSP	SETL	BOD	N	PO4
153	68	11	2	6	1	.08	.04	153.8	3592.5	106.7	861.0	231.3	23.0	103.9	3.1	24.9	6.69	.67
153	68	11	2	7	2	.13	.07	290.0	7177.1	207.8	1251.5	414.5	41.3	127.8	3.7	22.3	7.38	.74
153	68	11	2	8	3	.08	.04	153.8	3184.2	103.4	529.9	181.4	18.1	92.1	3.0	15.3	5.25	.52
153	68	11	2	9	4	.05	.03	96.1	1559.1	58.6	256.8	86.6	8.8	72.2	2.7	11.9	4.10	.41
153	68	11	2	10	5	.01	.01	19.2	208.4	11.2	38.7	12.3	1.2	48.2	2.6	8.9	2.85	.28
153	68	11	2	17	12	.04	.01	43.3	538.7	25.3	93.2	31.1	3.1	55.4	2.6	9.6	3.20	.32
153	68	11	2	18	13	.02	.01	38.5	463.4	22.4	78.4	26.6	2.7	5[illegible].6	2.6	9.1	3.08	.31
153	68	11	2	20	15	.04	.02	71.3	1009.1	42.2	154.4	56.2	5.6	63.0	2.6	9.6	3.51	.35
153	68	11	2	21	16	.01	.01	19.2	204.0	11.2	33.5	11.6	1.2	47.2	2.6	7.7	2.69	.27
153	68	11	2	22	17	.02	.01	38.5	454.0	22.3	70.0	25.4	2.5	52.5	2.6	8.1	2.94	.29
153	68	11	2	23	18	.01	.01	19.2	202.3	11.1	31.8	11.4	1.1	46.8	2.6	7.4	2.63	.26
153	68	11	2	24	19	.02	.01	38.5	450.1	22.2	66.8	24.9	2.5	52.1	2.6	7.7	2.88	.29
153	68	11	3	1	20	.04	.02	76.9	1083.8	45.2	147.0	58.5	5.8	62.7	2.6	8.5	3.38	.34
153	68	11	3	2	21	.08	.04	153.8	2849.6	100.1	346.0	149.5	14.9	82.4	2.9	10.0	4.32	.43
153	68	11	3	3	22	.08	.04	153.8	2781.5	99.1	318.6	142.4	14.2	79.3	2.9	9.2	4.12	.41
153	68	11	3	4	23	.03	.02	57.7	677.1	32.6	81.0	55.5	3.5	52.2	2.5	6.2	2.78	.27
153	68	11	3	5	24	.07	.04	134.6	2147.3	82.7	280.4	110.6	11.1	71.0	2.7	7.9	3.46	.37

NOTE:

T(0) = Hours since start of storm
RAINFALL = Inches of rainfall during indicated hour
RUNOFF = Inches of runoff during indicated hour
CFS-OFF = Average runoff rate in cfs during indicated hour

TABLE 7
San Francisco Hyetograph Storage/Treatment Analysis

Treatment[1]	Storage Capacity[2]	USWB 62 Yr. Rec. Event/yr.	Ovrflw/yr.	Quan/yr.
.02	.50	38.806	7.710	4.254
	1.00	36.661	2.823	1.907
	1.50	35.726	1.242	.926
	2.00	35.435	.597	.504
	2.50	35.194	.387	.265
	3.00	35.113	.242	.107
	3.50	35.048	.097	.033
	4.00	35.048	.032	.008
.04	.25	42.355	11.726	4.115
	.50	40.226	5.435	2.101
	.75	39.435	2.823	1.125
	1.00	39.113	1.419	.624
	1.50	38.903	.403	.231
	2.00	38.871	.177	.107
	2.50	38.790	.113	.031
	3.00	38.790	.016	.002
.06	.25	37.774	9.323	2.611
	.50	36.855	3.645	1.134
	.75	36.468	1.500	.524
	1.00	36.339	.661	.270
	1.50	36.290	.177	.095
	2.00	36.274	.113	.012
.08	.10	33.016	15.242	3.247
	.25	31.952	7.194	1.694
	.50	31.419	2.371	.633
	.75	31.274	.903	.264
	1.00	31.194	.371	.121
	1.50	31.145	.113	.025
	2.00	31.145	.016	.001
.10	.10	27.484	12.484	2.363
	.25	26.855	5.403	1.121
	.50	26.484	1.597	.369
	.75	26.435	.581	.128
	1.00	26.403	.177	.050
	1.50	26.387	.032	.007

[1]Treatment Rate in inches per hour.
[2]Storage Capacity in inches.

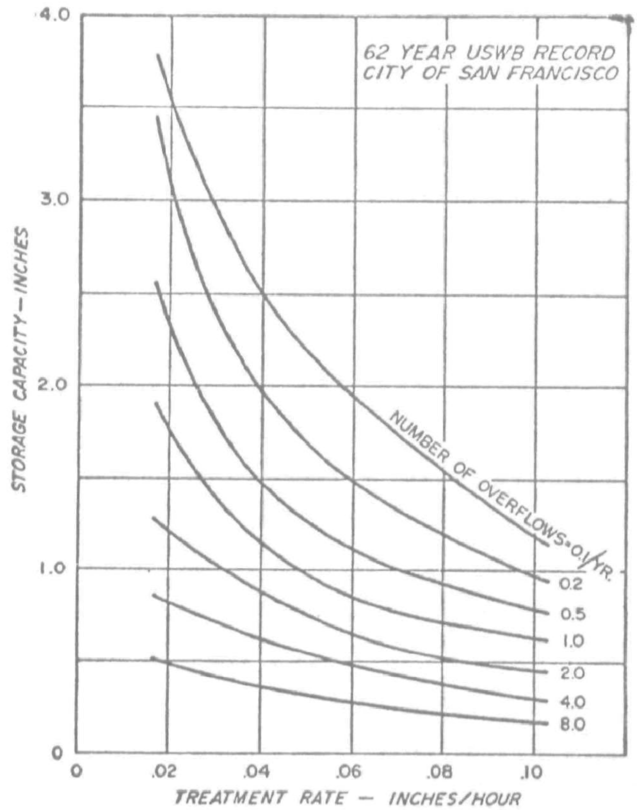


Figure 6. Overflow Frequencies.

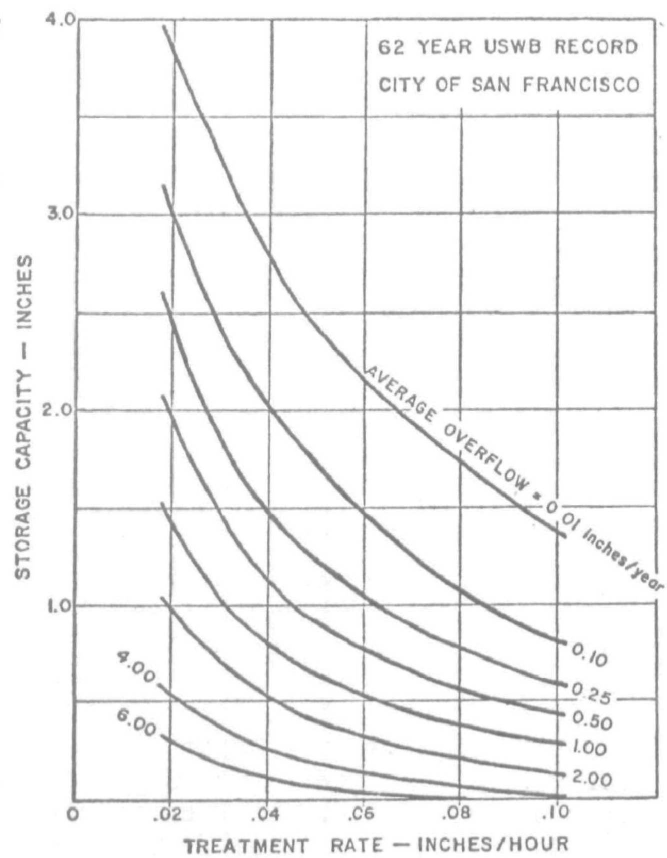


Figure 7. Overflow Volumes.

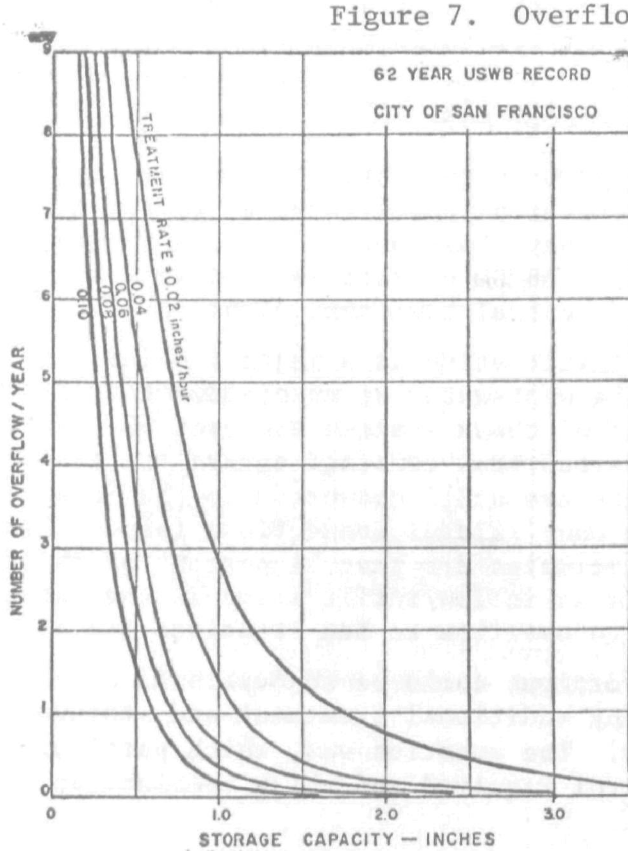


FIGURE 8. OVERFLOW OCCURRENCE RELATIONSHIPS.

scheme and exceeds by a factor of 2 the volume requirement of an all-treatment scheme.

The data presented above was necessarily based on two assumptions: that the runoff loss is 35 percent and that rainfall occurrence is uniform over the City. Each is a significant parameter in determining the total volumes of runoff. At the time the Master Plan was being developed no verified data existed on the losses experienced in the rainfall-runoff process, although some measurements have been made in more recent characterization studies.

The initial sizing of the Master Plan system was based upon the records available from the Weather Service gage. This data represented the best information available at the time and all other data indicated that any design based upon this gage would likely to conservative with regard to size and costs. Refinement of the design will take place as data accumulates from the City's extensive field information collection system.

Figure 8 shows a composite of the effects of various combinations of storage and treatment with regard to the frequency of uncontrolled overflow occurrences. It is apparent that, given a desired frequency of overflow occurrence, increasing the treatment rate decreases the storage requirements and that, for any given storage volume, increasing treatment capacity results in a lower occurrence frequency. Further inspection of the figure also indicates that the law of diminishing returns results in increasingly greater storage requirements for any treatment rate to attain a lower frequency of overflow occurrence. Through the application of cost factors for storage and treatment facilities, optimum design points for minimum cost for various levels of control can be derived.

East Bay Municipal Utility District

The East Bay Municipal Utility District, Special District No. 1 (EBMUD, SDI), provides wastewater disposal services to seven cities on the east side of San Francisco Bay, California. Figure 9 shows the location of the service area. The EBMUD systems is comprised of a 22 mile interceptor system plus a central treatment plant.

Wet weather inflow/infiltration is a major problem for EBMUD. The source of the problem is the wastewater sewer systems that drain into the interceptor system. Most of these systems are over 30 years old. A number of them have been converted from combined sewers to "sanitary" sewers, and a few combined sewers are still connected to the interceptor system. In addition, there are many illegal connections (especially to roof and yard drains). Present estimates are that 11 percent of the rainfall on the service area appears as inflow/infiltration in the interceptor system causing the system to overflow to San Francisco Bay about 11 times a year.

Reduction of the overflows could be accomplished by reducing extraneous inflows, by providing additional treatment and storage, or through some combination of both. The question was, which particular combination would give the control required and which combination would be the most cost-effective.

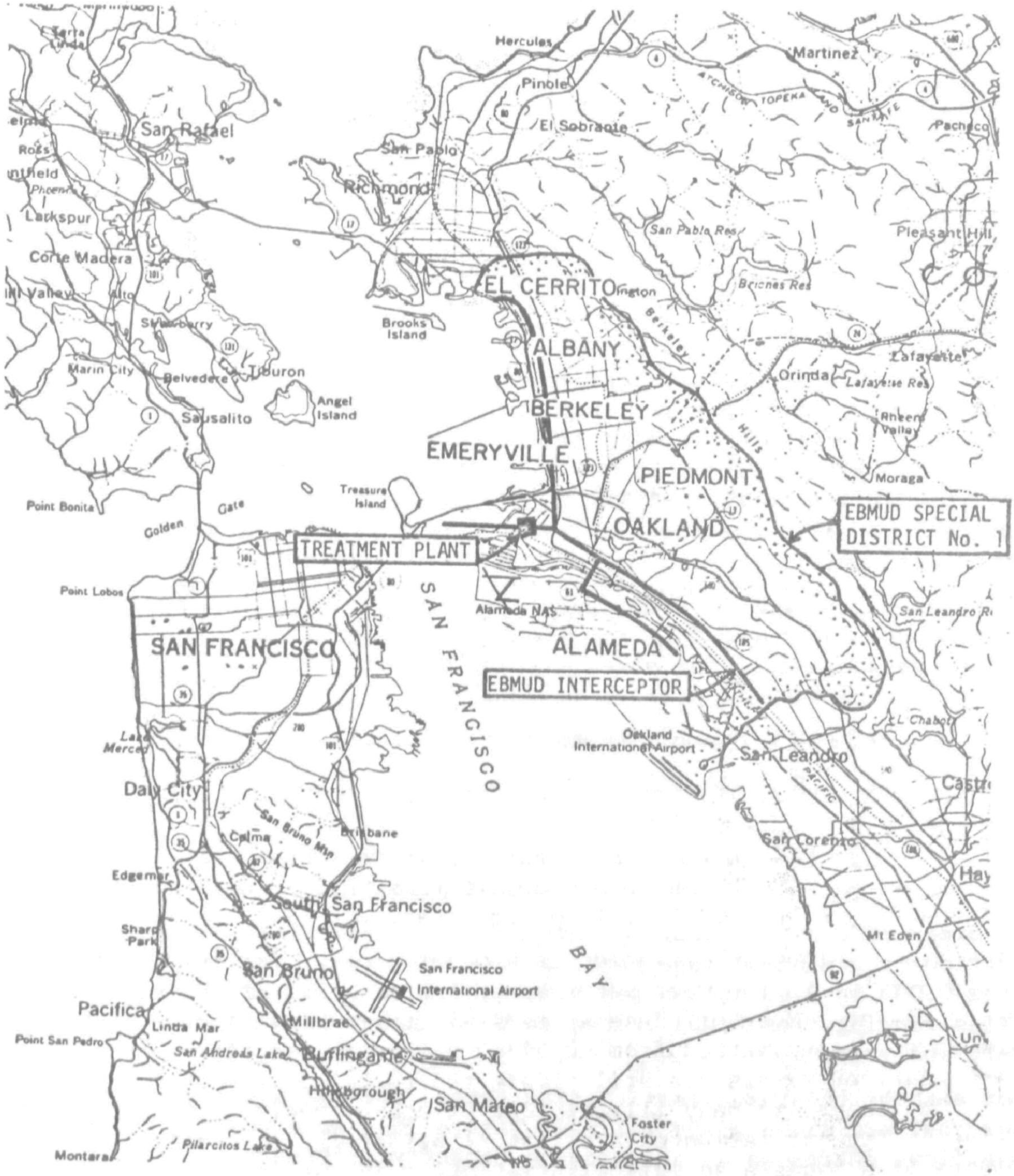


Figure 9. Vicinity Map of EBMUD Service Area.

It was estimated by the District staff that conversion of the remaining combined sewers to sanitary sewers would reduce the gross inflow/infiltration ratio (i.e., runoff coefficient) from 11 percent to 8 percent. If 80 percent of the direct connections to the sewer system could be eliminated (roof and yard drains, parking lots, catch basins connected in error, etc.) it was estimated that the inflow/infiltration ratio could be reduced another 3 percent, i.e., from 8 percent to 5 percent. Finally, by the additional removal of 50 percent of the percolation infiltration, it was estimated that the inflow/infiltration ratio could be reduced another 2 percent, i.e., from 5 percent to 3 percent.

For purposes of determining what combinations of treatment rate and storage capacity would meet the system requirements, the STORM Quantity Analysis portion was used to process 22½ years of hourly rainfall data recorded at the U. S. Weather Service station at Oakland International Airport.

An initial run was made with the STORM model set at existing Special District No. 1 treatment and storage capacities, assumed to be 0.0068 inches/hour and 0.017 inches, respectively, in excess of average dry weather flow requirements. For this run, an existing District-wide gross infiltration rate of 11.1 percent of rain, developed in an earlier study [6], was used. This infiltration rate includes the effect of combined sewers which drain approximately 4 percent of the total area. The run showed an average incidence of 10.9 overflows per year, which is in good agreement with historical data.

All subsequent runs of STORM were based on the assumption that all combined sewers were separated from the sanitary system. Treatment-storage combinations were examined for three alternative infiltration rates (expressed in percent of Oakland Airport rainfall):

1. 8% - Gross Infiltration Ratio without combined sewers
2. 5% - Removal of combined sewers plus 80% of "direct connections"
3. 3% - Additional reduction by removal of 50% of "percolation infiltration".

In order to provide enough points for curve plotting, a total of 49 infiltration-treatment-storage combinations were run. Treatment rates ranging from 0.003 to 0.03 inches per hour (107.5 to 1075 MGD) and storage capacities ranging from 0.001 inches to 0.08 inches (1.5 to 120 million gallons) were analyzed.

For each of the three infiltration rates, the average number of overflows per year are shown graphically in Figure 10. As expected, the number of overflows decreases as the storage capacity or treatment rate is increased or as infiltration is reduced by upstream extraneous control measures, as represented by the reduction in infiltration rates. Storage required to totally contain the inflow from the 22 year period of rainfall record is shown in Figure 11.

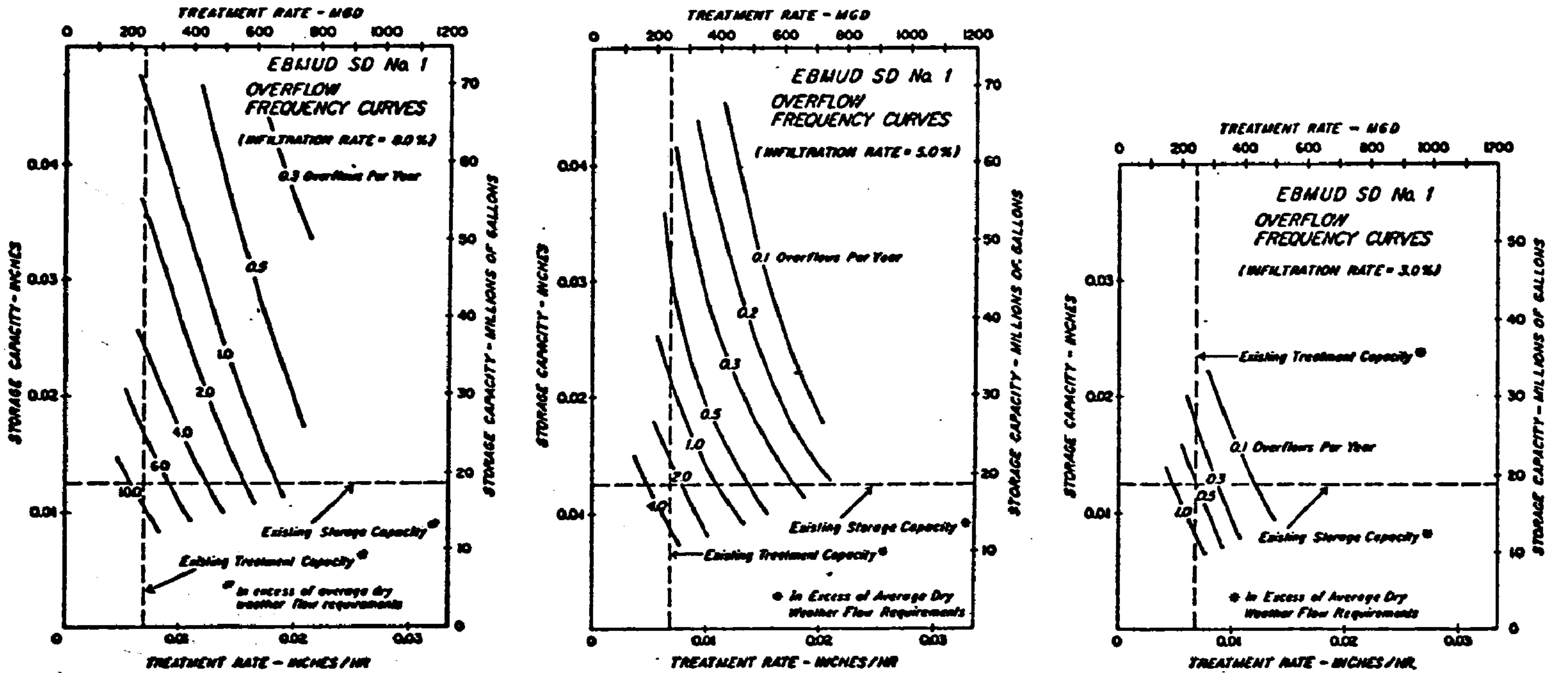


Figure 10. Overflows at Various Infiltration Rates.

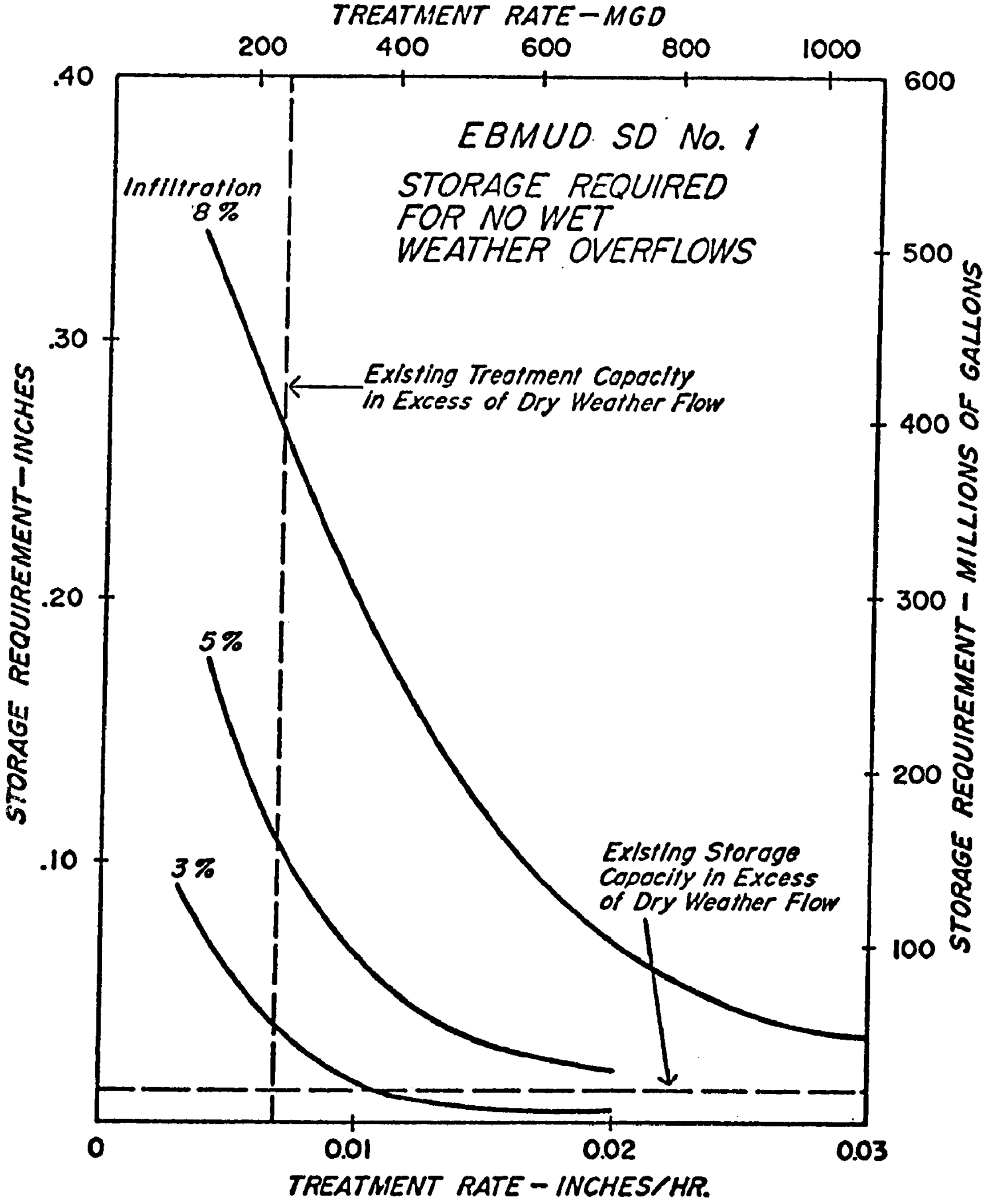


Figure 11. Zero Overflows Requirements.

APPLICATION OF "STORM" - EXAMPLES

The purpose of this section is to present some examples of other possible applictions of STORM in addition to those already described. Four applications will be shown:

1. Computation of the quantity of storm runoff by month and for single storm events.
2. Computation of pollutographs for single storm events.
3. Use of STORM to find the most economical treatment-storage combinations that meet system overflow constraints.
4. Analysis of changes in the quantity and quality of urban runoff due to alternative land use management schemes.

A prototype area was selected for these applications of STORM, i.e., the Castro Valley watershed near Oakland, California. Figure 12 shows the USGS map of the area with the watershed boundary and the location of the watershed relative to the San Francisco Bay area. Some land use information can be obtained from the map but an aerial photograph plus ground reconnaissance were necessary to obtain a better understanding of land uses in the basin. Table 8 gives a summary of the estimated hydrogeometric characteristics of the watershed. Runoff statistics for the watershed were generated by processing hourly rainfall data for the 17-month period from November 1971 through March 1973.

COMPUTATION OF THE QUANTITY OF STORM RUNOFF

In the initial application of STORM to Castro Valley, the program was calibrated to give the best comparison between computed and observed values of:

1. Average annual precipitation;
2. Average annual runoff;
3. Monthly runoff volumes; and
4. Individual storm event volumes.

It is not appropriate to make comparisons with the instantaneous measurements of discharge because the program computes runoff as hourly volumes only. The hourly volumes should reflect the general shape of the observed hydrograph and its volume, although the observed hydrograph will tend to lag behind the computed hydrograph in larger basins where the time of concentration is greater than one hour. The program can be applied to larger basins where this lag problem exists as long as one realizes the impact on the analysis. In storage analyses, however, this problem is generally not critical.

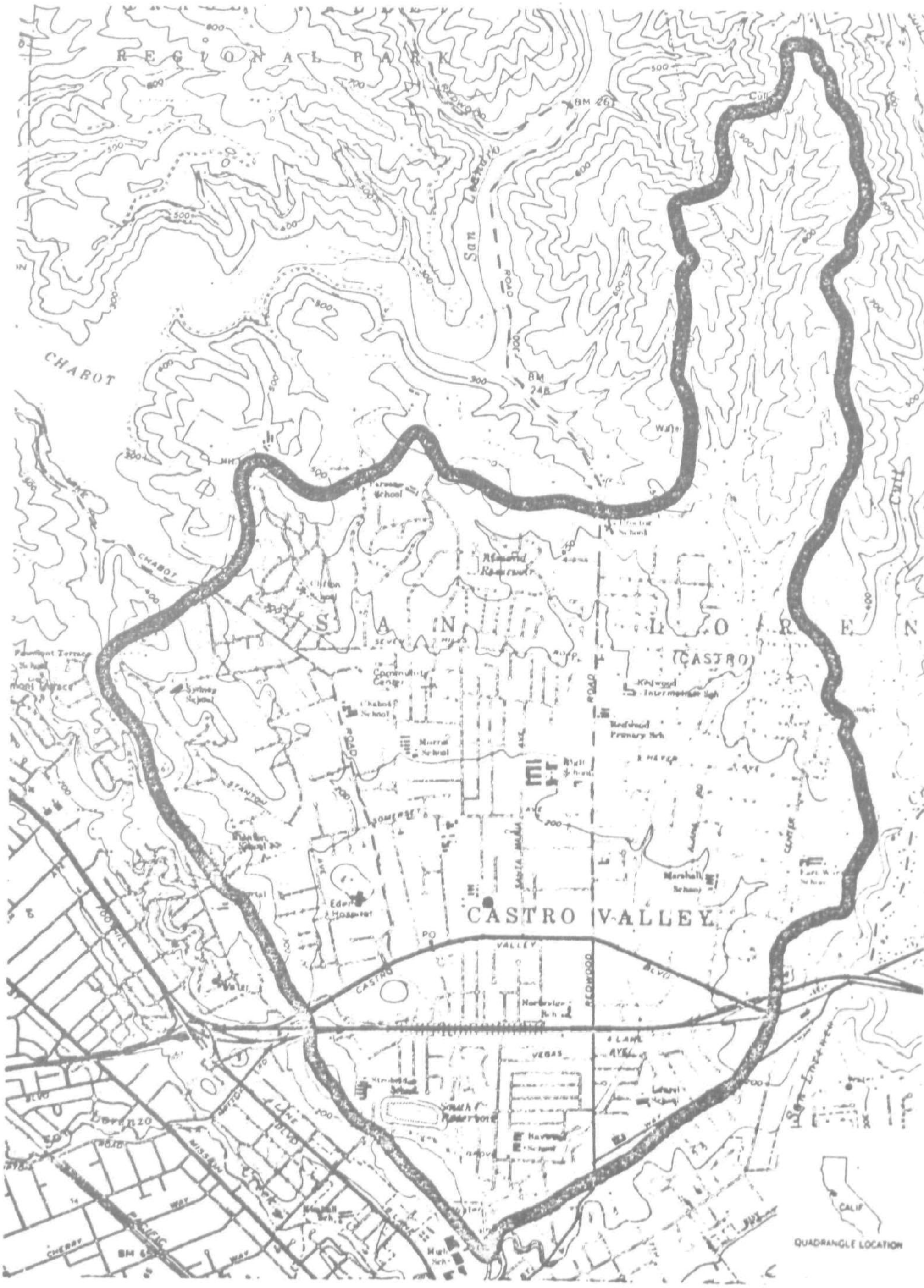


FIGURE 12. CASTO VALLEY WATERSHED.

TABLE 8
Hydrogeometric Data

Land Use	Percent of Area	Percent Impervious	Length of Street Gutters
Single Family Residential	70	40	275(ft/ac)
Multiple Family Residential	3	50	430
Commercial	7	80	400
Open or Park	20	2	20

Area of the watershed = 3136 acres
Depression Storage = .10 inch

Depression Storage Recovery, inches/day

January	0.05	July	0.28
February	0.07	August	0.25
March	0.12	September	0.20
April	0.17	October	0.13
May	0.23	November	0.07
June	0.26	December	0.05

Runoff Coefficient for Pervious areas = 0.45
Runoff Coefficient for Impervious area = 0.90

Area weighted basin average runoff coefficient = 0.61

The program parameters which were "tuned" in the calibration process were:

1. Rainfall factor relating basin average rainfall to the gage rainfall;
2. Depression storage and the rate of recovery of depression storage;
3. Imperviousness of the land uses; and
4. Pervious and impervious area runoff coefficients.

The recording rain gage for Castro Valley is centrally located in the watershed and was assumed to reflect basin average precipitation. Calibrated values of the other parameters are those listed in Table 9.

Table 9 shows the computed and observed data for monthly runoff and for total runoff over the 17-month period. Computed average annual rainfall and runoff information is obtained directly from the program's average annual summary. Monthly volumes were computed from the EVENT or POLLUTOGRAPH output. Figures 13 and 14 show the computed and observed hydrograph fit for two events in the 13-month record.

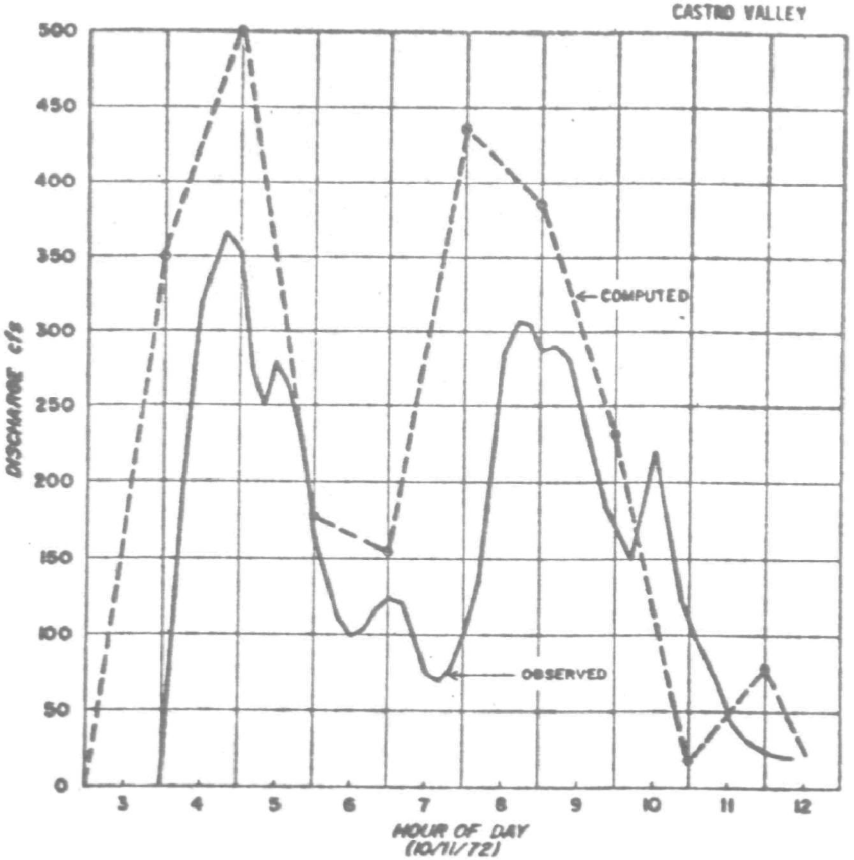


Figure 13. Comparison of Observed Hourly Values of Values of Runoff With Values Computed by "STORM" November 11, 1972.

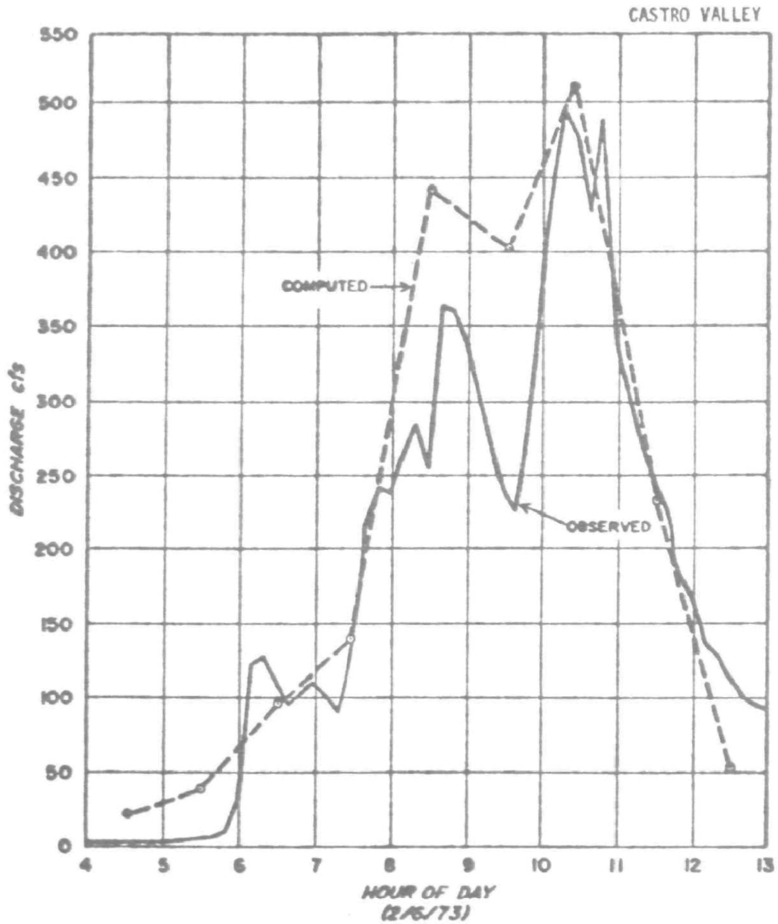


Figure 14. Comparison of Observed Hourly Values of Runoff With Values Computed by "STORM" February 6, 1973.

TABLE 9
Comparison of Monthly Runoff Volumes
From Castro Valley Computed by Runoff Model

Year	Month	Actual Runoff (Basin-In)*	STORM**, In
1971	11	.35	.73
	12	1.50	2.29
1972	1	.51	.58
	2	.60	.58
	3	.17***	0
	4	.34	.44
	5	.10***	0
	6	.17	.16
	7	.11***	0
	8	.08***	0
	9	.21	.48
	10	1.01	1.51
	11	2.85	2.82
	12	.91	1.03
1973	1	5.78	4.13
	2	4.34	2.45
	3	2.01	1.71
		=21.00	18.90
			+1.70 baseflow
			20.60

*From USGS Records.
**Does not include baseflow. Baseflow is approximately 0.10 inches per month.
***No rainfall recorded.

COMPUTATION OF THE QUALITY OF RUNOFF

The zero storage, zero treatment combination was also used for an initial calibration of the quality portion of STORM. The quality calibration was made difficult because of the lack of adequate data. It is usually not economically feasible to monitor every runoff event, thus monthly or average annual data generally does not exist. In most instances, as was the case for the Castro Valley data, the quality calibration must be made on data from several individual runoff events. However, unless care is taken to obtain a good sampling over the duration of the entire event, the comparison of total amounts of pollutant washoff will not be possible. This was also the case in Castro Valley; consequently, most of the calibration had to rely on a few data points per event. It would be highly desirable to have enough measurements during a runoff event to be able to trace the hourly performance of the pollutant washoff function.

The quality calibration for Castro Valley was made on the basis of comparisons of computed and observed concentrations for individual events. The parameters calibrated were:

1. Dust and dirt accumulation rates;
2. Pollutant composition of the dust and dirt; and
3. The exponent in the pollutant washoff equation.

The initial pollutant loading rates used in the calibration were those from the Chicago APWA study [3]. These data are programmed as default values to be used if other values for these parameters are not specified as input data. Table 10 shows the calibrated values of the pollutant loading and washoff parameters. A comparison of these values with the default values listed earlier reveals that the dust and dirt accumulation rates had to be increased by a factor of two except for the open or park area which was increased by a factor of about six. Pollutant composition of the dust and dirt was also increased by a factor of four for all parameters except suspended and settleable solids.

TABLE 10
Pollutant Loading and Washoff Parameters

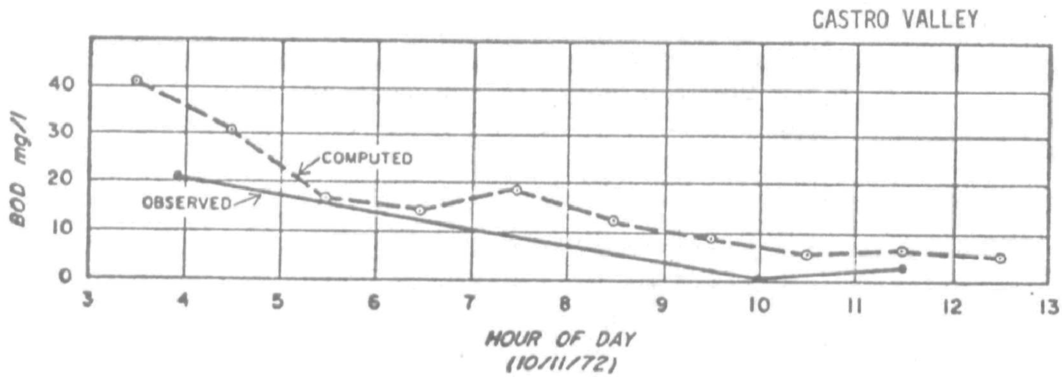
Land Use	Dust and Dirt Accumulation lbs/day/100 ft. gutter	Pounds Pollutant per 100 lbs Dust and Dirt: Susp. Solids	Sett. Solids	BOD	N	PO4
Single Family	1.4	11.1	1.1	2.0	.19	.02
Multiple Family	4.6	8.0	.8	1.44	.24	.02
Commercial	6.6	17.0	1.7	3.08	.16	.03
Open or Park	9.2	11.1	1.1	2.0	.19	.02

Street Sweeping Efficiency = 70%
Washoff exponent = 2.0

Comparisons of computed and measured BOD concentrations are shown in Figure 15. Agreement in the first case is very good, however, computed values tor the second storm are consistently low. Notice that the first storm occurred in November of 1972 which is the beginning of the winter rainy season. The good agreement in this case indicates that the accumulation of pollutant loads over the dry summer period is being computed well by STORM.

The second comparison shown was for a storm in February which is near the end of the winter. Since the computed values are consistently low, the implication is that the pollutant load on the watershed at the beginning of the storm was too small. This is quite likely the case because during the period between the November storm and the February storm, the computer program program has been performing a constant accounting of the pollutant load on the watershed, i.e., how much was there at the beginning of each storm, how much was left at the end of each storm, and how much accumulated between storms. If we assume, on the basis of Figure 15 (the November storm), that the pollutant load at the beginning of the rainy season is correct and that the computation of washoff rate is correct, then the rate of pollutant accumulation (dust and dirt accumulation) on the watershed must be larger for the prototype than the rate being used in the model. As a result, the watershed is being washed overly clean by STORM during the rainy season.

The next step in the calibration procedure, therefore, would be to increase the daily rate of dust and dirt accumulation during the winter period which would result in larger pollutant loads at the end of the rainy season.



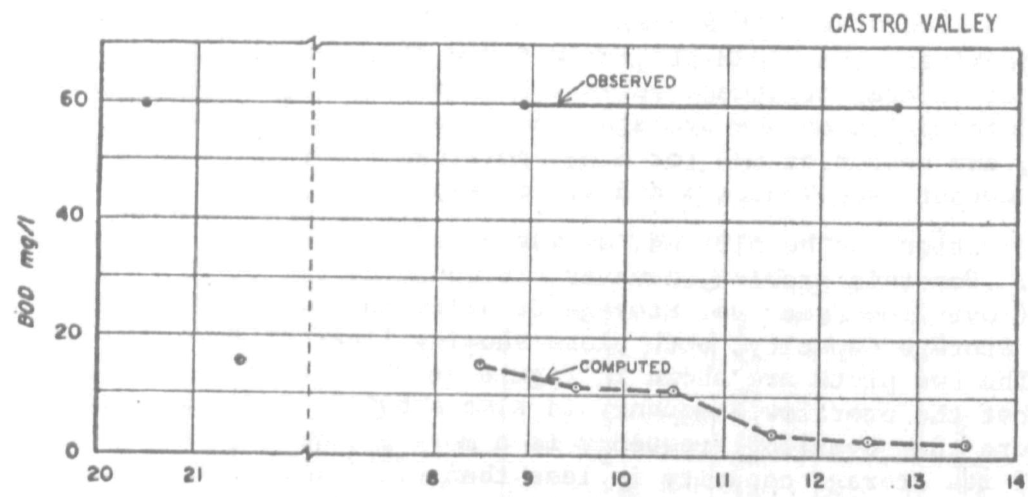


Figure 15. Comparison of Observed Hourly Values of BOD Concentration With Values Computed by "STORM" November 11, 1972 and February 6, 1973.

STORAGE-TREATMENT ANALYSIS EXAMPLE

The primary purpose of the STORM program is to analyze the effectiveness of storage and treatment facilities for use in controlling the quantity and quality of urban storm runoff. The criteria for control of the storm runoff may be in terms of maximum allowable:

1. Overflow events per year;
2. Volume of overflow per year;
3. Volume of overflow during some design storm event;
4. Pounds of BOD (or other pollutant) overflow per year; and/or
5. Pounds of BOD (or other pollutant) overflow during some design storm event.

It may be possible to achieve the desired control of runoff through a number of different combinations of storage and treatment. A cost analysis is then necessary to determine which storage-treatment alternative achieves the desired control at least cost.

The decision criteria for the purposes of this hypothetical example are as follows:

1. No more than five overflows per year; and
2. No more than 10,000 pounds of BOD overflow per year.

In order to determine what storage-treatment rate combinations will meet these objectives, the STORM program was run for several different storages for each of several treatment rates. For each storage-treatment combination, information on the average annual number of overflows, inches of overflow, and pounds of BOD (or other pollutant) was obtained directly from the output (see Tables 8 and 10 for example).

This information can be plotted, as was done in Figure 6, to enhance the analysis. For this problem, however, it would be better to first plot Number of Overflows/year vs. Storage Capacity and Pounds of BOD Overflow/ Year vs. Storage Capacity, both plots showing lines of equal treatment rates. The two plots are shown in Figure 16. Notice on the BOD overflow curves that the overflow frequency is also shown. It can be seen from this figure that overflow frequency is a more severe requirement for the system if its storage capacity is less than 0.3 inches. For greater storage capacities, however, the quality constraint is more limiting. Thus all treatment-storage combinations shown below the shaded line in Figure 17 are acceptable from the standpoint of meeting or exceeding the system performance criteria. However, it can be seen that for a given treatment rate, the smallest allowable storage will be that identified on the shaded performance line in Figure 17. Thus, in our example the most economical control system that meets the performance criteria will be one of the following three:

1. Treatment = 0.01 in/hr Storage = 0.46 in
2. Treatment = 0.03 in/hr Storage = 0.28 in
3. Treatment = 0.05 in/hr Storage = 0.20 in

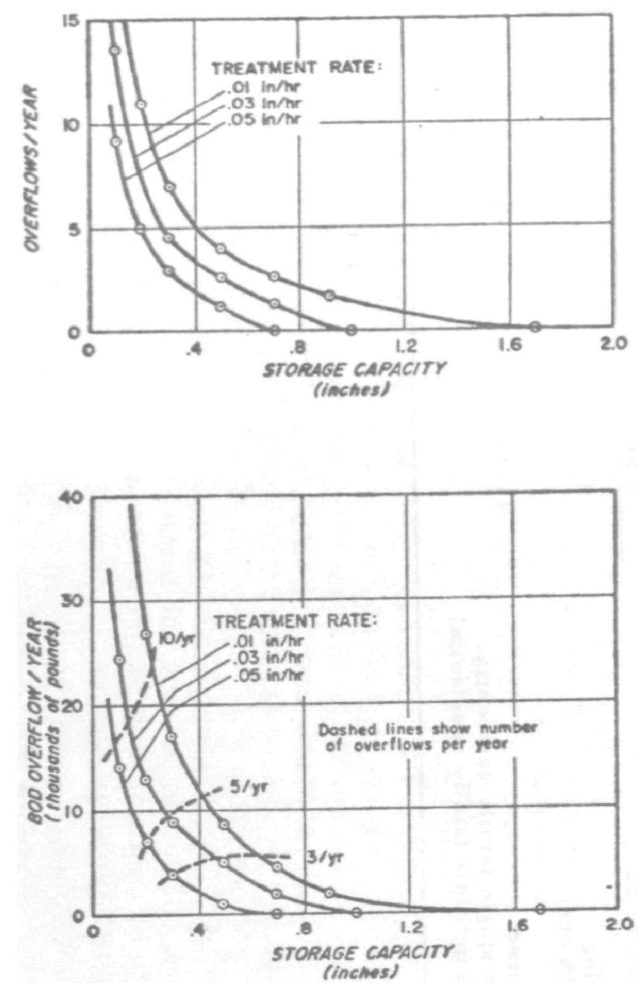


Figure 16. Relationship of Overflow Frequency and Pollutant Overflows to Treatment-Storage Alternatives Castro Valley.

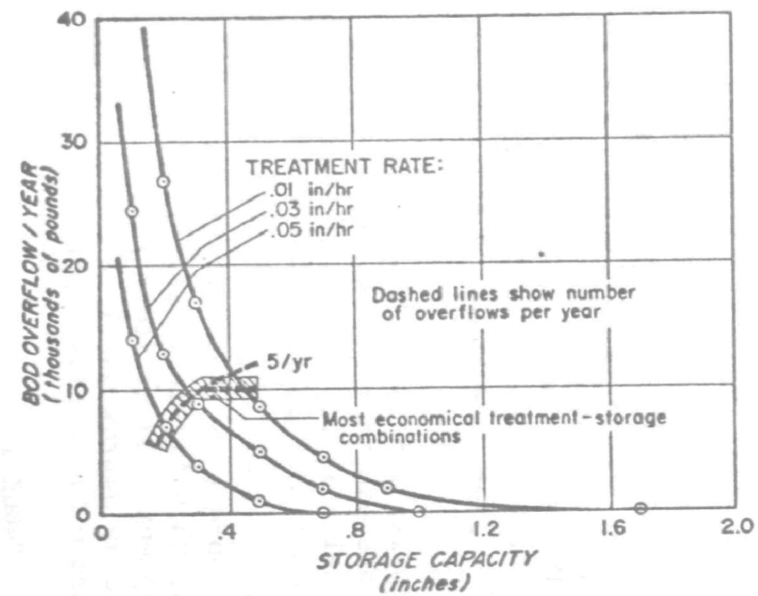


Figure 17. Most Economical Treatment-Storage Combinations To Meet System Overflow Constraints.

Economic Analysis of these three alternatives would identify the most economical system.

LAND USE MANAGEMENT ANALYSIS

Another possible application of STORM is to estimate the impact of proposed land use changes within a watershed on the quantity and quality of urban runoff. To illustrate this use, it was assumed that the upper arm of the Castro Valley watershed (see Figure 12) would be developed. This 470 acre area is presently open space. Under the assumed development plan, single family residential dwellings would be constructed on 67 percent of the area. The remaining 33 percent of the area would be developed as multiple family residences.

Table 11 shows the results of applying STORM to the 470 acre subarea in both its present undeveloped state and in the proposed developed state. In the developed state, the annual quantity of runoff can be expected to increase by 40 percent. The annual BOD, on the other hand will increase more than 400 percent.

The impact of the proposed development on the watershed as a whole is shown in Table 12. As expected, this impact is significantly less. As a result of the proposed development, annual storm runoff from the watershed can be expected to increase by approximately 5 percent. The annual BOD load washed off the watershed will increase approximately 14 percent.

TABLE 11
Effect of Changing Land Use on
Storm Runoff from 470 Acre Subarea

Land Use	Annual Runoff Inches	Annual BOD Load Pounds
Existing 100% open	8.34	5,700
Proposed 67% single family residential 33% multiple family residential	11.72	23,400

TABLE 12

Effect of Changing Land Use of 470 Acre Subarea on Storm Runoff From Entire Castro Valley Watershed (3140 Acres)

Land Use	Annual Runoff Inches	Annual BOD Load Pounds
Existing	11.08	130,500
70% single family residential		
3% multiple family residential		
7% commercial		
20% open		
Proposed	11.59	148,300
80% single family residential		
8% multiple family residential		
7% commercial		
5% open		

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COLLECTION OF FIELD DATA FOR STORMWATER MODEL CALIBRATION

By

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INTRODUCTION

This lecture on the collection of field data for stormwater management model calibration and verification is intended to stress the importance of data gathering procedures and techniques for data products that are to be used in conjunction with such models and to summarize the current state of the art with respect of wastewater sampling and flow measurement. It is assumed that the reader has a baccalaureate degree in one of the basic engineering disciplines (or the equivalent), an overall appreciation for urban hydrology and general stormwater characteristics, and at least a limited amount of field experience.

BACKGROUND

Hydrology today is in a period of transition due to the advent of the digital computer. This tool has made possible the development and use of complex mathematical models during the 1960s. Where once the "design storm" concept was the basic approach to design, near continuous simulation leading to estimates of flow probabilities has now become possible. The literature probably contains references to over 200 stormwater-related models. Very few of these, however, have been specially designed for urban areas. Most of these models can be classified into a few basic types, and many differ little from conventional hydrologic techniques of the past few decades.

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Precipitation Models

Although precipitation modeling and measurement lie outside the bounds of the present lecture, a few comments are appropriate for the sake of completeness since the more sophisticated urban stormwater models need a detailed knowledge of time and space variability of precipitation as inputs. For example, during the progress of a storm, the spatial distribution of precipitation for consecutive, specified time intervals (e.g., five minutes precipitation accumulation) might be required continuously on a real time basis. The procedure for computing the spatial distribution of a variable over some region from data at a limited number of sites is often referred to as objective analysis. Bergthorssen and Doos (1955) present the development of a typical objective analysis technique. The accuracy of any such analysis depends upon the variability of the analysis parameter and upon the density of observations.

Many researchers have used dense raingage networks to inquire directly of nature for particular regions. The time distribution characteristics of rainfall rates were investigated by Huff (1970b) who used data from a 60-storm sample on two dense networks in Illinois. He found variability parameters to fit closely a log normal distribution. Both absolute and relative variability showed a wide range within and between storms and between areas of different size. Little difference in variability properties was noted between rain and synoptic weather types associated most frequently with warm-season storms. In a study of the spatial distribution of rainfall rates, Huff (1970a) found it to be so great within and between convective storms that the equipment and operational demands for accurate rate measurements may be prohibitive for areas over 16 km square. In related work, Huff and Shipp (1969) found that a gage spacing of 0.3 minutes is needed for 1-minute rain rates compared with 7.5 minutes for total storm rainfall, a 625 to 1 increase.

Stormwater Models

Given the foregoing, it is no surprise that a number of different methods or models for the calculation of stormwater runoff from rainfall have arisen. However, there seem to be only a few basically different models that appear in the literature. The many so-called methods that are presented usually consist only of different procedures for evaluating the key parameters of one of the basic models, different computational procedures for solving similar equations, or sometimes only a name. Linsley (1971) has suggested that there are four basic techniques for calculating or estimating flows in a watershed; empirical formulae, statistical correlation with watershed characteristics (regression models), frequency analysis of streamflow, or computation from precipitation data (hydrolic synthesis). He admits, however, that the boundaries of the four can become quite blurred. A brief discussion of each technique follows.

Empirical techniques are in common use today, the most popular of which is the so-called rational method first put forth by Mulvaney (1851) and introduced in the United States by Kuichling (1889). Chow (1962) has given a good bibliography, and the method was reviewed recently by McPherson (1969). The real issue of how long must the rainfall continue for equilibrium to be reached so that the assumption holds that peak outflow will be equal to inflow rate (rainfall intensity) is not answered by the rational method. The best approach is to make use of information that is now available on overland flow. The technique of Izzard (1946) appears fairly satisfactory, although several other methods have been suggested in the more recent literature, e.g., Grace and Eagleson (1966) and Woolhiser and Liggett (1967).

Regression techniques seek to relate a causal factor (precipitation and/or watershed characteristics) with an effect (peak flow, storm runoff volume, etc.) by statistical correlation. One could view them merely as extensions of empirical techniques based upon the existence of more data and more sophisticated methods of analysis. Potter (1961) presents a typical example. Care must be exercised in such techniques, and Freeze (1972) notes, "This evidence for a wide range of watershed response functions leads to the development of a healthy skepticism toward black-box rainfall-runoff correlations, the concept of basin linearity, and the rationality of hydrograph separation."

Frequency analysis techniques can be used to generate probabilities of peak flow statistically if enough prior data are available and there have been no significant changes in characteristics over the data period. Methods of transposition to broaden the use of such techniques have been advanced, see Anderson (1970) for example. Dalrymple (1960) has presented an adaptation of regional frequency analysis.

Hydrologic synthesis techniques employ two basically separable relations; a rainfall-runoff relationship to estimate the volume of stormwater runoff (rainfall excess), and a procedure for determining the shape of the flow hydrograph having the estimated runoff volume. Rainfall-runoff relations can be broken down into three basic types - regression relations, infiltration indices, and water balance models. Runoff that reaches the receiving water has, in general, three components that follow different paths; surface (sheet or overland) flow, interface (through the upper soil layer) flow, and base (groundwater) flow. For urban drainage basins only surface flow is relevant in most instances. Regression techniques range from a simple plot of rainfall vs runoff to fairly complicated multivariable procedures. Kohler and Linsley (1951) proposed a coaxial correlation technique that has been used by the National Weather Service in flood forecasting. Cook (1946) introduced the use of infiltration indices which can be viewed as representing an average loss rate. These can be determined from selected storm hydrographs and rainfall data. The water balance method maintains a running account of the water in soil moisture storage by adding each new rainfall, less direct runoff and accretion to groundwater, and subtracting evapotranspiration. Thus, the amount of runoff and groundwater accretion are made functions of the prevailing soil moisture storage. This technique has been used in the Stanford Watershed Model as reported by Crawford and Linsley (1966).

Linear hydrologic synthesis has been popular since the unit hydrograph was first proposed by Sherman (1932). The black-box technique of deriving unit hydrographs from input and output data while ignoring physical basin processes is not uncommon and has been approached in many ways. Snyder (1938) used a simple least squares fit, O'Donnell (1960) used a Fourier expansion, and Dooge (1959) employed Laguerre functions. All such approaches seek to find the optimal shape of the response to a unit impulse of rainfall; the difference merely lies in the method of doing this.

As was noted earlier, Freeze (1972) has questioned the assumption of basin linearity, and there is no real dearth of nonlinear approaches. For example, Amorocho and Brandsetter (1971) investigated the determination of nonlinear response functions in the rainfall-runoff process, and Bidwell (1971) performed a regression analysis of nonlinear catchment systems.

TYPES OF FIELD DATA

Regardless of whether the model of interest is stochastic versus deterministic, on the one hand, or analytic versus synthetic, on the other, there are two uses that are generally made of field data, and to drive home the importance of distinguishing between the two, they will be referred to here as data types. Because of the complexity of the processes being modeled, most of the models that are popular today require field data both for estimating empirical parameters in their structure and for fitting other application-specific parameters (calibration). For example, one version of the Stanford Watershed Model has twenty parameters, two based on meteorological data, four on hydrograph separation, five are computed from physical measures, three are estimated from empirical tables, and six are fitted. All field data that are used within the model structure, i.e., to ready the model for specific application, will be referred to here as calibration data.

The chief concern of the model user is how well the model outputs (which are its sole reason for being) compare to reality, this comparison forming a measure of the predictive capability of the model. Here, also, input and output field data are required, but their fundamental use is quite different from that of calibration data. They will be referred to here as verification data, since they are used to verify the results of a particular model exercise.

This distinction between field data types is not made to suggest that different gathering techniques are required for calibration versus verification data; in point of fact they are the same. The reason for making the distinction is simply that calibration data must never (repeat never) be used for model verification. The importance of this simplistic statement cannot be overestimated. Having made this point, the matter will be dropped, with the proviso that everything said about the collection of calibration data applies equally to verification data.

FACTORS INVOLVED IN WASTEWATER CHARACTERIZATION

The various stormwater management models that are available today require data on the catchment, precipitation, and runoff quantity and quality. They vary widely, however, in the detail of temporal and spatial distribution of data required. For example, some models require time steps of less than one minute to satisfy numerical stability conditions, while others can be run with hourly, daily, and even up to semimonthly data.

Considerations other than the model structure are also involved. Both the quantity and quality of stormwater runoff are highly variable and transient in nature, being dependent upon meteorological and climatological factors, topography, hydraulic characteristics of the surface and subsurface conduits, the nature of the antecedent period, and the land use activities and housekeeping practices employed. It is this highly variable and transient nature of stormwater flows that makes their characterization so difficult. In addition to tremendous dynamic ranges, the poor quality of stormwater draining from the urban environment has a significant effect on the choice of suitable sampling and flow measurement equipment and methods as well as an impact in the analytical laboratory.

The three factors involved in stormwater characterization efforts and treated in this lecture are flow measurement, sampling and laboratory analyses of the samples taken. Sections II, III, and IV are devoted to a discussion of each of these factors in turn, and we will limit ourselves here to noting that although they represent three different efforts requiring different types of skills and training, they are also interdependent tasks and must be considered together in the establishment of a successful stormwater characterization program.

THE NEED FOR TIME-SYNCHRONOUS DATA

Measurements of stormwater characteristics are useful only with respect to their relationship with each other and with other phenomena. An assignment of time of occurrence to a data product makes possible a determination of its relationship to other parameters whose times of occurrence are known. The other parameters of interest may or may not be synchronous with the particular data. In some cases, definition of the time interval between events is sufficient but, generally, the true clock time of the value of the concerned parameter is required.

The required accuracy of the time element in stormwater characterization data is very different from requirement to requirement. An example is the use of peak flows for each year to determine their frequency. On the other hand, flow and quality data at intervals as short as one minute may be needed to define the discharge hydrograph from small urban areas.

A particular need for attention to the time element occurs in the measurement of flows from small urban storm sewers in order to define the hydrograph and to provide data for the development and verification of rainfall-runoff-quality models. Accurate definition of both the time and discharge elements of the hydrograph makes possible the computation of total volume of runoff during the storm by computing the area under the hydrograph, exclusive of base flow. By selecting a number of well defined hydrographs resulting from storms of similar rainfall characteristics, a typical hydrograph for the basin may be defined. In combination with time-synchronous water quality data, pollutographs can be generated. Shape of the unit hydrograph is determined by accurate timing as well as by discharge, although it is independent of clock time. The hydrograph as defined by clock time and discharge is often used to route flows along a stream channel or through a reservoir.

Peak flows, storm runoff volumes, daily flows, or other flow parameters are often correlated with similar flows at other points on a storm sewer or stream, or with flows of other storm sewers or streams, to provide a means for flow estimation. Also, correlations may be made with various physical characteristics of a basin, such as area, slope, population density, etc. Correlations with temperature, soil moisture, or antecedent precipitation may be made at times. In most cases, it is essential that the correlated variables be synchronous, so accurate timing of the data is often required.

Timing of measured flows and collection of quality samples can be useful in determining sources of pollution. For example, they can be related to time of release of pollutants from industrial plants, or to the time of accidental spills of pollutants. The time of travel of pollutants along a stream or storm sewer can be estimated from the time of travel of small rises or other flow changes in the channel.

DATA QUALITY

We are not yet overwhelmed with existing data on stormwater characteristics. As stated by Torno (1975), "One of the serious problems that faces either a new model developer or one who must evaluate several models is the incredible lack of good data...". To be effectively utilized, we need more than simple values as data products. We need to know something about the data quality, i.e., about the "goodness" or truthfulness of the data. Two terms that relate to the data gathering process are conventionally used to describe data quality, accuracy and precision. Accuracy refers to the agreement between the measurement and the true value of the measurand, with the discrepancy normally referred to as error; while precision refers to the reproducibility (repeatability) of a measurement when repeated on a homogenous time-stationary measurand, regardless of the displacement of the observed values from the true value, and thus indicates the number of significant digits in the result. We are therefore interested in establishing the best estimate of a measured quantity and the degree of precision of this estimate from a series of repeated measurements. Calibration, whether it be of a piece of flow measurement equipment, a chemical method for wastewater analysis, a stormwater management model, or whatever, is simply the process of determining estimates of accuracy and precision.

Discrepancies between the results of repeated observations, or errors, are inherent in any measurement process since it is recognized that the true value of an object of measurement can never be exactly established. These errors are customarily classified in two main groups, systematic and random (or accidental) errors. Systematic errors usually enter into records with the same sign and frequently with either the same magnitude (e.g., a zero offset) or with an establishable relationship between the magnitude of the measurement and the error. The methods of symmetry and substitution are frequently used to detect and quantify systematic errors. In the method of symmetry, the test is repeated in a symmetrical or reversed manner with respect to the particular condition that is suspect. In the method of substitution, the object of measurement is replaced by one of known magnitude (a calibration standard), an instrument with a known calibration curve is substituted for the measuring instrument in question, and so on. Thus, sys- systematic errors bear heavily on the accuracy of the measurement.

Random errors, on the other hand, are due to irregular causes, too many in number and too complex in nature to allow their origin to be determined. One of their chief characteristics is that they are normally as likely to be positive as negative and, therefore, are not likely to have a great effect on the mean of a set of measurements. The chief aim of a data quality assurance effort is to account for systematic errors and thereby reduce errors to the random class, which can be treated by simple probability theory in order to determine the most probable value of the object of observation and a measure of the confidence placed in this determination.

The statistical measures of location or central tendency (e.g., the various averages, mean, median, mode, etc.) are related to accuracy. The sta statistical measures of dispersion or variability (e.g., variance and standard deviation, coefficient of variation, and other measures derived from central moments of the probability density function) are related to precision.

Even lacking enough data for statistical treatment, there are some annotations that the data gatherer can make to increase the usefulness of the data. For example, inspection of equipment and records may indicate periods of instrument malfunction or failure (e.g., power interruptions).

These facts are important and should form a part of the total record. There may be circumstances discovered during site visits that would have an effect on preceding data that can not be readily determined, e.g., a partially blocked sampler intake, a rag caught in the notch of a weir, etc. These facts should also be noted and, where at all feasible, some qualitative notation as to expected data quality (e.g., poor, very good, etc.) should be made.

The importance of notations of data quality arises from the ultimate use of the data. At the risk of seeming ridiculous, ±50% data should not be used to calibrate a model whose outputs are desired within ±20%, nor should strong model verification judgements be made based upon a very small sample of data with a high variability, for example. The levels of data quality desired vary with the intended use of model outputs. The needs for overall basin planning, treatment plant design, plant operation, and research are all quite different and this must be kept in mind in designing the data gathering program (or system).

FLOW MEASUREMENT

The subject of flow measurement in general, and sewer flow measurement in particular, has received considerable attention in the past and continues to as reflected in the current literature and the introduction of new equipment into the commercial marketplace. Only a very sketchy treatment can be given here; for more detail, reference should be made to the recent monograph by Shelley and Kirkpatrick (1975), from which the bulk of the material, including all Figures and Tables, in this section was taken.

SITE SELECTION

The success or failure of selected flow measurement equipment or methods, with respect to accuracy and completeness of data collected as well as reasonableness of cost, depends very much on the care and effort exercised in selecting the gaging site. Except for a few basic requirements which are applicable to all types of equipment and methods and which will be discussed at this point, there are significant differences in site needs for various flow measurement devices.

A requirement which appears to be obvious, but which is frequently not sufficiently considered, is that the site selected be located to give the desired flow measurement. Does flow at the site provide information actually needed to fulfill project needs? Sometimes influent flows, diversions, or storage upstream or downstream from the selected site would bias the data in a manner not understood without a thorough study of the proposed site. Such study would include reference to surface maps and to sewerage maps and plans. Sometimes groundwater infiltration or unrecorded connections may exist. For these reasons, a thorough field investigation should be made before establishing a flow measurement site.

There are some situations where there is no choice of sites. Only a single site may be available where the desired flow measurement can be made. In this case, the problem is one of selecting the most suitable flow measurement equipment and methods for the available site.

A basic consideration in site selection is the possible availability of flow measurements or records collected by others. At times, data being collected by the U.S. Geological Survey, by the State, or by other public agencies can be used. There are locations where useful data, although not currently being collected, may have been collected in prior years. Additional data to supplement those earlier records may be more useful than new data collected at a different site. Other general site considerations include any history of surcharging, entry and backwater conditions, and intrusion from receiving waters.

Requirements that apply to all flow measurement sites are accessibility, personnel and equipment safety, and freedom from vandalism. If a car or other vehicle can be driven directly to the site at all times, the cost in time required for installation, operation, and maintenance of the equipment will be less, and it is possible that less expensive equipment can be selected. Consideration should be given to access during the periods of adverse weather conditions and during periods of flood stage. Sites on bridges or at manholes where heavy traffic occurs should be avoided unless suitable protection for men and equipment is provided. If entry to sewers is required, the more shallow locations should be selected where possible. Manhole steps and other facilities for sewer access must be carefully inspected, and any needed repairs made. Possible danger from harmful gases, chemicals, or explosion should be investigated. With respect to sites at or near streams, historical flood marks should be determined and used for placement of access facilities and measurement equipment above flood level where this is possible. Areas of known frequent vandalism should be avoided.

Selection of sites in open, rather than secluded, areas may help to reduce vandalism. Often, the only solution to prevent destruction of facilities is to place them in solid concrete or steel shelters, and to surround them with heavy fencing. Erection of warning signs is futile, as they often serve only to provide targets.

In development of a system or network of flow measurement stations, primary consideration must be given to cost if the maximum benefit is to result from available funds. Therefore, cost must be considered in the selection of each gaging site. Cost reduction can result from selection of sites where the less expensive types of equipment, which will fulfill project requirements of accuracy and completeness, can be installed. For example, if a site is selected where conditions are such that satisfactory records can be obtained with a weir installation, this would be preferable to selecting a site where the head loss required by a weir would not be available, and the expense of installing a Parshall flume must be met.

METHODS OF FLOW DETERMINATION

This brief discussion is intended to provide an overview of the physical principles that have been utilized in the design of equipment for the quantitative measurement of flows. For further reading see Shelley and Kirkpatrick (1975), ASME (1971), Replogle (1970), McMahon (1964), USDI Bureau of Reclamation (1967), Leupold and Stevens (1974), and any of the many standard texts on hydraulics and fluid mechanics.

Any flow measurement system can be considered to consist of two distinct parts, each of which has a separate function to perform. The first, or primary element, is that part of the system which is in contact with the fluid, resulting in some type of interaction. The secondary element is that part of the system which translates this interaction into the desired readout or recording. While there is almost an endless variety of secondary elements, primary elements are related to a more limited number of physical principles, being dependent upon some property of the fluid other than, or in addition to, its volume or mass such as kinetic energy, inertia, specific heat, or the like. Thus, the primary elements, or rather their physical principles, form a natural classification system for flow measuring devices and are so used in this discussion.

Flow measurement systems may be thought of as belonging to one of two rather broad divisions, quantity and rate. In quantity meters, the primary element measures isolated (i.e., separately counted) quantities of fluid either in terms of mass or volume. Usually a container or cavity of known capacity is alternately filled and emptied, permitting an essentially continuous flow of the metered supply. The secondary element counts the number of these quantities and indicates or records them, often against time. In rate meters, by contrast, the fluid passes in a continuous, uninterrupted stream, which interacts with the primary element in a certain way, the interaction being dependent upon one or more physical properties of the fluid. In the secondary element, the quantity of flow per unit time is derived from this interaction by known physical laws supplemented by empirical relations. A general categorization of flow meters by division, classification, type, and sub-type is presented in Table 1.

TABLE 1. FLOW METER CATEGORIZATION

DIVISION	CLASSIFICATION	TYPE	SUBTYPE
QUANTITY	GRAVIMETRIC	WEIGHER	
QUANTITY	GRAVIMETRIC	TILTING TRAP	
QUANTITY	GRAVIMETRIC	WEIGH DUMP	
QUANTITY	VOLUMETRIC	METERING TANK	
QUANTITY	VOLUMETRIC	RECIPROCATING PISTON	
QUANTITY	VOLUMETRIC	OSCILLATING OR RING PISTON	
QUANTITY	VOLUMETRIC	NUTATING DISC	
QUANTITY	VOLUMETRIC	SLIDING VANE	
QUANTITY	VOLUMETRIC	ROTATING VANE	
QUANTITY	VOLUMETRIC	GEAR OR LOBED IMPELLER	
QUANTITY	VOLUMETRIC	DETHRIDGE WHEEL	
RATE	DIFFERENTIAL PRESSURE	VENTURI	
RATE	DIFFERENTIAL PRESSURE	DALL TUBE	
RATE	DIFFERENTIAL PRESSURE	FLOW NOZZLE	
RATE	DIFFERENTIAL PRESSURE	ROUNDED EDGE ORIFICE	
RATE	DIFFERENTIAL PRESSURE	SQUARE EDGE ORIFICE	CONCENTRIC
RATE	DIFFERENTIAL PRESSURE	SQUARE EDGE ORIFICE	ECCENTRIC
RATE	DIFFERENTIAL PRESSURE	SQUARE EDGE ORIFICE	SEGMENTED
RATE	DIFFERENTIAL PRESSURE	SQUARE EDGE ORIFICE	GATE OR VARIABLE AREA
RATE	DIFFERENTIAL PRESSURE	CENTRIFUGAL	ELBOW OR LONG RADIUS BEND
RATE	DIFFERENTIAL PRESSURE	CENTRIFUGAL	TURBINE SCROLL CASE
RATE	DIFFERENTIAL PRESSURE	CENTRIFUGAL	GUIDE VANE SPEED RING
RATE	DIFFERENTIAL PRESSURE	IMPACT TUBE	PITOT-STATIC
RATE	DIFFERENTIAL PRESSURE	IMPACT TUBE	PITOT VENTURI
RATE	DIFFERENTIAL PRESSURE	LINEAR RESISTANCE	PIPE SECTION
RATE	DIFFERENTIAL PRESSURE	LINEAR RESISTANCE	CAPILLARY TUBE
RATE	DIFFERENTIAL PRESSURE	LINEAR RESISTANCE	POROUS PLUG
RATE	VARIABLE AREA	GATE	
RATE	VARIABLE AREA	CONE AND FLOAT	
RATE	VARIABLE AREA	SLOTTED CYLINDER AND PISTON	
RATE	HEAD-AREA	WEIR	SHARP CRESTED
RATE	HEAD-AREA	WEIR	BROAD CRESTED
RATE	HEAD-AREA	FLUME	VENTURI
RATE	HEAD-AREA	FLUME	PARSHALL
RATE	HEAD-AREA	FLUME	PALMER-BOWLUS
RATE	HEAD-AREA	FLUME	DISKIN DEVICE
RATE	HEAD-AREA	FLUME	CUTTHROAT
RATE	HEAD-AREA	FLUME	SAN DIMAS
RATE	HEAD-AREA	FLUME	TRAPEZOIDAL
RATE	HEAD-AREA	FLUME	TYPE HS, H, AND HL
RATE	HEAD-AREA	OPEN FLOW NOZZLE	
RATE	FLOW VELOCITY	FLOAT	SIMPLE
RATE	FLOW VELOCITY	FLOAT	INTEGRATING
RATE	FLOW VELOCITY	TRACER	
RATE	FLOW VELOCITY	VORTEX	VORTEX-VELOCITY
RATE	FLOW VELOCITY	VORTEX	EDDY-SHEDDING
RATE	FLOW VELOCITY	TURBINE	
RATE	FLOW VELOCITY	ROTATING ELEMENT	HORIZONTAL AXIS
RATE	FLOW VELOCITY	ROTATING ELEMENT	VERTICAL AXIS
RATE	FORCE-DISPLACEMENT	VANE	
RATE	FORCE-DISPLACEMENT	HYDROMETRIC PENDULUM	
RATE	FORCE-DISPLACEMENT	TARGET	
RATE	FORCE-DISPLACEMENT	JET DEFLECTION	
RATE	FORCE-DISPLACEMENT	BALL AND TUBE	
RATE	FORCE-MOMENTUM	AXIAL FLOW MASS	
RATE	FORCE-MOMENTUM	RADIAL MASS	
RATE	FORCE-MOMENTUM	GYROSCOPIC	
RATE	FORCE-MOMENTUM	MANGUS EFFECT	
RATE	THERMAL	HOT TIP	
RATE	THERMAL	COLD TIP	
RATE	THERMAL	BOUNDARY LAYER	
RATE	OTHER	ELECTROMAGNETIC	
RATE	OTHER	ACOUSTIC	
RATE	OTHER	DOPPLER	
RATE	OTHER	OPTICAL	
RATE	OTHER	DILUTION	
RATE	OTHER	ELECTROSTATIC	
RATE	OTHER	NUCLEAR RESONANCE	

DESIRABLE EQUIPMENT CHARACTERISTICS

It is obvious from Table 1 that not all types of flow meters are suitable for measuring stormwater flows. In fact, the severe conditions and vagaries of these flows place a number of very stringent design requirements on flow measurement equipment if it is to function satisfactorily in such an application. It should also be apparent that no single design can be considered ideal for all flow measurement activities in all storm flows of interest. Characteristics of the available sites, as well as the particular flows in question, make a device that might be acceptable for one location totally unsuitable for another. Despite this, one can set forth some equipment "requirements" in the form of primary design goals and some desirable equipment features in the form of secondary design goals.

Primary Design Goals

The following are considered to be primary design goals for equipment that is to be used to measure storm and combined sewer flows:

Range - Since flow velocities may range from 0.03 to 9 m/s (0.1 to 30 fps), it is desirable that the unit have either a very wide range of operation; be ablé to automatically shift scales; or otherwise cover at least a 100 to 1 range.

Accuracy - For most purposes, an accuracy of ±10% of the reading at the readout point is necessary, and there will be applications where an accuracy of ±5% is highly desirable. Repeatability of better than ±2% is desired in almost all instances.

Flow Effects on Accuracy - The unit should be capable of maintaining its accuracy when exposed to rapid changes in flow; e.g., depth and velocity changes in an open channel flow situation. There are instances where the flows of interest may accelerate from minimum to maximum in as short a time period as five minutes.

Gravity and Pressurized Flow Operation - Because of the conditions that exist at many measuring sites, it is very desirable that the unit have the capability (within a closed conduit) of measuring over the full range of open channel flow as well as with the conduit flowing full and under pressure.

Sensitivity to Submergence or Backwater Effects - Because of the possibility of changes in flow resistance downstream of the measuring site due to blockages, rising river stages including possible reverse flow, etc., it is highly advantageous that the unit be able to continue to function under such conditions or, at a minimum, be able to sense the existence of such conditions which would lead to erroneous readings.

Effects of Solids Movement - The unit should not be seriously affected by the movement of solids such as sand, gravel, debris, etc. within the fluid flow.

Flow Obstruction - The unit should be as non-intrusive as possible to avoid obstruction or other interference with the flow which could lead to flow blockage or physical damage to some portion of the device.

Head Loss - To be usable at a maximum number of measurement sites, the unit should induce as little head loss as possible.

Manhole Operation - To allow maximum flexibility in utilization, the unit should have the capability of being installed in confined and moisture-laden spaces such as sewer manholes.

Power Requirements - The unit should require minimum power at the measuring site to operate; the ability to operate on batteries is a definite asset for many installations.

Secondary Design Goals

The following are desirable features for flow measuring equipment, especially for use in a storm or combined sewer application:

Site Requirements - Unit design should be such as to minimize site requirements, such as the need for a fresh water supply, a vertical drop, excessive physical space, etc.

Installation Restrictions or Limitations - The unit should impose a minimum of restrictions or limitations on its installation and be capable of use on or within sewers of varying size.

Simplicity and Reliability - To maximize reliability of results and operation, the design of the unit should be as simple as possible, with a minimum of moving parts, etc.

Unattended Operation - For the majority of applications, it is highly desirable that the equipment be capable of unattended operation.

Maintenance Requirements - The design of the equipment should be such that routine maintenance is minimal and troubleshooting and repair can be effected with relative ease, even in the field.

Adverse Ambient Effects - The unit should be unaffected by adverse ambient conditions such as high humidity, freezing temperatures, hydrogen sulphide or corrosive gases, etc.

Submersion Proof - The unit should be capable of withstanding total immersion without significant damage.

Ruggedness - The unit should be of rugged construction and as vandal and theft proof as possible.

Self Contained - The unit should be self contained insofar as possible in view of the physical principles involved.

Precalibration - In order to maximize the flexibility of using the equipment in different settings, it is desirable that it be capable of precalibration; i.e., it should not be necessary to calibrate the system at each location and for each application.

Ease of Calibration - Calibration of the unit should be a simple, straightforward process requiring a minimum amount of time and ancillary equipment.

Maintenance of Calibration - The unit should operate accurately for extended periods of time without requiring recalibration.

Adaptability - The system should be capable of: indicating and recording instantaneous flow rates and totalized flows; providing flow signals to associated equipment (e.g., an automatic sampler); implementation of remote sensing techniques or incorporation into a computerized urban data system, including a multi-sensor single readout capability.

Cost - The unit should be affordable both in terms of acquisition and installation costs as well as operating costs, including repair and maintenance.

Evaluation Parameters

It is, of course, not necessary that all of the primary and secondary design goals be achieved for all flow measurement requirements. For example, "spot" measurements of all flow rather than continuous records are sufficient at times. Flow measurement devices used to calibrate others need not necessarily be self contained, nor would unattended operations be required. Furthermore, meeting all of the listed design goals for all installations and settings would be difficult, if not impossible, to achieve in a single design.

Nonetheless, the primary and secondary design goals can be used to formulate a set of evaluation parameters against which a given design or piece of equipment can be judged. Since application details may make certain parameters more or less important in one instance or another, no attempt has been made to apply weighting factors or assign numerical rank. It is hoped that the evaluation factors will prove useful, as a check list among other things, for the potential user who has a flow measurement requirement and who may require assistance in the selection of his equipment.

The evaluation parameters together with qualitative scales, are presented in the form of a flow measurement equipment checklist in Table 1.

GENERIC EVALUATIONS OF SOME PROMISING DEVICES

A slightly modified form of the flow measurement equipment checklist given in Table 2 has been used to evaluate the various flow measuring devices and techniques of Table 2, and a matrix summary is given as Table 3. It must be emphasized that these evaluations are made with a storm or combined sewer flow measurement application in mind and will not necessarily be applicable for other types of flows. They are necessarily somewhat subjective, and the writer apologizes in advance

TABLE 2. FLOW MEASUREMENT EQUIPMENT CHECKLIST

Designation: ______________________________

	Evaluation Parameter	Scale			Weight and Score
1	Range	☐ Poor	☐ Fair	☐ Good	
2	Accuracy	☐ Poor	☐ Fair	☐ Good	
3	Flow Effects on Accuracy	☐ High	☐ Moderate	☐ Slight	
4	Gravity & Pressurized Flow Operation	☐ No		☐ Yes	
5	Submergence or Backwater Effects	☐ High	☐ Moderate	☐ Low	
6	Effect of Solids Movement	☐ High	☐ Moderate	☐ Slight	
7	Flow Obstruction	☐ High	☐ Moderate	☐ Slight	
8	Head Loss	☐ High	☐ Medium	☐ Low	
9	Manhole Operation	☐ Poor	☐ Fair	☐ Good	
10	Power Requirements	☐ High	☐ Medium	☐ Low	
11	Site Requirements	☐ High	☐ Moderate	☐ Slight	
12	Installation Restrictions or Limitations	☐ High	☐ Moderate	☐ Slight	
13	Simplicity and Reliability	☐ Poor	☐ Fair	☐ Good	
14	Unattended Operation	☐ No		☐ Yes	
15	Maintenance Requirements	☐ High	☐ Medium	☐ Low	
16	Adverse Ambient Effects	☐ High	☐ Moderate	☐ Slight	
17	Submersion Proof	☐ No		☐ Yes	
18	Ruggedness	☐ Poor	☐ Fair	☐ Good	
19	Self Contained	☐ No		☐ Yes	
20	Precalibration	☐ No		☐ Yes	
21	Ease of Calibration	☐ Poor	☐ Fair	☐ Good	
22	Maintenance of Calibration	☐ Poor	☐ Fair	☐ Good	
23	Adaptability	☐ Poor	☐ Fair	☐ Good	
24	Cost	☐ High	☐ Medium	☐ Low	
25	Portability	☐ No		☐ Yes	

Comments:

TABLE 3. FLOWMETER EVALUATION SUMMARY

	Range	Accuracy	Flow Effects on Accuracy	Gravity & Pressurized Flow Operations	Submergence or Backwater Effects	Effect of Solids Movement	Flow Obstruction	Head Loss	Manhole Operation	Power Requirements	Site Requirements	Installation Restrictions or Limitations	Simplicity and Reliability	Unattended Operation	Maintenance Requirements	Adverse Ambient Effects	Submersion Proof	Ruggedness	Self Contained	Precalibration	Ease of Calibration	Maintenance of Calibration	Adaptability	Cost	Portability
Gravimetric-all types	G	G	H	Y	L	H	H	H	P	M	H	H	P	Y	H	M	-	F	Y	Y	G	F	-	H	N
Volumetric-all types	P	G	H	Y	L	H	H	M	P	L	H	H	F	Y	H	M	-	F	Y	Y	G	F	-	H	N
Venturi Tube	P	G	S	N	L	S	S	L	P	L	H	H	G	Y	M	M	-	G	Y	Y	G	G	-	H	N
Dall Tube	P	G	S	N	L	M	S	L	P	L	H	M	G	Y	M	M	-	G	Y	Y	G	F	-	H	N
Flow Nozzle	P	G	S	N	L	S	S	M	P	L	H	M	G	Y	L	M	-	G	Y	Y	G	G	-	M	N
Orifice Plate	P	F	S	N	L	H	H	H	P	L	H	S	G	Y	H	M	-	F	Y	Y	G	P	-	L	Y
Elbow Meter	P	F	S	N	L	S	S	L	P	L	H	S	G	Y	L	M	-	G	Y	N	F	G	-	L	N
Slope Area	F	P	H	N	H	S	S	L	G	L	M	S	G	Y	L	M	-	G	Y	N	F	G	-	H	N
Sharp-Crested Weir	F	F	M	N	M	H	H	H	F	L	M	M	G	Y	H	M	-	G	Y	Y	G	P	-	L	Y
Broad-Crested Weir	F	F	S	N	H	H	M	M	G	L	M	M	G	Y	L	M	-	G	Y	N	F	F	-	L	N
Subcritical Flume	F	F	S	N	L	S	S	L	F	L	M	S	G	Y	L	M	-	G	Y	Y	G	G	-	M	N
Parshall Flume	G	F	S	N	M	S	S	L	F	L	M	M	G	Y	L	M	-	G	Y	Y	G	G	-	M	Y
Palmer-Bowlus Flume	F	F	S	N	M	S	S	L	G	L	S	S	G	Y	L	M	-	G	Y	Y	G	G	-	L	Y
Diskin Device	F	F	S	N	M	M	H	L	G	L	S	S	G	N	H	H	-	F	Y	Y	F	F	-	L	Y.
Cutthroat Flume	G	F	S	N	L	S	S	L	P	L	S	S	G	Y	L	M	-	G	Y	Y	G	G	-	L	N
San Dimas Flume	G	F	S	N	L	S	S	L	F	L	S	S	G	Y	L	M	-	G	Y	Y	G	G	-	L	N
Trapezoidal Flume	G	F	S	N	L	S	S	L	F	L	S	S	G	Y	L	M	-	G	Y	Y	G	G	-	L	N
Type HS, H & HL Flume	G	F	S	N	H	M	S	H	G	L	M	M	G	Y	M	M	-	G	Y	Y	G	F	-	L	Y
Open Flow Nozzle	G	F	S	N	H	M	S	H	G	L	M	M	G	Y	M	M	-	G	Y	Y	G	F	-	L	Y
Float Velocity	G	P	H	N	L	S	S	L	G	L	S	S	G	N	L	H	-	G	N	-	-	-	-	L	Y
Tracer Velocity	F	F	M	Y	L	S	S	L	G	M	S	S	F	Y	M	S	-	F	N	N	G	G	-	H	Y
Vortex Velocity	P	F	S	N	L	H	H	L	P	L	H	H	F	Y	H	S	-	P	Y	Y	F	F	-	H	N
Eddy-Shedding	F	F	S	Y	L	M	M	L	G	L	S	S	F	Y	M	S	-	F	Y	Y	G	F	-	M	Y
Turbine Meter	P	F	S	N	L	H	H	M	P	L	H	M	F	Y	H	S	-	F	Y	Y	G	F	-	H	N
Rotating-Element Meter	F	F	S	Y	L	H	M	L	F	L	S	S	G	N	H	H	-	G	N	Y	G	G	-	L	Y
Vane Meter	P	F	S	N	L	M	H	L	F	L	S	M	G	Y	M	M	-	G	Y	Y	F	F	-	L	N
Hydrometric Pendulum	P	P	S	N	L	M	M	L	G	L	S	S	G	N	L	H	-	G	N	Y	F	F	-	L	Y
Target Meter	P	F	S	N	L	M	H	M	P	M	S	M	F	Y	H	S	-	P	Y	Y	G	F	-	H	N
Force-Momentum	P	G	S	N	L	M	M	L	P	H	H	H	P	Y	H	S	-	P	Y	Y	G	G	-	H	N
Hot-Tip Meter	F	P	S	Y	L	H	M	L	F	M	M	M	F	Y	H	M	-	F	Y	Y	G	F	-	H	N
Boundary Layer Meter	G	G	S	Y	L	S	S	L	P	M	M	M	F	Y	M	S	-	G	Y	Y	G	G	-	H	N
Electromagnetic Meter	F	G	S	Y	L	S	S	L	P	H	M	M	F	Y	M	S	-	F	Y	Y	G	G	-	H	N
Acoustic Meter	G	G	S	Y	L	M	S	L	F	M	M	M	F	Y	M	S	-	F	Y	Y	G	G	-	H	N
Doppler Meter	P	G	S	Y	L	H	S	L	F	M	M	M	F	Y	M	S	-	F	Y	Y	G	G	-	H	N
Optical Meter	F	P	S	N	L	S	S	L	F	L	S	S	G	N	L	H	-	G	N	Y	G	G	-	L	Y
Dilution	G	G	M	Y	L	S	S	L	G	M	S	S	F	Y	M	S	-	F	N	N	G	G	-	H	Y

Legend:

F - Fair	N - No
G - Goal	P - Poor
H - High	S - Slight
L - Low	Y - Yes
M - Medium or Moderate	

to the clever reader who has made a particular device work satisfactorily in such an application and, hence, feels that it has been treated unfairly.

Only a few of the evaluation parameters normally have numbers associated with them. To assist the reader in interpreting the ratings, the following general guidelines were used. If the normal range of a particular device was considered to be less than about 10:1, it was termed poor; if it was considered to be greater than around 100:1, it was termed good. The intermediate ranges were termed fair. The accuracy that might reasonably be anticipated in measuring storm or combined sewer flows was considered rather than the best accuracy achievable by a particular device. For example, although a sharp-crested weir may be capable of achieving accuracies of ±1.5% or better in clear irrigation water flows, accuracies of much better than ±4-7% should not necessarily be anticipated for a sharp-crested weir measuring stormwater or combined sewer discharges. If the accuracy of a particular flow measuring device or method was considered to be better than around ±1-2%, it was termed good; if it was considered to be worse than around ±10%, it was termed poor. The intermediate accuracies were termed fair.

The flow measuring devices and techniques were not rated on two evaluation parameters, submersion proof and adaptability, because these factors are so dependent upon the design details of the secondary element selected by the user.

Discussion

In comparison with Table 3, Table 4 offers a different (and even more subjective) comparison of the primary devices or techniques just described. Each method is numerically evaluated in terms of its percent of achievement of several desirable characteristics. Dilution techniques as a class appear to be the most promising of all. In view of

TABLE 4. COMPARISON OF MOST POPULAR PRIMARY DEVICES OR TECHNIQUES

3411-1A

Primary Device or Technique	Desirable Characteristic (% of Achievement)								Comments
	Range	Uncalib. Accuracy	Head Loss	Free From Upstream Effects	Free From Downstream Effects	Solids Bearing Liquids	Portability	Unattended Operation	
Dilution	100	100	100	100	100	100	100	80	Especially useful as a calibration tool.
Acoustic (Open Channel)	100	100	100	60	90	95	80	100	Good in large flows but expensive.
Parshall Flume	90	95	80	90	80	90	70	100	Requires drop in floor.
Palmer Bowlus Flume	80	90	85	90	85	90	90	100	Good overall.
Current Meter	90	95	100	100	100	90	100	0	Results are very operator dependent.
Electromagnetic	50	100	100	100	100	100	0	100	Generally requires pressure flow
Acoustic (Pressure Flow)	100	100	100	60	90	95	0	100	Wetted transducers recommended.
Open Flow Nozzle	60	95	70	80	75	80	80	95	Good if head drop is available.
Sharp-Crested Weir	60	95	70	80	80	50	80	90	Will require frequent cleaning.
Flow Tube	50	100	95	40	100	95	0	100	Pressurized flow only.
Venturi Tube	20	100	90	70	100	90	0	100	Pressurized flow only.
Trajectory Coordinate	80	70	50	100	70	100	100	0	Requires free discharge.
Slope Area	80	50	100	20	100	100	100	0	Use as last resort.

the current state of the art, however, their usefulness is probably greatest as a tool for in-place calibration of other primary devices. They have also been extremely useful for general survey purposes and have found some application as an adjunct to other primary devices during periods of extreme flow such as pressurized flow in a conduit that is normally open channel.

Acoustic open channel devices are also quite promising; but, because of their dependency upon the velocity profile and the frequently resulting requirement for several sets of transducers, they are presently only justifiable for very large flows in view of the expense involved. The usefulness of the Parshall flume is evidenced by its extreme popularity. The requirement for a drop in the flow is a disadvantage, and submerged operation may present problems at some sites. Known uncertainties in the head-discharge relations (possibly up to 5%) together with possible geometric deviations make calibration in place a vital necessity if high accuracy is required. Palmer-Bowlus type flumes are very popular overall. They can be used as portable as well as fixed devices in many instances, are relatively inexpensive, and can handle solids in the flow without great difficulty.

All point velocity measuring devices have been lumped together in the current meter category. In the hands of a highly experienced operator, good results can be obtained (the converse is also true, unfortunately), and they are often used to calibrate primary devices in place or for general survey work. They are generally not suited for unattended operation in storm and combined sewer flows, however.

Electromagnetic flowmeters show considerable promise where pressurized flow is assured as do closed pipe acoustic devices. Neither can be considered portable if one requires that the acoustic sensors be wetted, a recommended practice for most wastewater applications.

Open flow nozzles and sharp-crested weirs are often used where the required head drop is available. Weirs will require frequent cleaning and are best used as temporary installations for calibration purposes. Flow tubes and venturis are only suitable for pressurized flow sites such as might be encountered, for example, at the entrance to a treatment plant.

Trajectory coordinate techniques, such as the California pipe or Purdue methods, require a pipe discharging freely into the atmosphere with sufficient drop to allow a reasonably accurate vertical measurement to be made, a situation not often encountered in storm or combined sewers. Slope area methods, as explained earlier, must generally be considered as producing estimates only, and consequently should be considered as the choice of last resort (despite their apparent popularity).

REVIEW OF COMMERCIALLY AVAILABLE EQUIPMENT

The number of commercial firms that offer flow measuring equipment in the marketplace today is astoundingly large. Even when attention is limited to devices intended for liquid flowmetering, the number is still extremely large, probably well in excess of two hundred. Many manufacturers offer more than one type of primary device (and these typically in numerous models), and when combined with secondary device choices, the number is virtually overwhelming. Thus, no attempt to cover all available equipment can be made here, and this explains the great amount of space devoted to the generic descriptions in the preceding subsection. We will content ourselves here to note that two or more firms offer all devices that were described except for sharp-crested weirs, which are usually fabricated directly by (or for) the user in accordance with specifications for the particular measuring site.

The firms offering flow measuring equipment as at least a part of their product line range from very large, well known manufacturers that have offered a wide range of flow measuring equipment for over a century to relatively small organizations with a limited product line which has only recently been introduced. This latter category should not be excluded from consideration solely because of their seemingly novitiate status. The principals involved frequently have many years of experience, and their designs often reflect the most up-to-date expressions of the state of the art.

The revolution in the electronics industry, especially as regards solid-state designs and integrated circuitry, has not gone unnoticed by most flowmeter manufacturers; and as a result, many new, sophisticated secondary devices have recently appeared, and older equipment is frequently being upgraded in design to reflect the more modern technologies. Furthermore, many of these new secondary devices are of digital (rather than analog) design and are frequently computer compatible as supplied, offering tremendous possibilities for system structure.

A listing, by no means complete, of some manufacturers who offer flow measuring equipment in the categories listed in Table 4 is presented in Table 5. Under the heading "Company", the name, address, and telephone number has been provided. Under the heading "Products", only those products bearing on the flow measurement categories of Table 4 have been listed, even though the particular company may have a much more extensive flow measurement product line. The product emphasis was placed on primary devices, with secondary devices (in the form of level gages) indicated only where they are offered as "flowmeters". It can be generally assumed that each manufacturer offers a complete line of secondary elements for use with his primary devices.

Table 5 can be used to obtain direct, up-to-date information on all of the types of equipment discussed from at least two suppliers. Reference can be made to Shelley and Kirkpatrick (1975) for descriptions of the offerings of these and a number of other manufacturers.

TABLE 5. SOME FLOW MEASUREMENT EQUIPMENT MANUFACTURERS

COMPANY	PRODUCTS
AMERICAN CHAIN AND CABLE COMPANY, INC. ACCO BRISTOL DIVISION WATERBURY, CONNECTICUT 06720 TELEPHONE (203) 756-4451	Combination depth and velocity measuring device in a single unit
BADGER METER, INC. INSTRUMENT DIVISION 4545 WEST BROWN DEER ROAD MILWAUKEE, WISCONSIN 53223 TELEPHONE (414) 355-0400	Flow tubes, open flow nozzles, Parshall flumes
BADGER METER, INC. PRECISION PRODUCTS DIVISION 6116 EAST 15TH STREET TULSA, OKLAHOMA 74115 TELEPHONE (918) 836-4631	Acoustic (open channel)
BIF - A UNIT OF GENERAL SIGNAL 1600 DIVISION ROAD WEST WARWICK, R.I. 02893 TELEPHONE (401) 885-1000	Flow tubes, open flow nozzles, Parshall flumes, "universal" venturi tubes
BROOKS INSTRUMENT DIVISION EMERSON ELECTRIC COMPANY 407 WEST VINE STREET HATFIELD, PENNSYLVANIA 19440 TELEPHONE (215) 247-2366	Electromagnetic
CONTROLOTRON CORPORATION 176 CONTROL AVENUE FARMINGDALE, L.I., NEW YORK 11735 TELEPHONE (516) 249-4400	Acoustic (pressure flow)
CUSHING ENGINEERING INC. 3364 COMMERCIAL AVENUE NORTHBROOK, ILLINOIS 60062 TELEPHONE (312) 564-0500	Electromagnetic
C.W. STEVENS, INC. P. O. BOX 619 KENNETT SQUARE, PENNSYLVANIA 19348 TELEPHONE (215) 444-0616	Acoustic level gage
DREXELBROOK ENGINEERING COMPANY 205 KEITH VALLEY ROAD HORSHAM, PENNSYLVANIA 19044 TELEPHONE (215) 674-1234	Electronic level gage
ENVIRONMENTAL MEASUREMENT SYSTEMS A DIVISION OF WESMAR 905 DEXTER AVENUE NORTH SEATTLE, WASHINGTON 98109 TELEPHONE (206) 285-1621	Acoustic (open channel)
EPIC INC. 150 NASSAU STREET SUITE 1430 NEW YORK, NEW YORK 10038 TELEPHONE: (212) 349-2470	Current meters, level gages
FISCHER & PORTER CO. WARMINSTER, PENNSYLVANIA 18974 TELEPHONE (215) 675-6000	Electromagnetic, flow tubes, open flow nozzles, Parshall flumes, level gages
CARL FISHER AND COMPANY DIVISION OF FORMULABS, INC. 529 WEST FOURTH AVENUE P. O. BOX 1056 ESCONDIDO, CALIFORNIA 92025 TELEPHONE (714) 745-6423	Fluorescent dyes
FLUMET CO. P. O. BOX 575 WESTFIELD, NEW JERSEY N.Y. OFFICE: TELEPHONE (212) 227-6668	Palmer-Bowlus flumes
THE FOXBORO COMPANY FOXBORO, MASSACHUSETTS 02035 TELEPHONE (617) 543-8750	Electromagnetic, level gages
HINDE ENGINEERING COMPANY OF CALIFORNIA P. O. BOX 56 SARATOGA, CALIFORNIA 95070 TELEPHONE (408) 378-4112	Palmer-Bowlus flumes
INTEROCEAN SYSTEMS, INC. 3510 KURTZ STREET SAN DIEGO, CALIFORNIA 92110 TELEPHONE (714) 299-4500	Current meters, level gages
KAHL SCIENTIFIC INSTRUMENT CORPORATION P. O. BOX 1166 EL CAJON, CALIFORNIA 92022 TELEPHONE: (714) 444-2158	Current meters, fluorescent dyes
F. B. LEOPOLD COMPANY DIVISION OF SYBRON CORPORATION 227 S. DIVISION ST. ZELIENOPLE, PENNSYLVANIA 16063 TELEPHONE (412) 452-6300	Open flow nozzles, Palmer-Bowlus flumes, Parshall flumes
LEUPOLD & STEVENS, INC. P. O. BOX 588 600 N.W. MEADOW DRIVE BEAVERTON, OREGON 97005 TELEPHONE (503) 646-9171	Float level gages
MANNING ENVIRONMENTAL CORP. 120 DU BOIS STREET P. O. BOX 1356 SANTA CRUZ, CALIFORNIA 95061 TELEPHONE (408) 427-0230	Acoustic and "dipper" level gages
MARTIG BUB-L-AIR 2116 LAKEMOOR DRIVE OLYMPIA, WASHINGTON 98502 TELEPHONE (206) 943-2390	Bubbler level gage
METRITAPE, INC. 77 COMMONWEALTH AVENUE WEST CONCORD, MASSACHUSETTS 01742 TELEPHONE (617) 369-7500	Electronic level gage
NB PRODUCTS, INC. 35 BEULAH ROAD NEW BRITAIN, PENNSYLVANIA 18901 TELEPHONE (215) 345-1879	Portable V-notch weirs, level gages

TABLE 5. SOME FLOW MEASUREMENT EQUIPMENT MANUFACTURERS (Continued)

COMPANY	PRODUCTS
N-CON SYSTEMS COMPANY 308 MAIN STREET NEW ROCHELLE, NEW YORK 10801 TELEPHONE (914) 235-1020	Float and "dipper" level gages
NUSONICS, INC 9 KEYSTONE PLACE PARAMUS, NEW JERSEY 07652 TELEPHONE (201) 265-2400	Acoustic (pressure flow)
OCEAN RESEARCH EQUIPMENT, INC. FALMOUTH, MASSACHUSETTS 02541 TELEPHONE (617) 548-5800	Acoustic (open channel)
THE PERMUTIT COMPANY DIVISION OF SYBRON CORPORATION E49 MIDLAND AVENUE PARAMUS, NEW JERSEY 07652 TELEPHONE (201) 262-8900	Flow tubes, open flow nozzles, Parshall flumes, venturi tubes
PLASTI-FAB, INC. 11650 S.W. RIDGEVIEW TERRACE BEAVERTON, OREGON 97005 TELEPHONE (502) 644-1428	Palmer-Bowlus flumes, Parshall flumes, V-notch weir boxes
POLCON, INC. AN AFFILIATE OF CARL F. BUETTNER & ASSOCIATES, INC. 5106 HAMPTON AVENUE ST. LOUIS, MISSOURI 63109 TELEPHONE (314) 353-5993	Open channel flow tube
PORTAC MIN-ELL COMPANY, INC. 1689 BLUE JAY LANE CHERRY HILL, NEW JERSEY 08003 TELEPHONE (609) 429-0421	Current meter flow tube
ROBERTSHAW CONTROLS COMPANY P. O. BOX 3523 KNOXVILLE, TENNESSEE 37917 TELEPHONE (615) 546-0524	Parshall flumes, level gages
SARATOGA SYSTEMS, INC. 10601 SOUTH SARATOGA-SUNNYVALE ROAD CUPERTINO, CALIFORNIA 95014 TELEPHONE (408) 247-7120	Acoustic (pressure flow)
SCARPA LABORATORIES, INC. 46 LIBERTY STREET, BRAINY BORO STATION METUCHIN, NEW JERSEY 08840 TELEPHONE (201) 549-4260	Acoustic (pressure flow)
SIGMAMOTOR, INC. 14 ELIZABETH STREET MIDDLEPORT, NEW YORK 14105 TELEPHONE (716) 735-3616	Bubbler level gage
SINGER-AMERICAN METER DIVISION 13500 PHILMONT AVENUE PHILADELPHIA, PENNSYLVANIA 19116 TELEPHONE (215) 673-2100	Palmer-Bowlus flumes, Parshall flumes, level gages
SIRCO CONTROLS COMPANY 8815 SELKIRK STREET VANCOUVER 14, BRITISH COLUMBIA, CANADA TELEPHONE (604) 261-9321	Acoustic level gage
TAYLOR SYBRON CORPORATION TAYLOR INSTRUMENT PROCESS CONTROL DIVISION TELEPHONE (716) 235-5000	Electromagnetic
TRI-AID SCIENCES, INC. 161 NORRIS DRIVE ROCHESTER, NEW YORK 14610 TELEPHONE (716) 461-1660	Acoustic level gage
UNIVERSAL ENGINEERED SYSTEMS, INC. 7071 COMMERCE CIRCLE PLEASANTON, CALIFORNIA 94566 TELEPHONE (415) 462-1543	Palmer-Bowlus flumes
VICKERY-SIMMS, INC. P. O. BOX 459 ARLINGTON, TEXAS 76010 TELEPHONE (817) 261-4446	Parshall flumes, venturi
WALLACE-MURRAY CORPORATION CAROLINA FIBERGLASS PLANT P. O. BOX 580 510 EAST JONES STREET WILSON, NORTH CAROLINA 27893 TELEPHONE (919) 237-5371	Parshall flumes
WESMAR INDUSTRIAL SYSTEMS DIVISION 905 DEXTER AVENUE NORTH SEATTLE, WASHINGTON 98109 TELEPHONE (206) 285-2420	Acoustic level gages
WESTINGHOUSE ELECTRIC CORPORATION OCEANIC DIVISION P. O. BOX 1488, MAIL STOP 9R30 ANNAPOLIS, MARYLAND 21404 TELEPHONE (301) 765-5658	Acoustic (open channel)

In these days of inflation, little can be said about equipment costs except in a very cursory fashion. Recording level gages generally start around \$1K, and a full system for measuring flows at a difficult site could well run \$100K. Construction, installation, and (importantly) projected maintenance and repair costs must be considered in addition to equipment acquisition costs to arrive at true cost of ownership, which is the only real basis for comparison.

In closing this discussion of commercially available equipment, it must be noted that none of the devices completely measures up to the desirable equipment characteristics listed earlier, and much remains to be done before such an "ideal" device is available.

REVIEW OF RECENT FIELD EXPERIENCE

A brief review of flow measurement experiences from recent projects in the storm and combined sewer area will be given to allow a better appreciation of the application of some of the flow measuring devices and techniques in an actual field setting. The projects are reviewed in a generally chronological order, starting with the oldest (circa 1966) and coming up to the present time. The project title will be used for identification purposes. In some instances final reports have been issued, while in others project files or interviews with project engineers formed the source of the information presented.

Characterization and Treatment of Combined Sewer Overflows - was a study whose general objectives were: (a) to develop workable systems to manage overflows from the combined sewers of San Francisco, thereby alleviating pollution and protecting beneficial uses of local receiving waters, and (b) to provide the rationale and methodology for controlling pollution from combined sewer overflows in other metropolitan areas of the United States. Data collection for the project included measurement of dry weather flows and storm overflows of the Selby Street and Laguna Street trunk sewers. Monitoring included measurement of rainfall and discharge as well as the quality characteristics of the overflows. The tracer dilution method was selected for use in measurement of dry weather flows. Pontacyl Brilliant Pink B fluorescent dye was used for the tracer, and quantitative analyses for the tracer were made with a Turner Model 110 Fluorometer.

Although successful at the Laguna Street site, use of the dilution method did not prove satisfactory for measurement of the Selby Street overflow. Uneven distribution of the tracer when injected resulted from exposed sludge banks, and there was insufficient turbulence for adequate mixing of the dye. Because of the resulting lack of reliable data, a Palmer-Bowlus flume with a 1.22m (4 ft) throat and 15 cm (6 in.) invert slab, constructed from 16 gauge galvanized sheet metal was installed. A continuous record of the upstream water level was obtained by mounting a Stevens water level recorder, operated by float, in a stilling well constructed of sandbags.

Because of generally unsatisfactory conditions, several methods were used for the measurement of storm flow in the Selby Street outfall structure. These were:

a. Velocity determination with current meters at a point 15m (50 ft) above the outfall structure. Not considered to yield reliable data as the meters were immediately fouled with rags and other debris.

b. Differential head measurements over the broadcrested weirs of the outfall structure. Because of expected interference by tide gates, the theoretical head-discharge relationship for a broad-crested weir of similar shape was used for comparison purposes only.

c. Measurement of surface velocities in the outfall structure by timing the traverse of styrofoam floats across a measured control section. A factor of 0.64 was applied to surface velocities to convert to average velocity, thus accounting for both horizontal and vertical velocity distributions.

d. Measurement of vertical velocity profiles in the outfall structure with an especially designed current meter having low velocity sensitivity. This was to establish discharge values under low head conditions and to check the results of the surface velocity determinations.

Water levels in the outfall structure were continuously measured by means of a "bubbler" gage. A rating curve was developed from the results of the surface velocity determinations and the current meter measurements in the outfall structure. A theoretical curve computed from the broadcrested weir formula was approximately 10% larger than this developed curve. Flow determinations in the Laguna Street overflow were made by measuring the depth of flow at the outfall sewer and calculating the discharge by means of the Manning equation. Use of the Manning equation was said to be justified because the slope of the outfall sewer is known, and a uniform reach extends about 210m (700 ft) upstream from the point at which depths of flow were determined.

Engineering Investigation of Sewer Overflow Problem - Roanoke, Virginia - included investigations of approximately 25 percent of Roanoke, Virginia's separate sanitary sewer system concerning the amounts of infiltration for various storm intensities and durations and the amounts of sewage overflow from the system. Flows in three sanitary sewer interceptors, and streams draining the corresponding basins, were gaged during storm events to measure infiltration and runoff quantities to establish their relation to rainfall intensities and durations. After significant variation in dry weather flows was observed, continuous monitoring of flows in the interceptors was maintained. Overflows bypassing the water pollution control plant were measured during rainfall events.

Sharp-crested weirs were used to measure flows in two of the streams. In the third stream, a stage-discharge curve was developed from the Manning formula using the measured hydraulic characteristics of the stream. In two of the interceptor sewers, and in the water pollution control plant overflow, a stage-discharge curve based on the Manning formula and the measured hydraulic characteristics of the sewer were used to convert depth measurements to flow estimates. In the third interceptor sewer, dry weather flows were estimated using the Manning formula, but during rainfall events the sewer became surcharged and overflow was measured by means of a weir installed in the side of the manhole wall.

No problems with use of the streamflow measurement devices and methods were noted. However, accuracy of measurements with use of the Manning formula in the natural stream channel photographed must be considered very poor. A photograph of one of the weirs used for streamflow measurement shows a significant accumulation of trash and debris on the weir. Under this condition, accuracy must be considered poor. Hydrographs indicated that two of the interceptor sewers were surcharged during many of the storms, and a record of discharge was not obtained during those periods.

Water levels at the gaging sites were recorded by means of six float-actuated, continuous water-level recorders manufactured by the Instruments Corporation (now a part of Belfort Instrument Company), and one pressure type recorder manufactured by the Bristol Company. After the float-actuated recorders were serviced and supplied with an expanded time scale, they performed satisfactorily for the duration of the program. During dry-weather periods, the bubbler pipe in the Bristol recorder collected debris and required frequent cleaning. Because of this maintenance problem, it was replaced with float-actuated, continuous water-stage recorder.

Combined Sewer Overflow Abatement Alternatives - Washington D.C. - was a project whose objectives were to: (a) define the characteristics of urban runoff in the subject area, (b) investigate the feasibility of high-rate filtration for treatment of combined sewer overflow; and (c) develop and evaluate alternative methods of solution. Investigative activities included field monitoring of combined sewer overflows at two sites, and of separated stormwater discharges at one site. The monitoring program was conducted over a period of about six months, April 1 to September 23, 1969.

Selection of a satisfactory technique for flow measurement presented a problem. Weirs were not used because backwater elevations would have caused surcharging and flooding at the expected high flow rates. Depth of flow measurements with the use of a steady state empirical equation such as the Manning equation for calculating flow were not used because flow conditions were not steady state during periods of storm runoff. The method selected was use of lithium chloride as a tracer in a procedure similar to that of the salt dilution method (continuous injection type). Use of a lithium salt is said to improve the technique because the background of lithium in wastewater is usually low and because lithium concentrations at fractional parts per million levels can be accurately and conveniently determined by atomic absorption or flame emission spectroscopy. The slope-area method was used as a check.

A number of difficulties experienced in use of the equipment resulted in loss of flow record during several major storms. Greatest trouble was in clogging or damage to the submersible pump used to collect samples of wastewater required for measuring lithium chloride concentration. Flooding of one of the lithium chloride release stations caused damage to the bubbler instruments used to measure water stage, to the lithium chloride release system, and to other equipment. Flow rate estimates based on depth-of-flow measurements and the Manning formula were compared with results of the tracer method. Only a very general correlation with significant spread resulted. Large differences in flow were attributed to inaccurate measurement of depth of flow and the assumption of steady-state conditions inherent in Manning's formula.

Urban Runoff Characteristics - concerned investigations of a combined sewer watershed in Cincinnati, Ohio for the refinement of the US EPA Storm Water Management Model. Flows at three sites were monitored. At two of the sites, flow was actually measured in two tributary sewers immediately upstream from their junction. Third site was at the outlet of the watershed; thus, five sets of flow measuring equipment were used. The flow measuring equipment essentially consisted of a compressor, a manometer, and a pressure-type recorder. This recorder operated by measuring the pressure due to the depth and velocity of water flowing into a pipe by bubbling air through a long tube inserted into the water. The gage released air is at a constant pressure, and as the depth and the velocity of flow changed, the pressure differential was recorded on a circular chart in inches of water.

To obtain the discharge corresponding to the measured specific energy, Manning's equation was used to express the depth and the velocity of flow as functions of the discharge and the hydraulic radius of the sewer cross sections. Thus, curves have been calculated relating the measured specific energy and the corresponding discharge at the five measured locations.

Apparently, the value of slope used in the Manning formula was that of the sewer line at each of the five measuring sites. A photograph in the report shows a heavy, metal, top-hanging gate at the outlet of the sewer watershed. The outlet flow monitoring site is described as about 6m (20 ft) upstream from this gate. If this is the case, flow past the site probably would not be uniform, and the Manning formula would not be applicable. Flows in the two pairs of sewers just upstream from their junction with two sewers to be monitored would be controlled by the slopes of each of the two monitored sewers rather than the slopes of the four tributary sewers immediately upstream, which were the slopes used in the Manning formula to compute flow. In any case, water surface slope is more properly used with the Manning formula than is the sewer slope. Plotting of storm hydrographs for the measured sites discloses a number of serious inconsistencies in the data.

Storm and Combined Sewer Pollution Sources and Abatement - Atlanta, Georgia - was a study of six urban drainage basins within the City of Atlanta, Georgia, served by combined and separate sewers, to determine the major pollution sources during storm events. Rainfall frequency analysis and simulation techniques were utilized to obtain design criteria for alternative pollution abatement schemes. Measurements of flow were made of three major overflows, as well as of the interceptor sewers originating at these points of overflow. Three branches of South River were measured, as was the South River main stream at four points. A bypass at the interceptor entering one of the wastewater treatment plants was measured.

Data were collected from January 1969 until April 1970. Continuous flow monitoring was maintained at the river and its tributary stations, and in the interceptors where dry weather flow characteristics were of interest. Event monitoring only was conducted of overflows and of the bypass to the treatment plant. Rating curves for all gaging stations were developed by stage-discharge measurements with current meters. Either Price Type AA or Pigmy Type current meters were used. Some discharge measurements were verified by alternate methods or formulas, but results of these verifications were not given.

Stevens Type F water level recorders were used throughout. Gaging stations were reported to be constructed in accordance with established U.S. Geological Survey practice. For flow level recording at interceptors, scow floats were installed at manholes a short distance downstream from regulators. Although probable accuracy of the records collected was not reported, an indication of their accuracy is available, based on one of the gaging stations in the project which was operated by the U.S. Geological Survey. This station, having a drainage area of 3.86 sq km (1.49 sq mi), had been operated since October 1963, or for more than six years. The greatest flow measured by current meter during the period was 5947 ℓ/s (210 cfs), but the rating curve was extended to 23,220 ℓ/s (820 cfs) by computation of flow through a culvert. Records at the station are stated by the USGS as poor, with no quailification.

Stormwater Problems and Control in Sanitary Sewers - Oakland and Berkeley, California - was an engineering investigation conducted on stormwater infiltration into sanitary sewers and associated problems in the East Bay Municipal Utility District, Special District No. 1, with assistance from the cities of Oakland and Berkeley, California. Rainfall and sewer flow data were obtained in selected study sub-areas that characterized the land used patterns predominant in the study area. Results obtained were extrapolated over large areas. Palmer-Bowlus flumes were installed at three of the ten metering stations established specifically for the study. These flumes were constructed of stainless steel and were designed to fit the respective sizes and shapes of the sewers. They were mounted in the outlet sewer from the manholes so that head measurements could be made at the proper distances upstream from the throats. Wooden channel extensions through the manholes were installed to prevent water spreading out in the manhole as depth in the sewer increased.

At seven of the new metering stations, 90-degree, V-notch weirs were installed. These were constructed of marine plywood and covered for additional water resistance with two coats of polyurethane finish. For ease of installation, a channel closure was provided so that it could be easily slipped down into the flowing sewage and quickly bolted in position after installing and sealing the actual weir plate. Stevens type 2A35 water stage recorders were used at three weir locations where submergence of the weir was anticipated. These recorders were selected for the ability to record two liquid levels simultaniously, upstream and downstream from the weir. Taylor pressure recorders were installed at the other seven new metering stations. With these recorders, liquid level sensing consists of measuring the back pressure from a continuously purging nitrogen gas bubble system. In several cases, equipment was installed in manholes near the centers of streets, thus complicating the installation and use of equipment. Otherwise, no problems were noted with the use of the equipment at the newly established stations.

The flow rate at two pumping stations was determined by means of a system-head curve for the station which gives the discharge rate for each pump or combination of pumps. A recording ammeter was attached to the pump electrical leads to indicate the total number of pumps running at any given time. Relief overflow at one of these pumping stations was measured by means of a broad-crested weir and a wet-well liquid level recorder. A third pumping station was equipped with both a wet-well liquid level recorder and a flow recorder. The primary device for the flow recorder was a venturi tube mounted on the discharge manifold of the pumps. The influent pumping station at the water pollution control plant was equipped with individual flowmeters on each pump discharge which were connected to a combined flow recorder. The primary devices for the flow recorder were discharge weirs in the grit chambers which reflect the respective pump discharge rates.

A pumping station relief overflow structure was provided with a flow measuring device for measuring the volume of water that overflowed. The flow measuring arrangement consisted of measuring the liquid levels on both sides of a rectangular tide gate and extracting the flows from the manufacturers rating curve. Because of difficulties in installation and operation, no usable flow measurements were obtained during periods of overflow.

Dispatching System for Control of Combined Sewer Losses - concerned a regulator control system which is said to demonstrate impressive reductions in combined sewer overflow pollution of the Mississippi River in Minneapolis and Saint Paul. The project includes a computer-based data acquisition and control system that permits remote control of modified combined sewer regulators. Data from rain gages, regulator control devices, trunk sewers and interceptors, and river quality monitors provide real-time operating information.

Water surface elevations in the system were monitored at about 48 points by the installation of bubbler gages with telemetry units for transmitting data to a central location. This equipment provided information on the frequency and duration of overflows. Because flow rate was not measured, data on the volume of overflows were not thus determined.

Flow in each of the three Minneapolis interceptors at the Minniapolis-St. Paul city line was metered by dual venturi meters in each interceptor. This equipment, which was in use prior to the subject study, provides information on the effectiveness of the program to use the interceptors for temporary storage of combined sewage. Flow data in the interceptors served to provide a check on the accuracy of rainfall runoff modeling. Probable error of flow measurement by the venturi-meters was not discussed.

Preconstruction Evaluation of Combined Sewage Detention Facilities - was a lengthy study of combined sewer flows in Somersworth, New Hampshire prior to construction of detention facilities. In order to get reasonably accurate and reliable flow measurements a section of the outfall was replaced with a weir pit large enough to provide a fair amount of stilling behind the weir. This weir pit was 1.8 x 1.8 x 7.5m (6x6x24.75 ft), and the weir was located 5.6m (18.33 ft) from the inlet. Three different 0.63 cm (1/4 in.) thick steel plate rectangular weirs with crest lengths of 0.3, 1.2, and 1.7m (1, 4, and 5.6 ft) were used, the 1.2m (4 ft) one being employed for all but three months of the year-long study. The design was such that the different weirs could be changed easily in several minutes. The main difference in the 1.2m (4 ft) weir was that its crest was elevated 1.2m (4 ft) above the floor of the weir pit as opposed to 0.76m (2.5 ft) for the other two. This weir was constructed after initial operation with the 0.3 and 1.7m (1 and 5.6 ft) weirs, and observation of high flow rates using the 5.6' weir indicated that it was desirable to increase the stilling in the weir pit by raising the crest height.

Head measurements over the 0.3 and 1.7m (1 and 5.6 ft) weirs were made using an air operated Fischer and Porter recorder. A float operated transmitter and recorder manufactured by the Penn/Measure-Rite Division of Badger Meter Manufacturing Company was used for head measurements over the 1.2m (4 ft) weir. Both recorders were set up to use 24 hour, 30.5 cm (12 in.) diameter charts. The Fischer and Porter recorder charts had a range of 0-51 cm (0-20 in.) of head, and the Penn/Measure-Rite Recorder charts had a range of 0-76 cm (0-30 in.). Flow into the weir pit was such that some turbulence was created. This caused the flow recorders to print out head measurements in short vertical strokes instead of a smooth line, a condition that was corrected as much as possible by using the 1.2m (4 ft) weir. Sludge build-up behind the weirs did not appear to upset their hydraulic characteristics. However, before any sampling programs were undertaken, the sludge build-up was totally removed to obtain accurate chemical and biological characteristics of the combined sewage flow. The actual sludge build-up would occur within a couple of days after the installation of a weir. The majority of this sludge consisted of grit with a small percentage of organic matter.

Head measurements were converted to flows using "appropriate formulae for the rectangular weirs used in the weir pit." By this it is assumed that the Kindsvater-Carter equation was used rather than the Francis formula, which would require corrections for both less than standard contraction and velocity of approach much of the time. For example, at the maximum head of 0.6m (2 ft) on the 1.2m (4 ft) weir (any greater head would overflow the 1.8m deep weir pit) the Kindsvater-Carter equation indicates a discharge of around 91 MLD (24 MGD), whereas the uncorrected Francis formula yields approximately 83 MLD (22 MGD), a 10% difference. The maximum discharge for the 1.7m (5.6 ft) weir, as computed assuming the recorder's full 51 cm (20 in.) of head was used, is approximately 98 MLD (26 MGD). Maximum flow rates recorded with the 1.2m (4 ft) weir of from 99 to 148 MLD (26.2 to 39.0 MGD) are reported but not explained. It is possible that the 148 MLD (39.0 MGD) discharge was estimated but not so indicated. The point is that high accuracies should not necessarily be expected with the given installation at the higher flow rates.

Urban Storm Runoff and Combined Sewer Overflow Pollution - Sacramento, California - was a program to develop a general method for determining the extent of pollution resulting from stormwater runoff and combined sewer overflows occurring in an urban area, and the application of this method to the City of Sacramento, California. Combined sewage and stormwater in the system were characterized by collecting samples and measuring flows at each of 19 sampling locations during six wet weather periods. The intention was to collect (as nearly as practical) at the commencement of rainfall, three hours thereafter, and approximately 12 to 18 hours after the commencement of sampling. However, comparison with rainfall records indicated that the first data of each storm period were not collected until the time of maximum rainfall intensity, or later.

The wastewater flows were established at manhole sampling stations by use of the Manning formula. The coefficient of roughness was assumed to be 0.013, a design value used by the City of Sacramento Engineering Department. The value of S used was the measured slope of either the water surface or the invert. Because of difficulties in making the required measurements for determination of slope, flow data at three of the stormwater runoff sites are not considered to be reliable. None of the computed wastewater flows at manhole sampling stations was checked by means of flow measurement equipment. For design purposes, the peak stormwater runoff flow in each of the individual pipes comprising the Sacramento collection and conveyance system was estimated from rainfall records by use of the rational method. These estimated flows for a full pipe condition could be checked at three locations with computed flows at the manhole sampling stations. They differ with the computed flows by -6, +4, and -32 percent. This agreement is exceptional, particularly as these are the three locations where the computed flows are not considered to be reliable.

Storage and Treatment of Combined Sewer Overflows - was a project to demonstrate the feasibility and economic effectiveness of a combined wastewater overflow detention basin. Overflow from the combined sewer area to the detention basin was measured by a Palmer-Bowlus flume installed in a 198 cm (78 in.) reinforced concrete pipe. The flume was designed as outlined by Ludwig and Ludwig (1951). It was fabricated of steel, and installed in the concrete pipe as it was laid. Space between the flume and pipe was grouted. A Stevens Type A35 water-level recorder with a scow float was installed in the pipe to measure head on the flume. The chart time scale was 73.2 cm (28.8 in.) per day and the gage scale was 1:6. The volume discharged into the basin was computed from the recorded head on the flume and a rating curve for the flume, which was approximated in four linear sections. Discharges indicated by rainfalls on eleven different occasions were missed due to the recorder being out of order, amounting to eleven percent of the runoff events that were missed.

Overflow from the basin to the river was measured by a 6.7m (22 ft) long sharp crested weir located in the overflow structure. The weir head was measured by a Stevens Type A35 water-level recorder with a cylindrical float. The chart time scale was 24.4 cm (9.6 in.) per day and the gage scale was 1:6. The recorder was out of order during one of the three overflow events which occurred during the period of project operation.

Wastewater Flow Measurement in Sewers Using Ultrasound - concerns the use of newly-developed ultrasonic velocity measurement equipment in conjunction with ultrasonic level measurement equipment for the measurement for sewage flow. Each of two existing combined sewers in the Milwaukee, Wisconsin sewerage system, one 3.8m (12.5 ft) and the other 1.5m (5 ft) in diameter, were thus equipped initially. Subsequent discovery of an excessive amount of entrained air at the larger diameter sewer site necessitated the transferral of that equipment to a more favorable location upstream in a 3.7m (12 ft) diameter sewer. Performance of the ultrasonic meters was compared with other metering devices at each of the locations. Relationships between average volume flow, water level, and average velocity along selected horizontal chords of the sewer cross sections were determined. The unit installed in the samller sewer had operated for 18 months without failure and required only routine maintenance. Similarily, the relocated unit installed at the 3.7m (12 ft) sewer had operated without failure since its installation. No deterioration of the ultrasonic transducer probes had been detected, indicating their suitability for use in the sewer environment. The electronics of the ultrasonic velocity metering unit were modified to include peak protection, automatic gain control, and automatic trigger control to minimize the effects of variations in the solids loading. Further observations concerning use of the ultrasonic equipment are as follows:

a. Similar equipment has been used in Japan to successfully measure full pipe flow of return sludge with high solids loadings. For practical line diameters, say from 0.2 to 5m (0.5 to 16 ft), no operational limitations due to suspended solids would be expected.

b. Entrained air bubbles were found to cause operational problems because of dissipation of the ultrasonic pulse due to scattering by the bubbles. Therefore, it is recommended that measurement sites be selected which are reasonably free of severe upstream agitation that would cause air entrainment.

c. Difficulties with level gage performance resulted from standing ripples in the sewage surface which interfered with echo returns. This was alleviated by moving the level sensor a few feet to a point where the sewage surface was more still.

d. Comparative figures of flow as measured by the ultrasonic equipment with those measured by other metering devices are not given for the demonstration sites described.

e. Total system cost for each site was about $15,000. Future simplifications of the ultrasonic circuitry made possible through more extensive use of integrated circuits have the promise of reducing this system cost by a factor of two or three.

Biological Treatment of Combined Sewer Overflow - Kenosha, Wisconsin - concerns the design, construction, operation and two year evaluation of a biological process used for the treatment of potential combined sewer overflow. During 1970, while design and construction of the demonstration system facilities was occurring, a program to determine the quality of the combined sewer overflows in Kenosha was carried out.

This included measurement of rainfall, combined sewer overflow quantity and quality, and influent quality to the water pollution control plant during rainfalls.

Flow measurement equipment was installed in the outfall lines of three overflows, known as the 57th Street, 59th Street, and 67th Street overflows. Bubbler-type depth recorders were used. Depth-discharge relationships were developed for the three overflow lines by means of dye tests. However, no details of the test procedures were given. As a result of an unsuccessful attempt to correlate rainfall volumes with overflow volumes, it was disclosed that the overflow measuring devices were of questionable accuracy. In some cases, the volume of overflow measured exceeded the volume of rainfall over the combined sewer area. Apparently, the depth-discharge relationships were not accurate and so the overflow data were not used.

Measuring sites at the 57th Street and 59th Street overflows were abandoned in 1972. The depth recorder at the 67th Street site was moved upstream above a weir diverting flow to the interceptor sewer. The end of the bubble tube was placed just upstream from the overflow mechanism, and a formula for broad-crested weirs was used to convert level over the weir into flow rates and volumes. Flows computed in this manner were used to estimate the total volume of overflow to the demonstration treatment plant.

Surge Facility for Wet and Dry Weather Flow Control - concerns a 3-year demonstration project which encompassed the design, construction, operation, testing, and evaluation of a surge facility designed to provide flow equalization and some degree of treatment to all storm flows and to provide rate control of all wet weather and dry weather wastewater flows to interceptor sewers. The principle elements of the facility are a sedimentation-equalization basin, a clarifier, a storage pond, a chlorine contact basin, and a sludge digester. Flow and hydraulic measurements include: (a) influent to the sedimentation-equalization basin;

(b) underflow from the sedimentation-equalization basin to the clarifier and the storage pond; (c) overflow from the sedimentation-equalization basin directly to the storage pond; (d) water surface elevation of the sedimentation-equalization basin; and (e) effluent to the chlorine contact basin and the receiving stream.

The influent metering structure is a Parshall flume with a 0.30m (1 ft) throat width having a maximum capacity of approximately 30 MLD (8 MGD). The influent flow transmitter to the control building can be used to control a flow proportional sampler of the influent. Underflow from the sedimentation-equalization basin is measured with a 15.2 cm (6 in.) magnetic flowmeter. The underflow can be set at any desired rate up to 8.7 MLD (2.3 MGD). Overflows from the basin are measured by four sharp-crested rectangular weirs, totaling 2.54m (100 in.). Head on the weirs, and the water surface fluctuations in the sedimentation-equalization basin, are monitored with a Stevens Type F water-stage recorder. Time gears were selected to give an 8-day chart, and stage gears were selected to give an indication of 0.06m/cm (0.5 ft/in.) of chart or 0.012m/cm (0.1 ft/in.) of chart, depending on the depth variation expected during any particular testing period. Effluent from the facility is measured with a combination of 15.2 cm (6 in.) Parshall flume and a 137 cm (54 in.) sharp-crested rectangular weir. The two flow measuring devices are set to give a combined capacity of 22.7 MLD (6 MGD). The effluent flow transmitter can be used to control a flow proportional sampler of the effluent.

No discussion of problems with flow measurement equipment is given in the report. Flow data presented in the report appear to be complete and accurate.

Joint Construction Sediment Control Project - was a demonstration project concerned with sediment control. As part of the project, a gaging and sampling program was conducted to determine the effects of urbanization on storm runoff and water quality of natural areas. Four automatic flow gaging and water sampling stations were installed on small streams of the study area. Two of the stations were installed adjacent to each other on streams just upstream from their junction. One of these streams drains an experimental watershed and the other drains an adjacent reference watershed. Two other gaging and sampling stations were established immediately upstream and immediately downstream from a 1.6 Hectare (4 acre) pond. At three of the gaging sites, precalibrated, broad-crested, V-notch weirs developed and tested by the U.S. Department of Agriculture were installed. At two sites, the concrete weir caps have 2:1 side slopes; at the third site, side slopes of the weir cap are at 3:1. At the gaging site downstream from the four-acre pond, a sharp-crested, compound, 90°, V-notch and rectangular weir was installed.

A Stevens Duplex Water-Level Recorder Type 2A35 was used to simultaneously record water levels at the two adjacent stream sites. A Stevens Type A35 water level recorder was used upstream from the pond, and a Belfort liquid level recorder was used at the downstream site. A Gurley pygmy current meter was available to perform additional stream gaging, but was used primarily to check the calibration of the permanently installed weirs.

The weirs and level recording devices used are said to have proven to be accurate, reliable, and easy to maintain. However, it was found necessary to clean out sediment above the three U.S.D.A. weirs after each storm and sometimes under base flow conditions to maintain the calibration and accuracy of the weirs. No cleanout was required at the compound weir downstream from the pond. Because of the reported accumulation of sediment found after each storm, accuracy of runoff records at the three sites during periods of storm runoff may be questioned. An unknown pattern of alternate sediment accumulation and flushing during the storm runoff period could occur. Calibration may have been incorrect during the falling limb of the hydrograph, when sediment was accumulating.

Combined Sewer Overflow Abatement Plan, Des Moines, Iowa - was a project designed to provide engineering information regarding the volume, character, and impact of combined sewer overflows and urban stormwater discharges from a typical midwestern metropolitan area. Several different types of equipment and methods were used to measure flows and to provide secondary stage measurements. Weirs of the 90° V-Notch type and rectangular weirs, both suppressed and contracted, were used. At sites with larger volumes of flow, a stage-discharge relationship was determined using current meters. Natural controls of a permanent nature were found at these sites. The Manning formula, with water-surface slope and an "n" value of 0.018, was used to determine discharge in one conduit. The main outfall of the wastewater treatment plant was measured with a raw sewage flow totalizer of an undisclosed type. Stick gages painted with water soluble paint were used to detect overflow occurrences. Both Stevens Type F Recorder, Model 68, and project-designed and fabricated compressed-air bubbler recorders were used to record water levels.

Dry-weather sanitary sewer flows were measured in eight sewers and in the main outfall of the wastewater treatment plant. Weirs were used in all sewers and float recorders were used in seven of them. A bubbler recorder was used in the eighth. No problems in measuring dry-weather flows with this equipment were reported, although no check on accuracy is available. Because of the extended period of high river stage, flow measurements during wet weather were generally not obtained in the sewers or overflow points. During this period, most of the flow measuring sites were either submerged or at least intermittently affected by high water. Overflows at one point were estimated by use of the Manning formula, and at another point by use of a current meter.

Stormwater runoff was measured at four gaging sites on storm sewers. At three sites, flow was measured by means of weirs and bubbler recorders. During periods of high river stage, one of these sites was submerged or affected by backwater, when no record was obtained. Runoff at the fourth site was determined from a stage-discharge relationship established by current meter measurements. A concrete sewer line crossing the channel provided a permanent type control. The rating curve was extended an undisclosed amount above the highest current meter measurement.

Computer Management of a Combined Sewer System - concerns a computer-controlled "total systems management" complex, which affects much of the combined sewer system of Seattle, Washington. Computer-augmented treatment and disposal (CATAD) takes advantage of storage in the sewers to limit overflows, and selects overflow points based on water quality data. Development of the control system included the installation of 36 remote sensor stations. Work continues on a fully automatic, optimizing model to program decisions so the system can maintain an 80% overflow reduction.

Water levels at many locations are probably the most important single category of information used to calculate flows. By incorporating Manning's equations, various orifice and weir formulae, and pump efficiency curves, it is possible to calculate flow at almost any location in the system. Monitoring, control, and modeling of the entire system depends upon flow information from many locations, some of which cannot be obtained from water level measurements alone. In the CATAD system, other on-line flow measuring techniques employed are;

a. Flow measuring weirs are installed at various locations.

b. A calibrated propeller type flowmeter is monitored at the West Point Treatment plant site.

c. Force main pressure calibration methods are used at pump stations where there is sufficient friction head loss to calibrate the force main at various flow ranges.

In addition, more than 100 Palmer-Bowlus flumes are installed in manhole locations. A computer program has been applied for rating these measuring flumes, using data including sewer diameter, shelf height, flume side slopes, and elevation of vertical side slopes.

Level sensors used in the system are generally pneumatic bubblers with back pressure read by a differential pressure transmitter. The accuracy of measurements was checked by direct level measurements to bring instrument calibration of the various stations into agreement with the system datum. It was determined that the overall accuracy probably approaches about 2% of the full scale measurement. Project experience demonstrated that manufacturer's pump unit performance curves may be used to calculate reliable flows provided that critical analog sensors, particularly pump speed sensors, are reliable. Because flows calculated using performance curves had been considered dubious, force main pressure sensors were installed at many pump stations to provide alternative means of calculating station discharge. As a result of checking the pressure gages using the salt velocity method, it was found that flows thus calculated are not entirely reliable due to rapid fluctuation of analog pressure values.

Deficiencies in the flow calculation procedures for regulator stations were revealed. No allowance had been made for the effect of interceptor backwater affecting the tailwater at a regulator gate and no transition had been provided between fully-submerged and free discharge conditions. A backwater allowance was added, and a method of calculating the degree of gate submergence was developed.

Characterization and Treatment of Urban Land Runoff - was a project to characterize the runoff from a 4.3 sp km (1.67 sq mi) urban watershed in Durham, North Carolina with respect to annual pollutant yield. The U.S. Geological Survey operates a continuous stage recorder and two digital punch tape precipitation recorders within the basin. Streamflow control is provided by a shallow V-notch weir located on Third Fork Creek some 21m (70 ft) downstream from a bridge culvert and 11.3 km (7 mi) upstream from the mouth of Third Fork Creek. Water quality samples were taken from the center of the stream approximately 1.5m (5 ft) below the weir. Thus, rainfall, runoff, and water quality data were gathered for the basin and analyzed. The USEPA Storm Water Management Model (SWMM) was also evaluated with respect to actual conditions as measured in the field and "was judged to predict peak hydrograph flows and total hydrograph volumes with reasonable accuracy; however it was not judged effective for predicting pollutant concentrations".

In order to assess the impact of varying types of land use within the basin on urban runoff quality, five storms were manually sampled at sub-basin discharge locations. A control section, usually a pipe or box culvert, was utilized with Manning's equation to arrive at stage-discharge relationships for each basin sampled. The stage was manually read when a sample was taken. No accuracy estimates are available.

SAMPLING

The collection of a wastewater sample that is representative of the source in all respects of interest is a frequently underrated task; a situation that is unfortunate in view of the complexities involved, especially if suspended solids are present, as will be the case with most stormwater flows. This section, whose contents are mostly taken from Shelley (1974, 1975a, b, c, d) and Shelley and Kirkpatrick (1973), briefly treats the subject and automatic equipment for the purpose.

MECHANICS OF POLYDISPERSE SYSTEMS

The mechanics of polydisperse systems such as stormwater flows are among the most complex and least thoroughly understood of all aspects of science. This is not surprising when one considers that it covers dynamic processes ranging from such sedimentology subjects as the movement of soil to the thixotropic world of colloid chemistry. With complete descriptions having to account for such topics as electrokinetics, descriptive and structural rheology, sorption, flocculation, diffusion, and Brownian motion as well as hydraulic system influences, it is little wonder that empirical progress has outpaced analytical descriptive efforts. Even under well controlled laboratory conditions, the study of suspended solid laden flows remains very difficult, with considerable data scatter the rule and statistical treatment of results being usually required. The point of all of this is not to suggest that any attempt to seriously study the subject is doomed to failure but, rather, to point out that one should not approach it as though it is a $\pm$ 1 percent cosmos.

Table 6 has been prepared to give a better appreciation of suspended solids orders of magnitude and characteristics. Eight decades of particle sizes are covered, and nominal dimensions are given in millimeters, microns, and angstroms as some readers may have a better appreciation

TABLE 6. PARTICLE ORDERS OF MAGNITUDE AND CHARACTERISTICS

Millimeters	10	1	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
Microns	10,000	1,000	100	10	1	0.1	0.01	0.001	0.0001
Angstroms	10^{8}	10^{7}	10^{6}	10^{5}	10^{4}	10^{3}	10^{2}	10	1
Relative Weight of a Particle	10^{3}	1	10^{-3}	10^{-6}	10^{-9}	10^{-12}	10^{-15}	10^{-18}	10^{-21}
Relative Fall Velocity	4.8	1	5.6×10^{-2}	5.6×10^{-4}	5.6×10^{-6}	5.6×10^{-8}	5.6×10^{-10}	5.6×10^{-12}	-
Relative Brownian Displacement	-	1	10^{1}	10^{2}	10^{3}	10^{4}	10^{5}	10^{6}	-

Classification

sand	silt	clay	ultra clay	solution

Phase	bed load	coarse suspension	coll. susp.	colloidal solution	molecular soln.
Appearance	on bottom	very cloudy	turbid	virtually clear	clear
Observed w/	naked eye	naked eye	microscope	electron or ultra-microscope	-
Separated w/	screen	filter paper	clay filter	ultrafilter	-
Form after evaporation	granular	loose powder	powder or gel	gel	crystal

<u>Notes</u>:

1. The fall velocity of a 1 mm diameter quartz sphere is 16.0 cm/sec. Similar particles much below 10^{-4} mm in diameter essentially do not settle, their fall velocities being better stated in centimeters per century.
2. The time for average Brownian displacement of a 2 mm diameter sphere by one centimeter is around 7,300 yr. Brownian motion starts as a practical consideration for diameters smaller than 10^{-2} mm.
3. The resolution limit of an ordinary microscope is around 2000A compared to 10A for electron or ultramicroscopes.
4. The size of particles passing the finest practical sieves (300 mesh) is around 0.05 mm. The limit of an ultrafilter is approximately 10^{-6} mm.

for size in one set of units rather than another. The changes in particle weight (or mass or volume) with size are indicated relative to a 1 mm particle. Changes in hydraulic size with mean particle diameter are indicated relative to a quartz sphere (s.g. = 2.65) 1 mm in diameter. Within the range of validity of Stokes' Law they vary with the square of the diameter. The displacement due to Brownian motion relative to a 1 mm diameter particle is also indicated.

The major divisions of particle size classification set forth in Table 6 are indicated, as is the physical nature or phase of the mixture. Several other characteristics of particle-water mixtures are also given, including the visual appearance, methods of particle observations, separation techniques, and the form of the solids after evaporation.

There are characteristics of particles other than size which are of interest to explain certain phenomena. For example, the charge on a particle attracts counterions and repels similiions. Some of the counterions may be immobilized in the Stern layer, while the remainder form a diffuse Gouy layer. Within this double layer, the decay of potential is measured by its thickness, which is very sensitive to the concentration and valence of the counterions as is the surface change density. Potentials at the particle surface, at the boundary between the Stern and Guoy layers, and at the hydrodynamic plane of shear are of special importance.

As an example, clay minerals usually have a negative charge. This may be due to preferential adsorption of anions, especially hydroxyl ions; cationic substitutions within the crystal lattice; and residual valences (broken bonds) at particle edges (Postma; 1967). The double layer of counterions (hydrated cations) described above balances the negative charge. If the electrolytic potential (thickness of the double layer) decreases below a critical value, coagulation occurs. In this process, while the double layer is present, two clay particles approaching each other by Brownian movement are repelled because their charges are equal.

Other forces, however, may tend to cause the particles to approach each other. The electrolytic content of the water is a strong factor here as it affects the thickness (and hence electrolytic potential) of the double layer. Increases in pollution may cause the flocculation process to be completed, perhaps due to the presence of polyvalent cations in industrial wastes and excessive amounts of particulate organic sewage, which can act as a binding substance for fine-grained particles. On the other hand, dissolved organic solids may inhibit flocculation, at least to some degree.

The diameter of a floccule may be considerably larger than the diameters of its constituent particles, with the result that the fall velocity of flocculated clays is much greater than that in peptized form. Much water is included in the floccules, however, with the result that fall velocity increases are not as great as might otherwise be expected. For example, as pointed out by Postma (1967), a unit particle with a specific weight of 2.7 and a diameter of 5 microns has a fall velocity in seawater of about 0.002 cm/s. A particle of the same density but a 500 micron diameter would sink at a rate of approximately 20 cm/s. A floccule of clay particles of the same size but containing 95 percent water, on the other hand, would have a fall velocity of only 0.4 cm/s, the hydraulic size of a 20 micron quartz sphere.

The transport of solid particles by a fluid stream is an exceedingly complex phenomena and no complete theory which takes into account all of the parameters has yet been formulated. It normally occurs as a combination of bed movement, saltation, and suspension. Although these are interrelated, they are usually discussed separately because the phenomena are not understood sufficiently to allow one to satisfactorily consider them together.

The distribution of suspended solids or sediment in a transport stream is expressed in terms of concentration in one of two ways. Spatial concentration is defined (FIASP; 1963) as "the quantity of sediment relative to the quantity of fluid in a fluid-sediment mixture". Thus, it could be expressed as the dry weight of solids per unit volume of water-solids mixture. There are three such concentrations that may be of interest: spatial concentration at a point in the cross section, which is the concentration in a small volume at the point; spatial concentration in a vertical, which is the concentration in a small water column extending from the stream bed to the water surface; and spatial concentration in a cross section, which is the concentration of the mixture contained in a unit length of channel at the cross section. Turbidity, density, and other fluid properties of the water-solids mixture are related to the spatial concentration.

On the other hand, the discharge-weighted concentration is defined as the quantity of suspended solids relative to the discharge of the fluid-solids mixture. Thus, it could be expressed as the dry weight of solids in a unit volume of discharge, or the ratio of the dry weight of solids discharge to the weight of the water-solids discharge. Again there are three such concentrations that may be of interest: discharge-weighted concentration at a point in the cross section which is the concentration in the water-solids discharge through a small cross-sectional area at the point; discharge-weighted concentration in a vertical, which is the concentration in the water-solids discharge through a unit width cross-sectional area centered on the vertical; and discharge-weighted concentration in a cross section, which is the concentration in the discharge through the entire cross section. The discharge-weighted concentration may be multiplied by the overall stream discharge to obtain suspended solids discharge.

Although a number of theories on the suspension of solids in flowing water have been proposed, it is now generally recognized that it is directly related to turbulence of the flow as explained by Lane and Kalinske (1939, 1941). In turbulent flow the instantaneous current vector has vertical and lateral components as well as the horizontal one, and these are constantly changing in magnitude and direction with time in a random fashion. As expressed in FIASP (1963) for sediment:

"Sediment carried in suspension is acted on in the vertical direction by momentary currents which move upward or downward in the stream. Because the water level in the stream remains unchanged, the quantity of upward and downward flow must be equal. If the upward and downward currents were the only forces affecting the vertical movement of sediment, complete mixing would soon take place and the concentration of sediment would become uniform throughout the depth. However, all particles of specific gravity greater than that of water settle steadily downward. Under the combined action of vertical currents and gravitational force, a particle caught in a current moving upward at a rate greater than the settling velocity of the particle should be transported upward, but if it is suspended in water moving downward, or moving upward at a rate less than its settling velocity, the particle should move downward. It might seem that the downward currents would take down as much sediment as the upward ones carry up, with the result that all the material finally would settle to the bottom. However, as settling takes place the sediment concentration increases toward the bottom, and the upward currents travel from a region of higher concentration to one of lower concentration, whereas for the downward currents the opposite relation prevails. As the amounts of water moving upward and downward are equal and the sediment concentration in the rising currents is potentially greater than in the downward currents, more sediment must be acted upon by the rising than by the falling currents. The settling action superimposed on the fluctuating upward and downward currents tends to produce a balanced suspension in which the rate of increase in sediment concentration toward the bottom depends upon the degree of turbulence in the stream and the settling velocity of the suspended particles."

Since coarse particles settle faster than fine particles of the same specific gravity and the vertical water motion due to turbulence will not be appreciably different for particles of different sizes, equilibrium can only be obtained if the vertical concentration distribution varies, with the concentration increasing more towards the bottom for coarse than for fine particles. As particle size decreases, the vertical concentration gradient becomes less and less until it is essentially zero, the case for most silts and clays. As an example, Figure 1 depicts typical distribution curves for various sizes of sediment in the Missouri River at Kansas City. Of course, for buoyant particles the reverse is true.

With all of the analytical difficulties discussed or alluded to in the foregoing, it is obvious that there must be great reliance upon empirical data in the study of the sampling of suspended solids. A wide variety of sampling equipment designs is available (Shelley; 1974); but none of them is universally acceptable for representatively sampling all flows of interest (Shelley; 1975a), and differences in designs can produce marked differences in results as reported by Harris and Keffer (1974) and Shelley (1975c).

Let us consider the ability of a sampler intake probe to gather a representative sample of dense suspended solids in the sediment range, say up to 0.5 mm with specific gravity of 2.65. The results of a rather thorough examination of relatively small diameter intake probes (0.63 and 0.32 cm) are given in FIASP (1941b). The argument is developed that, for a nozzle pointing directly into the flow, the most representative sample of a fluid/suspended-solids mixture will be obtained when the sampling velocity is equal to the flow velocity at the sampling point. Using this as the reference criteria, investigations were conducted to determine the effects of a) deviations from the normal sampling rate, b) deviations from the straight-into-flow position of the probe, c) deviations in size and shape of the probe, and d) disturbance of sample by nozzle appurtenances. The effect of the sampling velocity on the representativeness of the sample is indicated in Figure 2 which presents the results for 0.45 mm and 0.06 mm sand. For the latter size,

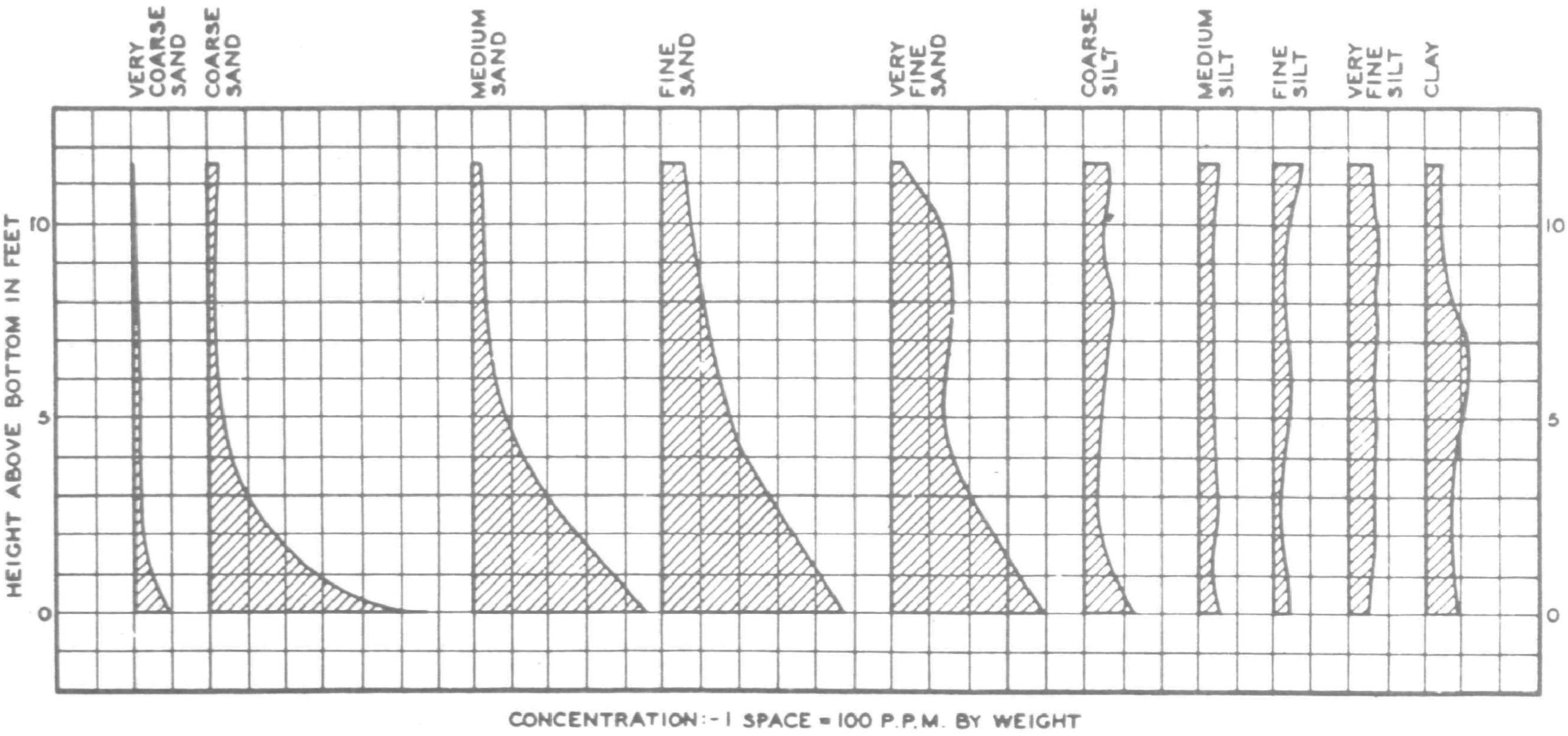

Figure 1. Vertical Distribution of Sediment in the Missouri River at Kansas City*

* Taken from FIASP (1948).

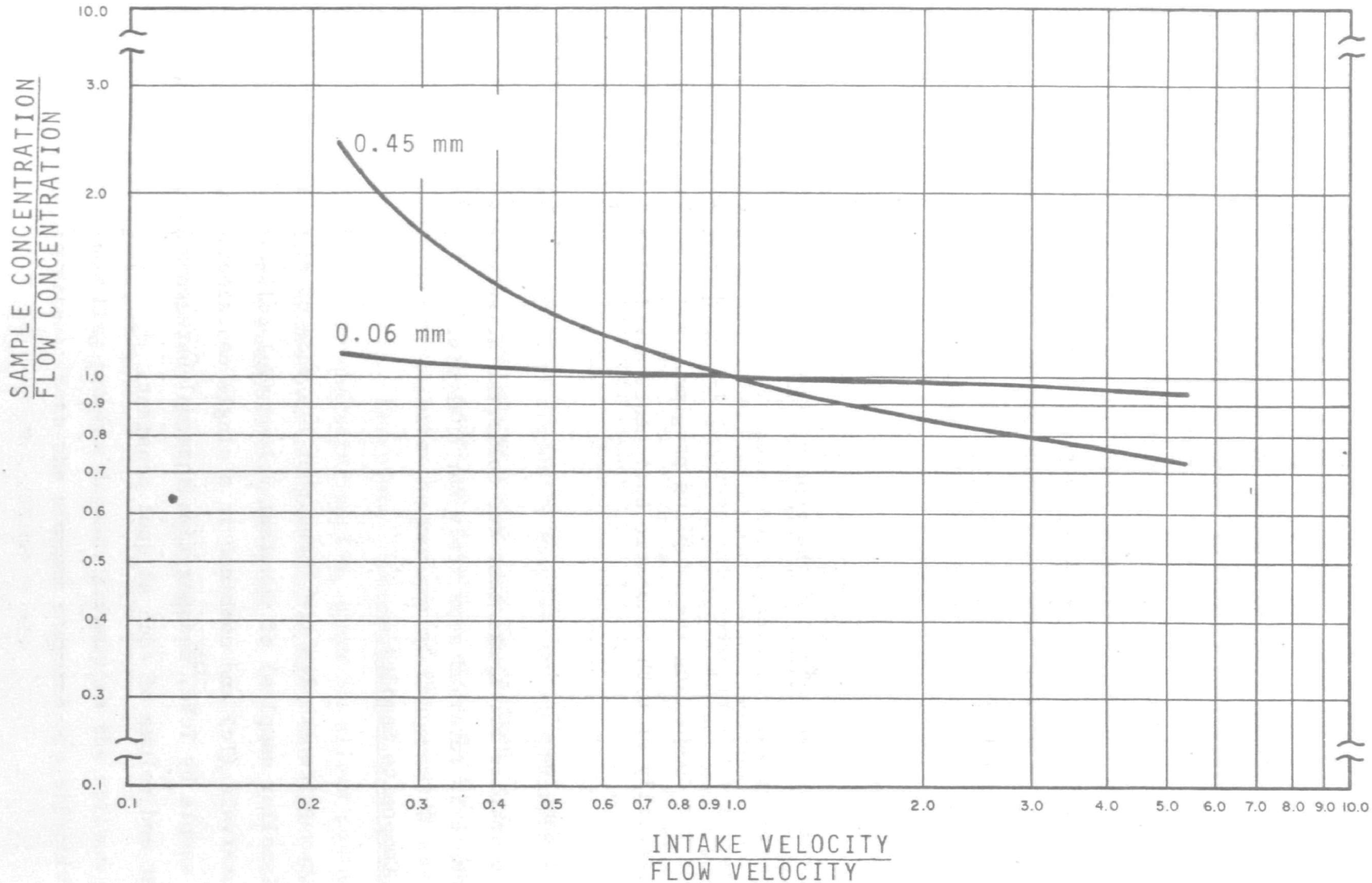

Figure 2. Effect of Sampling Velocity on Representativeness of Suspended Solids*

* Data from FIASP (1941b).

which falls within the Stokes' Law range, less than $\pm$ 4 percent error in concentration was observed over sampling velocities ranging from 0.4 to 4 times the stream velocity. For the 0.45 mm particles, the error at a relative sampling rate of 0.4 was +45 percent, and at a relative sampling rate of 4 the error was -25 percent.

For probe orientations up to 20^{o} to either side of head-on, no appreciable errors in concentration were observed. Probe diameter and inlet geometry (beveled inside, beveled outside or rounded edge) showed comparatively little effect on the representativeness of the sample. In summary, it was found that for any sampler intake facing into the stream, the sampling rate is the primary factor to be controlled.

TYPES OF SAMPLES

In general, the selection of the type of sample to be collected depends on a number of factors such as the rates of change of flow and of the character of the wastewater, the accuracy required, and the availability of funds for conducting the sampling program. All samples collected, either manually or with automatic equipment, are included in the following types.

Descrete Samples

Discrete samples are those that are collected at selected intervals, and each sample is retained separately for analysis.

Simple Composite Samples

Simple composite samples are those that are made up of a series of aliquots (smaller samples) of constant volume (Vc) collected at regular time intervals (Tc) and combined in a single container. One could denote such a sample by TcVc, meaning time interval between sucessive aliquots constant and volume of each aliquot constant.

Flow Proportional Composite Samples

Flow proportional composite samples are those collected in relation to the flow volume during the period of compositing, thus indicating the "average" waste condition during the period. One of the two ways of accomplishing this is to collect samples of equal volume (Vc), but at variable time intervals (Tv) which are inversely proportional to the volume of the flow. That is, the time interval between samples is reduced as the volume of flow increases. Alternatively, flow proportioning can be achieved by increasing the volume of each sample in proportion to the flow (Vv), but keeping the time interval between samples constant (Tc).

Sequential Composite Samples

Sequential composite samples are composed of a series of short-period composites, each of which is held in an individual container. For example, each of several samples collected during a 1-hour period may be composited for the hour. The 24-hour sequential composite is made up from the individual 1-hour composites.

Although the various types of composite samples are useful when an overall picture of "average" wastewater conditions is desired (e.g., monitoring industrial discharges, checking treatment plant efficiencies, etc.), they do nothing to describe the pattern of changes that may occur over the compositing period. Therefore, discrete sampling is generally more useful for stormwater characterization, since it allows pollutant parameters to be determined over a known time history and this provides information about their variability with time.

PRACTICAL CONSIDERATIONS OF A SAMPLING PROGRAM

The adequacy of a sampling program depends largely on the optimum selection of sampling sites. Both the program cost and its effectiveness in collecting samples representative of the character of sewer flows are seriously affected by the care exercised in site selection. Awareness of the general character of sewer flows and of flow modes in storm sewers and combined sewers, and knowledge of the variability of pollutant concentration, leads to an understanding of how best to select sites for sampling. Some of the considerations in making such selections are:

a. Maximum accessibility and safety - Manholes on busy streets should be avoided if possible; shallow depths with manhole steps in good condition are desirable. Sites with a history of surcharging or submergence by surface water, or both, should be avoided if possible. Avoid locations which may tend to invite vandalism.

b. Be sure that the site provides the information desired - Familiarity with the sewer system is necessary. Knowledge of the existence of inflow or outflow between the sampling point and point of data use is essential.

c. Make certain the site is far enough downstream from tributary inflow to ensure mixing of the tributary with the main sewer.

d. Locate in a straight length of sewer, at least six sewer widths below bends.

e. Locate at a point of maximum turbulence, as found in sewer sections of greater roughness and of probable higher velocities. Locate just downstream from a drop or hydraulic jump, if possible.

f. In all cases, consider the cost of installation, balancing cost against effectiveness in providing the data needed.

A decision as to whether manual or automatic sampling should be employed involves more than just cost considerations. Experience has indicated that operator training is necessary if manual sampling is to produce reproducible results. Instances have been noted wherein two different operators were asked to obtain a sample at a particular site with no other guidance given. Analyses of samples taken at almost the same instants in time have shown variances up to 50%. Other work conducted solely to compare manual sampling methods has indicated such discrepancies in results that suspicion must be cast upon manual methods which involve dipping of samples out of raw waste sources, and has raised questions regarding the suitability of such manual grab sampling as a yardstick against which to measure other techniques.

The preferred method of gathering manual samples from a raw waste stream is to use a nonclogging submersible pump to actually extract the fluid and tubing of appropriate size to transport it up to the sample container. Pump and tubing sizes should be such that effective collection and transport of all suspended solids of interest is assured. Both small, flexible impeller centrifugal pumps and progressive cavity screw pumps have been successfully used with good repeatability of results. It should be noted, however, that the collection of flow proportional composite or sequential samples can become quite tedious if performed manually at the sampling site.

The decision to use automatic sampling equipment does not represent the universal answer to wastewater characterization, however. For initial characterization studies, proper manual sampling may represent the most economical method of gathering the desired data. It is also prudent from time to time to verify the results of an automatic sampler with samples obtained manually. In this regard, because of the strict chain of custody procedures that can be exercised with manually collected samples, they can be used to support those data resulting from the use of automatic sampling equipment. It should be noted, however, that more and more use is being made of automatic samplers as time goes by, and this trend is expected to continue.

Presently available automatic sewage samplers have a great variety of characteristics with respect to size of sample collected, lift capability, type of sample collected (discrete or composite), materials, of construction, and numerous other both good and poor features. A number of considerations in selection of a sampler are:

a. Rate of change of sewage conditions
b. Frequency of change of sewage conditions
c. Range of sewage conditions
d. Periodicity or randomness of change
e. Availability of recorded flow data
f. Need for determining instantaneous conditions, average conditions, or both
g. Volume of sample required
h. Need for preservation of sample
i. Estimated size of suspended matter
j. Need for automatic controls for starting and stopping
k. Need for mobility or for a permanent installation
l. Operating head requirements.

Because of the variability in the character of storm and/or combined sewage, and because of the many physical difficulties in collecting samples to characterize the sewage, precise characterization is not practicable, nor is it possible. In recognition of this fact, one must guard against embarking on an excessively detailed sampling program, thus increasing costs, both for sampling and for analyzing the samples, beyond costs that can be considered sufficient for conducting a program which is adequate for the intended purpose.

A careful study of costs should be made prior to commencing a program of sampling, balancing cost against the number of samples and analyses required for adequate characterization of the wastewater. As the program progresses, current study of the results being obtained may make it reasonable to reduce or increase the number of samples collected.

ELEMENTS OF AN AUTOMATIC SAMPLER SYSTEM

In a system breakdown by functional attributes, an automatic liquid sampler may be divided into five basic elements or subsystems. Each of these will be discussed in turn.

Sample Intake Subsystem

The operational function of the sample intake is to reliably allow gathering a representative sample from the flow stream in question. Its reliability is measured in terms of freedom from plugging or clogging to the degree that sampler operation is affected and invulnerability to physical damage due to large objects in the flow. It is also desirable, from the viewpoint of sewer operation, that the sample intake offer a minimum obstruction to the flow in order to reduce the possibility of blockage of the entire pipe by lodged debris, etc.

The sample intake of many commercially available automatic liquid samplers is often only the end of a plastic suction tube, and the user is left to his own ingenuity and devices if he desires to do anything other than simply dangle the tube in the stream to be sampled. Some manufacturers provide a weighted, perforated plastic cylinder that screens the hose inlet from the unwanted material that might cause choking or blockage elsewhere within the sampler. Typical hole sizes are around 1/3 cm (1/8 inch) in diameter and, if there are sufficient holes to assure free flow, results have been satisfactory in some applications. Samplers that employ pneumatic ejection have their own intake chambers that must be used in order for the equipment to function properly. There will be some sampling sites where the use of custom designed intakes is indicated. However, their individual attributes will be a function of the flow characteristics; e.g., is the stream trash or debris ladened, is there any flow stratification, are suspended solids high, etc.?

Sample Gathering Subsystem

Three basic sample gathering methods or categories can be identified; mechanical, forced flow, and suction lift. Several different commercial samplers using each method are available today. The sample lift requirements of the particular site often play a determining role in the gathering method to be employed.

Mechanical Methods - There are many examples of mechanical gathering methods used in both commercially available and one-of-a-kind samplers. One of the more common designs is the cup on a chain driven by a sprocket drive arrangement. In another design, a cup is lowered within a guide pipe, via a small automatic winch and cable. Other examples include a self closing pipe-like device that extracts a vertical "core" from the flow stream, a specially contoured box assembly with end closures that extracts a short length (plug) of the entire flow cross-section, a revolving or oscillating scoop that traverses the entire flow depth, etc.

Some of the latter units employ scoops that are characterized for use with a particular primary flow measurement device such as a weir or Parshall flume and extract an aliquot volume that is proportional to the flow rate. Another design for mechanically gathering flow proportional samples involves the use of a sort of Dethridge wheel with a sample cup mounted on its periphery. Since the wheel rotation is proportional to flow, the effect is that a fixed volume aliquot is taken each time a certain discharge quantity has passed, and total discharge can be estimated from the size of the resultant composite sample.

The foregoing designs have primarily arisen from one of two basic considerations. First, site conditions that require very high lifts have dictated the use of mechanical gathering units due to the limitations of suction lift pumps and space considerations. Some mechanical units are capable of lifts of 61m (200 ft) or more. Second, the desire to gather samples that are integrated across the flow depth has led to some of the different mechanical approaches mentioned above. Unless vertical velocity and pollutant gradients are quantified and accounted for, their presence makes the results of such depth integrated samples questionable, at least in a mass discharge sense.

One of the penalties that one must trade-off in selecting a mechanical gathering unit is the necessity for some obstruction to the flow, at least while the sample is being taken. The tendency for exposed mechanisms to foul, together with the added vulnerability of many moving parts, means that successful operation will require periodic inspection, cleaning, and maintenance.

<u>Forced Flow Methods</u> - All forced flow gathering methods require some obstruction to the flow, but usually it is less than with mechanical gathering methods. It may only be a small inlet chamber with a check valve assembly of some sort or it may be an entire submersible pump. The main advantage of submersible pumps is that their high discharge pressures allow sampling at greater depths, thereby increasing the flexibility of the unit somewhat insofar as site depth is concerned. Pump malfunction and clogging, especially in the sizes often used for samplers, is always a distinct possibility and, because of their location in the flow stream itself, maintenance is much more difficult and costly to perform than on above ground or more easily accessible units. They also necessarily present an obstruction to the flow and are thus in a vulnerable position as regards damage by debris in the flow.

Pneumatic ejection is a forced flow gathering method used by a number of commercial samplers. The gas source required by these units varies from bottled refrigerant to motor driven air compressors. The units that use bottled refrigerant must be of a fairly small scale to avoid an enormous appetite for the gas and hence a relatively short operating life before the gas supply is exhausted. Furthermore, concern has recently been expressed about the quantities of freon that are being discharged into the atmosphere. The ability of such units to backflush or purge themselves is also necessarily limited. The advantages of few moving parts, inherent explosion proof construction, and high lift capabilities must be weighed against low or variable line velocities, low or variable sample intake velocities, and relatively small sample capacities in some designs. Another disadvantage of most pnuematic ejection units is that the sample chamber fills immediately upon discharge of the previous sample. Thus, it may not be representative of flow conditions at the time of the next triggering and, if paced by a flowmeter, correlation of results may be quite difficult.

Suction Lift Methods - Suction lift units must be designed to operate in the environment near the flow to be sampled or else their use is limited to a little over 9m (30 ft) due to atmospheric pressure. Several samplers that take their suction lift directly from an evacuated sample bottle are available today. Vacuum leaks, the variability of sample size with lift, the requirement for heavy glass sample bottles to withstand the vacuum, difficulty of cleaning due to the requirement for a separate line for each sample bottle, the necessity of placing the sample bottles near the flow stream (and hence in a vulnerable position), the varying velocities as the sample is being withdrawn, etc., are among the many disadvantages of this technique

Other units are available that use a vacuum pump and some sort of metering chamber to measure the quantity of sample being extracted. These units in some designs offer the advantages of fairly high sample intake and transport velocities, the fluid itself never comes in contact with the pump, and the pump output can easily be reversed to purge the sampling line and intake to help prevent cross-contamination and clogging. Their chief disadvantage, shared with certain other suction lift designs arises from the following consideration.

With all suction lift devices a physical phenomenon must be borne in mind and accounted for if sample representativeness is to be maintained. When the pressure on a liquid (such as sewage) which contains dissolved gases is reduced, the gases will tend to pass out of solution. In so doing they will rise to the surface and entrain suspended solids in route. (In fact, this mechanism is used to treat water; even small units for aquariums are commercially available.) The result of this is that the surface layer of the liquid may be enhanced in suspended solids, and if this layer is a part of a small sample aliquot, the sample may not be at all representative. In the absence of other mitigating factors, the first flow of any suction lift sampler should therefore be returned to waste.

A variety of positive displacement pumps have been used in the design of suction lift samplers, including flexible impeller, progressive cavity rotary screw, roller or vane, and peristaltic types. Generally these pumps are self-priming (as opposed to many centrifugal pumps), but some designs should not be operated dry because of internal wearing of rubbing parts. The desirability of a low-cost pump that is relatively free from clogging has led many designers to use peristaltic pumps. A number of types have been employed including finger, nutating, and two- and three-roller designs using either molded inserts or regular tubing. Most of these operate at such low flow rates, however, that the representativeness of suspended solids is questionable. Newer high-capacity peristaltic pumps are now available and should find application in larger automatic samplers. The ability of some of these pumps to operate equally well in either direction affords the capability to blow down lines and help remove blockages. Also, they offer no obstruction to the flow since the transport tubing need not be interrupted by the pump, and strings, rags, cigarette filters and the like are passed with ease.

All in all, the suction lift gathering method appears to offer more advantages and flexibility than either of the others for a storm or combined sewer application. The limitation on sample lift can be overcome by designing the pumping portion of the unit so that it can be separated from the rest of the sampler and thus positioned within 6m (20 ft) or so of the flow to be sampled. For many sites, however, even this will not be necessary.

Sample Transport Subsystem

The majority of the commercially available automatic samplers have fairly small line sizes in the sample train. Such tubes, especially at 1/3 cm (1/8 in.) inside diameter and smaller, are very vulnerable to plugging, clogging due to the build-up of fats, etc. For many applications, a better minimum line size would be 1 to 1.3 cm (3/8 to 1/2 in.) inside diameter.

For flows that are high in suspended solids, it is imperative that adequate sample flow rate be maintained throughout the sampling train in order to effectively transport them. In horizontal runs, the velocity must exceed the scour velocity, while in vertical runs the settling or fall velocity must be exceeded several times to assure adequate transport of solids in the flow. Sharp bends and twists or kinks in the sampling lines should be avoided if there is a possibility of trash or debris in the lines that could become lodged and restrict or choke the flow. The same is true of some valve designs. In summary, the sampling train must be sized so that the smallest opening is large enough to give assurance that plugging or clogging is unlikely in view of the material being sampled. However, it is not sufficient to simply make all lines large, which also reduces friction losses, without paying careful attention to the velocity of flow. For many applications, minimum velocities of 0.6 to 1 m/s (2 to 3 fps) would appear warranted, and even higher velocities are required for some applications.

In few equipment designs the sample is delivered under pressure all the way to the sampler container. It is more often the case, however, that a break occurs at a distribution funnel or some similar appurtenance, and flow from this point on into the sample container is by gravity alone. Field experience with some such designs has been less than satisfactory, with instances of clogging, accumulation of fats, etc., being noted.

Sample Storage Subsystem

For storm and combined sewer applications, discrete sampling is generally desired. This allows characterization of the sewage throughout the time history of the storm event. If the samples are sufficiently large, manual compositing can be performed based on flow records or some other suitable weighting scheme. Although the quantity of samples required will be a function of the subsequent analyses that are to be performed, in general at least 1 liter and preferably 2 liters will be desired. An additional benefit arises because such relatively large samples are less vulnerable to errors arising from cross-contamination.

The sample container itself should either be easy to clean or disposable. The cost of cleaning and sterilizing makes disposable containers attractive, especially if bacteriological analyses are to be performed. Although some of today's better plastics are much lighter than glass and can be autoclaved, they are not so easy to clean or inspect for cleanliness. Also the plastics will tend to scratch more easily than glass and, consequently, cleaning a well-used container can become quite a chore. The food packaging industry, especially dairy products, offers a wide assortment of potential disposable sample containers in the larger sizes. Both the 1.9ℓ (1/2 gal) paper and plastic milk cartons can be considered viable candidates, and their cost in quantity is in the pennies-each range.

The requirements for sample preservation are enumerated in MDQARL (1974) and will not be repeated here except to note that refrigeration is stated as the best single preservation and will, in all likelihood, be required unless the sampling cycle is brief and samples are retrieved shortly after being taken. It should be mentioned, however, that if the samples are allowed to become too cold, they may no longer be representative.

For example, destruction of the organisms necessary for the development of BOD may occur, or freezing may cause serious changes in the concentration of suspended solids. Light can also affect samples and either a dark storage area or opaque containers would seem desirable. Unless disposable containers are used, however, it will be difficult to inspect an opaque container for cleanliness. Again the paper milk carton is attractive since not only is it relatively opaque, but its top opens completely allowing visual inspection of its contents.

Controls and Power Subsystem

The control aspects of some commercial automatic samplers have come under particular criticism. It is no simple matter, however, to provide great flexibility in operation of a unit while at the same time avoiding all complexities in its control system. The problem is not only one of component selection but packaging as well. For instance, even though the possibility of immersion may be extremely remote in a particular installation, the corrosive highly-humid atmosphere which will, in all likelihood, be present makes sealing of control elements and electronics desirable in most instances.

The automatic sampler for storm and combined sewer application will, in all likelihood, be used in an intermittent mode; i.e., it will be idle for some period of time and activated to capture a particular meteoroligical event. If field experience to date is any indication, the greatest need for an improved control element is for an automatic starter. While the sensor is not a part of the sampler proper, its function is essential to successful sampler utilization. Although remote rain gages, etc., can be used for sensing elements, one of the most attractive techniques would be to use the liquid height (or its rate of increase) to start a sampling cycle. This will avoid the difficulties associated with different run-off times due to local conditions such as dryness of ground, etc.

The controls determine the flexibility of operation of the sampler, its ability to be paced by various types of flow measuring devices, etc. Built-in timers should be repeatable and time period should not be affected by voltage variations. The ability to repeatedly gather the required aliquot volume independent of flow depth or lift is very important if composite samples are to be collected. Provisions for manual operation and testing are desirable as is a clearly laid out control panel. Some means of determining the time when discrete samples were taken is necessary if synchronization with flow records is contemplated. An event marker could be desirable for a sampler that is to be paced by an external flow recorder. Reliability of the control system can dominate the total system reliability. At the same time, this element will, in all likelihood, be the most difficult to repair and calibrate. Furthermore, environmental effects will be the most pronounced in the control system. The power switching function of the control system may be required to deal with multiple switching of inductive loads and must achieve the switching of these loads without the possible damage associated with transfer of energy interruptions.

The above tasks can probably be best executed, in the light of the current electronics state-of-the-art, by a solid state controller element. In addition to higher inherent reliability, such an approach will allow switching of high level loads in a manner that eliminates RFI emissions and destructive results. In addition, the unit should be of modular construction for ease of modification, performance monitoring, fault location, and replacement/repair. Such an approach also lends itself to encapsulation which will minimize environmental effects. Solid state switching eliminates the possibility of burned or welded contacts either of which will cause complete sampler breakdown.

Solid state controllers can be easily designed with sufficient flexibility to accept start commands from a variety of types of remote sensors, telephone circuits, etc.

One of the attributes essential to the control system of an automatic sampler to be used in a storm or combined sewer application is that it be able to withstand power outages and continue its program. Such power interruptions appear to be increasingly common as demand for electricity continues to grow. Although desirable in some instances, the provision of a random interrogate signal to be coupled with a sequence sample mode generates programming problems, especially when coupled with power interrupt possibilities.

The foregoing discussion as it relates to problems associated with interruptions in electrical service is, of course, directed to samplers that rely upon outside power for some aspect of their operation. The need for high sample intake and transport velocities, larger sample lines and capacities, together with the possible requirement for mechanical refrigeration make it unlikely that such a sampler can be totally battery operated today. Although recent break-throughs have resulted in 1 kw dry-cell batteries, their cost is prohibitive for this sort of an application. Other approaches to self-contained power such as custom designed wet-cell packs, diesel generators, etc., while within the current state-of-the-art, introduce other problems and complexities that must be carefully weighed before serious consideration can be given to their incorporation in an automatic sampler design.

REVIEW OF COMMERCIALLY AVAILABLE EQUIPMENT

The foregoing discussion of the elements of an automatic sampler can be viewed as equipment requirements. In addition to these requirements, there are also certain desirable features which will enhance the utility and value of the equipment. For example, the design should be such that maintenance and troubleshooting are relatively simple tasks. Spare parts should be readily available and reasonably priced. The equipment design should be such that the unit has maximum inherent reliability. As a general rule, complexity in design should be avoided even at the sacrifice of a certain degree of flexibility of operation. A reliable unit that gathers a reasonably representative sample most of the time is much more desirable than an extremely sophisticated complex unit that gathers a very representative sample 10 percent of the time, the other 90 percent of the time being spent undergoing some form of repair due to a malfunction associated with its complexity.

It is also desirable that the cost of the equipment be as low as practical both in terms of acquisition as well as operational and maintenance costs. For example, a piece of equipment that requires

100 man-hours to clean after each 24 hours of operation is very undesirable. It is also desirable that the unit be capable of unattended operation and remaining in a standby condition for extended periods of time.

The sampler should be of sturdy construction with a minimum of parts exposed to the sewage or to the highly humid, corrosive atmosphere associated directly with the sewer. It should not be subject to corrosion or the possibility of sample contamination due to its materials of construction. The sample containers should be capable of being easily removed and cleaned; preferably they should be disposable.

For portable automatic wastewater samplers, the list of desirable features is even longer. Harris and Keffer (1974) give a number of features of an "ideal" portable sampler which are based upon sampler comparison studies and over 90,000 hours of field experience. Included were:

- Capability for AC/DC operation with adequate battery energy storage for 120-hour operation at 1-hour sampling intervals.

- Suitable for suspension in a standard manhole and still provide access for inspection and sample removal.

- Total weight including batteries under 18 kilograms (40 pounds).

- Sample collection interval adjustable from 10 minutes to 4 hours.

- Capability for collecting both simple and flow-proportional composite samples.

- Capability for multiplexing repeated aliquots into discrete bottles (i.e., sequential composite).

- Intake hose liquid velocity adjustable from 0.61 to 3 m/sec (2.0 to 10 fps) with dial setting.

- Minimum lift of 6.1 meters (20 feet).

- Explosion proof.

- Watertight exterior case to protect components in the event of rain or submersion.

- Exterior case capable of being locked and with lugs for attaching steel cable to prevent tampering and provide some security.

- No metal parts in contact with waste source or samples.

- An integral sample container compartment capable of maintaining samples at 4 to 6°C for a period of 24 hours at ambient temperatures up to 38°C.

- With the exception of the intake hose, capable of operating in a temperature range between -10 to 40°C.

- Purge cycle before and after each collection interval and sensing mechanism to purge in event of plugging during sample collection and then collect complete sample.

- Capable of being repaired in the field.

Although some types of automatic liquid sampling equipment have been available commercially for some time, project engineers continue to design custom sampling units for their particular projects due to a lack of commercial availability of suitable equipment. In the last few years, however, there has been a proliferation of commercial sampling equipment designed for various applications. New companies are being formed and existing companies are adding automatic sampling equipment to their product lines. In addition to their standard product lines, most manufacturers of automatic sampling equipment provide special adaptations of their equipment or custom designs to meet unique requirements of certain projects. Some designs which began in this way have become standard products, and this can be expected to continue.

The products themselves are rapidly changing also. Not only are improvements being made as field experience is gathered with new designs, but attention is also being paid to certain areas that have heretofore been largely ignored. For example, one company is introducing sampling probes that allow gathering oil or various other liquids from the flow surface; solid-state electronics are being used more and more in sampler control subsystems; new-type batteries are offering extended life between charges and less weight; and so on. Table 7 lists the names and addresses of some 32 manufacturers who are known to offer standard lines of automatic wastewater sampling equipment. In view of the burgeoning nature of this product area, it is inevitable that some omissions have been made.

An overall matrix, which summarizes the equipment characteristics to facilitate comparisons, is presented in Table 8. There are several column headings for each sampler model (or class of models). "Gathering Method" identifies the actual method used (mechanical, forced flow, suction lift) and type (peristaltic-, vacuum-, centrifugal-pump, etc.). Depending upon the gathering method employed, the sample flow rate may vary while a sample is being taken, vary with parameters such as lift, etc. Therefore, the "Flow Rate" column typically lists the upper end of the range for a particular piece of equipment and values significantly less may be encountered in a field application. "Lift" indicates the

TABLE 7. AUTOMATIC WASTEWATER SAMPLER MANUFACTURERS

Bestel-Dean Limited
92 Worsley Road North,
Worsley
Manchester, England M28 5QW

BIF Sanitrol
P.O. Box 41
Largo, Florida 33546

Brailsford and Company, Inc.
Milton Road
Rye, New York 10580

Brandywine Valley Sales Co.
20 East Main Street
Honey Brook, PA 19344

Chicago Pump Division
FMC Corporation
622 Diversey Parkway
Chicago, Illinois 60614

Collins Products Co.
P.O. Box 382
Livingston, Texas 77351

Environmental Marketing
Associates
3331 Northwest Elmwood Dr.
Corvallis, Oregon 97330

ETS Products
12161 Lackland Road
St. Louis, Missouri 63141

Fluid Kinetics, Inc.
3120 Production Drive
Fairfield, Ohio 45014

Horizon Ecology Company
7435 North Oak Park Drive
Chicago, Illinois 60648

Hydra-Numatic Sales Co.
65 Hudson Street
Hackensack, NJ 07602

Hydraguard Automatic
Samplers
850 Kees Street
Lebanon, Oregon 97355

Instrumentation Specialties
Company
Environmental Division
P.O. Box 5347
Lincoln, Nebraska 68505

Kent Cambridge Instrument
Company
73 Spring Street
Ossining, New York 10562

Lakeside Equipment Corp.
1022 East Devon Avenue
Bartlett, Illinois 60103

Manning Environmental Corp.
120 DuBois Street
P.O. Box 1356
Santa Cruz, California 98061

Markland Specialty Eng. Ltd.
Box 145
Etobicoke, Ontario (Canada)

Nalco Chemical Company
180 N. Michigan Avenue
Chicago, Illinois 60601

Nappe Corporation
Croton Falls Industrial Complex
Route 22
Croton Falls, New York 10519

N-Con Systems Company
308 Main Street
New Rochelle, New York 10801

Paul Noascono Company
805 Illinois Avenue
Collinsville, Illinois 62234

TABLE 7. AUTOMATIC WASTEWATER SAMPLER MANUFACTURERS (Cont'd)

Peri Pump Company, Ltd.
180 Clark Drive
Kenmore, New York 14223

Phipps and Bírd, Inc.
303 South 6th Street
Richmond, Virginia 23205

Protech, Inc.
Roberts Lane
Malvern, PA 19355

Quality Control Equipment
Company
P.O. Box 2706
Des Moines, Iowa 50315

Rice Barton Corporation
P.O. Box 1086
Worcester, MA 01601

Sigmamotor, Inc.
14 Elizabeth Street
Middleport, New York 14105

Sirco Controls Company
8815 Selkirk Street
Vancouver, B.C.

Sonford Products Corporation
100 East Broadway, Box B
St. Paul Park, MN 55071

Testing Machines, Inc.
400 Bayview Avenue
Amityville, New York 11701

Tri-Aid Services, Inc.
161 Norris Drive
Rochester, New York 14610

Williams Instrument Co., Inc.
P.O. Box 4365, North Annex
San Fernando, California 91342

TABLE 8. SAMPLER CHARACTERISTIC SUMMARY MATRIX

Sampler	Gathering Method	Flow Rate (ml/min)	Lift (m)	Line Size (mm)	Sample Type	Instal-lation	Cost Range ($)	Power
Bestel-Dean Mk II	S-Watson-Marlow	690	6.1	6.4	D, TcVc, TvVc	Portable	Unk.	AC/DC
Bestel-Dean Crude	S-screw type	Unk.	6.1	19.1	D, TcVc, TvVc	Portable	Unk.	AC
BIF 41	M-cup on chain	NA	4.9	25.4	TcVc, TvVc	Fixed	~1,000	AC
Brailsford DC-F & EP	S-piston type	10	<2	4.8	Continuous	Portable	296-373	DC
Brailsford EVS	S-vacuum pump	5	3.7	4.8	TcVc, TvVc	Portable	520-672	AC/DC
Brailsford DV-2	S-piston type	10	<2	4.8	TcVc, TvVc	Portable	373	DC
BVS PP-100	F-pneumatic	*	85	3.2	TcVc, TvVc	Portable	853-1,525	AC/DC
BVS PE-400	F-submersible pump	7,600	9.8	12.7	TcVc, TvVc	Portable	1,500-2,510	AC/DC
BVS SE-800	F-submersible pump	7,600	9.8	12.7	D, TcVc, TvVc	Fixed	5,650	AC
BVS PPE-400	F-pneumatic	*	85	3.2	TcVc, TvVc	P or F	1,450-3,350	AC/DC
Chicago Pump	user supplied	~133,000	NA	25.4	TcVc, TvVc	Fixed	2,600-3,200	AC
Collins 42	user supplied	>3,785	NA	2.4	TcVc, TvVc	P or F	985-2,478	AC
Collins 40	user supplied	~5,000	NA	2.4	TcVc, TvVc	P or F	835-2,328	AC
EMA 200	F-piston type	Unk.	<1	9.5	TcVc, TvVc	Portable	199-456	AC/DC
ETS FS-4	S-peristaltic	~20	8.8	6.4	Continuous	Portable	1,095-up	AC
Horizon S7570	S-peristaltic	100	9.1	0.8	Grab	Portable	~410	AC/DC
Horizon S7576	S-peristaltic	100	9.1	0.8	TcVc	Portable	~220	AC
Horizon S7578	S-peristaltic	100	9.1	0.8	Continuous, TcVc	Portable	595	DC
Hydraguard HP	F-pneumatic	*	>9	6.4	TcVv	Portable	246-541	Air
Hydraguard A	F-pneumatic	*	>9	6.4	TvVc	Portable	286-668	Air & AC
Hydra-Numatic	S-centrifugal	5,700	4.6	6.4	TcVc, TvVc	Portable	1,800	AC
ISCO 1392	S-peristaltic	1,500	7.9	6.4	D, TcVc, TvVc, S	Portable	1,095-1,498	AC/DC
ISCO 1480	S-peristaltic	NA	7.9	6.4	TcVc, TvVc	Portable	645-1,020	AC/DC
ISCO 1580	S-peristaltic	1,400	7.9	6.4	TcVc, TvVc	Portable	750-1,130	AC/DC
Kent SSA	S-peristaltic	150	4.9	6.4	Discrete	Portable	1,240	AC/DC
Kent SSB	S-peristaltic	200	4.0	6.4	D, TcVc, TvVc, S	Fixed	2,354	AC
Kent SSC	S-screw type	33,000	5.0	25.4	D, TcVc, TvVc, S	Fixed	2,354	AC
Lakeside T-2	M-scoop	NA	0	12.7	TcVv	Fixed	~700-up	AC
Manning S-4000	S-vacuum pump	3,800	6.7	9.5	D, S	Portable	1,290	DC
Markland 1301	F-pneumatic	*	18.3	6.4	TcVc, TvVc	Portable	1,095-1,350	Air & DC
Markland 101 & 102	F-pneumatic	*	18.3	6.4	D, TcVc	Fixed	594-2,189	Air & DC
Markland 104T	F-pneumatic	*	18.3	6.4	D, TcVc, TvVc	Fixed	1,094-2,644	Air & AC
Nalco S-100	F-submersible pump	28,400	7.6	12.7	TcVc, TvVc	Portable	Unk.	AC
Nappe Porta-Positer	S-flexible impeller	11,400	1.8	6.4	TcVc	Portable	225-285	AC/DC
Nappe Series 46	S-flexible impeller	13,200	4.6	9.5	TcVc, TvVc	Fixed	1,100-1,800	AC
Noascono Shift	S-peristaltic	8	9.1	4.8	Continuous	Portable	Unk.	AC
N-Con Surveyor II	S-flexible impeller	20,000	1.8	6.4	TcVc, TvVc	Portable	290-590	AC
N-Con Scout II	S-peristaltic	150	5.5	6.4	TcVc, TvVc	Portable	575-935	AC/DC
N-Con Sentry 500	S-peristaltic	150	5.5	6.4	Sequential	Portable	1,125-1,205	AC/DC
N-Con Trebler	M-scoop	NA	0	12.7	TcVv	Fixed	1,050-1,350	AC
N-Con Sentinel	user supplied	63,000	NA	25.4	TcVc, TvVc	Fixed	~2,600	AC

TABLE 8. SAMPLER CHARACTERISTIC SUMMARY MATRIX (Cont'd)

Sampler	Gathering Method	Flow Rate (ml/min)	Lift (m)	Line Size (mm)	Sample Type	Instal-lation	Cost Range ($)	Power
Peri 704	S-peristaltic	160	7.6	6.4	TcVc	Portable	Unk.	DC
Phipps and Bird	M-cup on chain	NA	18.3	NA	TcVc, TvVc	Fixed	~1,000-up	AC
ProTech CG-110	F-pneumatic	1,000	9.1	3.2	TcVc	Portable	485	-
ProTech CG-125	F-pneumatic	1,000	9.1	3.2	TcVc	Portable	695-1,205	-/AC
ProTech CG-125FP	F-pneumatic	1,000	9.1	3.2	TcVc, TvVc	Portable	925-1,610	AC/DC
ProTech CEG-200	F-pneumatic	1,000	16.8	3.2	TcVc, TvVc	P or F	1,345-2,445	Air/AC
ProTech CEL-300	F-submersible pump	~6,000	9.1	12.7	TcVc, TvVc	P or F	1,495-2,750	AC
ProTech DEL-4005	F-submersible pump	~6,000	9.1	12.7	Discrete	Fixed	3,995-4,765	AC
QCEC CVE	S-vacuum pump	3,000	6.1	6.4	TcVc, TvVc	Portable	570-1,030	AC/DC
QCEC CVE II	S-vacuum pump	3,000	6.1	6.4	TcVc, TvVc	Portable	~1,000-up	AC/DC
QCEC E	M-cup on chain	NA	18.3	NA	TcVc, TvVc	Fixed	~1,000-up	AC
Rice Barton	S-vacuum pump	Unk.	3.7	25.4	TcVc	Fixed	Unk.	AC
SERCO NW-3	S-evacuated jars	Varies	~3	6.4	Discrete	Portable	~1,000	-
SERCO TC-2	user supplied	42,000	NA	~19	TcVc, TvVc	Fixed	~2,500	Air & AC
Sigmamotor WA-1	S-peristaltic	60	6.7	3.2	TcVc	Portable	430-730	AC/DC
Sigmamotor WAP-2	S-peristaltic	60	6.7	3.2	TcVc, TvVc	Portable	650-870	AC/DC
Sigmamotor WM-3-24	S-peristaltic	60	6.7	3.2	Discrete	Portable	975-1,525	AC/DC
Sigmamotor WA-5	S-peristaltic	80	5.5	6.4	TcVc	Portable	750-990	AC/DC
Sigmamotor WAP-5	S-peristaltic	80	5.5	6.4	TcVc, TvVc	Portable	850-1,215	AC/DC
Sigmamotor WM-5-24	S-peristaltic	80	5.5	6.4	Discrete	Portable	1,225-1,775	AC/DC
Sirco B/ST-VS	S-vacuum pump	12,000	6.7	9.5	TcVc, TvVc	P or F	1,900-3,000	AC/DC
Sirco B/IE-VS	M-cup on cable	NA	61	9.5	TcVc, TvVc	Fixed	1,500-3,000	AC
Sirco B/DP-VS	user supplied	-	NA	9.5	TcVc, TvVc	P or F	1,600-3,000	AC/DC
Sirco MK-VS	S-vacuum pump	6,000	6.7	9.5	D, TcVc, TvVc, S	Portable	~1,300-up	AC/DC
Sonford HG-4	M-dipper	NA	0.5	19.0	TcVc, TvVc	Portable	325-495	AC/DC
Streamgard DA-24S1	user supplied	NA	NA	6.4	Discrete	Portable	775	-
TMI Fluid Stream	F-pneumatic	*	7.6	12.7	TcVc	Fixed	~800	Air & AC
TMI Mk 3B (Hants)	S-evacuated jars	Varies	~3	3.2	Discrete	Portable	~700-up	-
Tri-Aid	S-peristaltic	500	7.5	9.5	TcVc, TvVc	P or F	650-985	AC
Williams Oscillamatic	S-diaphragm type	60	3.6	6.4	TcVc	P or F	438	-

<u>Legend</u>: M - Mechanical
F - Forced Flow
S - Suction Lift
* - Depends on pressure and lift
NA - Not Applicable
Unk - Unknown at time of writing

maximum vertical distance that is allowed between the sampler intake and the remainder of the unit (or at least its pump in the case of suction lift devices).

"Line Size" indicates the minimum line diameter of the sampling train. "Sample Type" indicates which type or types of sample the unit (or series) is capable of gathering. Not all types can necessarily be taken by all units in a given model class; e.g., an optional controller may be required to enable taking a TvVc type sample, etc. The "Installation" column is used to indicate if the manufacturer considers the unit to be portable or if it is primarily intended for a fixed installation. "Cost Range" indicates either the approximate cost for a typical unit or the lowest price for a basic model and a higher price reflecting the addition of options (solid state controller, battery, refrigerator, etc.) that might enhance the utility of the device. Finally, the "Power" column is used to indicate whether line current (AC), battery (DC), or other forms of power (e.g., air pressure) are required for the unit to operate.

In general, the commercially available automatic samplers have been designed for a particular type of application. In the present work, however, they are being considered for application in a storm or combined sewer setting. Because of the vagaries of such an application, it is altogether possible that a particular unit may be quite well suited for one particular application and totally unsuitable for use in another. The following 12 points should be considered in the selection of a piece of automatic sampling equipment:

1. Obstruction or clogging of sampling parts, tubes and pumps.

2. Obstruction of flow.

3. Operation under the full range of flow conditions peculiar to storm and combined sewers.

4. Operation unimpeded by the movement of solids such as sand, gravel and debris within the fluid flow; including durability.

5. Operation automatic (during storm conditions), unattended, self-cleaning.

6. Flexibility of operation allowed by control system.

7. Collection of samples of floatable materials, oils and grease, as well as coarser bottom solids.

8. Storage, maintenance and protection of collected samples from damage and deterioration as well as the sample train and containers from precontamination.

9. Amenability to installation and operation in confined and moisture laden places such as sewer manholes.

10. Ability to withstand total immersion or flooding during adverse flow conditions.

11. Ability to withstand and operate under freezing ambient conditions.

12. Ability to sample over a wide range of operating head conditions.

REVIEW OF RECENT FIELD EXPERIENCE

In order to assess the efficacy of both commercially available samplers and custom engineered units in actual field usage, a survey of recent USEPA projects, many of which were in the storm and combined sewer pollution control area, was conducted. None of these projects was undertaken solely to compare or evaluate samplers, but all required determination of water quality. In the following, difficulties encountered with various elements of the liquid samplers are described.

The small diameter, low intake velocity probes found in several commercial units were felt to be unable to gather as representative a sample of the flow as could be obtained manually. There were many instances of inlet tube openings being blocked by rags, paper, disposable diapers, and other such debris. Although less a fault of the equipment than an installation practice, there were several instances of intake tubes being flushed over emergency overflow weirs, up on to manhole steps, etc., during periods of high flow and left high and dry and unable to gather any samples when the flow subsided.

There were numerous instances of pre-evacuated bottle samplers losing their vacuum in 24 to 48 hours, resulting in little or no data. Furthermore, personnel find these units with their 24 individual intake tubes virtually impossible to clean in the field. The low suction lifts on many commercial units render some sites inaccessible. In one project, three sites required manual sampling because none of the samplers on hand could meet the 5 to 6m lifts required at these sites. There were several instances of sample quantity varying with sewage level as well as with the lift required at the particular site. On at least two occasions, submersible pumps were damaged or completely swept away by heavy debris in the flow.

Within the sampling train itself, line freezing during winter operation was a problem in several projects with instances of up to 60% data loss reported. In one project, the intake line was too large, which allowed solids to settle out in it until it ultimately became clogged. There were numerous instances of smaller suction tubes becoming plugged with stringy and large sized material. A very frequent complaint, applied especially to discrete samplers, was that they gathered inadequate sample volumes for the laboratory analyses required.

Although not directly the fault of the sampling equipment itself, on one project data were lost for 14 storms due to improper sterilization of non-disposable sample bottles.

The control subsystems of commercial units probably came in for more criticism than any other. Comments on automatic starters ranged from poor to unreliable to absolutely inadequate. There were instances where dampness deteriorated electrical contacts and solenoids causing failure of apparently well-insulated parts. The complexity of some electrical systems made them difficult to maintain and repair by field personnel. Inadequate fuses and failures of microswitches, relays, and reed switches were commonly encountered. The minimum time between extraction of samples for some commercial units was too long to adequately characterize some rapidly changing flows.

Collected USEPA experience in one region, involving over 90,000 hours use of some 50 commercial automatic liquid samplers of 15 makes and models, has indicated that the mean sampler failure rate is approximately 16% with a range of 4% to 40% among types. They have found that the ability of an experienced team to gather a complete 24-hour composite sample is approximately 80%. When one factors in the possibility of mistakes in installation, variations in personnel expertise, excessive changes in lift, surcharging, and winter operation, it is small wonder that projects on which more than 50 or 60 percent of the desired data were successfully gathered using automatic samplers were, until recently, in the minority.

In fairness to present day equipment, it must be pointed out that some of the above cited complaints stem from equipment designs of up to six years ago, and many commercial manufacturers, properly benefitting from field experience, have modified or otherwise improved their products' performance. The would-be purchaser of commercial automatic samplers today, however, should keep in mind the design deficiencies that led to the foregoing complaints when selecting a particular unit for his application.

LABORATORY ANALYSIS

As an integral part of any wastewater characterization effort, the role of the analytical laboratory must not be underestimated. To be of value, the data it provides must correctly describe the characteristics or concentrations of the constituents of interest in the sample submitted to it. In many cases, an approximate answer or incorrect result is worse than no answer at all, since it could lead to faulty interpretations and incorrect decisions. The Methods Development and Quality Assurance Research Laboratory (MDQARL) (1972) has prepared a handbook for analytical quality control that provides sufficient information to allow inauguration or reinforcement of a program that will emphasize early recognition, prevention, and correction of factors leading to breakdowns in the validity of data products from water and wastewater laboratories.

It is especially important that the laboratory analyst be involved in the initial design of any wastewater characterization program. The requirements for analysis of constituents of interest vary widely and must be accounted for in the program design if it is to be sucessful. Nothing is more frustrating to the analyst, for example, than to be given a 100 ml sample and asked to perform a large number of analyses, any one of which would require about all of the available sample quantity to run correctly.

For any program, 500 ml should be considered the bare minimum sample quantity, and at least a liter will be required in many instances. It is always safer to err on the side of collecting too much sample rather than too little. Furthermore, such larger samples are less sensitive to slight pre- and cross-contamination effects that might result from some sampling equipment designs, improper sample collection and handling techniques, etc.

SAMPLE PRESERVATION AND HANDLING

Having collected a representative sample of the fluid mixture in question, there remains the problem of sample preservation and analysis. It is a practical impossibility either to perform instant analyses of the sample on the spot or to completely and unequivocally preserve it for subsequent examination. Preservative techniques can only retard the chemical and biological changes that inevitably continue following extraction of the sample from its parent source. In the former case, changes occur that are a function of the physical conditions - metal cations may precipitate as hydroxides or form complexes with other constituents; cations or anions may change valence states under certain reducing or oxidizing conditions; constituents may dissolve or volatize with time, and so on. In the latter case, biological changes taking place may change the valence state of an element or radical; soluble constituents may be converted to organically bound materials in cell structures; cell lysis may result in release of cellular material into solution, etc.

Preservation methods are relatively limited and are generally intended to retard biological action, retard hydrolysis of chemical compounds and complexes, and reduce volatility of constituents. They are generally limited to pH control, chemical addition, refrigeration, and freezing. MDQARL (1974) has compiled a list of recommendations for preservation of samples according to the measurement analysis to be performed. Since it is frequently of interest to examine a sample for a number of parameters, this list has been reproduced here as Table 9.

Standard methods for the examination of water and wastewater have been set forth by the American Public Health Association (1971) and MDQARL (1974). As regards the solids content in particular, termed residue in these publications, methods are given for determining total residue (upon evaporation), total volatile and fixed residue, total suspended matter (nonfilterable residue), volatile and fixed suspended matter,

TABLE 9. RECOMMENDATION PRESERVATION OF SAMPLES ACCORDING TO MEASUREMENT [1]

Measurement	Vol Req (ml)	Container	Preservative	Holding Time [6]
Acidity	100	P,G [2]	Cool, 4°C	24 Hrs
Alkalinity	100	P,G	Cool, 4°C	24 Hrs
Arsenic	100	P,G	HNO_3 to pH <2	6 Mos
BOD	1000	P,G	Cool, 4°C	6 Hrs [3]
Bromide	100	P,G	Cool, 4°C	24 Hrs
COD	50	P,G	H_2SO_4 to pH <2	7 Days
Chloride	50	P,G	None Req	7 Days
Chlorine Req	50	P,G	Cool, 4°C	24 Hrs
Color	50	P,G	Cool, 4°C	24 Hrs
Cyanides	500	P,G	Cool, 4°C NaOH to pH 12	24 Hrs
Dissolved Oxygen				
Probe	300	G only	Det on site	No Holding
Winkler	300	G only	Fix on site	No Holding
Fluoride	300	P,G	Cool, 4°C	7 Days
Hardness	100	P,G	Cool, 4°C	7 Days
Iodide	100	P,G	Cool, 4°C	24 Hrs
MBAS	250	P,G	Cool, 4°C	24 Hrs
Metals				
Dissolved	200	P,G	Filter on site HNO_3 to pH <2	6 Mos
Suspended			Filter on site	6 Mos
Total	100		HNO_3 to pH <2	6 Mos
Mercury				
Dissolved	100	P,G	Filter HNO_3 to pH <2	38 Days (Glass) 13 Days (Hard Plastic)
Nitrogen				
Ammonia	400	P,G	Cool, 4°C H_2SO_4 to pH <2	24 Hrs [4]
Kjeldahl	500	P,G	Cool, 4°C H_2SO_4 to pH <2	24 Hrs [4]
Nitrate	100	P,G	Cool, 4°C H_2SO_4 to pH <2	24 Hrs [4]
Nitrate	50	P,G	Cool, 4°C	24 Hrs [4]
NTA	50	P,G	Cool, 4°C	24 Hrs
Oil and Grease	1000	G only	Cool, 4°C H_2SO_4 to pH <2	24 Hrs
Organic Carbon	25	P,G	Cool, 4°C H_2SO_4 to pH <2	24 Hrs
pH	25	P,G	Cool, 4°C Det on site	6 Hrs [3]
Phenolics	500	G only	Cool, 4°C H_3PO_4 to ph <4 1.0g $CuSO_4$/l	24 Hrs
Phosphorus				
Orthophosphate, Dissolved	50	P,G	Filter on site Cool, 4°C	24 Hrs [4]
Hydrolyzable	50	P,G	Cool, 4°C H_2SO_4 to pH <2	24 Hrs [4]
Total	50	P,G	Cool, 4°C	24 Hrs [4]
Total, Dissolved	50	P,G	Filter on site Cool, 4°C	24 Hrs [4]
Residue				
Filterable	100	P,G	Cool, 4°C	7 Days
Nonfilterable	100	P,G	Cool, 4°C	7 Days
Total	100	P,G	Cool, 4°C	7 Days
Volatile	100	P,G	Cool, 4°C	7 Days
Settleable Matter	1000	P,G	None Req	24 Hrs
Selenium	50	P,G	HNO_3 to pH 2	6 Mos
Silica	50	P only	Cool, 4°C	7 Days
Specific Conductance	100	P,G	Cool, 4°C	24 Hrs 5)
Sulfate	50	P,G	Cool, 4°C	7 Days
Sulfide	50	P,G	2 ml zinc acetate	24 Hrs
Sulfite	50	P,G	Cool, 4°C	24 Hrs
Temperature	1000	P,G	Det on site	No Holding
Threshold Odor	200	G only	Cool, 4°C	24 Hrs
Turbidity	100	P,G	Cool, 4°C	7 Days

NOTES:

1. Taken from MDQARL (1974).
2. Plastic or Glass.
3. If samples cannot be returned to the laboratory in less than 6 hours and holding time exceeds this limit, the final reported data bould indicate the actual holding time.
4. Mercuric chloride may be used as an alternate preservative at a concentration of 40 mg/l, especially if a longer holding time is required. However, the use of mercuric chloride is discouraged whenever possible.
5. If the sample is stabilized by cooling, it should be warmed to 25°C for reading, or temperature correction made and results reported at 25°C.
6. It has been shown that samples properly preserved may be held for extended periods beyond the recommended holding time.

dissolved matter (filterable residue), and settleable matter. These tests do not determine specific chemical substances, but classes of matter that have similar physical properties and similar responses to ignition. The tests are basically empirical in nature and, as a result, the constituents of each form of residue are defined to a large extent by the procedures employed. For this reason, close adherence to the established procedures is necessary to insure reproducibility and comparability of results. It is interesting to note that MDQARL (1974) states that precision and accuracy data are not available for methods used to determine total dissolved solids (total filterable residue), total suspended solids (total nonfilterable residue), total solids (residue), or settleable matter.

Sampling for certain basic wastewater parameters is essential. In general these include:

a. Biochemical oxygen demand (BOD) - Used to determine the relative oxygen requirement of the wastewater. Data from BOD tests are used for the development of engineering criteria for the design of wastewater treatment plants.

b. Chemical oxygen demand (COD) - Provides additional information concerning the oxygen requirement of wastewater. It provides an independent measurement of organic matter in the sample, rather than being a substitute for the BOD test. For combined sewer overflows and stormwater, COD may be more representative of oxygen demand in a receiving stream because of the presence of metals and other toxicants which are relatively nonbiodegradable.

c. Total oxygen demand (TOD) - A recently developed test to measure the organic content of wastewater in which the organics are converted to stable end products in a platinum-catalyzed combustion chamber. The test can be performed quickly, and results have been correlated with the COD in certain locations.

d. Total organic carbon (TOC) - Still another means of measuring the organic matter present in water which has found increasing use in recent times. The test is especially applicable to small concentrations of organic matter.

e. Chloride - One of the major anions in water and sewage. The concentration in sewage may be increased by some industrial wastes, by runoff from streets and highways where salt is used to control ice formation, salt water intrusion in tidal areas, etc. A high chloride content is injurious to vehicles and highway structures, and may contaminate water supplies near the highway.

f. Nitrogen Series - A product of microbiologic activity, is an indicator of sewage pollution, or pollution resulting from fertilizers, automobile exhausts, or other sources. Its presence may require additional amounts of chlorine, or introduction of a nitrogen fixation process, in order to produce a free chlorine residual in control of bacteria.

g. pH - The logarithm of the reciprocal of hydrogen ion activity. State regulations often prescribe pH limits for effluents from industrial waste treatment plants. Provides a control in chemical and biological treatment processes for wastewater.

h. Solids (Total, Suspended, Volatile, and Settleable) - Usually represent a large fraction of the pollutional load in combined sewage. Inorganic sediments, in a physical sense, are major pollutants, but also serve as the transporting or catalytic agents that may either expand or reduce the severity of other forms of pollution.

i. Oil and Grease - Commonly found in sanitary sewage, but also appear in industrial wastes as a result of various industrial processes. Present a serious problem of removal in wastewater treatment facilities.

j. Bacterial Indicators (Total Coliform, Fecal Coliform, Fecal Streptococcus) - Indicate the level of bacterial contamination.

Where more exotic wastes are combined with stormwater and sanitary sewage, additional treatment facilities may be required for the removal of industrial byproducts and nutrients such as cyanide, fluoride, metals, pesticides, nitrogen, phosphorus, sulfate and sulfide. For planning and design of such treatment facilities, additional analyses are required in accordance with the pollutant material expected in the wastewater. This may, in turn, require significant expansion of the analysis program.

SAMPLING EQUIPMENT CLEANING

The proper cleaning of all equipment used in the sampling of wastewater is essential to assuring valid results from laboratory analyses. Cleaning protocols should be developed for all sampling equipment early in the design of the stormwater characterization program. Here also, the laboratory analyst should be consulted, both to assure that the procedures and techniques are adequate as well as to avoid including practices that are not warranted in view of the analyses to be performed.

As an example, Lair (1974) has set down the standard operating procedures for the cleaning of sample bottles and field equipment used by USEPA Region IV Surveillance and Analysis field personnel engaged in NPDES compliance monitoring. They are reproduced below for a typical automatic sampler and related sampling equipment.

<u>2-1/2 Gallon Pyrex Glass Composite Bottles</u>

1. Rinse twice with spectro grade acetone.

2. Rinse thoroughly with hot tap water using a bottle brush to remove particulate matter and surface film.

3. Rinse thoroughly three times with tap water.

4. Acid wash with at least 20 percent hydrochloric acid.

5. Rinse thoroughly three times with tap water.

6. Rinse thoroughly three times with distilled water.

7. Rinse thoroughly with petroleum ether and dry by pulling room air through bottle.

8. Dry in drying oven overnight.

9. Cap with Aluminum foil.

ISCO Glass Sample Bottles

1. One spectro grade acetone rinse.

2. Dishwasher cycle (wash and tap water rinse, no detergent).

3. Acid rinse with at least 20 percent hydrochloric acid.

4. Dishwasher cycle, tap and distilled water rinse cycles, no detergent.

5. Replace in covered ISCO bases.

Sample Tubing (1/4, 3/8 or 1/8 pexcon or tygon)

1. Do not reuse sample tubing. No cleaning required. New sample tubing is to be used for each new sampling setup.

2. Use teflon tubing where samples for organics are to be collected.

ISCO Pump Tubing

1. Rinse by pumping hot tap water through tubing for at least 2 minutes.

2. Acid wash tubing by pumping at least a 20 percent solution of hydrochloric acid through tubing for at least 2 minutes.

3. Rinse by pumping hot tap water through tubing for at least 2 minutes.

4. Rinse by pumping distilled water through tubing for at least 2 minutes.

Teflon Sample Tubing

1. Teflon sample tubing should be cleaned in the same manner as the 2-1/2 gallon pyrex sample containers.

ISCO Rotary Funnel and Distributor

1. Clean with hexane to remove any grease deposits.

2. Rinse thoroughly with hot water and a bottle brush to remove particulate matter and surface films.

3. Use a squeeze bottle of 20 percent hydrochloric acid and rinse thoroughly, rinse funnel as well as funnel and distributor depressions.

4. Rinse thoroughly with tap water.

5. Rinse thoroughly with distilled water.

6. Replace in sampler.

ISCO Sample Headers

1. Rinse entire header with hexane or petroleum ether.

2. Disassemble header and rinse thoroughly with hot tap water, using a bottle brush to remove particulate matter and surface films.

3. Rinse the plastic portion of the header with at least a 20 percent solution of hydrochloric acid. Do not use acid on the metal parts.

4. Rinse thoroughly with tap water.

5. Reassemble header.

6. Rinse all header parts thoroughly with distilled water.

<u>One Gallon Plastic Sample Containers</u>

1. Use only new bottles when sampling wastewater sources.

<u>One Quart Wide Mouth Bottles for Organics, Pesticides and Oil and Grease Samples</u>

1. Use only new bottles with teflon liners.

2. Rinse twice with petroleum ether and allow to dry.

<u>One Pint Narrow Mouth Bottles for Phenol Samples</u>

1. Use new bottles only.

<u>One Pint Narrow Mouth Mercury Sample Bottles</u>

1. Use only new bottles.

2. Rinse with at least 20 percent nitric acid.

3. Rinse at least three times with distilled water.

One Liter Plastic Storemore Cyanide Sample Bottles

1. Use only new bottles.

SUMMARY

1. There are numerous stormwater management models available and their data needs vary both as to time and spatial coverage. Therefore a knowledge of model structure and requirements is desirable to facilitate a data gathering program.

2. Regardless of the type of model involved, two types of data are required; calibration, for fitting a model to a particular basin, and verification, for model testing. There is no real difference in the data gathering requirements, but the distinction of their uses must be kept in mind. In particular, no data used for calibration should ever be used for verification.

3. The three major factors in the field data considered here are flow measurement, sampling, and laboratory analysis of the samples taken. Although they represent three different efforts, requiring different types of skills and training, they are also interdependent tasks and must be considered together in the establishment of a successful stormwater characterization program.

4. The required accuracy of the time element in stormwater characterization data is application dependent. Time synchronization of data is required, however, to allow parameter interrelationship to be determined.

5. The required quality of the field gathered data, expressed as accuracy and precision, will depend upon the ultimate use. Records should include all information that would bear on data quality, as well as estimates of accuracy and precision, to maximize data utility.

6. A flowmeter is one tool of several that must be employed for the characterization of a wastewater stream. Its selection must be based upon consideration of the overall flow measurement program to be undertaken, the nature of the flows to be measured, the physical characteristics of the flow measurement sites, and the degree of accuracy required, among other factors.

7. In view of the large number of highly variable parameters associated with the storm and combined sewer application, no single flowmeter can exist that is universally applicable with equal efficacy. Some requirements are conflicting, e.g., an open drainage ditch versus a closed conduit deep underground, and a careful series of trade-off studies is required in order to arrive at a "best" selection for a particular program and site.

8. The proper selection of flow measurement sites can be as important as the selection of methods and equipment. A clear understanding of the data requirements and ultimate use is necessary, as is a familiarity with the sewer system to be examined. There are measurement sites where no presently available equipment can operate unattended for long at a high degree of accuracy (better than ±5% of full scale).

9. Where large flows are to be measured with fairly high accuracy, considerable expense in terms of initial equipment cost, site preparation and installation, and operator training and maintenance is involved; fifty to one hundred thousand dollars should not be considered atypical.

10. The most consistently reliable flow measurement data have been taken at sites where the equipment has been calibrated in place over the entire range of flows anticipated.

11. In an extensive review of field experience in wastewater flow measurement, it was found that in most instances errors of greater than 10% seem to be the rule. It is not at all uncommon to find readings that differ from spot field checks by from 50 to 200 percent. Some wastewater discharge data are of such poor quality as to be virtually useless.

12. An automatic liquid sampler is one tool of several that is often employed for the characterization of a flow stream. Its selection must be based upon consideration of the overall sampling program to be undertaken, the characteristics of the flows to be sampled, the physical characteristics of the sampling sites, and the sample analyses that are available and desired.

13. In view of the large number of highly variable parameters associated with the storm and combined sewer application, no single automatic sampler can exist that is universally applicable with equal efficacy. Some requirements are conflicting, and a careful series of trade-off studies is required in order to arrive at a "best" selection for a particular program. Such a selection may not be well suited for a different program, and a systems approach is required for either the selection or design of automatic sampling equipment for storm and combined sewer application.

14. The proper selection of sampling sites can be as important as the selection of sampling methods and equipment. A clear understanding of the data requirements and ultimate use is necessary as is a familiarity with the sewer system to be examined.

15. Field experience with automatic sampling equipment, with emphasis on recent USEPA projects, revealed leaks in vacuum operated units; faulty automatic starters; inlet blockage and line plugging; limited suction lift; low transport velocities; complicated electrical systems; and failures of timers, micro-switches, relays and

contacts, and reed switches among the difficulties frequently encountered.

16. There is a plethora of sampling devices available in the marketplace today. These automatic samplers are of various designs and capabilities and incorporate both good and poor features. There are numerous claims (and counter-claims) made by the various manufacturers and their representatives, including limited data in certain instances, as to the efficacy of one particular piece of equipment (i.e., design approach) or another. The present state of affairs can be summarized as follows:

    (a) Comparisons of water quality data gathered using different commercially available samples demonstrate without question that there can be marked differences in results obtained with different types of equipment;

    (b) Different wastewater flow characteristics call for different equipment requirements in order to assure representative sampling; and

    (c) The results of manual sampling are extremely methodologically dependent, and data strongly indicate that they may or may not be representative of the wastewater flow in question.

17. The laboratory analyst should be involved early in the design of the stormwater characterization program. It is especially important to determine the constituents that are to be examined early in the program design to assure that sufficient quantity of sample for the analysis to be performed is delivered to the laboratory.

18. Complete and unequivocal sample preservation is a virtual impossibility. However, sample preservation guidelines are given for a number of parameters and should be closely adhered to if the results of laboratory analysis are to be meaningful.

19. It is important that cleaning protocols be developed for all sampling equipment used in a stormwater characterization program. Here also the laboratory analyst should be consulted, both to assure that the procedures and techniques are adequate and to avoid including unnecessary practices in view of the analysis to be performed.

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16. ABSTRACT

This Short Course on applications of stormwater management models is a follow-up to a course sponsored by the U.S. EPA and now available as EPA Report 670/2-75-065. The proceedings contained herein represent an entirely new set of contributions from participating speakers. The objective of this Short Course is to provide practitioners with the capability to apply specific models directly. Toward this goal, a discussion of the common components of stormwater management models first gives an overview of modeling needs. The U.S. EPA Stormwater Management Model (SWMM) is then described in detail and an illustrative case study presented. The methodology for data preparation is outlined and sample input and output data given for the Rainfall-Runoff, Transport, Storage/Treatment and Receiving Water Blocks of the EPA SWMM. A discussion of criteria for selecting models for application as either planning or design tools is then presented along with illustrations of the use of two simplified models. Finally, the techniques for collecting field data for model calibration are presented and the performance of commercially available sampling equipment assessed.

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