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DRY-WEATHER DEPOSITION AND FLUSHING FOR COMBINED SEWER OVERFLOW POLLUTION CONTROL

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FOREWORD

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

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory developes 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 for 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 deleterious effects of storm sewer discharges and combined sewer overflows upon the nation's waterways have become of increasing concern in recent times. Efforts to alleviate the problem depend in part upon the development of integrated technologies involving non-structural best management practices with structural storage and treatment concepts.

This report presents the summary results of a two year field-oriented data collection effort aimed at assessing the feasibility and effectiveness of flushing small diameter combined sewer laterals. Manual methods using a flush tanker were used to effectively remove pollutants that deposit during dry weather periods. These deposits containing substantial organic, nutrient and heavy metal pollutants would otherwise be suspended during wet weather periods and overflow into our nation's waterways. The world's first automated sewer flushing module was designed, fabricated, installed and successfully operated yielding comparable pollutant removal effectiveness as manual flushing.

Francis T. Mayo, Director
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ABSTRACT

This report summarizes the results of a two year study aimed at addressing the feasibility, cost-effectiveness and ease of application of upstream solids control as an integral part of overall combined sewer management. The project was functionally divided into four major phases. The first three phases were intensive field engineering investigations, while the fourth phase was relegated to data reduction and desk-top analytical efforts.

In the first phase of field work, four test segments on different streets in the Boston sewerage system were field flushed over an extended period using different flushing methods. External sources of fresh water, as well as sewage, were used. The experiments were aimed at quantifying the effectiveness of flushing deposition accumulations from a single pipe segment on a routine basis, as well as roughly estimating deposition characteristics within collection system laterals. Removals of 75 to 90% for grit, organic and nutrient contaminants can be expected for single manhole to manhole small diameter combined and separated sewer laterals. All flushing methods yielded comparable flushing pollutant removals. The most effective flushing method was an application of about 50 cubic feet (1.42 cubic meters) of water, injected at discharge rates exceeding 0.5 cfs (14.4 liters per second).

The second phase of field work was concerned with the problem of flushing a long flat stretch of combined sewer laterals. Flushes were injected into the uppermost manhole and pollutant levels in the flush wave passing three downstream manholes were monitored. Work was divided into two subphases. Initially, pollutant removals over the three segments were determined for different flushing conditions established in the first manhole, providing insights into flushing effectiveness over three segments of pipe. The results of these experiments indicated that a single flush at the upper end of the street was reasonably effective in removing most of the deposited load along the 675-foot (206 m.) stretch of 12-inch (30.5 cm.) combined sewer lateral. Next, settleability tests were performed for the purpose of crudely extrapolating how far beyond the flushing monitoring manholes would the materials be carried. The experiments showed that heavier grit fractions would quickly resettle, leaving the lighter solid fractions in suspension. Roughly 20 to 30% of suspended solids and about half of the BOD and nutrient loads would remain in suspension after 30 minutes of settling time. Analysis of the heavy metals results from the settleability experiments indicated that about 20 to 40% of the heavy metals would not settle within two hours of settling.

In the final phase of field operation, an automatic sewer flushing module was designed, fabricated, installed and operated on a single segment

for an extended period. Flushed pollutant loads were determined for seven flushing events, and are comparable to removals noted in the first phase of work, where flushing was accomplished by manual means using a flush truck. The purpose of this work was to begin to develop operational experience using automated flushing equipment. The state-of-the-art with respect to operational automated flushing methods, equipment and sensing interfaces has not been fully demonstrated at this point in time. The effort in this study is viewed as a pilot prototype investigation.

In the fourth phase, various predictive deposition loading and flushing criteria were generated from the large data base developed during the field programs. These formalisms allow for scanning of large-scale sewer systems to identify problem pipes with respect to deposition. The refined tools will allow for comparative analysis of upstream solids control vs selected structural options to compare program efficiencies.

This work was submitted in fulfillment of Grant No. R804578 by Northeastern University, under the joint sponsorship of the U.S. Environmental Protection Agency and the Division of Water Pollution Control, State of Massachusetts. This report covers the period July, 1976 to February, 1979, and work was completed February, 1979.

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LIST OF ABBREVIATIONS AND SYMBOLS

ABBREVIATIONS

CDF	- cumulative distribution function
cf	- cubic feet
cfs	- cubic feet per second
cm	- centimeter
DMH	- downstream manhole
EMT	- electrical metal tubing
fpm	- feet per minute
fps	- feet per second
ft	- feet
ft^2	- square feet
gpad	- gallon per acre per day
gpcd	- gallons per capita per day
gpd	- gallons per day
gpd/sf	- gallons per day per square foot
ha	- hectare
ID	- inner diameter
kg	- kilograms
km	- kilometer
l	- liter
l/m	- liters per minute
lb/cap/day	- pounds per capita per day
lb/day	- pounds per day
log	- logarithm
m	- meter
mg	- milligram
mg/l	- milligram per liter
mi	- mile
ml	- milliliter
mm	- millimeter
NPDES	- National Pollution Discharge Elimination System
ppb	- part per billion
psf	- pounds per square foot
psi	- pounds per square inch
psig	- pounds per square inch-gauge
pvc	- Polyvinyl chloride
S.D.	- standard deviation
WRNDB	- sewerage system within the area covering portions of West Roxbury, Dedham, Newton and Brookline in metropolitan Boston.
µg	- microgram

SYMBOLS

a	- Flow cross-sectional area (ft^2)
a_i	- Flow cross-sectional area, ith measurement
A	- Area of collection system (acres)
A_c	- Flow cross-sectional area (ft^2)
A_s	- Flow surface area (ft^2)
B^s	- Set of flushes performed at one pipe segment during a given phase
BOD	- Biochemical Oxygen Demand (5 day)
BOD_{SETT}	- Fraction of BOD removed by settling
COD	- Chemical Oxygen Demand
$\bar{D}$	- Mean pipe diameter of a collection system, (in)
D_i	- Pipe diameter of sewer segment i, (in)
e	- Base of the natural logarithms
E^n	- Euclidean η dimensional space
$F(h_i/D)$	- Function of flow depth i relative to diameter, D
FC	- Fecal coliform
$F_{i,j}$	- Complex stage/discharge function

F_S - Indicates the cumulative probability of a value s of the pipe slopes

G_S - Indicates complementary cumulative probability distribution

h - Water depth (ft)

$h_{i,j}$ - jth flush wave depth, flush i

h_o - Background flow depth

h_{max} - Maximum depth of flow during flush

L - Length downstream from point of flush (ft)

L' - Total length of the collection system (ft)

l_i - Length of sewer segment i

L_{PD} - Estimated length of pipe over which 80% of the total loads deposit in the collection system

L_{PM} - Estimated length of pipe over which the percentage PM of the total loads deposit in the collection system

m - Summation index

n - Manning's roughness coefficient

n_f - Manning's roughness coefficient at full pipe

n_i - Manning's roughness coefficient for ith measurement

N - Nitrogen

NH_3 - Ammonia

OF( ) - Objective functional

OP - Ortho Phosphate

OR - Overflow rate (gpd/ft^2)

p - Particle size (mm)

P - Percent TSS remaining in suspension

P(a) - Indicates the probability of a

PL - Percentage of pipe length corresponding to a percentage of PM of the loads depositing in the collection system

PL_D - Percentage of pipe length corresponding to 80% of the loads depositing in the collection system

$PL_{D/4}$ - One fourth of PL_D

PM - Any given percentage of the solids deposited in a collection system

PP - Population in service area

q - Discharge per capita, including infiltration, (gpcd)

Q - Flow rate (cfs)

$Q_{i,j}$ - Flow rate corresponding to jth sample of ith flush

QAV - Average daily dry weather flow, (cfs)

QMAX - Peak daily dry weather flow (cfs)

r - Hydraulic radius (ft)

r_i - Hydraulic radius of ith measurement

R - Multiple regression coefficient on the regression analysis

R^2 - Portion of the total variation about the mean (predicted by the regression equation) which is explained by the regression

$\bar{S}$ - Mean pipe slope of the collection system

s - A particular value of pipe slope

S - Energy slope

S_o - Pipe slope

$\hat{S}$ - Estimated energy slope

$\overset{*}{S}$ - Starting value of slope, S, in computations

S_i - Slope of sewer segment i

S_G - Mean ground slope

S_{PD} - Slope corresponding to P_{LD} in the CDF of the pipe slopes

$\bar{S}_{PD}$ - Average of pipe slopes below S_{PD} in the CDF

$S_{PD/4}$ - One fourth of S_{PD}

S_{PL} - Slope corresponding to PL in the CDF of the pipe slopes

TC - Total coliform

TKN - Total Kjeldhal Nitrogen

TKN_{SETT} - Fraction of TKN removed by settling

TP - Total Phosphorous
TS - Indicates the total mass of solids that deposit in collection system (lb/day)
TS_{a-b} - Indicates the total mass of solids that deposit in the collection system, assuming pipe bottom sediment varying from a to b (inches)
TSS - Total Suspended Solids
TSS_{SETT} - Fraction of TSS removed by settling
V - Average foreword velocity (fps)
V_L - Forward wave velocity at distance L (fps)
V_{SETT} - Settling velocity (fps)
Ve_i - Estimated flush volume i
Vm_i - Measured input volume of flush i
VSS - Volatile Suspended Solids
X - Major dimension of non-circular pipe
Y - Minor dimensions of non-circular pipe
Z_i - Percentage daily solids deposition rate in pipe segment i
Z_k - Objective functional value at kth iteration
ZS_i - Amount of daily dry weather sewage solids input along pipe segment i
α - Slope of regression line
ξ - Pre-set objective functional tolerance for terminating computation
ρ - Specific weight of water
ρ_s - Specific weight of solids
τ - Fluid shear stress
τ_c - Critical shear stress
Δt_j - jth time interval

CHEMICAL SYMBOLS

Cd - Cadmium
Cr - Chromium
Cu - Copper
Hg - Mercury
Ni - Nickel
Pb - Lead
Zn - Zinc

NOTATION

$\sum$ - Summation
$\forall$ - For all values
$\supset$ - Contained within the set

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The authors would like to commend so many of the staff of EDP and the four graduate students funded by this grant who worked so hard to make this program a success. This study, through the dilligent efforts of many, collected the largest data base of its kind available, through conditions of severe weather, workload and stress.

We would like to particularly express appreciation to members of the EDP field crew, including Paul Soper, Paul Williams, Karlson Greene and David Walsh, for their many hours of difficult times endured. Appreciation is also expressed to members of the EDP analysis team, particularly David Watson for his many hours of toil in data assembly. Certainly vital to the study, and to be commended for their efforts, are the four graduate students, Conrad Desrosiers, Robert Noonan, Kevin McBrien and Anthony Inniss, for their many hours spent collecting and analyzing the thousands of samples taken as part of this effort.

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SECTION 1

INTRODUCTION

Control of combined and storm sewer overflow is a problem of increasing importance in the field of water quality management. The control of combined sewer overflows employing structural measures such as sewer separation, storage and treatment have been used for a number of major cities in the United States. Nationwide application of these techniques for the control of combined sewer overflows would require expenditures critically taxing present and forseeable future resource allocations. New strategies are needed to reduce these costs to tolerable limits. Non-structural controls such as sewer system upgrading and active maintenance, improved catchbasin operation, street sweeping and sewer flushing are upstream collection system management practices that collectively can reduce total combined sewer pollutant loadings and accordingly the costs of downstream structural controls.

From a national standpoint, the costs of implementing controls on storm and combined sewer overflow sources of pollution are estimated to be $102.7 billion.(1) Of this total, ·66.5 billion would be required to control stormwater runoff, and $36.1 billion to control combined sewer overflows.Quite obviously, amounts of this magnitude cannot feasibly be raised. Another consideration in this area of concern is that structurally-oriented solutions for controlling combined sewer overflows generally imply periods of disruption in major urban areas. As such, the real social costs of applying these approaches may even exceed the current survey estimates that include only direct construction costs. The question then arises whether this is a viable tradeoff for the attendant water quality improvements.

The answer to this dilemma is not clear. The estimated monies will probably never be available and a more realistic view would be to spend limited dollars in maximizing the potential of existing capital outlays. The NPDES permit program recognized that the first and most logical step in fully utilizing our nation's municipal pollution control expenditures is to maximize existing treatment plant performance via nonstructural options such as increases in O&M dollars and chemical addition. (2) Similar management concepts as applied to sewer collection systems may provide large savings in controlling wet-weather pollution from combined sewer overflows. Sewer flushing is a potential low-cost, non-structural control alternative which should not be viewed as a substitutable alternative for structural control. Sewer flushing can significantly reduce overall costs when integrated together with other upstream management practices and downstream structural options as required where necessary.

The deposition of sewage solids during dry weather in combined sewer systems has long been recognized as a major contributor to first-flush phenomena occurring during wet weather run-off periods. Another manifestation of first flush, in addition to the scouring of materials already deposited in the lines, is the first flush of loose solid particles on the urban ground surface that are transported into the sewerage system. These particulates may settle out in the system and be available for flushing during periods of larger flows. The magnitude of these combined loadings during runoff periods has been estimated to range up to 30 percent of the total daily dry weather sewage loadings.

Studies in Buffalo, N.Y., have shown that 20 to 30 % of the annual collection of domestic wastewater solids settle and eventually are discharged during storms(3). Other studies have indicated similar rates of domestic wastewater solids deposition (4, 5, 6 and 7). As a result, a large residual sanitary pollution load over and above that normally carried is discharged over a relatively short interval of time, often resulting in what is known as a "first flush" phenomenon. This can produce shock loadings detrimental to receiving water characteristics.

One of the underlying reasons for considerable sewage solids deposition is the combined sewer hydraulic design. Combined sewers are sized to convey many times the anticipated peak dry weather sewage flow. Combined sewer laterals can carry up to 1000 times the expected background sewage flow. Ratios of peak to average dry weather flow usually range from 2 to 10 for interceptor sewers. The oversized combined sewer segments possess substantial sedimentation potential during dry weather periods. Dry weather flow velocities are typically inadequate to maintain settleable solids in suspension, and substantial solids tend to accumulate in the pipes. During rainstorms, the accumulated solids may resuspend and, because of the limited hydraulic capacity of the interceptor, overflow to receiving waters. Suspended solids concentrations of several thousand parts per million are not uncommon for combined sewer overflows.

Some simple calculations illustrate the potential impact of overflow on the receiving waters. If twenty-five percent of the daily pollution loading accumulates in the collection system, an intense rainstorm lasting two hours after four days of antecedent dry weather may wash the equivalent of a full day's flow of raw sewage overboard to the receiving waters. The average antecedent dry period between storm events is about four days for many areas of the United States, especially along the eastern seaboard. Furthermore, one day's equivalent of raw sewage is discharged within a two-hour period, or at twelve times the rate at which raw sewage is entering the collection system. The shock pollution loading potential of combined sewer overflow can be substantial. It is clear that the sewage treatment plant simply never sees a substantial portion of the polluting materials entering a combined sewer collection system. Furthermore, the rate at which the accumulated pollutional loads are discharged to receiving waters can represent shock loadings several orders of magnitude greater than the rate at which raw sewage is being generated by the community.

The concept of sewer flushing is to either scour and transport deposited pollutants to the sewage treatment facility during dry weather and/or to displace solids deposited in the upper reaches of large collection systems closer to the system outlet. The idea is either to reduce depositing pollutants that may be resuspended and overflow during wet events and/or to decrease the time of concentration of the solids transport within the collection system. During wet weather events these accumulated loads may then be more quickly displaced to the treatment headworks before overflows occur or would be more efficiently captured by wet weather storage facilities.

Flushing of sewer lines, although widely used around the turn of the century as a maintenance practice, is still in its infancy in regard to being viewed as a viable pollution control alternative for combined sewer systems. The concept of sewer flushing is a controversial issue since it involves generally low capital first-cost investments but high operational and maintenance costs. Federal funding mechanisms for sewerage conveyance and treatment facilities presently favor high first-cost programs with low operational costs. Federal funding does not cover operational costs. Moreover, the notion of increasing the municipal commitment for greater manpower investments runs counter to the historical trend toward decreasing public works' budgets in the area of sewerage system management. Another situation compounding the problem is that municipalities in many cases have one department involved with sewage collection and another for treatment and sewer flushing is collection system oriented. In another vein, little applied research has been performed to develop and quantify criteria for estimating deposition loadings and for flushing sewers. These criteria are necessary to quantify the need for and the extent of potential sewer flushing management programs. As a consequence, planners were heretofore reluctant to investigate flushing as a pollution control alternative in the context of overall combined sewer management. Recently, however, Congress has mandated that all alternative forms of wet weather pollution control with emphasis on the non-structural Best Management Practices (BMP) such as sewer flushing and street sweeping, be thoroughly considered in any new combined sewer management facility planning effort (8).

The tasks of identifying those portions of a sewerage system where deposition may occur and developing control policies to eliminate these conditions are indeed non-trivial. There has been simply no quantitative information available on the locations of depositing materials, their characteristics, or the hydraulic conditions minimally necessary to dislodge and transport them. The literature is sparse in this regard. Inaba (9) reported that deposition in a combined sewer district in Tokyo was limited to pipe diameters less than 21 inches (0.53 m), and that greater volumes of depositions were found in smaller diameter pipes. In 1898 Ogden (10) reported on flushing experiments conducted in Ithaca, New York.

The Storm and Combined Sewer Research Program, Federal Water Pollution Control Administration (now U.S. Environmental Protection Agency), initiated research efforts in 1966 through a contract with the FMC Corporation, Santa Clara, California, to demonstrate the feasibility of reducing pollution from combined sewer overflows by means of periodic flushing during dry weather. It was contemplated at that time to have three phases of work in this area. The first phase included a study of the overall flushing concept, small-scale hydraulic modeling, and design and development of cost estimates for constructing test equipment. The results of this work appeared in a final report entitled "Feasibility of a Periodic Flushing System for Combined Sewer

Cleansing" (11), and set the stage for the second phase, which allowed the effectiveness of flushing under various conditions be be determined.

This second phase (another contract to FMC) was completed in 1972 and the work is described in a final report entitled "A Flushing System for Combined Sewer Cleansing" (12). This work produced a flushing evaluation facility at FMC, consisting of 12 and 18 inch diameter test sewers about 1600 feet (488 m) long, supported above ground, thus allowing slope adjustments, and including holding tanks at three points along the test sewers for the flushing experiments. This phase of work developed limited experience in periodic flushing of simulated combined sewer laterals within a limited size range (12 and 18 inches diameters). The report recommended that further studies (the third phase) be made for flushing of larger sizes of pipe, of wave sequencing, and of solids buildup over longer time periods. It was also suggested that a demonstration in an operating combined sewer system be performed.

In 1974 a combined sewer management study aimed at assessing alternative strategies for the abatement of combined sewer overflows discharging to portions of Boston Harbor was completed (4,13). As part of the research work conducted during that study, a number of theoretical formalisms for prediction of dry weather deposition and flushing criteria for sewers were developed. The development of the deposition analysis techniques stemmed from critical shear stress considerations. The theoretical formalisms developed were roughly checked in the field using visual inspection techniques to assess solids accumulation. The results of that anlysis although admittedly crude, were encouraging. This model was used to analyze deposition problem segments within a service area of 3000 acres (1215 ha) entailing roughly 0.5 million lineal feet (152 km) of sewer. Roughly 3000 manhole to manhole segments were analyzed for deposition loadings and it was determined that roughly 17% of the segments contained about 75% of the estimated daily dry weather sewage depositions. It turned out that most of these segments were small diameter combined sewer laterals. Flushing criteria were empirically developed using data generated by FMC (11,12) and then were used to estimate flushing volumes.

The research results reported in this document can be considered as the envisioned third phase of the two FMC studies. Much of the theoretical work of the aforementioned Boston study was used as the starting point for this research.

1.1 Conceptual Overview of Project

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

Research Objectives

1. Test the feasibility of applying various solids control techniques as a method of deposition control in combined and sanitary sewer lines on test segments in the Boston sewer system.

2. Monitor deposition rates on a number of test segments.

3. Monitor pollutant removals including solids, organics and nutrients associated with the various solids control techniques.

4. Assess pollution oriented characteristics of both the flushed and remaining materials versus maintenance problems of grit, sand and gravel accumulations.

5. Recommend most favorable solids control techniques for operational testing by both automated and manual means.

6. Develop, test and evaluate automated control system in a field operational testing program.

7. Develop, test and evaluate manual sewer flushing techniques utilizing specially equipped water tankers in a field operational testing program.

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

9. Assess the effects of stormwater washoff on the characterization of combined sewer solids.

10. Refine existing deposition model and flushing criteria.

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

Figure 1 is an overview schematic of the effort. The program was broken into three distinct components: 1) a field feasibility analysis of various manual flushing techniques to test the feasibility of applying various techniques to single manhole to manhole sewer segments; 2) an operational testing program to assess serial flushing effectiveness using manual methods over a long combined sewer lateral and to assess effectiveness of an automated flushing; and 3) a detailed data analysis and costing phase to develop a reasonable deposition model and flushing criteria, and analyze the concept of upstream solids control as an integral part of combined sewer abatement schemes.

The feasibility analysis was aimed at answering the question of what are typical deposition rates in sewerage collection systems, what are

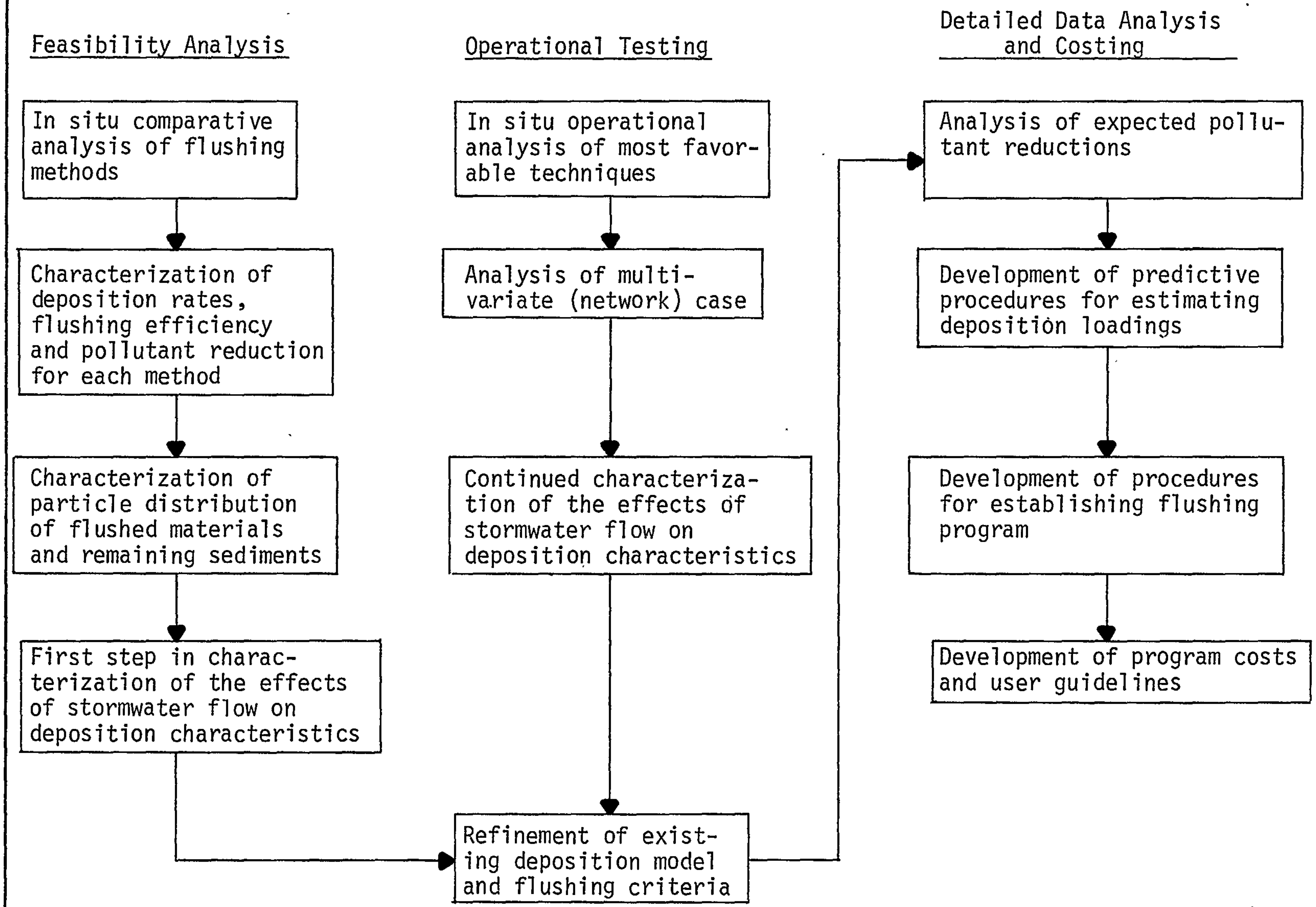


FIGURE 1: CONCEPTUAL OVERVIEW OF PROJECT

the best flushing techniques to use, and what pollutant reductions can be reasonably expected as well as supplying a wealth of data for the refinement of the existing deposition model and flushing criteria. In the operational testing subprogram the more promising strategies developed in the feasibility analysis were utilized in a field program aimed at continuing data development as well as testing the operational feasibility of such a program by both manual and automated means.

From the large data base developed during the two field programs, procedures for estimating collection system deposition leadings and flushing criteria were generated. These formalisms allow for scanning of large-scale sewer systems to identify problem pipes with respect to deposition.

1.2 Synopsis of Work Completed

The work performed can be functionally divided into four major phases. The first three phases were intensive field engineering investigations while the fourth phase was relegated to data reduction and desk-top analytical efforts.

In the first phase of field work, four small diameter (12 and 15 inch, or 0.31 and 0.39 m) test segments on different streets in the Boston sewerage system were field flushed over an extended period using different flushing methods. External sources of fresh water, as well as sewage, were used. The experiments were aimed at quantifying the effectiveness of flushing deposition accumulations from a single pipe segment on a routine basis, as well as roughly estimating deposition characteristics within collection system laterals.

The second phase of field work was concerned with the problem of flushing a long flat stretch of a 12-inch sewer lateral. The street contains five manholes and is roughly 1000 feet (305 m) in length. Flushes were injected into the uppermost manhole and pollutant levels in the flush wave passing three downstream manholes were monitored. Work was divided into two subphases. Initially, pollutant removals over the three segments were determined for different flushing conditions established in the first manhole. These results provided insights into flushing effectiveness over three segments of pipe. Next, settleability tests were performed on samples taken from flushes conducted in a similar manner for the purpose of crudely extrapolating how far beyond the flushing monitoring manholes would the materials be carried.

In the final phase of field operation, an automatic sewer flushing module was designed, fabricated, installed and operated on a single segment for an extended period. The purpose of this work was to begin to develop operational experience using automated flushing equipment. The effort in this study should be viewed as a pilot prototype investigation.

In the fourth phase, various predictive deposition loading and flushing criteria were generated from the large data base developed during the field programs. These formalisms allow for scanning of large-scale sewer systems to identify problem pipes with respect to deposition. Simplified desk-top procedures were also prepared for assessing the magnitude of deposition loadings within combined sewer collection systems and for quickly establishing the extent of flushing programs. The refined tools permit comparative analysis of upstream solids control vs selected structural options to compare program efficiencies.

Accomplishments

I. First Phase Field Program

- 86 Flushing experiments with samples analyzed;
- 5600 Analytical determinations (solid, organic, nutrient and bacterial levels) of flush wave samples;
- 150 Physical analyses of sediment scrapings;
- 600 Heavy metals determinations of flush wave samples and numerous flow measurements.

II. Second Phase Field Programs

- 36 Serial flushing experiments with samples analyzed for pollutant levels;
- 10 Serial flushing experiments - flush wave characteristics noted;
- 10 Flush wave discharge experiments using florimetric methods;
- 18 Settleability experiments;
- 5000 Analytical determinations (solids, organics, nutrients and heavy metals) of flush wave samples;
- 100 Physical analyses of sediment scrapings.

III. Third Phase Field Programs

- 1 Automated Sewer Flushing Module designed, fabricated, installed and operated;
- 7 Flushes sampled;
- 3 Storm events sampled;
- 500 Analytical determinations of flush wave samples.

IV. Analysis Phase

- All results of flushing experiments data processed and computer files prepared;
- Optimization procedure developed to compute flush wave discharge from stage information;
- Pollutant masses computed for all experiments;
- Simplified procedures developed for estimating dry weather deposition within collection systems;
- Cost analysis of flushing program impacts on sewage treatment; and
- General guidelines and cost information for sewer flushing.

1.3 Report Format

This report contains sixteen chapters. Summary conclusions and recommendations are presented in Chapters 2 and 3. Examples of procedures used and developed throughout the report are given in Chapter 16. The remaining chapters can be classified under three major headings, including: a) experimental methodologies and procedures, b) reporting of experimental results, and c) analysis and analytical desk top extension of field results into user guidelines. Table 1 shows the three major groupings and the relevant chapters.

TABLE 1

OUTLINE OF REPORT

Chapter	Title
A. Experimental Methodologies and Procedures	
4	Details of Test Segments
5	Field Procedures and Equipment Details
6	Analytical Laboratory Procedures
7	Data Processing and Computational Methods
B. Experimental Findings	
8	Single Segment Flushing Results
9	Serial Segment Flushing Results
10	Settleability Testing Results
11	Automative Flushing Results and Related Topics
C. Analysis and Synthesis	
12	Predictive Tools
13	Simplified Procedures For Estimating Deposition Loadings
14	Flushing Guidelines
15	Assessment of Treatment Costs With Sewer Flushing

The first major grouping includes all the methodological procedures used to gather experimental data. Pertinent information describing the selection of test segments, their characteristics and pre-cleaning procedures are described in Chapter 4. All field flushing procedures and details of various equipment fabricated for use in the project are described in Chapter 5. Analytical laboratory procedures and techniques used to determine pollutant characteristics of the materials flushed during the various experimental programs are presented in Chapter 6. Data processing details and computational procedures used to compute flush wave hydraulic characteristics and estimates of the flushed pollutant loadings are given in Chapter 7.

All experimental flushing results are presented in the second portion of the report. In Chapter 8 tabulations and statistical summaries of pollutant loadings flushed during the first phase of the field program are given. Serial flushing results from the second phase are given in Chapter 9. Summary findings of the settleability experiments conducted during the serial flushing program are presented in Chapter 10. These results are useful in extrapolating flushing effectiveness for pipe lengths longer than the test segments. Automated module flushing results are given in Chapter 11. An experiment is also described where flushing was immediately conducted follow-a wet weather runoff event. Pollutant loadings transported during the storm event were monitored along with flushed loadings permitting comparative assessment of runoff versus flushed pollutant loadings. The computational procedures described in Chapter 7 were used to compute the flushed pollutant loads reported in Chapters 8 through 11, using raw field and analytical laboratory data. All background sewage sampling conducted during the project is summarized in Chapter 11.

The final portion of this report contains the analysis and synthesis of the field data into user guidelines and tools. Chapter 12, entitled "Predictive Tools", presents the details of an existing multi-segment predictive deposition model (4) and calibration efforts using field flushing results described in Chapters 8 and 9. This chapter also contains an empirical procedure useful in extrapolating flushing effectiveness for segment lengths in excess of 1000 feet (305 m). This procedure was prepared using the settling column findings presented in Chapter 10, and the results of the serial segment flushing program described in Chapter 9. Simplified procedures for estimating daily deposition loadings within collection systems are given in Chapter 13. Flushing guidelines and costs are presented in Chapter 14. The last chapter presents a comparative cost assessment of the impacts of flushing on dry weather sewage treatment costs and a rudimentary analysis of the impact of dry weather sewer flushing on wet weather storage and treatment combined sewer overflow abatement programs.

SECTION 2

CONCLUSIONS

Conclusions derived from this investigation are divided into three broad categories including: general overview; technical flushing removal conclusions; and equipment and methodological conclusions.

A. General Overview

1. Sewer flushing has been shown to be an effective means for substantially reducing dry weather sewage pollutant related deposition materials in small diameter combined sewer laterals. Removals of 75 to 90% for organic and nutrient contaminants can be expected for single manhole to manhole segments. Removals of 65 to 75% for organic and nutrient deposits can be expected for serial segments up to 700 feet (213 m) and roughly 35-45% removals are projected for segment lengths greater than 1000 feet (305 m).

2. Sewer flushing is a reasonably effective means for reducing dry weather sewage grit/inorganic related deposition materials in small diameter combined sewer laterals, with removals of about 75% for single manhole to manhole flushing segments. Removals of 55-65% for solids can be expected for serial segments up to 700 feet (213 m) and roughly 18 to 25% removals are projected for segments lengths greater than 1000 feet (305 m).

3. Sewer flushing is a most effective means for suspending and transporting great distances heavy metals associated with light colloidal solids particles. Approximately 20-40% of heavy metals contained within sewage sediments would be transported by flush waves at least 1000 feet (305 m) and probably much further, including cadmium, chromium, copper, lead, nickel and zinc.

4. Extensive field experience has indicated that sewer flushing by manual means (water tank truck) is a simple reliable method of combined sewer solids control for smaller diameter laterals and trunk sewers.

5. Recommendations 1-4 are based on extensive field flushing experience using manual flush methods with a water tanker. An average of 300 gallons per flush were used during the experiments, representing about 0.5% of the total daily water consumption in the area. Various flushing methods were investigated, including different combinations of externally supplied flush volumes and rates together with backup and release using sewage. All methods yielded comparable flushing pollutant removals. The most effective flushing method for 12-15 inch (0.31-0.39 m) lateral sewers was an application of about 50 cubic feet (1.42 cubic meters) of water injected at discharge rates exceeding 0.50 cfs (14.4 liters per second).

6. Initial experiments with an automated sewer flushing module indicated great operational promise with comparable removals to other techniques for flushing. The module was designed to backup sewage with quick release for establishing the flush wave and requires no external water supply.

7. Sewer flushing has an additional inestimatable benefit, in that the "hands-on" presence of field crews continuously surveying the collection systems would encounter and note possible malfunctions that would otherwise go undetected. Correction of these malfunctions, such as broken, clogged or constricted pipes, or inoperative regulators and/or tide gates, could substantially reduce potential overflow volumes and pollutant loads, and possibly upstream flooding.

8. There were substantial sediment beds in the test segment sewer laterals prior to commencing the flushing program. Except for the heaviest grit particles, sediment deposits in the sewers were maintained at minimum levels by flushing during the experimental period. Inspection of the laterals several months after the flushing programs were terminated revealed that sediment layers returned to pre-project conditions. Sewer flushing is therefore a viable means for minimizing grit accumulations, which can reduce hydraulic capacity.

9. An analysis of the potential impact of nominal sewer flushing programs vs the costs of primary and conventional secondary treatment, including solids handling, indicated that annual operational and maintenance costs would rise about 3 to 6% depending on the type of treatment plant. Therefore, sewer flushing would not significantly increase treatment costs.

B. <u>Technical Flushing Removal Conclusions</u>

1. During the first phase of operation 86 separate flushing experiments were conducted during the period of August 30, 1976 to November 12, 1976. Roughly 20 flushes on a 3-4 day interval were accomplished for each of 4 test manhole to manhole segments. Three different methods of manual flushing were performed. The first method consisted of backing up the upper end of the flushing manhole with an inflatable rubber stopper with quick release. The other two methods were gravity and pressurized dump discharge into the flush manhole. Pollutant removals for the flushing experiments indicated that all methods provide about the same degree of removal. The best method is an external source high volume/high rate flush. Average flush volume during this experimentation period was 300 gallons (1.13 cubic meters). A minimum flush volume of 225 gallons (0.85 cubic meters) is recommended for a single manhole to manhole segment of a small diameter (12-15 inch, or 0.31-0.39 m) sewer lateral. The periodic flushing removed the domestic sewage deposits that accumulated between flushing events and maintained minimal levels of grit, rock and debris.

2. The average flushing pollutant removal rates normalized by both antecedent days between the flushing events, and tributary population during the first phase program, were the following for separated sewer segments: COD = 7.78 (1.71×10^{-2}), BOD = 3.43 (7.56×10^{-3}), TKN = 0.18 (3.96×10^{-4}), NH_3 = .05 (1.10×10^{-4}), TP = .05 (1.10×10^{-4}), TSS = 8.89 (1.96×10^{-2}), and VSS = 6.5 (1.43×10^{-2}) grams/capita/day(lbs/capita/day). Similar average results for the

combined sewer flushing laterals were: COD = 22.0 (4.84×10^{-2}), BOD = 7.98 (1.76×10^{-2}), TKN = 0.64 (1.41×10^{-3}), NH_3 = 0.22 (4.84×10^{-4}), TP = 0.14 (3.08×10^{-4}), TSS = 19.03 (4.19×10^{-2}), and VSS = 12.21 (2.69×10^{-2}) grams/capita/day (lbs/capita/day). The flushed removals on the combined sewer segments were about two to four times the levels found on the separated sewer streets.

3. Sediment scrapings of sanitary deposits prior to flushing during the first phase program ranged form .026 to .037 lbs per linael foot (39.4 to 55.8 grams per meter).

4. Composited flush wave samples from the first phase were allowed to settle for periods of approximately four hours prior to heavy metals analyses. Heavy metals analyzed included: nickel, chromium, cadmium, lead and mercury. Heavy metals concentrations in the supernatant of settled flush wave were very low. Heavy metals levels in the settled fraction were high. Average heavy metal results for the combined sewer test segments were roughly twice those of the separate sanitary sewered streets, indicating the impact of wet weather street wash load.

5. During the second phase field flushing program, 6 serial flushing experiments were performed on three flat combined sewer segments, 675 feet (206 m) in length. Three flushes were accomplished per experiment and pollutant masses were determined at each of three downstream sampling manholes per flush. The last flush per experiment was always a maximal flush meant to completely remove any residual pollutants. The average results for all six experiments indicated that most of the loads for all three segments were removed during the first flush. Nearly 88% of the total BOD load transported by the first sampling manhole was accomplished by the first flush. The removals slightly decreased for two other sampling manholes further downstream. The experiments indicate that a single flush at the upper end of the street was reasonably effective in removing most of the deposited load along the 675 foot (206 m) stretch of 12 inch (0.31 m) combined sewer lateral.

6. Settleability tests were performed on composited flush wave samples from the three sampling manholes in the second phase flushing operation. The experiments showed definite shifts in suspended solids/settling velocity distribution from the first to the third downstream sampling manholes, indicating that heavier grit fractions would quickly resettle leaving the lighter solid fractions in suspension. About 20 to 30 percent of suspended solids would remain in suspension after 30 minutes of settling time. The fractions of volatile solids relative to the suspended solids increased both with settling time during the experiment and with the distance downstream from the flushing manhole. Distribution of COD and BOD versus the settling time showed the similar characteristics as the suspended solids settling behavior. About half of the initial BOD levels would remain in suspension after 30 minutes of settling. Organic and nutrients concentrations correlated extremely well with both TSS concentrations and settling velocities (correlations ranged from 0.6 to 0.8).

7. Analysis of the heavy metals results from the settleability experiments indicated that about 20 to 40% of the heavy metals present in the composited flush waves would not settle within two hours of settling. The balance of the metals were associated with heavier solids particles and rapidly

settled. The metals associated with the lighter colloidal fractions could easily be transported by flushing during dry weather to treatment facilities or, alternatively, would be transported to receiving waters during overflow periods. Sewer flushing is therefore a viable means for reducing an important and significant source of heavy metals in overflows.

8. An automated sewer flushing module consisting of an oil-on-air hydraulic gated device triggered by an automatic time clock was designed, fabricated, installed and successfully operated on a daily basis for a 5 1/2 month testing period. The device backed up sewage to predetermined levels and then, retracting, induced a flush wave. Flushed pollutant loads were determined for 7 flushing events, and are comparable to removals noted in an earlier phase of work where flushing was accomplished by manual means using a flush truck.

9. A special field measurement program was conducted to monitor pollutant loads during storm events and then to immediately flush the segment following the end of the storm event. The results indicated that runoff from slight to moderate rainfall events only partially removed fresh sanitary deposits, whereas the flushing removed significant organic loads. Significant organic deposits were observed after the storm event and none after the flushing event. Several inferences can be drawn from the data. The flush wave provided the necessary turbulence to suspend, entrain and transport fresh organic deposits. Flushing frequency intervals need not be determined by return periods defined by slight to moderate rainfall events. "First flush" runoff loads during intense storms can be the results of long-term sewage solids deposition accumulations occurring over both dry and moderately wet runoff periods.

10. Background sewage concentrations were measured at four upstream sewer laterals over a two year period. Pollutant concentrations found in the laterals were much higher and with far more variability than levels normally encountered further downstream at treatment plants. This phenomenon has been observed on numerous occasions in other locations.

11. Field flushing results from the second phase program and the analytical results of the settling column tests were used to develop an empirical model relating the percentage of flushed masses remaining in suspension at downstream points as a result of upstream flushing. This model was favorably compared with other flushing pollutant removal data from the serial flushing experiments. The model indicates that at least 20% of flushed solids would be transported at least 1000 feet (305 m) from a point of flush. Similar estimates for organic and nutrient flushed loadings are 45 to 50% for the same distance. Most combined sewer laterals do not exceed this distance and may discharge into trunk sewers with high shear stress characteristics, that is, good solids carrying capacity. These results imply that sewer flushing of combined sewer laterals could result in significant reductions of dry weather deposits containing pollutant related contaminants.

12. An existing generalized procedure for estimating daily dry weather sewage solids deposition loadings within each manhole to manhole segment of an entire collection system network was roughly calibrated using field flush-

ing pollutant removal results. This procedure is therefore recommended for application where detailed segment-by-segment estimates of deposition rates are desired. This procedure can only be applied when the hydraulic characteristics, such as the pipe length, size, shape and slope, are known for each segment.

13. A simplified methodology was prepared for providing first-cut assessments of the total amounts of solids (lb/day) that deposit in a sewerage collection system; and the extent of the collection system over which the deposition takes place. The complex distribution-parameter dry weather sewage deposition model described in conclusion 12 was applied to 75 separate and combined sewer collection systems in eastern Massachusetts to generate estimates of solids deposited daily per system (lb/day). These estimated loads were then regressed with selected variables representing the physical characteristics of these collection systems including total pipe length, service area, average collection system pipe slope, average pipe diameter and other more complicated variables representing various points on the lower end of the collection system pipe slope cumulative density function. Four alternative predictive single term power functions were developed from the regression analysis. The degree of fit of the non-linear functions to the data set were remarkably high. The R^2 values of the alternative models ranged from 0.85 for the simplest approach requiring little external data analysis and preparation, up to 0.95 for the most complex model requiring substantial external engineering and data reduction analyses. These simplified procedures are recommended for general application in combined sewer management planning.

14. In addition to the general predictive procedures for estimating solids deposition within collection systems as a function of sewer shed characteristics (as described in conclusion 13), the effects of sewer system age and maintenance on solids deposition was simulated by considering prior sediment deposits to develop multiplicative coefficients to the predictive equations mentioned in conclusion 13.

15. The first phase field flushing results were used to develop mean ratios between other pollutants, such as BOD, COD, TKN, TP, NH_3 and VSS with suspended solids. This therefore permits the use of the predictive equations for total solids deposited (described in conclusions 13 and 14) to be used for the estimation of other pollutants.

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

C. Equipment & Methodological Conclusions

1. A specially designed water tanker equipped with two 1000-gallon tanks mounted on a steel I-beam skid was fabricated for delivering flush waters under controlled discharge conditions. The tanker was equipped with a

pneumatic system to pressurize the tanks. The operation under gravity conditions provided a controlled flush release of 35 to 50 cubic feet (0.99 to 1.42 cubic meters) at a rate of 0.25 to 0.50 cfs (7.2 to 14.4 liters per second). Under pressurized conditions the same volumetric range of flush was accomplished at a rate of 0.5 to 1.25 cfs (14.4 to 35.4 liters per second). The water tanker was successfully used for 300 different experiments over a 1 1/2 year period.

2. An automated sewer flushing module was designed, fabricated, installed and successfully operated for an extended period. The device developed was an air-operated gate capable of backing up sewage flows to predetermined levels and then suddenly retracting, inducing a flush wave. The flushing gate was controlled from a master control timer capable of pre-programmed flushing of varying sequences with flushing intervals ranging from 6 to 72 hours. The entire unit was powered by a 12 volt automobile battery. Air supply to the system was by means of a small high pressure air cylinder requiring replenishment every 150 flushes. The device worked remarkably well with very little down time for repairs. The device is amendable to package fabrication and installation.

3. A special dye injection procedure was developed to provide a practical and reliable method of measuring steady state and non-steady state discharge for waste streams containing suspended solids levels up to 10,000 mg/l. Non-steady state discharge levels of flush waves were determined. The procedure consisted of three distinct operational units including: a) the high pressure injection nozzle system; b) the pumping and air separation unit; and c) a flow-through cell equipped florimeter with recording readout. The dye injection experiments utilized Uracine dye with a special filter system to eliminate any background interference. The procedure is recommended for similar difficult-to-measure waste streams.

4. A specially designed settling column and procedures were developed to perform settleability tests on flush wave samples. A yoke-frame installation was devised to permit axial and transverse mixing of the column before experimentation since the settling velocities for a considerable portion of the flush wave solids were extremely rapid. U.S. Environmental Protection Agency is currently investigating refined adaptation of this design for settling column analyses of combined sewer overflow samples.

5. A mathematical programming procedure was developed and utilized to determine the parameters of a non-steady state loop-rating curve approach for estimating flush wave discharge from recorded stage levels. The approach minimized the variance between computed and measured flush truck volumes, and substantially reduced the error in variance between measured and computed flush volumes in defining parameters of stage discharge curves. The methodology can be logically extended to estimate stage/discharge rating curve parameters for a multi-flow measurement site system.

SECTION 3

RECOMMENDATIONS

Results of the present study indicate that sewer flushing effectiveness dislodges and resuspends deposited pollutants in small diameter combined sewer laterals. Most of the flushing experiments were conducted by manual means in which a water tanker was used to deliver the flush volumes. One automated flushing module was fabricated and tested for a five and one-half month period.

The study yielded a massive amount of data on a subject primarily a source of conjecture in the past. As noted in Chapter 2, many significant conclusions were reached as to the effectiveness of sewer flushing as a combined sewer central measure, as well as indications generated as to the source and nature of combined sewer transport and subsequent overflow during storm events. As in many research and demonstration efforts, the sewer flushing project also yielded a number of interesting and important questions. The most significant of these questions have been coalesced into a series of recommendations for further study. Additionally, since this sewer flushing effort was aimed primarily at testing the potential of sewer flushing as a combined sewer pollution central method, and since the results generated were quite positive, a more comprehensive study aimed at overall development and testing of an automated sewer flushing network is proposed. The following are the recommendations of this effort.

1) The scope of this study was limited to a maximum flush length, carefully sampled and analyzed, of approximately 675 feet (206.1 m). Longer flush length studies were conducted, but were limited to visual observation of flush wave characteristics. This study did generate some "rough-cut" wave/solids travel predictions, but actual field investigations should be conducted to verify their results.

2) Under any circumstance, single input flushing removes and should successfully transport a minimum of 20% of the solids and up to 50% of BOD, COD, nutrients and metals a distance in excess of 1000 feet (305 m). This distance should be sufficient in most systems to transport that fraction of the deposited load far enough to reach a trunk sewer or interception capable of retaining the materials in suspension until they reach the wastewater treatment plant. To further improve flushing efficiency, especially in small upstream networks or areas where upstream offline storage is considered viable, booster flushing to further push solids to a downstream location might be advantageous. This type of additional or booster flush was not tried as part of this effort and should be investigated, especially to further define the movement potential within a long, highly depositing sector.

3) Flush waves were shown to remove significant masses of heavy metals with large percentages, up to 50 %, remaining in suspension after extended quiescent periods. Initial settling column tests conducted as part of this study indicated that most of the heavy metals analyzed including nickel, chromium, cadmium and lead, showed little tendency to settle in the column in periods extending to 2 hours. Tests conducted in an earlier phase of the project where flush samples were allowed to settle for periods of approximately 4 hours indicated large percentage settling of heavy metals after the time period. A definite question exists, possibly due in part to difficulties with existing settling column procedures, especially pertaining to light near-colloidal fractions. As part of an ongoing effort, EPA has funded a study to develop an improved settling column and procedure, and test the unit on combined sewer overflows. Due to the importance of heavy metals as a potential health hazard and the potential for sewer flushing to remove large percentages of the metals from the combined system between storm events, further analysis of the settleability of flush wave entrained heavy metals should be conducted using the new procedure if it proves successful. The metals program should be coupled with the long-distance flushing program mentioned as recommendation 1 to assess metals movement and subsequent overflow pollution source.

4) To the authors' knowledge, the automated sewer flushing module designed and built for use in this project, represents a first of its kind ever actually applied and tested in the field. Results of approximately 5 1/2 months of operation were very positive. The device as tested was quite simplistic, aimed primarily toward prototype concept testing rather than rigorous proofing of equipment. The automated module represents only one of a spectrum of different types of simple devices that could be developed for sewer flushing. Prior to widespread application of sewer flushing technology, a further effort should be expended testing differing automated flushing approaches in as rigorous a fashion over an extended time period.

5) Sewer flushing over a limited time period proved to be a technical viable combined sewer abatement method. Comparative assessments made between sewer flushing and removals due to wet weather indicated that flushing was far more efficient than runoff from low to moderate intensity storms in moving deposited combined sewer solids. This fact was further iterated by the continuous decrease in sediment levels in the flushing segments during the active program. Unfortunately, the program was of quite limited duration, leaving the long-term question of dry vs wet weather control still somewhat unresolved. Based on the results of this study a long-term program assessing dry/wet weather flushing/abatement performance should be instituted to generate real operational data on effectiveness and cost to compare to other abatement technologies.

Proposal for an Overall Assessment of Long-Term Automated Sewer Flushing

Results of the sewer flushing research project indicated that sewer flushing by manual or automated means yielded similar pollutant removals. Due to the large number of 201 facilities plans being developed nationally for combined sewer systems, and the results of several major cost effectiveness studies recently released indicating the need for alternative, realistic combined sewer abatement technologies, the authors recommend that the following study be immediately implemented. In this plan, concept of a network of automatic flushing modules would be operationally tested for an extended period of time. The purpose of this study would be to develop operational experience with long-term automated flushing for making sound technical and economical comparative assessments with other forms of combined sewer overflow abatement technology, as well as to provide empirical evidence of long-term collection system pollutant removal effectiveness over both dry and wet weather periods.

Flushing by manual means has been shown to be an effective, reliable pollution control mechanism, subject to the unresolved issues previously mentioned. Initial testing of an automated flushing technique has been shown to be equally as successful as manual flushing.

The widespread and successful use of process computers coupled with the advanced state of telemetric technology would suggest that a network of automatic flushing modules could be centrally controlled to operate in-line storage devices and/or to trigger external water sources for inducing flush waves.

Unfortunately, a number of implicit underlying assumptions violate present knowledge, including: (a) perfected automated flushing systems do exist; (b) the state-of-the-art in long-term sewer monitoring in sewer lines is reasonably perfected allowing for totally automated control; and (c) an automated sewer flushing system would take care of debris accumulations such as sticks and rags commonly occurring within sewer sustems.

First of all, perfected automated flushing systems do not presently exist. Except for the conceptual effort and well-controlled experiments of the FMC Corporation (12) which never actually demonstrated automated flushing modules in the field, and the limited experience with one flushing module in this study, no real work has ever been performed of the magnitude required for proper system evolution. Automated flow regulation of large interceptors and trunk lines is being conducted in Seattle and Detroit using movable sluice gates, and in Minneapolis - St. Paul employing inflatable dams (14), but no significant work has been done in small pipes where the pollutant/mechanical difficulties lie. The nature of deposited solid movement and flow patterns in upstream networks is completely different than that found in major collection pipes requiring considerably different sensing and control devices. Debris clogging and grit accumulations can change dramatically from day to day. Maintenance of a complex automated computer controlled flushing system would be a near impossibility for most municipalities. The current trend in municipal waste treatment is away from complex facilities entailing complicated automative equipment because of the lack of resources and skilled labor required for successful operation.

Secondly, the state-of-the-art in automated flow monitoring of sewer systems is not perfected. There presently exists literally dozens of different kinds of automated sewer flow monitoring devices ranging from a liquid-level sensor using floats, or moveable probes to sense the liquid surface, to pressure tranducers and various bubble sensors to sense pressure and indirectly liquid depth, to continuous velocity meters using dyes, ultrasonics and strain gages. Newly developed sonic level sensors which are non-intrusive could be applied for the determination of bed load deposition accumulation. These devices are as of yet untried in this situation, and could possibly function with little trouble in a limited application. However, in a larger flushing program requiring hundreds of these sensors, operational difficulties might arise, crippling the effectiveness of the telemetry system. In essence, no system has been properly tested, especially in small pipes. The state-of-the-art in development of automated sewer flushing systems and control devices is presently a considerable distance from the point of sophisticated computerized telemetric feedback systems.

In order to address these points, as well as answer the many questions raised in the conclusions of this study, a dry/wet weather program is envisioned integrating recommendations 1-5 with a step-by-step development of automated flushing systems. This proposed program would emphasize testing, evaluation and development of feasible system components and assess overall system performance. The recommended study would demonstrate approximately five automated dry-weather sewer flushing modules in a branch sewer network or a system of five modules encompassing 1-3 miles (1.69-4.3 km) of sewer length, coupled with wet-weather first-flush control by upstream off-line storage.

In the dry-weather deposition oriented program 3-5 automated flushing modules would be constructed and installed in an enclosed subsystem network to allow for assessment of overall pollutant reduction efficiency. A representative scheme for the envisioned program is depicted in Figure 2. Types of modules could probably include a hydraulically operated gate system, an expandable diaphragm positioned above the flow and inflating down into the sewer, an auto-siphon type system and a jetted siphon. Various types of flow sensing devices would be used to attain optimum results. Flushing would be operated as a staged-sequential network controlled by a central controller located within the network. The concept would assess long duration flushing, long distance flushing and booster flushing as illustrated in Figure 2 where, for example, the deposits just downstream of module 1 are first moved down to area A, then are displaced to area B by the automatic siphon at mode 2, and finally flushed out to the trunk sewer by the forced jet of module 3.

It is envisioned that 4-6 storm events would be monitored before the installation of the flushing system, and approximately 10 while the system is in operation. In addition, flushes and outputs from the system would be regularly monitored during dry weather. The program would be split into four seasons whereby six weeks would be monitored during each season. On a monthly basis, all lines within the subsystem would be TV-inspected and recorded on video tape to visually assess overall results of the flushing program.

In order to provide maximum wet weather control, an upstream off-line storage module could be placed downstream of the flushed network to capture residual wet weather first flushes emanating during wet weather. In this system, outputs from the various flow sensors would be transmitted to a central controller located at the storage module or other remote site.

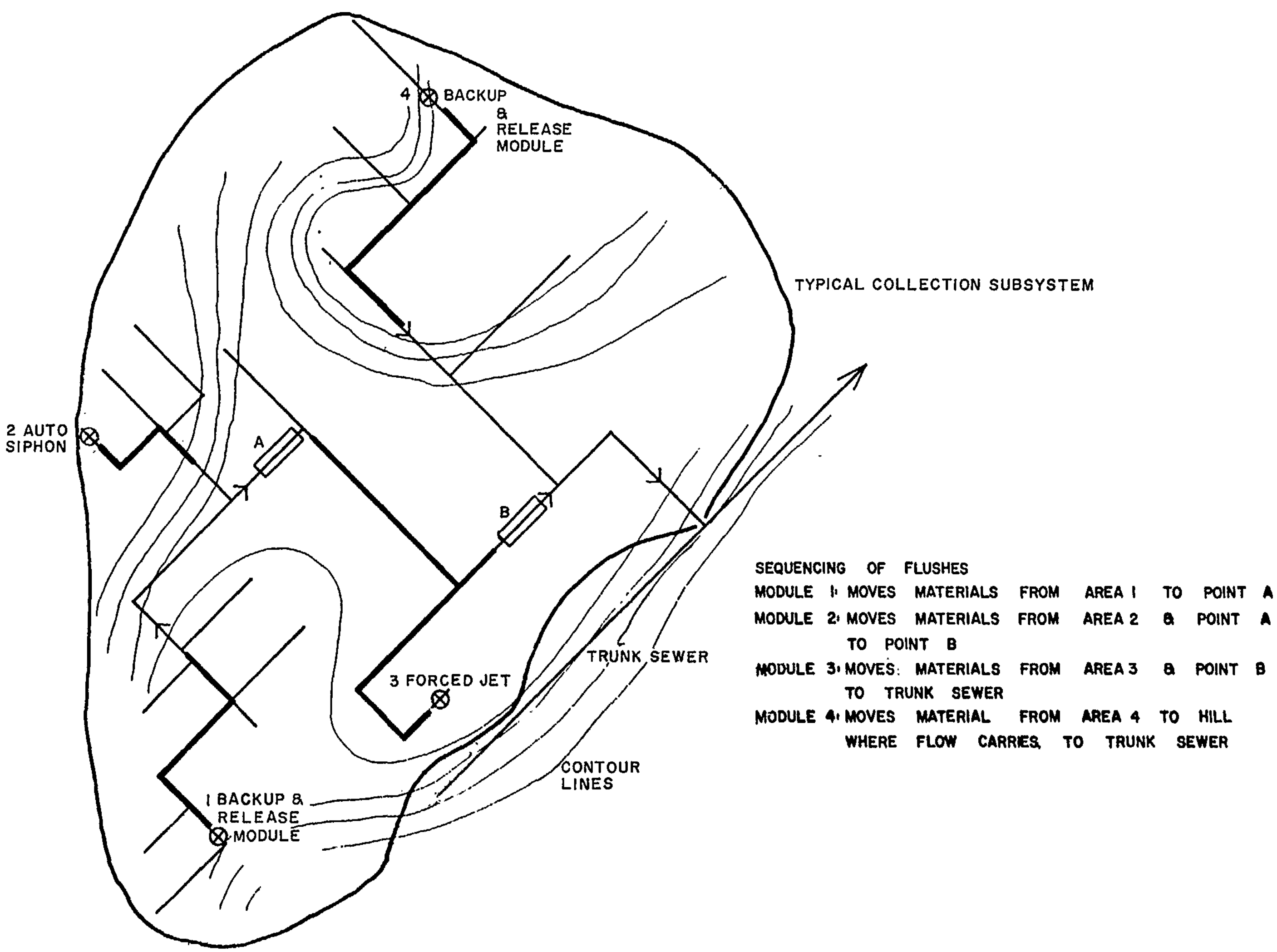


FIGURE 2 PROPOSED AUTO FLUSHING CONCEPT

During wet weather, the flushing gates or dams would be used to concentrate a major first flush and duct it into an adjacent off-line storage tank for detainment until after the storm has subsided. When flow levels in the intercepting line had sufficiently decreased, the storage tank would discharge back into the system where the waste would continue to the treatment facility. Equipment development and testing would follow a step-by-step concept utilizing simple, proven pieces of equipment as much as possible. Monitoring of the wet weather program would include input into the storage tank as well as overall network emissions.

The proposed program would provide a means for EPA to develop, test and evaluate flushing as a realistic combined sewer pollution control alternative capable of incorporation directly into 201 facilities plans. Project duration would total 2 years, at which time all aspects of sewer flushing, as well as testing of sewer flushing (upstream capture), would be assessed, operation performance evaluated and real costs generated.

SECTION 4

DETAILS OF TEST SEGMENTS

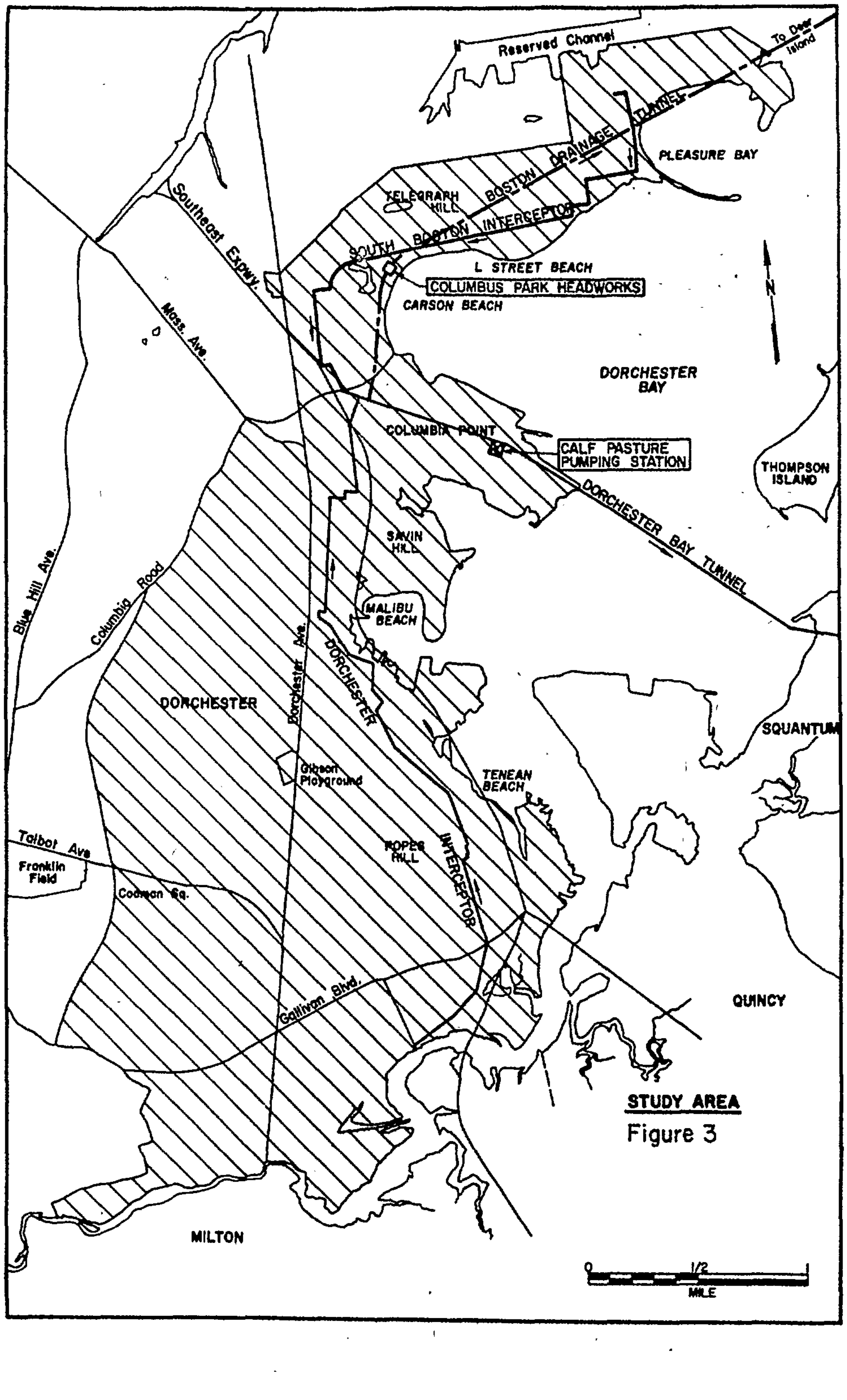
4.1 Foreword

Discussion of the selection process used to choose the experimental flushing segments is presented in Section 4.2. Descriptions, maps and photographs of the test segments flushed in the three phases of experimentation are given in Section 4.3. Activities relating to the preparation of the test segments for the flushing experiments are described in Section 4.4.

4.2 Selection of Test Segments

The combined sewerage system servicing the high-density residential communities of Dorchester and South Boston in the metropolitan Boston area was investigated to select potential flushing test segments. The topographical relief of the 3600 acre (1458 ha) area encompassing portions of both communities is moderate to hilly. This area is indicated by crosshatching in Figure 3. This particular sewerage system was chosen for investigation because the area had been thoroughly mapped and intensive field physical surveys had been accomplished in a prior study (4). In addition, a computerized sewer atlas had been previously prepared, consisting of nearly 3000 manhole to manhole segments representing about 0.5 million lineal feet of sewer. This information had been used as input for a deposition model to estimate daily dry weather deposition loadings. A number of pipe segments with high dry weather sewage deposition rates were identified in that study and physically surveyed. Rudimentary flushing experiments were conducted during that study in an attempt to dislodge heavy sanitary deposits. This particular sewerage system was chosen because a wealth of detailed information existed. Other areas in Boston would have been equally as suitable for conducting the flushing experiments, but the cost of basic mapping inventory and acquiring site-specific knowledge would have been prohibitive.

It was decided at the onset of the project that experimentation would be limited to small diameter (12-18 inch) pipe segments. This limitation on pipe size was imposed for three reasons. First of all, the underlying motivation of the project was to investigate the feasibility of flushing upstream pipe segments as an integral component of an overall source control management program. Secondly, prior experience with this system indicated that most of the predicted daily dry weather deposition loadings were contained within the small diameter laterals. Thirdly, project budgetary constraints precluded flushing large diameter sewer pipe segments.



STUDY AREA

Figure 3

Existing large scale base maps, small scale street maps, topographic maps, and pertinent existing sewerage system maps for the selected areas were collected and visually analyzed to detect probable candidate sites for the field flushing experiments. Copies of plan maps and City of Boston detailed sewer maps (1"=79') were obtained for a number of candidate areas to verify pipe detail. This information in conjunction with past experience with the sewerage system enabled selection of ten potential candidate sites.

The existing deposition model results for the area were further utilized to study in detail the selected pipes with respect to deposition loadings. The candidate sites were then field inspected to roughly assess adequacy for field flushing experiments. Assessments were made on the basis of existing deposits, pipe characteristics, access, traffic and safety.

4.3 Description of Test Segments

After a careful review and inspection program, four streets were selected for the flushing experiments. These streets are all in Dorchester characterized by high density, 3-story multi-family dwellings. General location of the segments are shown in Figure 4. Two of the test segments located on Port Norfolk and Walnut Street are served by flat combined sewer laterals of 12 and 15-inch circular pipe, respectively. The other two test segments on Shepton and Templeton Streets, are serviced by separate sewer laterals of 12 and 15-inch circular pipe, respectively. There are downspouts on both streets connected to the sanitary sewer. Although these two segments are separated, considerable stormwater inflow occurs during storm events.

A plan map of the two combined sewer laterals on Port Norfolk and Walnut Streets is shown in Figure 5. The map was prepared using City of Boston assessor maps and relevant detailed sewer plan and profile maps. The map shown in Figure 5 also contains the number of residences and occupants per dwelling. This information was gathered from recent census tract information. Due to the residential nature of the community, the number of dwellings and occupants were reasonably stable but did vary over the course of the study as a result of several fires and ensuing demolition.

The combined sewer lateral test segment on Port Norfolk Street was used for both the first and second phase experiments. Starting from the uppermost manhole on the west end of Port Norfolk Street, the first phase experiments were conducted using the sewer segment located between the third and fourth manholes. Flushes were initiated at the third manhole and sampled at the fourth manhole. Further details of the flushing and sampling procedures are given in Chapter 5. During the second phase of work, the second manhole on the west end of Port Norfolk Street was used as the flush injection point and the flush waves at the next three downstream manholes were sampled. The 15-inch combined sewer lateral test segment on Walnut Street, used only during the first phase program, is located on the westerly side of Walnut Street. Photographs of both segments are shown in Figure 6. The photo on Port Norfolk Street was taken from the easterly end of Port Norfolk Street, that is, opposite the direction of flow. The photo on Walnut Street was taken at the most westerly end of Walnut Street again in the direction opposite to the direction of flow. Street grades are flat for both streets. Characteristics of these

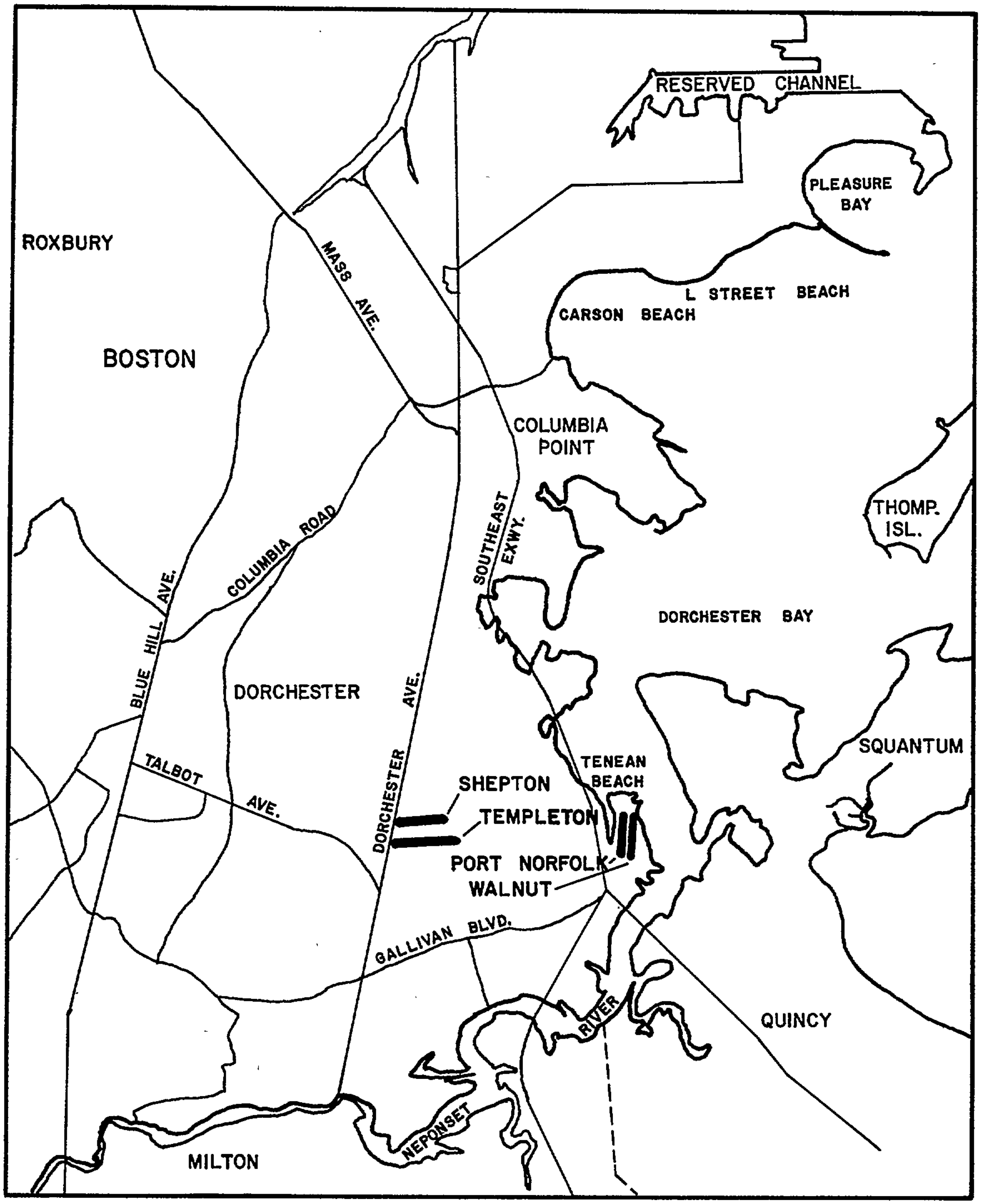


FIGURE 4 LOCATION OF FLUSHING SEGMENTS

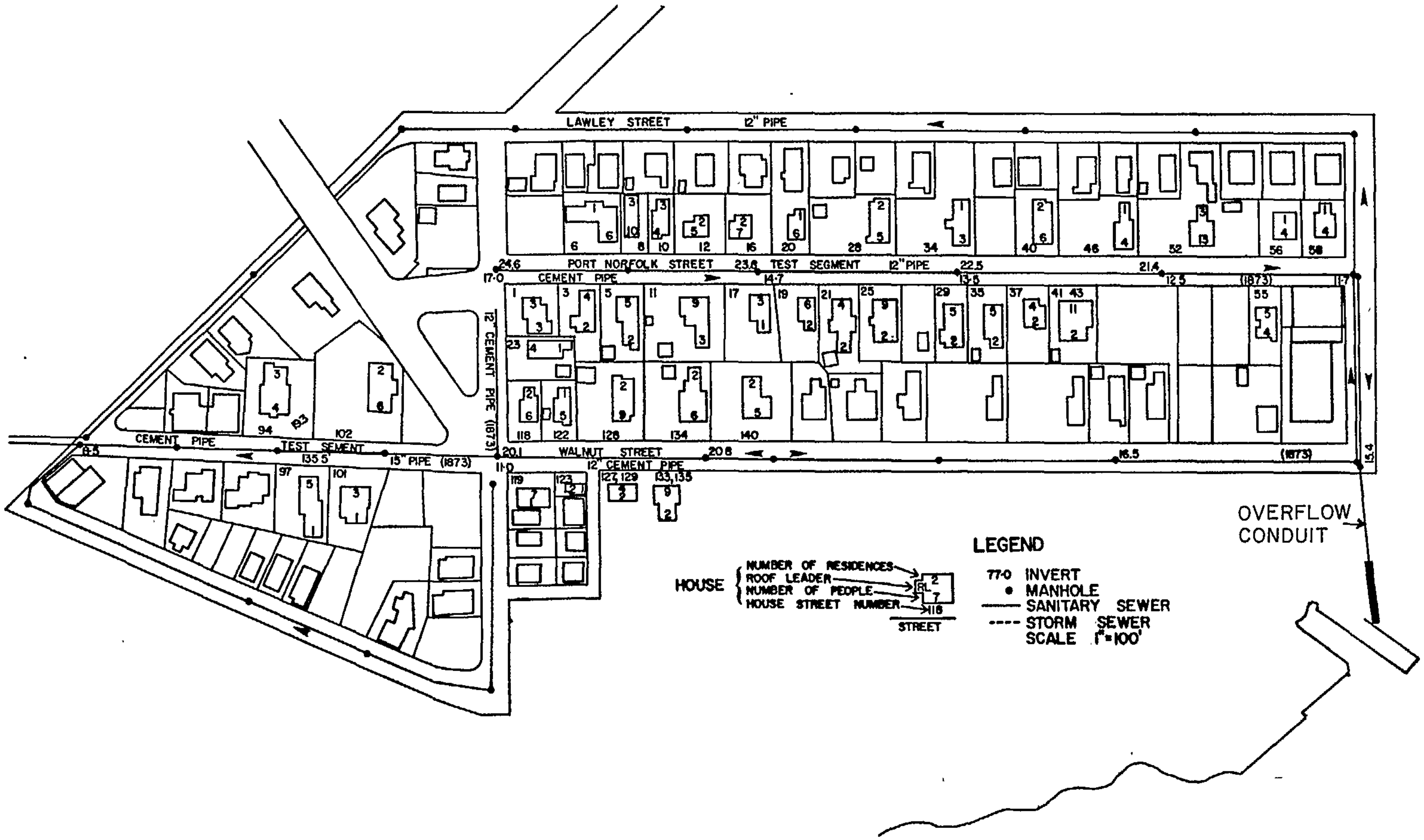


FIGURE 5
PLAN MAP OF SEWER FLUSHING TEST SEGMENTS
PORT NORFOLK & WALNUT STREETS

Port Norfolk Street, Dorchester

Walnut Street, Dorchester

FIGURE 6: PHOTOGRAPHS OF PORT NORFOLK AND WALNUT STREET TEST SEGMENTS

flushing segments are summarized in Table 2. Estimates of contributing population cited in Table 2 include all upstream input and tributory capita along the segment down to the sampling manhole. Slopes of the other segments along Port Norfolk Street used in the serial flushing experiments are similar to the first phase test segment.* The pipe diameter along Port Norfolk Street remains unchanged.

The plan map of the flushing test segments on Shepton and Templeton Street is shown in Figure 7. The flushing test segment on Shepton Street was used during both the first and third phases of experimentation and is located between the second and third manholes from the intersection of Florida and Shepton Streets. The segment on Templeton Street was flushed during the first phase and is situated between the second and third manholes from the intersection of Florida and Templeton Streets. The resident population along Shepton St. appeared stable during the study because of the percentage of long term residences and families. It appeared that the population along Templeton St. may have varied considerably because of the transient nature of inhabitants. The location of each dwelling, number of residences and inhabitants per dwell- (from census tract information) are also shown.

Photographs of both streets are shown in Figure 8. The photos were taken from Florida Street in a westerly direction. There is a hill crest in the middle of both streets with the street grades flattening near the intersection at Florida Street. The test segments are in the foreground in both photos. General characteristics of both streets are given in Table 2.

4.4 Pre-Cleaning Test Segments

Prior to initiation of the flushing program, visual inspections of the test segments indicated roughly 4-6 inches of deposited sanitary wastes mixed with long term accumulations of gravel, sand and grit. The segment on Shepton Street contained mostly domestic waste deposits. The deposits in Templeton Street and Port Norfolk Street contained substantial quantities of sand and gravel. The deposits along the segment on Walnut Street contained domestic waste deposits, sand and gravel, and considerably quantities of grease.

It was desired to remove these sediments to clean pipe conditions for the purpose of starting the program at zero base-line deposition conditions. Intensive water jetting cleaning for over a week was tried using both fire hydrant discharges and injections from the specially designed water tanker described in Chapter 5. Several inches of material were removed at the Shepton Street segment but the effort was futile at the other locations. The City of Boston Public Works Department then provided mechanical cleaning rodding devices in an attempt to remove these sediments. These efforts were again of little use. Precleaning efforts loosened bricks in the manhole table on Port Norfolk Street and large quantities of sand flowed into the segment.

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* Virtual pipe slopes of all pipe segments used in this research effort were determined by application of least squares/mathematical optimization techniques to flush wave data. These procedures and results are described in Chapter 7.

TABLE 2: DESCRIPTION OF FLUSHING SEGMENT CHARACTERISTICS PRIOR TO FLUSHING EXPERIMENTS

Characteristics	Port Norfolk	Shepton	Templeton	Walnut
Pipe Shape & Size (inches)	12 circ.	12 circ.	15 circ.	15 circ.
Service Type	Combined	Separated w/ connected roof leaders	Separated w/ connected roof leaders	Combined
Length of Flush Segment (feet)	247 *	226	187	136
Sewer Map Pipe Slope	.0049	.0035	.0032	.0048
Contributing Population	94	230	221	71
General Sediment Appearance	Heavy septic sanitary deposits & fine sand	Fresh sanitary deposits	Septic sanitary deposits & grit	Septic sanitary deposits & some sand/gravel
Dry Weather Flow Appearance	Impounded very sluggish	Slight meandering movement	Impounded slowly moving	Impounded very sluggish
Street Surface Appearance	Good surface w/ considerable surface trash	Good surface, clean	Poor surface dirty	Good surface, clean

* Phase I segment only

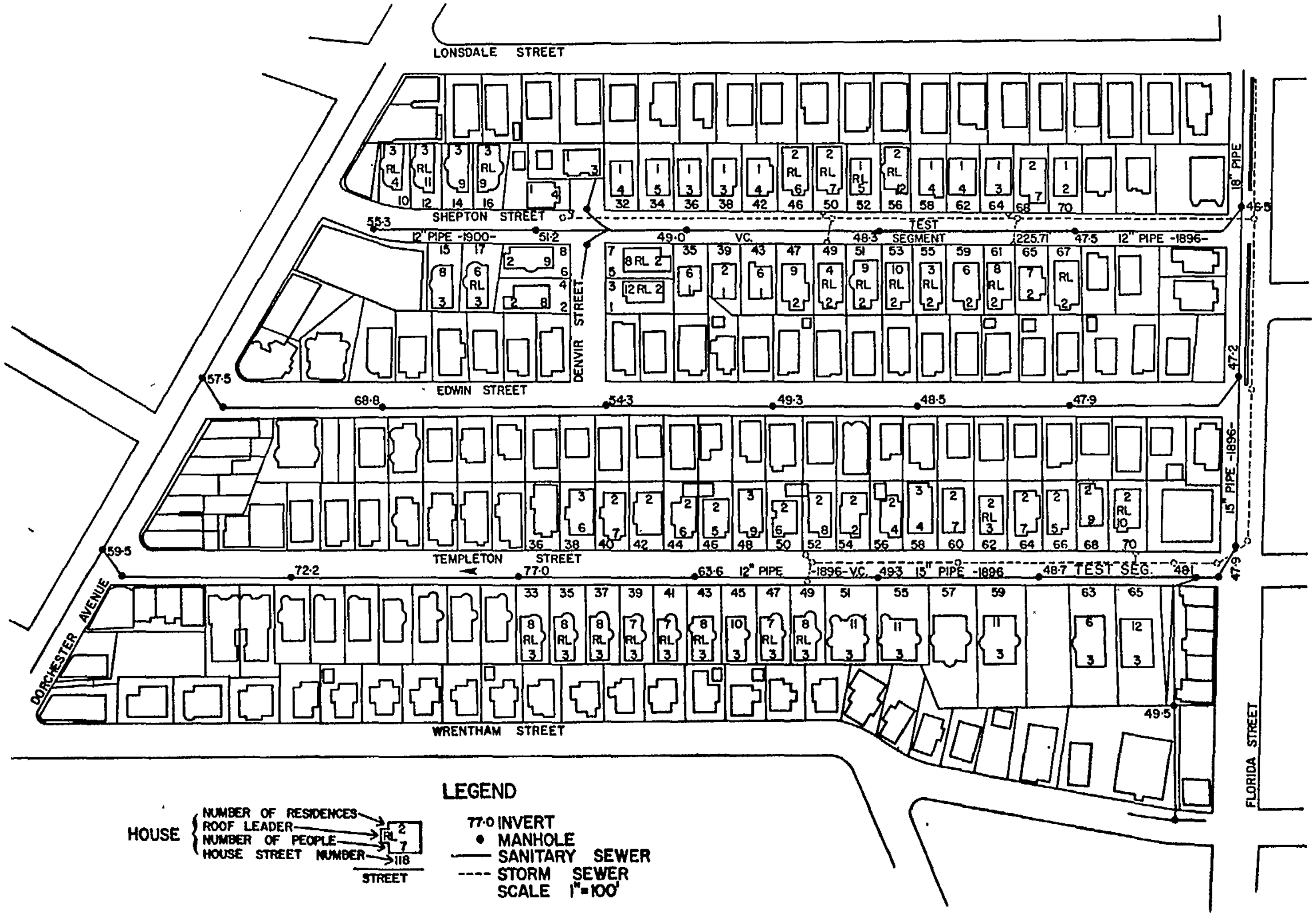


FIGURE 7 PLAN MAP OF SEWER FLUSHING TEST SEGMENTS
SHEPTON & TEMPLETON STREETS

Shepton Street, Dorchester

Templeton Street, Dorchester

FIGURE 8: PHOTOGRAPHS OF SHEPTON AND TEMPLETON STREET FLUSHING TEST SEGMENTS

Field crews then repaired the manholes. Finally, a professional sewer cleaning truck equipped with a 2000 psi water jetting nozzle cleaning device was hired to remove these materials. The intent was to remove long-term accumulations from the entire upstream pipe length as well as from the test segment at each street. Shepton Street was thoroughly cleaned with the exception of a few large rocks, brick fragments and pockets of gravel. Several inches of gravel and sand remained along the other three segments. All segments were reasonably free of residual organic deposits. The cleaning and repair operation took several weeks to accomplish.

The sediment beds were maintained at constant levels over the course of the first phase flushing program which was conducted over a three month period during the summer and fall of 1976. The second phase of experimentation began in the spring of 1977. Over the course of the winter the deposits again accumulated to pre-project conditions primarily due to sand from winter deicing practices. The professional sewer cleaning contractor was re-hired to clean the Port Norfolk and Shepton Street segments. The second phase program was conducted during the spring and summer of 1977 solely on Port Norfolk Street. The segments along the entire street were maintained nearly free of any sand and gravel accumulations during this period as a result of the repeated flushing experiments. The third phase automated flushing experiments were conducted over the summer and fall of 1977. No substantial sediment layers were noted during this period. The automatic module on Shepton Street was inspected in the early spring of 1978 and substantial pre-project heavy organic and grit deposits were observed. In general, sediment beds in the test pipe segments were maintained at fairly constant levels during any sequence of flushing operations. Once the flushing operations were terminated the deposition characteristics returned to pre-project conditions.

SECTION 5

FIELD PROCEDURES AND EQUIPMENT DETAILS

5.1 Foreword

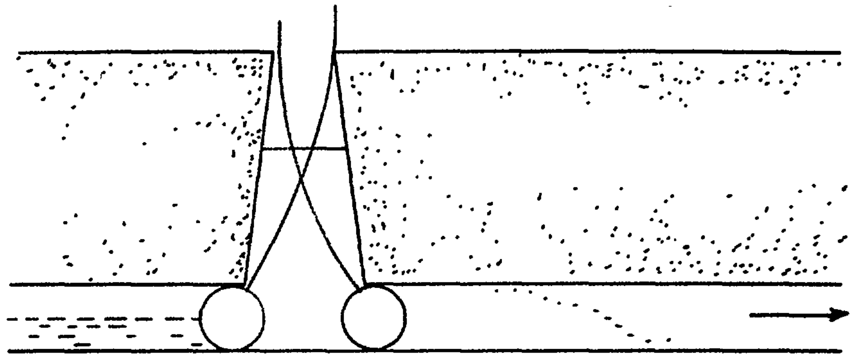
This chapter presents a summary of all field procedures, activities and equipment used during the sewer flushing research study. A brief discussion of the types of flushing methods considered in the study is presented in section 5.2. Details of the specially equipped flushing truck designed, fabricated and used during the first two phases of experimentation are given in section 5.3. Field operations procedures used in the first phase of experimentation are described in section 5.4. Operational procedures used during the second phase experiments are given in section 5.5. Equipment details of the automatic sewer flushing module used in the third phase of work are described in section 5.6. Details of the flush wave flow monitoring procedures are given in section 5.7.

5.2 Manual Flushing Approaches

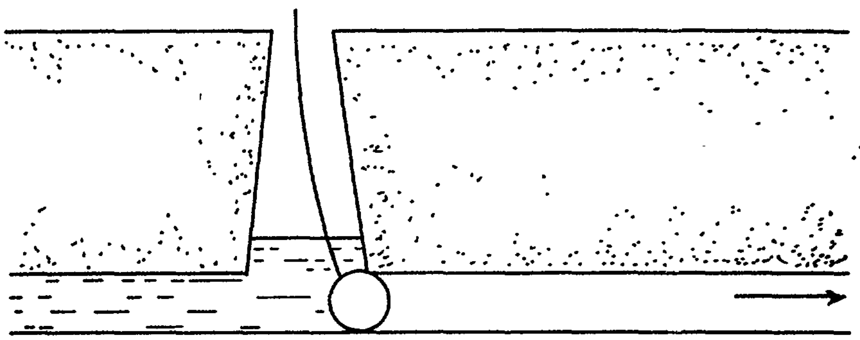
A number of different manual methods for inducing flush wave in the test segments were considered in this study. Manual methods were solely used in the first two phases of work while the third phase dealt solely with an automated approach. Two general categories of manual methods were considered: backup and release using sewage and external flush water injection using fresh water.

Backup and release methods considered in the study are shown in Figure 9. Figure 9-A represents a situation where both the upstream and downstream side of a manhole are stoppered, the manhole surcharged with a predetermined volume of water and then the water rapidly discharged by releasing the downstream stopper. Figures 9-B and 9-C represent two conditions of upstream backup and release; case B utilizes both the upstream pipe capacity and a fraction of the manhole capacity, and case C utilizes the upstream pipe capacity only.

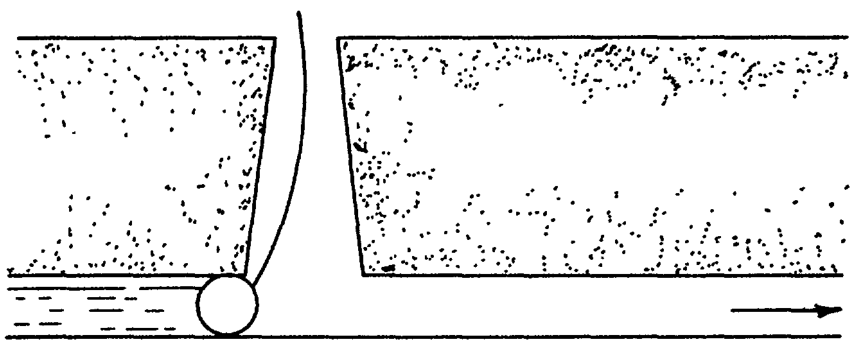
Representative external source flush injection methods are shown in Figure 10. Figure 10-A depicts a gravity flush feed at a low discharge rate while Figure 10-B depicts a flush injected into the manhole at a high rate of discharge. The flush volume in either case can be small or large. High flush rates induce high velocity heads and the ability to scour sediments, while large volumetric flushes provide fluid momentum and the capacity to dilute and transport scoured materials. One of the objectives of the first phase experimentation was to determine the flushing pollutant removal effectiveness for



A. Manhole Surcharge - Rapid Release (External Source)

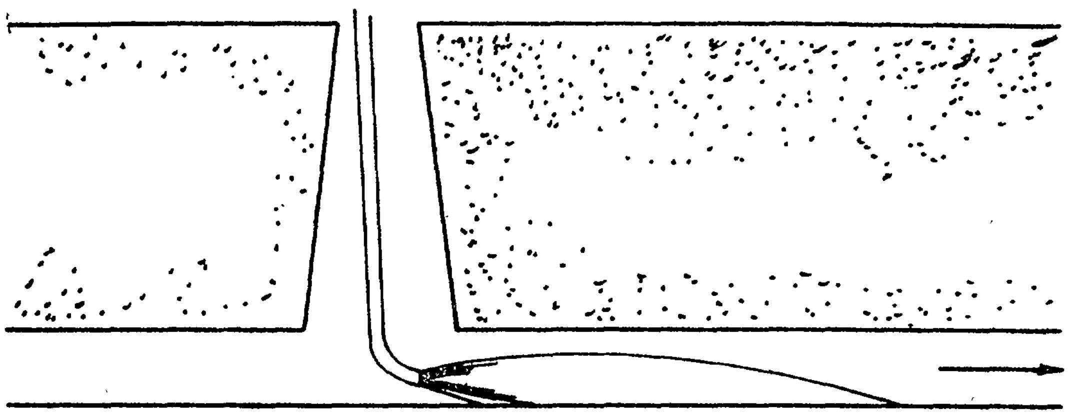


B. Back and Release - Collection Pipe and Manhole (Higher Head)

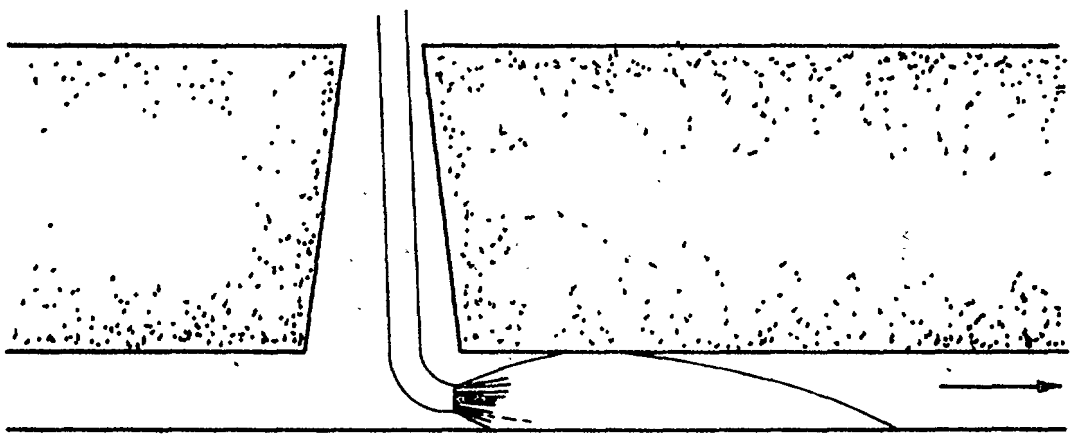


C. Backup and Release - Collection Pipe

FIGURE 9 REPRESENTATIVE BACKUP AND RELEASE FLUSH METHODS



A. Low Rate Flush (Gravity Feed) (Volume High or Low).



B. High Rate Flush (Pressurized Feed) (Volume High or Low).

FIGURE 10 REPRESENTATIVE EXTERNAL SOURCE FLUSH INJECTION METHODS differing combinations of flush volume and flush rate. Minimization of the quantity of flush water used may be important for communities faced with water shortages in the future, while in other applications the issue of increased mechanical equipment complexity and energy cost considerations may be more important. The pre-project apriori hypothesis was that flush volume was more important since it was believed that the high velocity head induced by high entry rates would be rapidly dissipated by friction losses within the wave as it proceeded downstream. The results of the first phase program described in Chapter 8 showed high volume with high rate flushes to be the most effective.

Four combinations of flush volume and flush rate were considered in this study. The external flush rate and volume inputs were delivered by a specially equipped flush tank truck which was prepared for the project. Details of the tanker are given in Section 5.3. The operation under gravity conditions provided a controlled flush release of 35 to 50 cubic feet (.99 to 1.98 cubic meters) at a rate of 0.25 to 0.5 cubic feet per second (7.08 to 14.16 liters per second). Under pressurized conditions the same volumetric range of flush was accomplished at a range of 0.5 to 1.25 cubic feet per second (14.16 to 35.4 liters per second). These ranges of flush rates and volumes were chosen from prior experience with flushing segments in this study area (4) and moreover were chosen to be reasonably representative of the type of flushing operation which could be implemented by most communities. Various sized discharge nozzles (1-3 inch) were also considered early in the first phase experiments to maximize velocity head effects in the flush wave.

5.3 Details of Flush Truck and Ancillary Equipment

Prior to the start of the actual flushing efforts many pieces of specialized equipment had to be designed and fabricated for use in the study. These in general included the manual sewer flushing module, discharge nozzles, and sampling equipment. The major single piece of equipment used during the first two phases of the sewer flushing program was the manual sewer flushing module or flushing tank truck. The nature of the sewer flushing research work necessitated a wide range of discharge capabilities to be provided by the flushing module. Since the first phase flushing program was aimed at assessing the preformance of various methods which in essence translated primarily to varying flush injection rates, a wide range of rate flexibility was required. The unit also had to be mobile and capable of conducting several flushes before refilling.

Figure 11 is a mechanical schematic showing the plan, side and end views of the flushing truck. Figure 12 is a photograph of the flush truck. In essence the unit consisted of two 1000 gallon (3.7 cubic meters) pressure tanks mounted in parallel on a steel I beam frame. Fluid transmission was conducted through a 4 inch (10.2 cm) I.D. steel piping manifold allowing complete separation or interconnection of tanks and discharge routes. Final discharge was made through a 3 inch (7.6 cm) quick-action gate valve, to allow for rapid on and off as well as good throttling characteristics. Accurate measurement of discharge volumes and subsequently discharge rate was accomplished using a 4 inch turbine water meter, manufactured by Hersey Sparling Co. and supplied to the study by the City of Boston Public Works Department. The meter provided accurate volumetric measurement of the

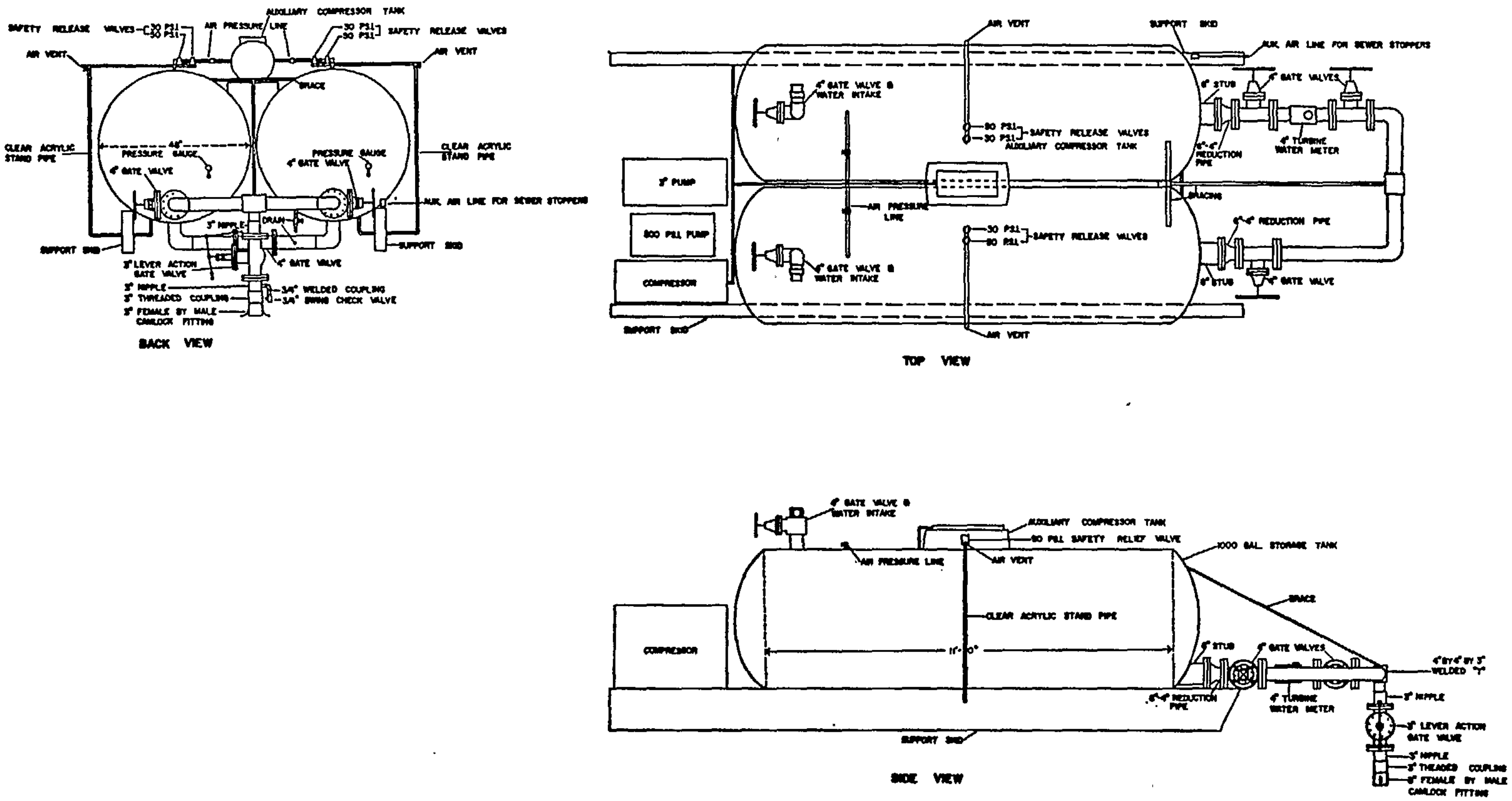


FIGURE II MECHANICAL PIPING SCHEMATIC DIAGRAM
MANUAL SEWER FLUSHING UNIT

SCALE: 1" = 5'6"

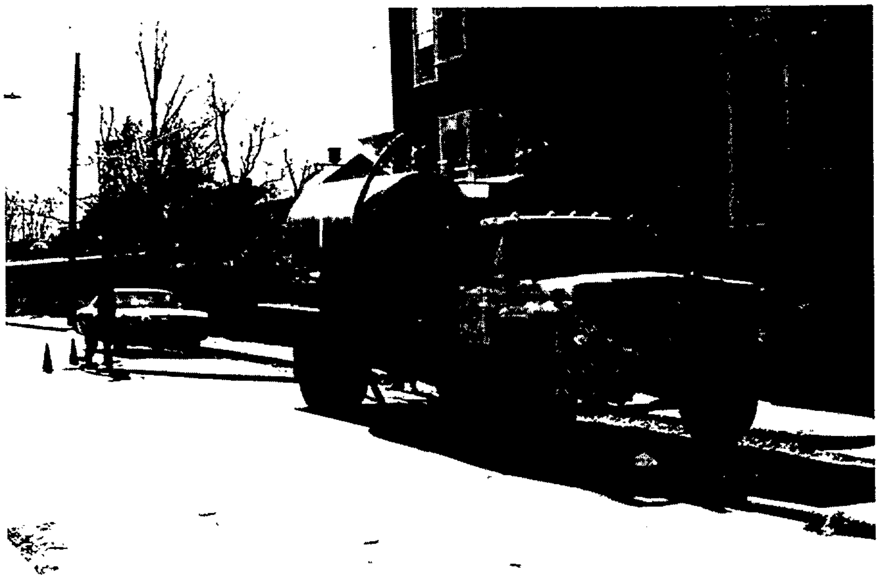


FIGURE 12: PHOTOGRAPH OF FLUSH TRUCK

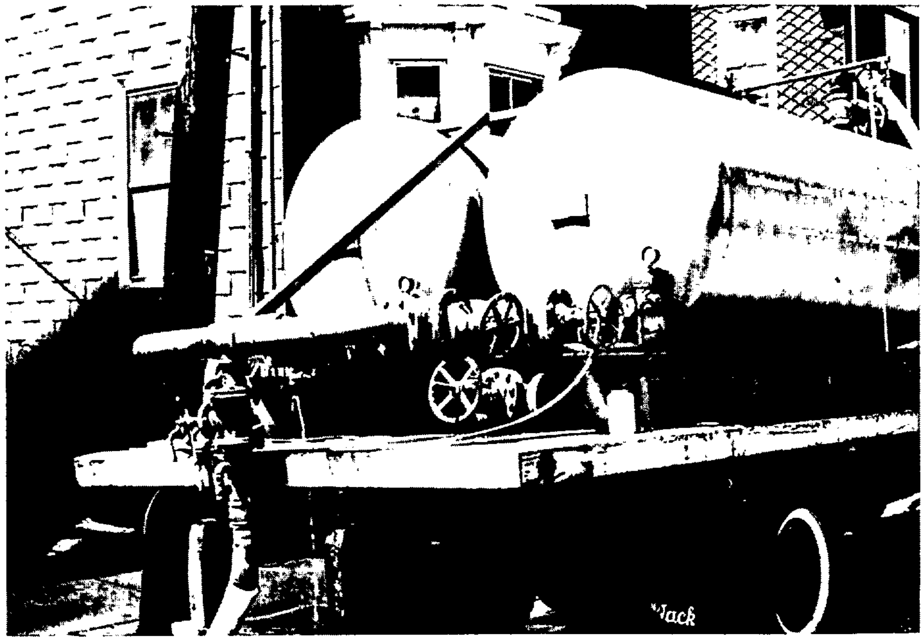
discharge stream at rates from 0-1000 gpm (0-3750 l/m). Figure 13A shows a photograph of the manifold piping and equipment at the rear of the truck. The entire sewer flushing module as assembled was 22 feet (6.7 m) in length and 8 feet (2.4m) wide. To enable mobile operation the entire unit was attached to a 20 foot (6.1m) flat bed truck. The tanker was equipped with a pneumatic system to pressurize the tanks to 70 psi. Tank pressurization and control was accomplished by a gasoline powered compressor mounted on the unit. Figure 13B shows the pressurization equipment located on top of the tank units. Typically, during the study the flushing truck would be filled by means of a 3-inch fire hose from a hydrant through one of the 3-inch quick-connect fill valves located on top of each tank.

Flush injection was accomplished by means of a 3-inch fire hose connected to a discharge nozzle mounted on a support rod. An assortment of nozzles was assembled from 90° pvc electrical sweep ells ranging from 1-3 inches in inside diameter. Each nozzle could be readily coupled to the discharge hose by means of a 3-inch camlock fitting. All nozzles and hoses used in the sewer flushing study were equipped with camlock fittings to enable rapid assembly. The pvc sweep ells provided an ideal nozzle for use during the study in that the 90° bend is made over a large radius thereby minimizing energy losses at the discharge. The nozzle support rod was made from a 10-foot length of 1-inch electrical tubing. The electrical tubing was outfitted with a welded point in one end for anchoring the nozzle during discharge as well as a movable nozzle holder coupling that could be adjusted to allow for nozzle centering on any size pipe from 8-30 inches in diameter. Figure 14A shows a photograph of the flushing nozzles, nozzle support and hoses. Figure 14B shows a photograph of the inflatible rubber stoppers (children's toy called "Hippity Hop") which were lowered into place by a rope installed in place in the sewer segment and rapidly inflated using the flush truck pressurization system. These devices were inexpensive and worked extremely well. The entire flushing system so developed could be easily operated by two people from the street surface.

5.4 First Phase Flushing Procedures

The flushing program in this phase was concerned with only the effects of flushing a single manhole to manhole segment. Four test segments were flushed in this program and were described in section 4.3. Three different methods of manual flushing were performed. The first method consisted of backing up the upper end of the flushing manhole with an inflatable rubber stopper, followed by quick release. The other two methods were gravity and pressurized dump discharge into the flush manhole with the upper end of the flush manhole generally blocked off. Different flush volumes were used during the external source injection experiments. All dump discharge flushes were performed using the water tanker. All flush volumes were measured by a water meter which was repeatedly calibrated to ensure accurate monitoring of the delivered flush volumes.

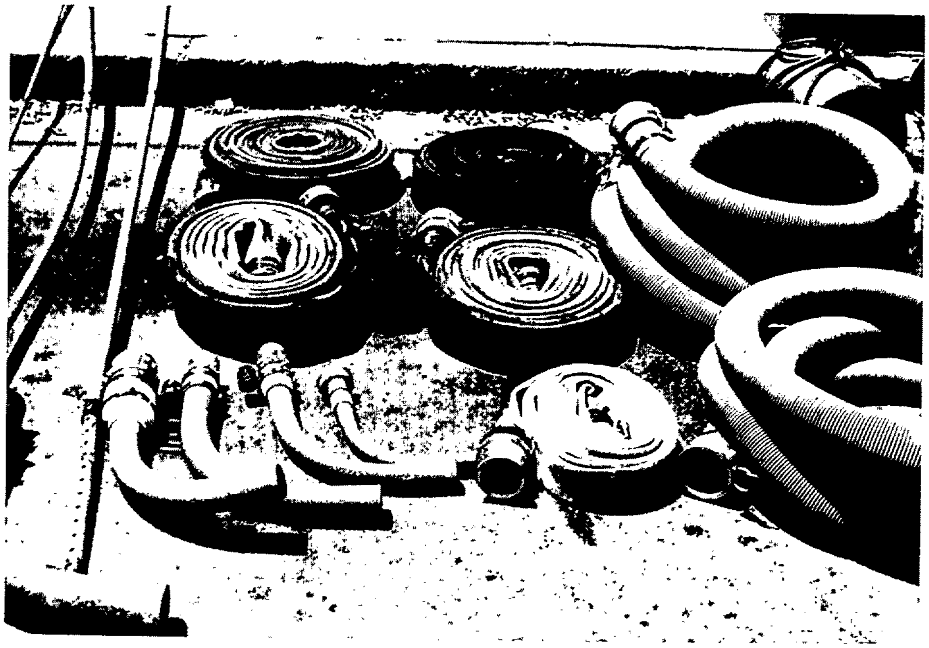
A total of 86 separate flushing experiments were performed during the period of August 30, 1976 through November 12, 1976. Roughly 20 flushes on a 3-4 day basis were accomplished for each of the four test segments. The



A. Manifold Delivery System

B. Pressurization Equipment

FIGURE 13: PHOTOGRAPHS OF MANIFOLD DELIVERY SYSTEM AND PRESSURIZATION EQUIPMENT, FLUSH TRUCK.



A. Flushing Nozzles, Support Rod and Hoses

B. Inflatable Sewer Stoppers

FIGURE 14: PHOTOGRAPHS OF FLUSHING NOZZLES AND INFLATABLE SEWER STOPPERS

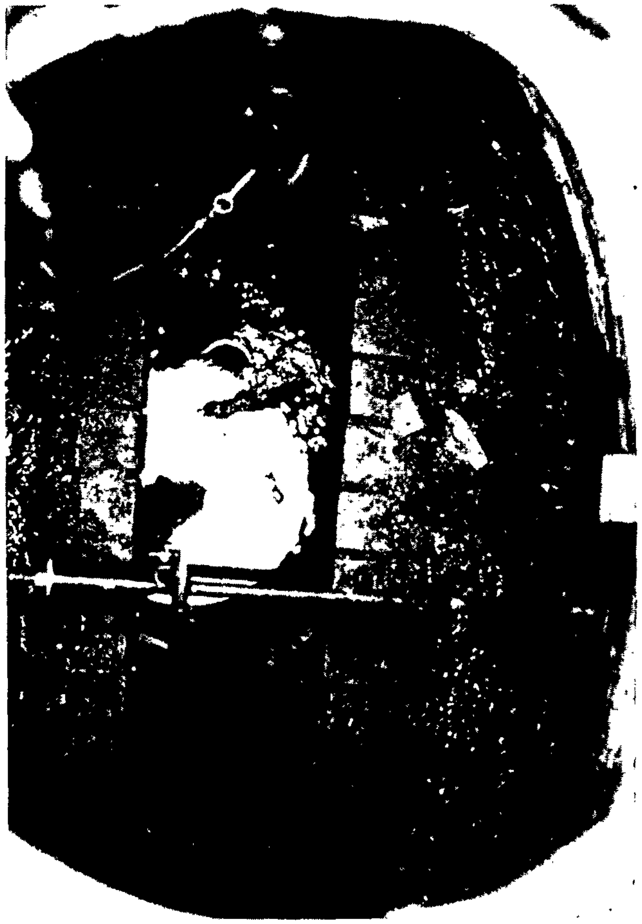
method of flushing was rotated per segment per flush so that all methods were applied to each segment over the test period.

The sequence of pertinent operations during a given flushing experiment was the following. After the safety equipment was set-up, the segment was then visually inspected by lamping to assess solids buildup and debris as well as to characterize the depositing matter (fine sand and organic matter, toilet paper, rags, small rocks and sticks). Several liquid background samples and depth flow were taken at five minute intervals. Next, the upper end of the flush manhole was blocked-off (in most cases) and sediment samples over a prescribed unit length was taken in both the flush and sampling manholes. The scraped materials were visually inspected to assess solids characteristics, collected in a suitable container and brought back to the laboratory. A photograph of the inflated rubber sewer stopper in the upper end of the flush injection manhole on Port Norfolk Street is shown in Figure 15A. Figure 15B shows a photograph of a field engineer with safety equipment for entering the flushing manhole to take sediment scrappings. Ventilation equipment and the flush truck are shown in the background of the photograph in Figure 15A. Figure 16 shows two photographs of the sediment scrapping operation. A specially designed pipe squeegee was used to scrape the sediments of a lineal foot of sewer.

The flushing experiment was then conducted either using backup sewage or fresh water injection from the flush tank. Dye was injected in the wave and at the instant of arrival, a one-liter aliquot was taken with a specially designed hand scoup for obtaining a reasonable cross-sectional sample of the solids within the flush wave at the downstream sampling manhole. The device specifically excluded bed load materials. After the first sample was taken at the first visual sighting of the wave, 8 grab samples were taken at 10 second intervals, and then an additional 8 samples were taken at 20-second intervals. Wave heights were taken at each interval of time which were later used to determine the instantaneous flow rate for computing mass pollutants removed by the flushing experiment. A total of 17 flush wave grab samples were taken during a given flushing operation totalling 4 minutes.

Figure 17A shows a photograph of a low rate flush feed at 0.25 cubic feet per second into the flush manhole on Shepton Street. Figure 17B shows a similar photo on Shepton Street with a high feed rate of 0.75 cubic feet per second. The jet is continuing well into the flushing segment. Hydraulic entry of the flush feed into the pipe segments for the Shepton and Port Norfolk Street segments generally exhibited the patterns shown in Figure 17. Figure 18 shows a photograph of a high rate (0.75 cfs) and large volume flush (75 cubic feet) at the Walnut Street test segment. The flush wave in this case has induced an extremely turbulent backwater effect in the flush manhole. Flushes at Templeton and Walnut Streets generally followed this pattern. Figure 19A shows a photograph of the flush wave sampling hand scoups. Figure 19B shows the flush wave sampling operation. The field engineer is taking a grab sample with the hand scoup near the end of a sampling sequence at the Walnut Street segment. In the far left hand side of the photograph is the staff gage where liquid level depths to the nearest eighth inch were recorded at the appropriate time intervals signaled by an observer at the top of the

B. Field Engineer Equipped with Safety Gear.



A. Inflatable Sewer Stopper in Place.

FIGURE 15: PHOTOGRAPHS OF INFLATABLE SEWER STOPPER AND FIELD ENGINEER EQUIPPED WITH SAFETY GEAR

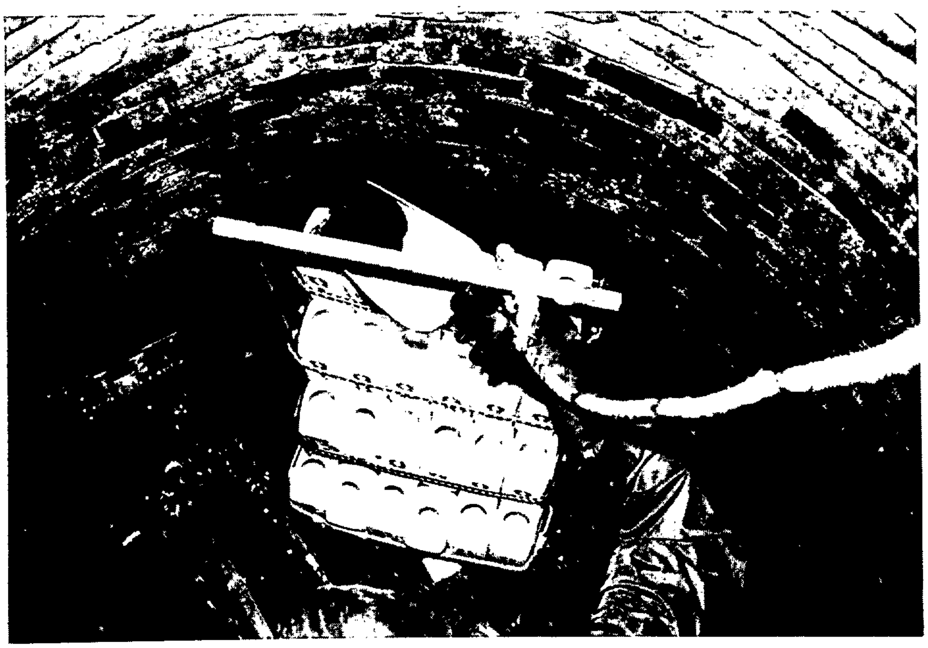
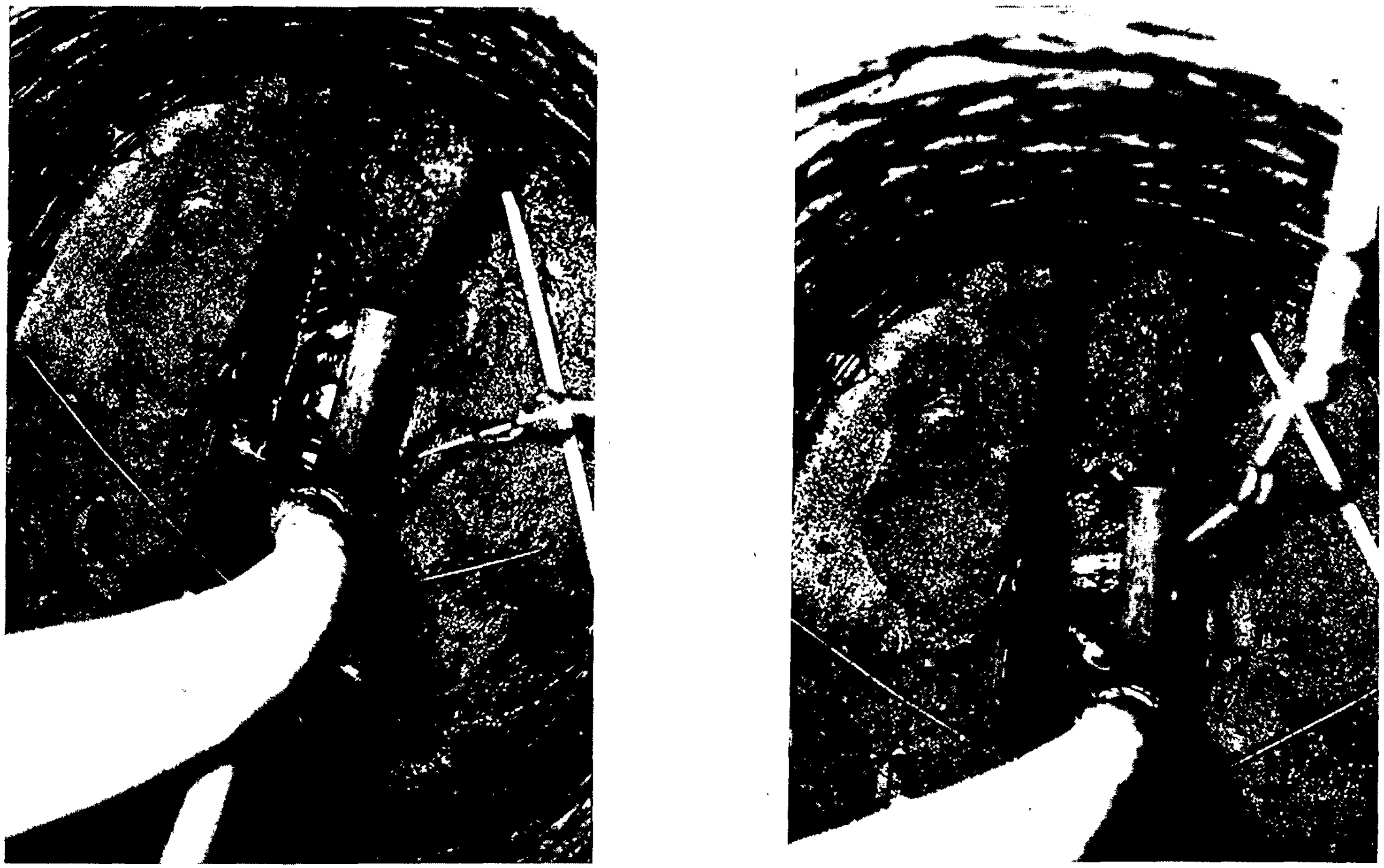


FIGURE 16: PHOTOGRAPHS OF SEDIMENT SCRAPING OPERATION

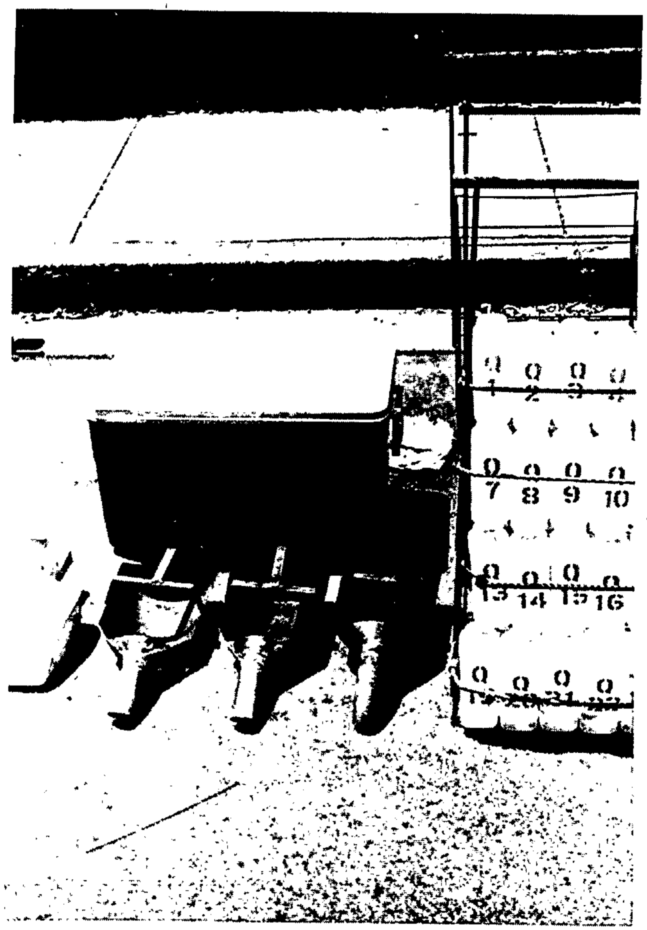


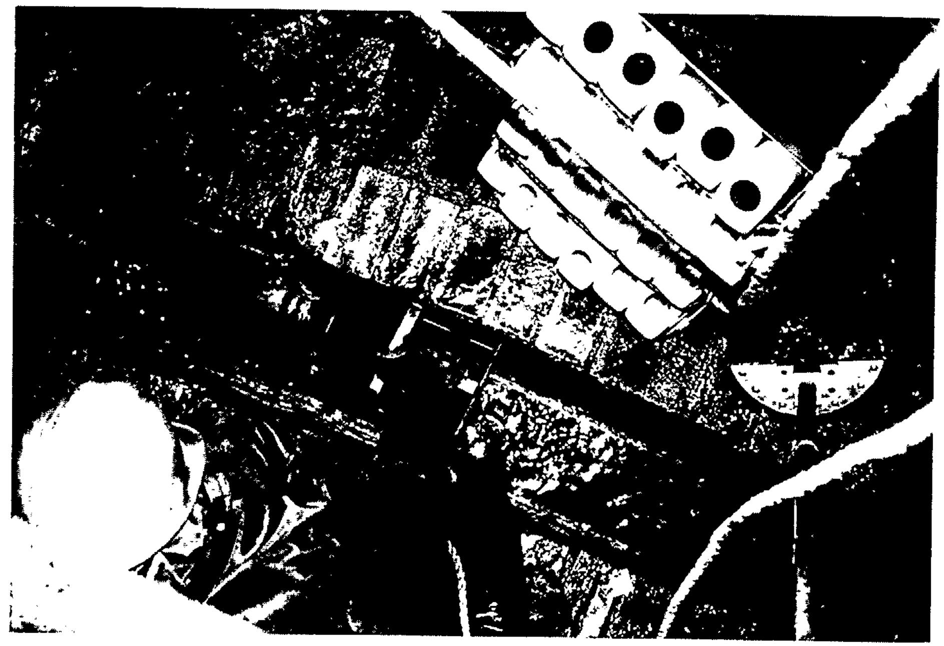
A. Low Flush Rate (0.25 cfs)

B. High Flush Rate (0.75 cfs)

FIGURE 17: PHOTOGRAPHS OF FLUSH WAVE INJECTIONS AT DIFFERENT FEED RATES ON SHEPTON STREET, DORCHESTER

FIGURE 18: PHOTOGRAPH OF FLUSH WAVE INJECTION AT WALNUT STREET, DORCHESTER





A. Flush Wave Hand Sampling Scoups

B. Flush Wave Grab Sampling Operation, Walnut Street

FIGURE 19: PHOTOGRAPHS OF FLUSH WAVE HAND SAMPLING SCOUPS AND GRAB SAMPLING OPERATION

manhole (not shown on the photograph). The pipe squeegee used to scrape pipe sediments is shown on the right hand side of the photograph. After the sampling sequence was completed the pipe segment was then flushed for five minutes at a maximal flush rate of about 1.25 cfs. The purpose of this final operation was to flush clean any residual organic matter remaining in the segment. The segment was assumed to be clean for the next period of solids accumulation. The final washing operation was conducted from the onset of the first phase program till the end of October, 1976.* The balance of the first phase program was conducted without the final flushing operation to ascertain in an indirect way whether there were residual pollutants remaining from a given flushing experiment.

Flow monitoring of dry weather flow characteristics, stage/discharge calibration efforts at the sampling manholes and the special dye injection procedure used to directly estimate flush wave discharge are described in Section 5.7.

5.5 Second Phase Flushing Procedures

The experimental field work in this phase was concerned with the problem of flushing a long flat stretch of combined sewer lateral. The street contains five manholes and is roughly 1000 feet in length. Flushes were injected into the upper most manhole and pollutant levels in the flush wave passing three downstream manholes were monitored. A diagram of the second phase flushing and sampling manholes along Port Norfolk Street is shown in Figure 20. Work was divided into two subphases. Initially, pollutant removals over the three segments were determined for different flushing conditions established in the first manhole. These flushing experiments provided insights into flushing effectiveness over three segments of pipe. Details of the field operation for this subphase of work are described in Section 5.5.1. In the next subphase of work, settleability tests were performed on samples taken from flushes conducted in a similar manner for the purpose of crudely extrapolating how far beyond the flushing monitoring manholes would the materials be carried. Sampling procedures and sample preparation for settling column tests are presented in Section 5.5.2.

5.5.1 Serial Flushing - Pollutant Removals

The purpose of these experiments were to ascertain the pollutant removal effectiveness over three consecutive combined sewered segments on Port Norfolk Street by flushing the uppermost manhole using the flush truck. These experiments were also intended to provide additional information for assessing the flushing effectiveness of the first phase flushes and to provide further information for determining rates of dry weather sewage deposition.

The backup and release method of flushing was not considered in this phase since there was no appreciable contributary population at the upstream flushing manhole. Most of the flushes conducted during this period were delivered at high feed rates with high volumes since this mode of flushing proved to be the most effective during the first phase operation. Booster

* At that point 69 flushes had been conducted or approx. 17 at each segment.

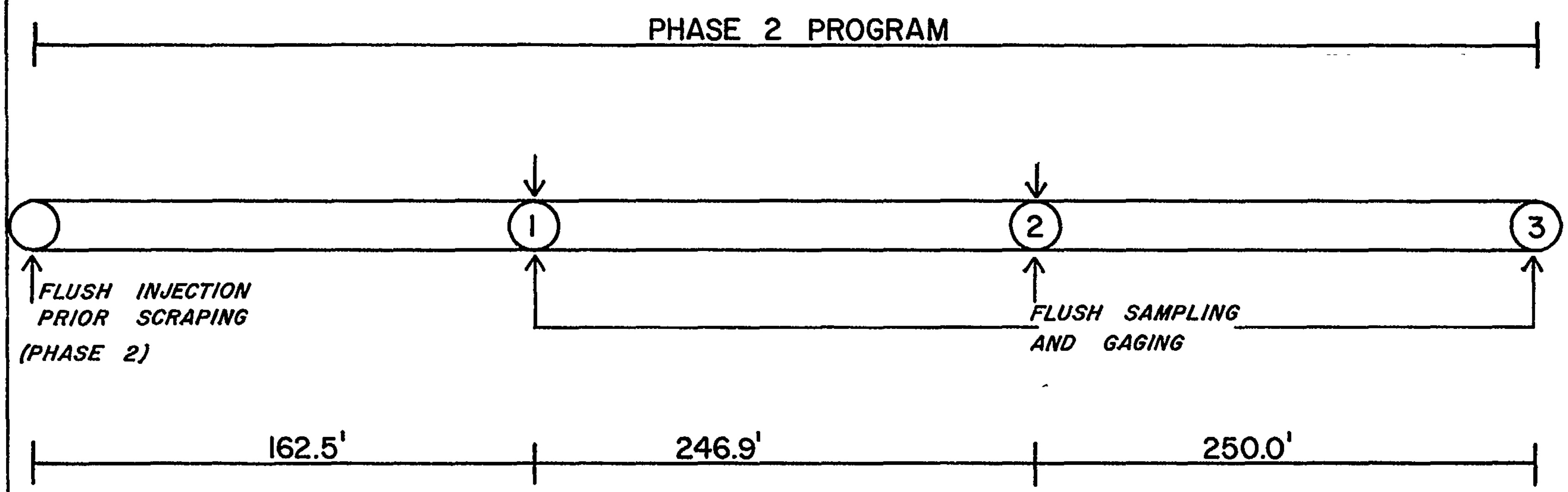


FIGURE 20 SCHEMATIC OF SECOND PHASE FIELD OPERATIONS

flushing along the segment was not considered in this study. This concept entails sequencing multiple flushes along a segment such that the resuspended pollutants and grit do not resettle. Initial flushing experiments in this phase indicated extremely favorable pollutant removal results using a single upstream injection. In view of the limited project resources, the principal investigators believed that replication of these initial favorable findings was more important from the standpoint of establishing technical credibility of simple flush methods, than in pursuing the effectiveness of multiple sequenced flushing operations. The experimentation period began in December 1976 and extended through March, 1977, entailing two replicate sets of three flushing rate/volume experiments. Each experiment consisted of three flushes conducted within a short period of each other. The first two flushes on a given day were the same while the final flush was maximal volume/rate flush meant to remove any remaining pollutant load in the segments. Different combinations of flush volumes (35 to 75 cubic feet) and delivery rates (0.3 cfs to 1 cfs) were considered. Three crews sampled the flush wave passing the downstream manholes. Samples were taken at the same frequency as in the first phase of experimentation described in Section 5.4. Sediment scrapings and sampling of background sewage levels were also accomplished as in the first phase. Results of this flushing program are reported in Chapter 9.

A special effort was initiated in the phase of work to develop a dragging device meant to scrape clean any residual matter remaining in the segments either after a flushing experiment, or alternatively, prior to flushing so that both the efficiency of flushing and the rates of deposition could be accurately monitored. Rates of deposition could be determined either by primary measurement using a scraping operation on an undisturbed and pre-cleaned pipe segment or alternatively, by summing the pollutant masses transported by the flushing to the mass measured by post-flush scraping. Flushing pollutant removal effectiveness could be better estimated if either the total rates of deposition or the quantity of residual materials remaining after flushing were known. Sediment scrapings over a unit length of pipe (one foot) had been taken before and after flushing at both ends of the test segment during the first phase program. There were a number of difficulties with this approach. First of all, the segments were never cleaned to zero base-line clear pipe conditions despite two intensive weeks of cleaning using three different procedures. This work is described in Section 4.4.4. Secondly, there was no way of ascertaining the longitudinal profile of sediment in the segments. FMC (12) reported that most of the deposited material occurred within the first quarter of the experimental test segment length and depending on the duration of the accumulation period, the sediment profile would progressively move downstream along the segment. Accurate delineation of deposition characteristics along the length of the segment was beyond the scope of this research work. The envisioned scraping operation was viewed as a compromise in which estimates of the total quantity in the segment would be obtained respective of the actual profile in the segment.

A special scoup fitted with a nylon catch bag was fabricated and powered by a low-speed winch system which was connected by cables between consecutive manholes. The operation was tried several times and failed due to the presence of cast iron house laterals protruding into the combined sewer lateral along Port Norfolk Street. The approach was terminated and the manual sediment scraping operation of a unit foot of pipe was continued in the second phase.

5.5.2 Serial Flushing - Settleability Analyses

The purpose of these experiments on Port Norfolk Street was to roughly assess the transport of flushed pollutants beyond the test segments using information derived from various settleability tests. Six different flushing experiments were performed in the period of April, 1977 through August, 1977 in which settleability tests were performed for samples taken at each of the three sampling manholes. Three flushes were conducted per experiment. Settling column tests were conducted on samples taken from first flush and Imhoff cone tests were performed on second and third flush samples. External source flush volumes were injected into the uppermost manhole on Port Norfolk Street using similar volumes/rates as used in the first half of phase two, described in Section 5.5.1.

Special equipment was fabricated to obtain "undisturbed" samples of flush waves at the three downstream manholes on Port Norfolk Street. Four special sampling devices were constructed to collect representative "undisturbed" flush wave samples at various intervals during a flush wave. A diagram of the sampling device is shown in Figure 21. Each sampling device consists of a 14-inch section of schedule 40 pvc pipe to which snap-action end gates were attached with an approximate 5 gallon capacity when full. The two end gates were made of one-eighth inch circular aluminum plates attached to sliding support rods. To operate the sampling device, the two end gates are first raised to the open position and are held in place by a spring loaded latch. All four open tubes were placed in the bottom of one of the downstream sampling manholes. As the flush wave passed the sampling point, each of the tubes were rapidly lifted at a pre-selected times. Rapid lifting caused immediate closure of the end gates and the capture of a segment of the actual flush wave. The aforementioned samples were augmented using hand dipped plastic buckets.*

Sample collection for the column tests and Imhoff Cone tests required approximately 10 gallons of the flush wave. For an accurate representation of the flush wave, samples were collected at various times after the flush wave first appeared at each of the three sampling manholes. These individual samples were then composited into the one sample representative of the flush wave that passed through each sampling manhole. The flush wave was sampled at six instants as it passed through the sampling manholes. These times varied from manhole to manhole, as the characteristics of the flush wave changed the further downstream it progressed. The sampling

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* Length of the manholes along Port Norfolk Street allowed placement of only four sampling devices in a manhole. Since the depth of flow attained averaged less than half depth of the samples, the volume had to be augmented with two additional samples.

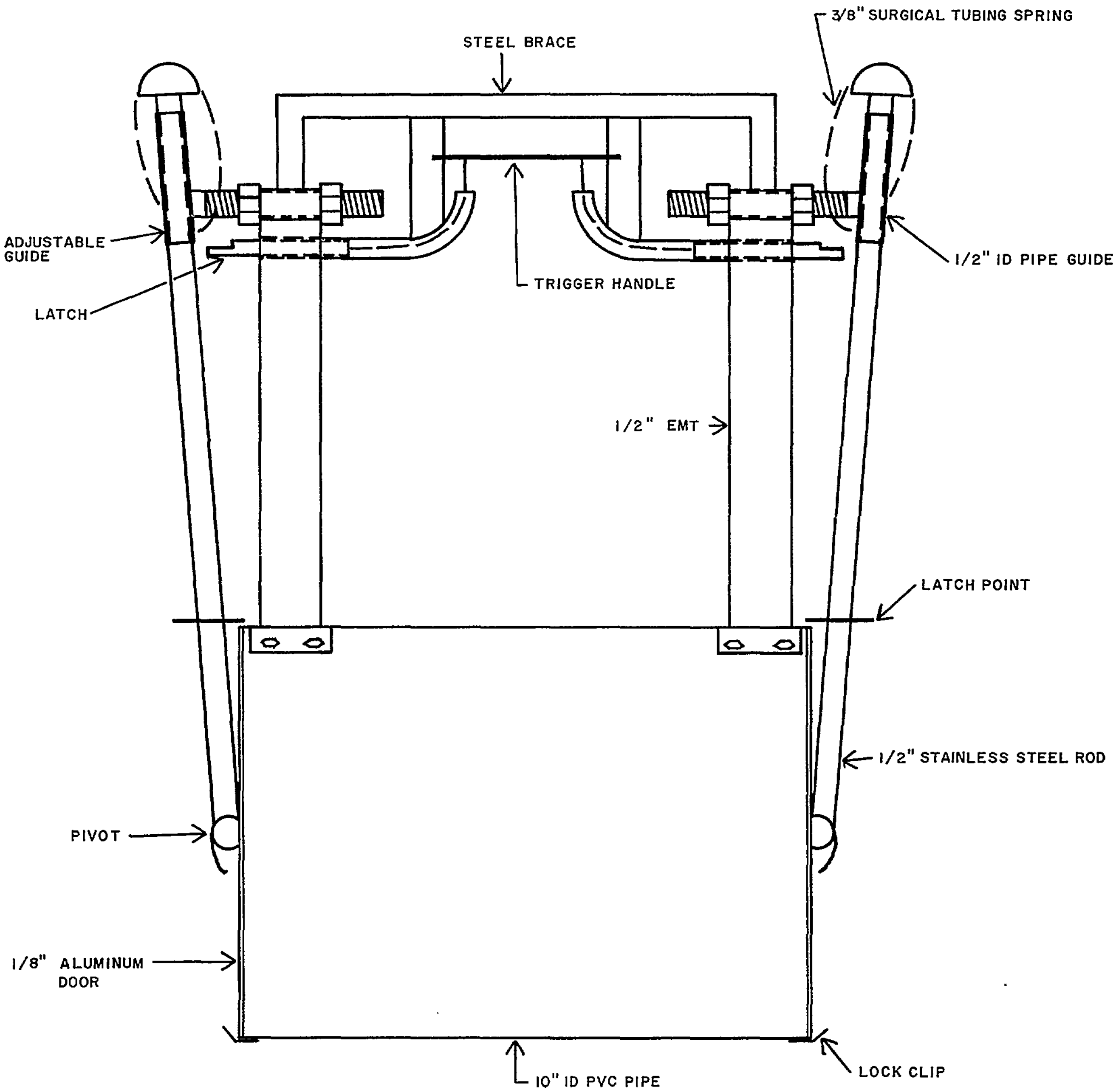


FIGURE 21 SETTLING COLUMN FLUSH SAMPLER

times used attempted to divide the wave into six parts for ease of collection and to minimize the sampling effects on the wave. The time taken for the sample collection started when the wave first appeared at the upstream end of the manhole. Sampling times were determined from inspection of the first phase flushes and are shown below for each sampling manhole.

Sampling Manhole	Sampling Intervals (seconds) Referenced to First Arrival of Flush Wave at Manhole
1	10, 20, 30, 40, 60 and 80
2	20, 30, 40, 60, 80 and 100
3	20, 40, 60, 80, 100 and 120

The depths of the flush wave passing each manhole were measured and recorded at 10 second intervals. A flow-composited sample was prepared from the six grab samples for each sampling manhole per flush using flush wave depth to pipe area considerations accounting for the presence of sediments within the 12 inch lateral. Special settling column equipment and procedures were established in order to perform settleability tests of the flush waves. A special yoke-frame installation was devised to permit axial and transverse mixing of the column before settleability experimentation. This ad hoc procedure was necessary since the settling velocities for a considerable portion of the composited flush wave solids were extremely rapid. Gentle mixing using air agitation was initially performed but resulted in solids bulking because of the high solids content of the flush samples. Details of the settling column and procedures are described in Chapter 6. Results of the settling column and Imhoff Cone testing are presented in Chapter 10.

5.6 Automated Sewer Flushing Module

The phase III portion of the sewer flushing research project dealt with the development and operation of a simplistic automated sewer flushing module. The device developed was in essence an air operated gate capable of backing up sewage flows to a predetermined level and then suddenly retracting, inducing a flushing wave. This type of backup and release device was well suited to the situation as found on Templeton and Shepton Streets in Dorchester. The sewer lines in the Templeton/Shepton Street area were previously described in chapter 4. Upstream of the flushing manhole is a hill allowing development of sufficient static head for a clean discharge. The module so developed was installed on 8/30/77 and operated on a regularly serviced basis from 8/31 - 10/31/77. During this period the module was checked at least 3 times per week to ensure performance as well as conduct sampling runs. Automated flushes were sampled seven times during the period of 9/22 - 10/13/77 with the results and discussion of overall performance presented in chapter 11 of this report. Module operation was continued after 10/31/77 until mid January 1978 on a periodic inspection basis to assess long term operation and serviceability, as well as visual performance with respect to flushing of both the upstream or reservoir segment and the downstream flushed segments.

The automated sewer flushing module, as all other specialized pieces of equipment used during the sewer flushing study, was designed and fabricated by EDP Inc. Due to project constraints and the prototype nature of the flushing module, construction was kept as simplistic as possible to allow for maximum flexibility. Figures 22 and 23A and B present a schematic representation of the sewer flushing module, as well as photographs of the installed unit taken after months in service.

5.6.1 Design Details of the Automated Sewer Flushing Module

The automated flushing module depicted in Figure 22 was constructed primarily of wood with a final epoxy coating for water-proofing. The module was designed for ease of installation and maintenance and could be readily adapted for packaged installation. As previously noted the module consisted of an air-on-oil cylinder operated gate controlled from a master control timer capable of pre-programmed flushing of varying sequences, with flushing intervals ranging from 6-72 hours. The entire unit was powered by a 12 volt automobile battery that was capable of 6 months operation flushing daily before recharge. Air supply to the system was by means of a small high pressure air cylinder requiring replenishment every 150 flushes.

The automated module was constructed in two major parts: 1) the flushing gate assembly;and 2) the master controller. The flushing gate assembly was constructed so as to sit upon the table of the sewer within the manhole. In order to provide maximum flexibility with respect to backup volumes and static head of the flush wave, the sewer was extended upward from its vertical centerline to slightly above its crown. The support frame gate guide for the hydraulic cylinder was then coupled with the sidewall extentions to form the completed assembly. Control of the gate action was provided by an EDP-designed crystal controlled clock timer mechanism which emitted a signal to the 2-way air valve, which then activated a spring returned air-piloted 4-way hydraulic valve causing the gate to go down. The gate would then stay in the down position until the backed-up flow reached the preset level sensor float or was down for a predetermined amount of time. The time function acted simply as a safety mechanism in case of sensor malfunction. This safety feature was not entirely necessary in that the design of the module was such that even if the gate was stuck in the down position, the sewage would simply overflow the top of the gate.

The actual flushing gate was made of plywood set into the support frame with a positive seal on the sides. Initially, gate sealing on the sewer invert was provided by a closed cell foam seal ring attached to the contour fitted gate. This sealing mechanism proved to be somewhat unreliable due to the irregular contour of the brick sewer channel in the manhole. After a few weeks of initial testing, a polyethylene/foam floating seal was added to the support frame forming a band along the bottom of the sewer channel. This

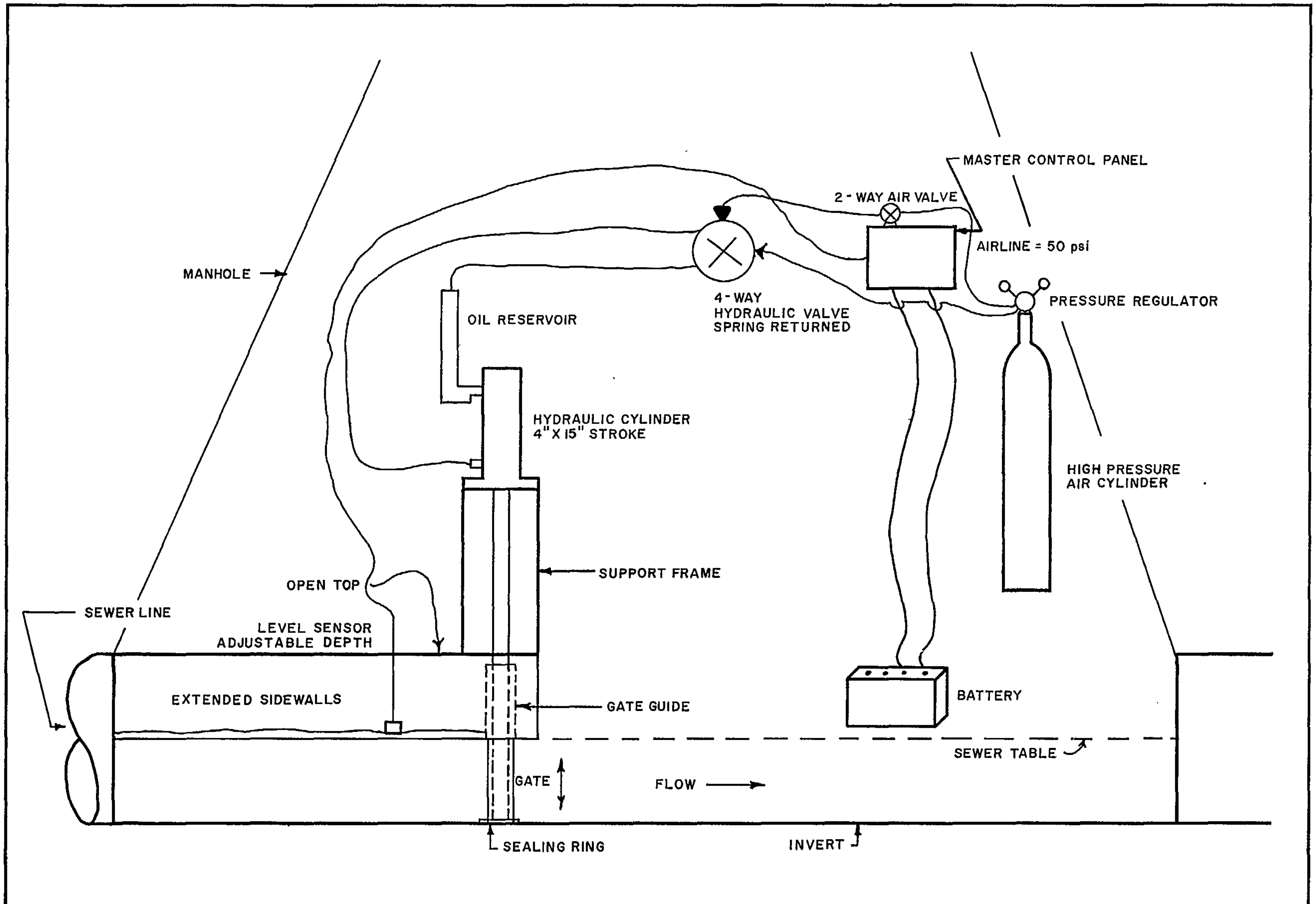
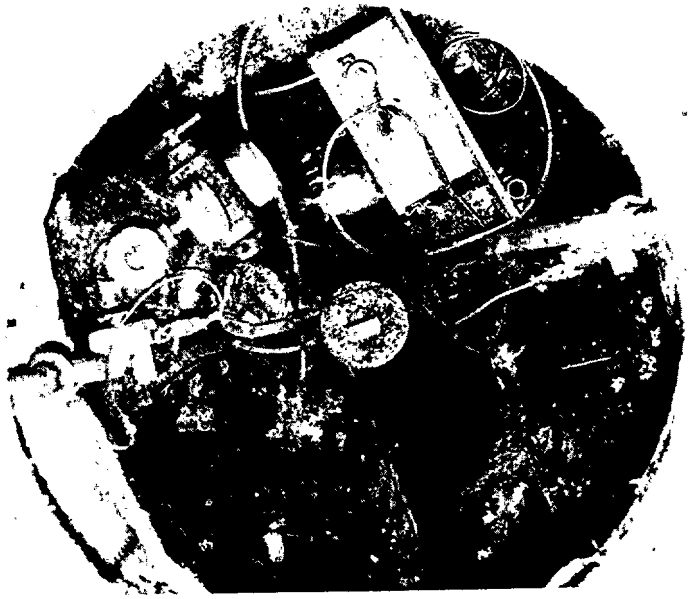


FIGURE 22 AUTOMATED SEWER FLUSHING MODULE



A. Top view of installed module showing high-pressure cylinder, oil reservoir, control panel and hydraulic cylinder

B. Angled view of installed module

FIGURE 23: PHOTOGRAPHS OF AUTOMATED SEWER FLUSHING MODULE

mechanism worked very well throughout the testing period, providing close to 100 percent stoppage under any sediment condition.

5.6.2 Operational Details of the Automated Flushing Module

The automated sewer flushing module designed for this study proved to be an effective flushing unit requiring minimal maintenance during the testing period. Initially, there were problems with gate sealing and manhole leakage during the first few weeks of operation. After the gate seal was modified the unit became practically service-free. The module battery and air supply were never replenished during almost five months of operation. The results of the testing program were considered very positive. Once the module was initially set at a 24-hour interval, it was left on for the entire program. All flushes were conducted automatically. In order to provide tracking of the backup behind the gate and a crosscheck on gate timing, a continuous recording liquid level sensor was installed in the well behind the gate and operated from 8/31 - 10/31/78. The level sensor recorded both dry and wet weather flow and gate operation. Several storms occurred during the automated flushing operation. No problems were encountered with module performance or with sewer backups during rainfall events.

5.7 Flow Gaging Methodologies

This section describes the various field procedures used in generating data for flow gaging of flush waves, as well as dry and wet weather. Procedures for utilization of the field generated data are presented in chapter 7. Accurate and reliable flow measurement of the unsteady state, turbulent flush waves, proved to be one of the more difficult field tasks encountered during the sewer flushing program. Several procedures were attempted, including: 1) steady state calibration of the flush segments using the flushing truck; 2) high pressure dye injection utilizing a special system developed during the study; and 3) utilization of measurement flumes specially constructed to minimize backwater and upstream sedimentation.

5.7.1 Dry and Wet Weather Flow Gaging

One of the parameters of interest to the sewer flushing study was establishment of baseline flow which could then be translated into per capita waste rates for later computation. A continuous recording liquid level sensor was installed in each of the sewer flushing test segments for extended periods to monitor liquid level during both dry and wet weather. This procedure was adequate for most of the segments, with the primary exception being Port Norfolk Street. The main difficulty encountered was the relatively low depth of flow in the order of 1/4-3/4 inch (0.6-1.9 cm) and small level variations making accurate resolution of liquid levels difficult. In order to increase level variability and therefore resolution of readings, a special constrictive flume was constructed using 4 inch (10.3 cm) pvc pipe with extended sides and a special polyethylene inlet section. The flume so constructed induced critical flow conditions with minimal head loss due to the nature of the inlet and outlet sections. This was particularly important to avoid biasing upstream sedimentation rates as would the use of a Palmer Bowlus or Parshall flume. Calibration of the flume so constructed was done with time of travel studies using Uracine dye, as well as direct inflow measurement using the water meter on the flush truck as an input source. In such a manner reasonably reliable determinations of dry weather flow rates were made possible.

For those segments with enough depth of water to allow adequate resolution of dry weather liquid levels, time of travel studies utilizing dye were conducted to develop calibration data used in the procedures described in chapter 7.

5.7.2 Steady State Flush Wave Calibration Procedures

Initially, attempts were made to use a Manning's equation based rating curve as a steady state surrogate for determining the flow of the passing flush waves. Although the procedure did not work well in approximating flush wave flow due to the non-steady state turbulent flow characteristics present, the methodology provided reliable dry weather flow rating curves. The following is a synopsis of the approach used in the field to generate calibration data for the procedure presented in chapter 7 of this report.

On arrival at a given test segment the depth of flow and depth of sediment, if any, were carefully noted. Substantial sediment layers of dry weather deposition were noted at some of the measurement sites and proved to be a problem in the curve fitting process since they varied over the course of the study. Velocity determinations were made by measuring the time of travel of dye injected immediately upstream from the measurement site. Dye (Rodamine B or Uracine) was used in all cases. The length of the segment was measured in the field at all sites. At least 3 separate time of travel measurements were taken for each flow condition, and, if necessary, more were taken until they coverged narrowly to one value which would then be used in computing the velocity at that stage.

The following procedure was used for the four sites where the water tanker was used for calibration purposes. Upon arrival at the site, the time of travel was recorded for the background flow. The tanks on the truck were filled from nearby fire hydrants and pressurized while still connected to the hydrants. Water from the truck was discharged into the upstream manhole of the test segment and the flow rate maintained for a sufficient period to allow stabilization of both the pressure in the tanks and the water depth in the downstream end of the pipe. The input flow rate from the trunk was then recorded from the flow meter on the truck. The flow depth in the sewer was recorded and the dye tests (time of travel) experiments were completed. Flows at lower depths were determined by progressively lowering the delivery rate from the truck. Using the input flow rate from the truck and an estimate of the background flow rate the total flow rate being routed through the pipe was determined within a small margin of error. These flow rates were used to check the velocity and flow determinations in the sewer. All data so generated was then input into the procedure described in chapter 7 to develop final rating curves for each of the four test segments.

5.7.3 Dye Injection Calibration

One of the more interesting aspects of the field efforts conducted during the sewer flushing study included the development and utilization of a practical, reliable method of calibration of steady state and more importantly non-steady state flows using dye injection. Flow calibration of the flush waves was of vital importance to the results of the study. During flushing experiments the concentrations of solids downstream from the injection point ranged up to 10,000 mg/l as suspended solids. Those conditions precluded the use of any direct velocity measurement device. Attempts were made to generate steady state based rating curves to allow for translation of the depth measurements taken during the flush waves, but they proved quite unreliable due to varying velocities encountered during the flush wave passage. Typically, velocities on the front side of the wave before and up to the peak were much greater for the same depth of flow than those encountered after the peak.

An alternative procedure using high pressure dye injection was developed. The procedure basically consisted of three distinct operational units including: 1) the high pressure injection nozzle system; 2) the pumping and air separation unit; and 3) a flow-through cell equipped fluorometer with recording readout. Figure 24 is a diagram depicting the dye injection system used. Development of the dye injection procedure was a fairly complex process. The utilization of fluorescent dyes in tracer studies coupled with fluorometer readouts has been widely applied to river and stream gaging studies. Similar applications as applied to sewer systems have typically failed to provide reliable results primarily due to significant fluorescence interference from compounds in the sewerage. This is especially true with respect to rhodamine compounds whose fluorescence peak is similar to that of phenolic compounds often found in sewerage systems.

The dye injection experiments conducted during this program utilized Uracine dye with a special filter system to eliminate any background interference. This filtering system was necessary since a Turner model 111 fluorometer (15) was used. Initially, samples were taken from all the sewer segments and analyzed on a Perkin Elmer Hitachi model 204 research spectrofluorometer. This unit has extreme selectivity and the capacity to scan both the exciter and analyzer wavelengths independently. Using a scanning procedure all fluorescent peaks of the sewerage were identified for both the exciter and analyzer. Samples of both Rhodamine B and Uracine dye were then subjected to the same procedure. The results of all scans were then compared and zero interference peaks identified. As it turned out, Uracine dye had strong fluorescent peaks in zero interference bands with the proper exciter and analyzer wave lengths. This fact was verified utilizing dye samples spiked with sewage. Once the optimum exciter and analyzer peaks were established, a Kodak filters manual (16) was used to identify combinations of filters that would provide the proper exciter and analyzer wavelengths on the Turner fluorometer.

The next step of the process was the development of the high pressure injection system itself. The system was constructed of a dye injection bar 6 feet (18.9 cm) in length, with a filter and control valve on one end and the nozzles on the other. The nozzles used on the dye bar were actually

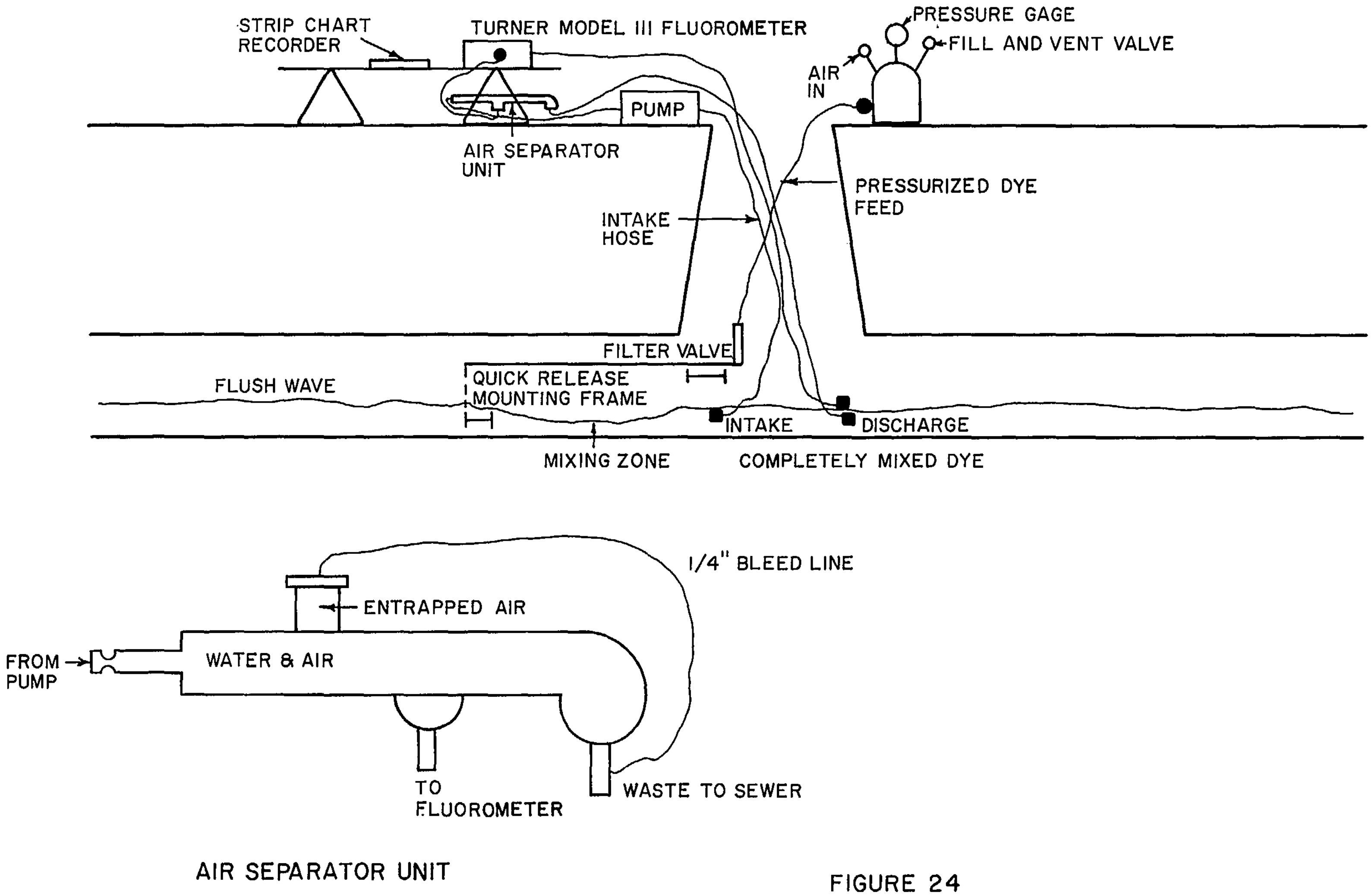


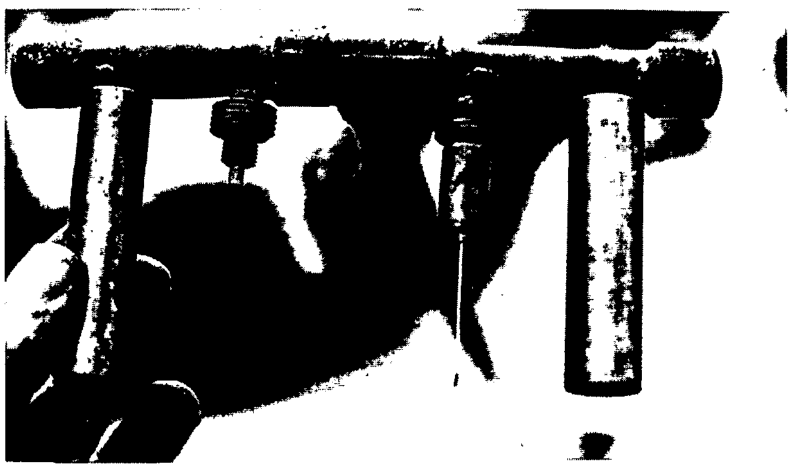
FIGURE 24
DYE INJECTION FLOW CALIBRATION
METHODOLOGY

stainless steel hypodermic needles. Needles were used because they were an ideal injection nozzle that can be readily varied in size, and which deliver a fine solid jet of dye at high pressures. Figure 25 shows photographs of the dye injection system. Figure 25B is a close-up photograph of the dye injection nozzles , mounting system and guard. Experiments were conducted to generate the ideal nozzle size/flow rate to maximize dye penetration and minimize input flow. Ultimately, 20 gage needles with 100 psi injection pressure was used, giving nozzle jet penetration of approximately 12 inches (30.5 cm). This penetration distance allowed the dye to penetrate the flush wave stream, hit the sewer invert and totally disperse. The result tested in a hydraulics laboratory flume produced homogeneous dye mixing within 3 inches (7.6 cm) of the injection point. The dye injection bar was coupled to a 5 gallon (18.75 liter) reservoir that was connected to the flush truck's pressurization system.

Dye detection was accomplished via a continuous flow-through system utilizing a high pressure pump through an air separater device, through the Turner fluorometer as shown in Figure 24. The net result was a continuous concentration time track of the flush wave passing the sampling manhole. Figure 25A and C show a view of the street level apparatus and an actual dye injection experiment respectively.

The dye injection procedure so developed proved to be extremely reliable in measuring the transient flush wave flow. Computed comparisons of injected volumes versus volumes detected downstream match within a maximum of 3 percent. The dye injection procedure so developed was used in the field to generate calibration data for the optimization procedure presented in chapter 7.

A. Photograph of Street Level Apparatus Including Fluorometer and Recorder, Injection Reservoir, Recirculating Pump and Lines.



B. Dye Injection Nozzles.

C. Actual Dye Injection Experiment.

FIGURE 25. DYE INJECTION APPARATUS

SECTION 6

LABORATORY ANALYSES AND PROCEDURES

6.1 Foreword

Field procedures used during the three phases of sewer flushing experimentation were described in Chapter 5. Descriptions of the various laboratory analyses, special experiments and procedures are given in this chapter. Parameter coverage for the discrete flush wave samples collected during all three phases of work are described in section 6.2. Discussion of heavy metals analyses of first phase composited flush wave samples is given in section 6.3. Various analyses of pipe sediment scrapings are described in section 6.4. Settleability experiments conducted during the latter half of phase two are described in section 6.5. Finally, a listing of all analytical procedures used in the analysis is given in section 6.6.

6.2 Discrete Flush Wave Samples

The flush wave pollutant characteristics were determined by analysis of 17 discrete liquid samples. A background sample was taken and analyzed for each flushing experiment. Figure 26 presents an overview of the sample handling during the first phase operations. Each of the discrete samples for all phases were analyzed for both Total and Volatile Suspended Solids. Analyses of BOD_5, COD, Ammonia, Total Kjeldahl Nitrogen, Ortho and Total Phosphate were performed for all samples for about half of the first phase flushing experiments. In addition, Total and Fecal Coliform bacterial levels were determined for all samples from the initial first phase flushing experiments. Analyses of COD, BOD_5, nitrogen and phosphorus levels were determined for selected samples from the second and third phase flushing experiments. Samples analyzed for these parameters were taken at the onset of the flush wave where peak concentrations occurred, and from the tail of the wave. These determinations together with estimated levels obtained by regression with Volatile Suspended Solids concentrations were used to characterize pollutant profiles for these flushes. The regression procedures used to fill-in missing data are discussed in the next chapter.

6.3 Heavy Metals Analysis of First Phase Flow Composited Flushing Samples

During the first phase, an approximate flow-proportioned one liter sample was made from the 17 discrete flush wave samples. Depth of flow and cross-sectional area characteristics were used in proportioning the samples to the one liter sample. The composite samples were allowed to settle for four hours and a supernatant sample collected. The remaining supernatant was carefully decanted and a representative sample of of the settled material collected.

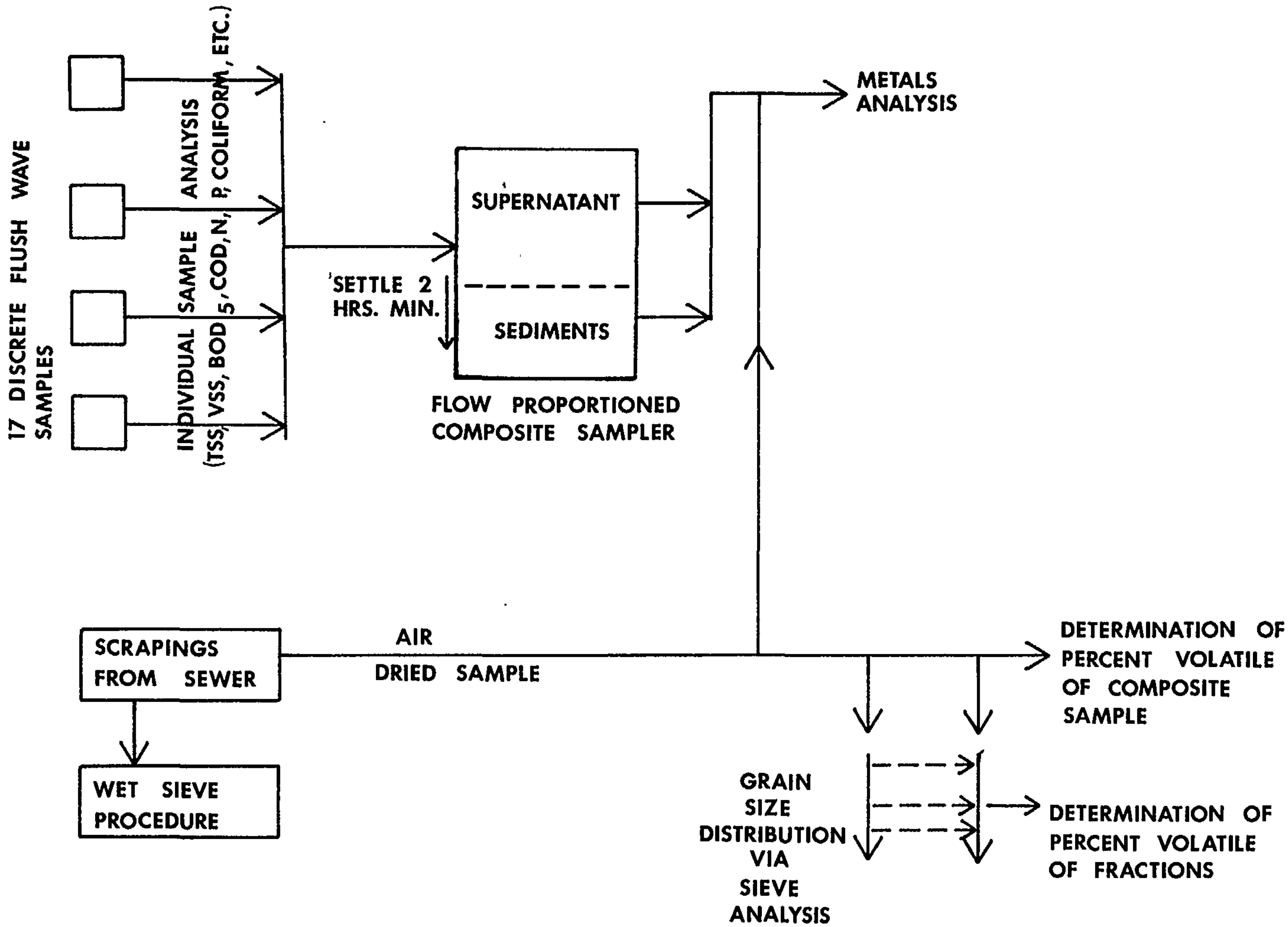


FIGURE 26 : OVERVIEW OF SAMPLE HANDLING DURING PHASE I

The supernatant sample and the bottom settled material (sediment represented materials which could eventually settle from the flush wave under ideal quiescent conditions)* are unlikely to occur in a sewer system.

Initially, all supernatant and sediment samples were analyzed for heavy metals including Cadmium, Chromium, Copper, Iron, Lead, Mercury, Nickel and Zinc. Shortly after commencement of the program, the results of the supernatant metals analyses indicated concentrations in the range of less than a part per billion. These levels were considered to be so low, especially in relation to the high concentrations found in the sediments, that subsequent heavy metals analyses were only conducted on the sediment fraction.

6.4 Analysis of Solids Scrapings

Before and after flushing during the first and second phase, sediments scraped from the sewer segments were evaluated using several methods. The primary concern of the effort was to conduct, on a representative basis, sieve analyses of the scrapings and determine the grain size distribution of the material as well as organic and inorganic content.

6.4.1 Wet Sieving Techniques. Initially a wet sieving technique was employed to determine grain size distribution of the scraped materials. The procedure was to simply mix the sample and pour it through a standard sieve series of sieve numbers 8, 16, 30, 50, 100, 200 and pan. This range of sieves yielding mesh openings ranging from 2.38 to 0.84 millimeters or coarse to fine sand/coarse silt on the Massachusetts Institute of Technology (M.I.T.) classification system. Unfortunately, levels of rag and paper meterials present in the flushed solids were very high, causing clogging of the coarse sieves. The procedure was discontinued after a short period of testing.

6.4.2 Dry Sieving Techniques. Two separate dry sieving techniques were evaluated. First the scraped material was placed on pre-weighed drying trays, and initial weight recorded. Samples were then air-dried at 68°C for several days and dry weight recorded. A portion of the dried sample was then saved for the heavy metals analyses. Approximately 1000 grams of dried sample was placed in a series of preweighed sieves and shaken on a standard sieve shaker for five minutes. The sieve series used was the same as that outlined for the wet sieves. The percent of sample retained on such sieve and pan was determined by weighing. The portion retained after ashing at 550°C for one hour permitted determination of the percent volatile solids of the sample.

The other dry sieving technique used involved splitting the air-dried sample in half. One half of the dried sample was placed through the sieves and total weights retained on each sieve determined. The other half of the sample was ashed at 550°C for one hour, and the ashed residue sieved used the same procedure as previously outlined.

The latter method appeared to be more accurate since most of the paper and cloth materials were removed prior to sieving. Since paper and cloth residues were common to most of the scraping samples, the accuracy of the sieve analysis on the organic portion of the sample could not be evaluated. The technique of splitting the air dried sample and conducting two separate

*This program with its extended settling period was aimed primarily at assessing dissolved versus potentially settleable fractions.

sieve analyses was used on the majority of samples in both the first and second phases of the study. All sieving data generated was plotted on standard M.I.T. classification sieve analysis paper.

6.5 Settleability Experiments

Settling characteristics of flush wave pollutants were evaluated during the latter portion of the second phase experiments. Six different flushing experiments were conducted in which samples were taken at each of three monitoring manholes for three consecutive flushes. These experiments were conducted in the Port Norfolk Street test segment. Three flow-composited samples were prepared for each flush. Details of the experiments including sample collection procedures and flow-compositing techniques were previously described in section 5.5.2. Each composite sample from the first flush on a given day was subjected to a settling column test while the samples from the second and third flushes were settled in an Imhoff cone for one hour. Table 3 lists the analytical analyses performed on samples collected during the six experiments.

TABLE 3: ANALYTICAL PARAMETERS MEASURED FOR THE DIFFERENT COLUMN AND IMHOFF CONE TESTS, SECOND PHASE PROGRAM

ANALYTICAL PARAMETERS	DATE					
	7/27/77	8/4/77	8/22/77	8/25/77	8/29/77	9/7/77
TSS	X	X	X	X	X	X
VSS	X	X	X	X	X	X
COD	X	X	X	X	X	X
BOD	X	X				X
TKN	X	X				X
NH_3			X			
OP				X		
TP				X		
Cd	X	X			X	X
Cr					X	X
Cu	X	X	X	X	X	X
Hg						
Ni					X	X
Pb					X	X
Zn	X	X	X	X	X	X

6.5.1. Imhoff Cone Testing. The procedure used for the Imhoff cone tests is described in Standard Methods, Section 208F, "Settleable Matter" (16).

One liter of well-mixed sample was poured into the Imhoff cone, while another completely-mixed sample was analyzed for the pollutants cited in Table 3. The sample in the Imhoff cone was allowed to settle for one hour. At the end of one hour the volume occupied by the settled material was noted and a portion of the supernatant was withdrawn for analysis. The fractions of pollutant mass removals were estimated using the results of the two analytical tests.

6.5.2. Settling Column Procedures. Three alternative methods were used for the settling column analyses including a small column, a large column with aeration mixing, and a large column with special gravity pre-mixing apparatus.

The first settling column analysis method utilized a series of 3" diameter, 18" deep columns with a single sampling port at mid-depth to establish removals for greater vertical velocities than those that could be feasibly measured in a larger column. A range of vertical velocites from 6 to 0.05 fpm could be evaluated. Several attempts using the small columns indicated that the small vessels were not suited for evaluating the flush wave samples. The major problem was that often the sample collected from the single sample port was in a zone of hindered or compression settling. Flocculent settling characteristics in the upper portion of the column were not noted. This method was thereafter discontinued.

The larger settling column employed during the testing program was a 6 foot long (1.6 meters) by 6 inches (15 cm) inside diameter acrylic tube. The 6 inch by 6 foot column size was chosen because of the volumetric constraint in obtaining the flush wave samples. Each test using this column would require approximately 10 gallons (375 liters) of sample. As explained in section 5.5.2, the sample used in the settling column testing program was a composite formed from six individual grab samples at each sampling manhole during the flush. Since a flow proportional composite sample was desired a minimum of 50 percent excess sample or 15 gallons (56.25 liters) would be required. A 15 gallon (56.25 liter) sample was arbitrarily chosen as the upper limit of sample that could be withdrawn from any given manhole without disturbing the flush wave as it proceeded downstream.

The basic procedures used during the column testing program followed those outlined by Zanoni and Blomquist (17). Six settleability experiments were conducted and their dates are listed in Table 3. The first two column tests used diffused air to uniformly suspend particulate matter throughout the column prior to the tests. The introduction of air resulted in the floatation of some material altering the settling characteristics of the column test. In order to eliminate this problem the column was modified so that mixing could be accomplished without the use of air. The construction of the column modification is shown in Figure 27. The last four column tests were conducted within two hours of sample collection. No attempts were made to mechanically mix the materials placed in the settling column prior to testing. Mechanical mixing would break-up organic solids, thus potentially biasing the column testing results in relation to settling within actual sewer lines. Samples collected from the settling column experiments were analyzed for solids, organics, heavy metals, nitrogen and phosphates. The parameter coverage for each of the experimentations is cited in Table 3.

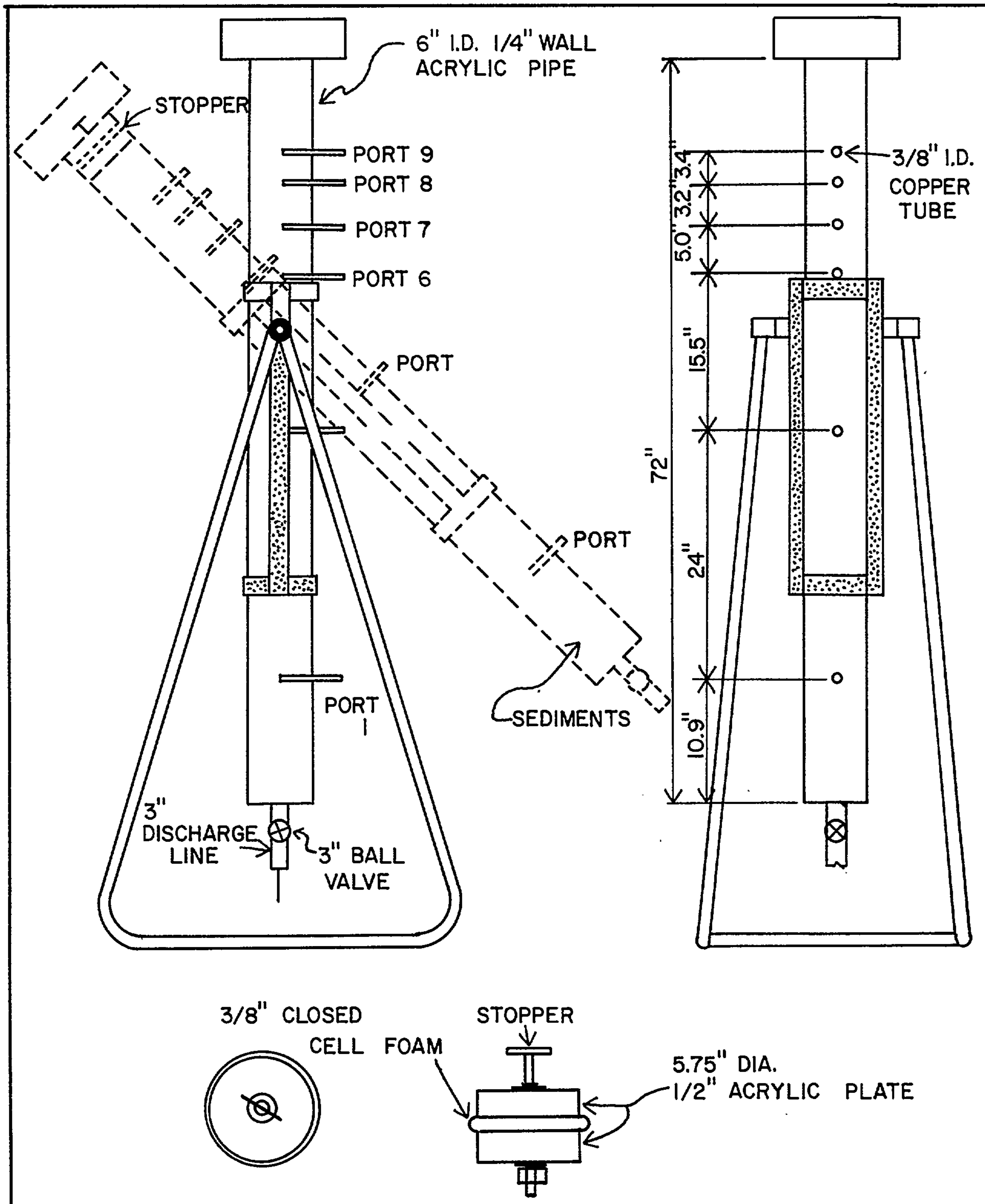


FIGURE 27 DIAGRAM OF SETTLING COLUMN

During the initial column tests compressed air was used for mixing. The air supply was introduced as the column was being filled prior to the start of the test run. The upper ports used for sampling were approximately one foot from the water surface to avoid the extraction of the air-entrained solids. During the filling of the column, heavy grit and stones rapidly settled out of the sample and deposited on the base of the column despite the continuous aeration.

Two sets of column tests were performed using this type of mixing. The samples collected on 7/27/77 were taken at 10, 20, 30, 60, 90 and 120 minutes. The results obtained from the 7/27/77 tests indicated a reduction of approximately 65-75 percent of the TSS concentration in the first 10 minutes of quiescent settling. In an attempt to characterize this rapid settling, the column runs performed on the second set of samples, collected on 8/4/77 were initiated immediately after stoppage of aeration. The other samples were taken at 5, 10, 20, 30, 60, 90 and 120 minutes. Results of these tests are discussed in Chapter 10.

The settling column was modified after the first two sets of settling column tests to permit mechanical mixing of the contents. In addition to the "flip" mixing system, the aeration stones were eliminated and larger diameter sample ports were installed to prevent clogging due to high solids content. The sample ports were changed to 3/8 inch from 1/4 inch, which resulted in a shorter time period for sample withdrawal.

The column filling procedure was similar to the two previous column tests with the exception that the initial sample was taken from the mixing barrel. The timing of the column run began as soon as the column was returned to the upright position.

The modified column was used for four sets of column tests conducted on 8/22/77, 8/25/77, 8/29/77 and 9/7/77. The sampling program for the experiments conducted on 8/22/77 and 8/25/77 entailed a two hour time period with samples collected at 5, 10, 20, 30, 60, 90 and 120 minutes. In an attempt to better define characteristics of the faster settling particles, sampling times of 2, 4, 6, 8, 10, 20 and 30 minutes were used on the latter two sets of experiments conducted on 8/29/77 and 9/7/77. The results obtained are discussed in Chapter 10.

6.6 Analytical Methods

The procedure used for the various parameters are as described in Standard Methods (18). These procedures were:

Suspended Solids (TSS)	Section 208D "Total Non-Filtrable Residue"
Volatile Suspended Solids (VSS)	Section 208D "Total Non-Filtrable Residue"
Chemical Oxygen Demand (COD)	Section 508 "Oxygen Demand (Chemical)"
Bio-Chemical Oxygen Demand (BOD)	Section 507 "Oxygen Demand (Bio-Chemical)"
Total Kjeldahl Nitrogen (TKN)	Section 421 "Nitrogen (Organic)"
Ammonia-Nitrogen (NH_3)	Section 102-8-f "Selective Ion Electrodes and Probes"
Ortho-Phosphate (OP)	Section 425E "Stannous Chloride Method"

Total-Phosphate (TP)	Section 425C	"Persulfate Digestion" followed by Section 425E, "Stannous Chloride Method"
Total Coliform (TC)	Section 909A	"Standard Total Coliform Membrane Filter Procedure"
Fecal Coliform (FC)	Section 909C	"Fecal Coliform Membrane Filter Procedure"

Metals - Cadmium, Chromium, Copper, Iron, Lead, Nickel and Zinc concentrations were determined from a single sample which had been pretreated following Standard Methods, Section 301A-VI. A separate sample and sample handling procedure was used for mercury analysis.

6.6.1. Pretreatment of Liquid Samples for Heavy Metals Determinations

Liquid samples collected during the first phase of the study and used for mercury analysis were not pre-digested. As a result only free mercury results were recorded. Liquid samples analyzed for mercury from all other phases of the study followed the procedure in Section 301A-VI of Standard Methods which measured total mercury. Pretreatment of liquid samples for other metals analysis followed Standard Methods using the method outlined in Section 301C, subsections II-5 and 6.

6.6.2. Pretreatment of Sediment and Scraping Samples

Pretreatment of solids samples followed the method outlined in Standard Methods, Section 301C, subsection II-6, except for the mercury samples. Sediment samples used in mercury analysis were subjected to a concentrated nitric acid leaching period of two hours. Throughout the leaching period the samples were gently agitated. The nitric acid leachate was then analyzed for mercury content. This mercury pretreatment method was followed for the first phase samples, all other mercury samples were pretreated using the digestion procedure in Standard Methods, Section 301A-VI.

6.6.3. Heavy Metals Determination

Mercury determinations followed Standard Methods, Section 301A-VI using cold vapor atomic absorption. Copper, zinc, and iron determinations were performed using flame ionization atomic absorption, Standard Methods, Section 301A. Direct aspiration into an air-acetylene flame was used. A carbon cup atomizer was used for determination of cadmium, chromium, lead and nickel because of their low concentrations. A Varian A-6 atomic absorption spectrophotometer fitted with a carbon cup atomizer was used.

SECTION 7

COMPUTATIONAL METHODS

7.1 Introduction

This chapter describes various computational procedures used to estimate the quantities of pollutants transported from the flushing segments. Conversion of the non-steady state flush wave stage levels into discharge proved to be a difficult but essential detail in converting discrete flush wave pollutant concentrations into quantities of mass transported. The motivation for investigating the alternative flow computational procedures presented in this chapter was to develop reasonably accurate estimates of the non-steady state flush wave flow rates from the field depth of flow measurements.

Data processing of the analytical laboratory results of samples taken during the flushing experiments and pertinent physical field information are described in section 7.2. Approaches used in the estimation of missing flush wave pollutant concentration levels are described in section 7.3. This step was necessary since total flushed mass transport estimates were computed using discrete values of discharge and flush wave pollutant concentrations at fixed intervals in time. Samples were not always analyzed for all analytical parameters and during several experiments an incomplete set of samples was taken.

Alternative procedures are described in section 7.4 for estimating instantaneous flush wave discharge values from stage level readings. Three alternative approaches are presented: 1) the application of Manning's equation using plan pipe slope and variable roughness coefficients; 2) utilization of Manning's equation with a virtual slope derived from least squares fitting of that equation to steady-state field flow calibration points; and 3) utilization of mathematical programming techniques for determining parameters of complex loop-rating curves. Comparison of integrated flush volumes from predicted flow rates with known flush delivery volumes indicated that the first approach grossly misestimated the actual flow rates. The second approach substantially improved the predicted results but neglected to account for the unsteadiness of the flow regime because of the strict application of the steady uniform flow rating curve. Actual field observations and measurements suggested that the flush wave hydraulic characteristics are best described by a looping stage-discharge curve with higher flow rates in the front of the wave than in the back of the wave for similar flow depths. The third approach described this phenomena in an extremely reasonable way and was therefore selected for converting flush wave stage recordings into discharge. Computed flush rates are compared with field measured flush rates using dye injection procedures described in Chapter 5.

Section 7.5 describes the procedure for estimating the flushed pollutant masses for all phases of work.

7.2 Raw Data Handling

Subsequent to laboratory analysis of the samples collected during each flushing experiment, relevant information were data processed for further handling and analysis. This information consisted of: a) the street name, date and hour of the flush; b) a brief characterization of the flushing technique used indicating whether the flush was a pressure flush, a gravity flush or a backup and release flush; c) whether or not the upstream manhole was blocked during the flush; d) whether time of travel measurements using dye has been performed; e) the truck delivery volume, in cubic feet, used in flushing the pipe segment(s); f) the flush duration, that is, the time, in seconds, during which the flush volume was introduced in the pipe; and g) the recorded time, in seconds, elapsed between the opening of the quick-action gate valve in the truck and the instant a sudden rise in the water level in the downstream manhole was noted. Next, data cards for each collected sample were prepared containing: a) an order number of the sample collected; b) the time in seconds between the present and previous sample; c) the flow depth in the pipe at the time the sample was collected; and d) concentrations, in mg/l, of COD, BOD, TKN, NH_3, TP, OP, TSS, VSS, and Total Coliform and Fecal Coliform bacteria, in colonies/100 ml. The last data card per flushing experiment contained: a) pipe diameter; b) the average of the sediment depths at the downstream manhole measured prior to and after the flush; c) the pipe slope; and d) the Manning's roughness coefficient believed to be appropriate for the particular flush. Information contained on this last card was subsequently reassessed and improved through an optimization technique described in Section 7.4. Typically, cards were prepared for each flush. A sample of the data cards showing the results of the 10/04/76 experimental flush for the Walnut Street test segment is shown in Table 4.

Once all the data cards for one particular phase of the project were punched and edited, disk files were created for further editing, processing, and analysis.

7.3 Missing Data Fill-in Procedure

In general, TSS and VSS were determined for all samples collected during all three phases of the field program. Determination of COD, BOD TKN, NH_3, TP and PO_4 were only conducted for selected flushes. Furthermore analytical determinations were not always performed for all 18 samples collected during each flush. As a consequence there were gaps in the data requiring filling-in before the flushed pollutant masses could be computed. The number and time distribution of the missing data varied during the three phases of experimentation, requiring different criteria for estimating missing flush wave pollutant concentration points.

TABLE 4. SAMPLE OF DATA CARDS FOR THE FLUSH OF 10/04/76 AT WALNUT STREET

4
WALNUT STREET 10 04 76 11:05 AM
GRAVITY FLUSH WITH 3 INCH NOZZLE AND HOSE, UPSTREAM MH NOT BLOCKED
NO 0 35.0 71.6 54.0

A	B	C	D	E	F	G	H	I	J	K	L	M
* 0	0.		920.	960.	83.	35.	28.2	22.0	2202.	1847.	14000000.	
** 1	0.	5.5	840.	1290.	95.	30.	29.4	20.7	5551.	3855.	49000000.	7000000.
2	10.	5.75	2120.	1140.	100.	31.	29.0	24.1	3698.	2792.	41000000.	7000000.
3	20.	6.	2880.	840.	95.	32.	30.2	27.2	3155.	2347.	14500000.	9500000.
4	30.	6.25	2120.	720.	127.	35.	34.2	22.3	2584.	1876.	10000000.	1100000.
5	40.	6.5	208.	720.	102.	31.	28.2	22.6	1943.	1328.		
6	50.	6.75	1680.	795.	83.	26.	25.0	17.1	1947.	1271.	11000000.	1800000.
7	60.	6.5	2720.	750.	81.	24.	20.4	14.5	1883.	1266.	13500000.	1100000.
8	70.	6.25	2240.	740.	71.	23.	20.0	14.0	1509.	981.	14600000.	500000.
9	80.	6.25							1519.	984.		
10	100.	6.25							1477.	998.		
11	120.	6.25										
12	140.	6.	1680.	577.	53.	10.	13.6	7.6	1252.	853.	7400000.	270000.
13	160.	6.							1554.	1101.		
14	180.	6.							1466.	1045.		
15	200.	5.75	1840.	585.	45.	7.	11.6	6.2	1627.	1154.	7200000.	
16	220.	5.5							922.	663.		
17	240.	5.5	1200.	315.	7.	3.	5.2	3.0	664.	493.	4000000.	320000.

***15. .5 0.00248 0.015

KEY: A - Sample number, where (*) implies background sample and (**) implies sample taken at first occurrence of wave.
B - Time interval of sampling.
C - Depth of flush wave (inch).
D through M - COD, BOD, TKN, NH_3, TP, OP, TSS, VSS, TC and FC, respectively.
*** - Pipe diameter (inches), average sediment depth (inch), pipe slope and estimate of Manning's coefficient.

7.3.1 Estimation of Missing Data - First Phase

Out of the 83 flushing experiments 44 were analysed for pollutants other than TSS and VSS and, of these, 19 were analysed for only BOD. In general, sample numbers 1 through 9, 12, 15 and 18 were anlyzed for pollutants other than TSS and VSS. The approach was to monitor over the initial peak concentration period and then at the tail of the wave. In a few cases gaps existed between the first nine samples and/or in the later part of the flush wave. Linear interpolation of the missing values between two known values was used to estimate the gaps in the first phase flush data. Interpolation of missing data was only used for computing the pollutant masses removed, so that the original data files remained unaltered.

7.3.2 Estimation of Missing Data - Second Phase

In the second phase 18 serial flushing experiments were performed on Port Norfolk Street, with samples collected at three successive manholes downstream from the manhole where the flush wave was introduced. Eighteen sample sets were collected at the first and second downstream manholes and 15 sample sets were taken for the third downstream manhole. TSS and VSS determinations were performed for all samples collected at each manhole during this phase of work. Samples depicting the frontal portion of the flush wave were analyzed for COD for all flushes. For several flushes samples from the frontal portion of the flush wave were analyzed for BOD and for a few flushes, TKN determinations were performed in a similar manner. Typically, 5 to 6 samples out of the first 9 collected were analyzed for COD and BOD. The number of TKN analytical determinations were too few to justify presentation of results in terms of the masses of TKN removed by the flushes. Only missing COD and BOD values were estimated using the following regression relationships:

$$\frac{\text{COD (mg/}\ell\text{)}}{\text{Port Norfolk St.}} \qquad \text{COD} = 6.018\ \text{VSS}^{0.795} \qquad (R = 0.92) \qquad (1)$$

$$\frac{\text{BOD (mg/}\ell\text{)}}{\text{Port Norfolk St.}} \qquad \text{BOD} = 1.907\ \text{VSS}^{0.811} \qquad (R = 0.86) \qquad (2)$$

The COD and BOD equations were derived on the basis of 100 and 110 pairs of measured COD/BOD and VSS flush concentrations from the Phase two flushing program results. Linear regression equations derived from the same data resulted in slightly inferior fits than the logarithmic linear equations presented above. Linear and log-linear regressions of both BOD and COD on TSS were also inferior. Estimation of missing gaps was only performed in the course of computing the masses removed, leaving the original data files free from estimated values.

7.3.3 Estimation of Missing Data - Third Phase

The third phase program consisted of seven flushes on Shepton Street using the automatic device to backup and release the stored volume of sewage when the liquid behind the blocking gate reached a prespecified level. Seventeen samples were collected in the first downstream manhole for each flush. TSS and VSS determinations were performed on all samples for all seven flushes. COD analyses were also performed for all flushes on samples collected from the front part of the flush wave and a few toward the end of the wave. Although the missing COD concentrations could be estimated by interpolation, the time gaps between two successive values at the tail, for half of the flushes, were large (150 seconds out of 230 seconds for the entire flush) precluding the use of simple linear interpolation. Regression of the missing COD data on VSS was again utilized. The regression equation used to complete the missing COD values is:

$$\text{COD} = 3.1995\ \text{VSS}^{0.9141} \quad (R = 0.91) \qquad (3)$$

This relationship was derived from flush data at Shepton Street.

7.4 Stage Discharge (H-Q) Relationships

In chapter 5 procedures for determining flush wave stage levels were described. These recorded flow depths required translation into flow rates in order to convert pollutant concentrations associated with the discrete estimates of flow rate into mass rates transported from the flushed pipe segment. Several alternative procedures for converting stage into discharge were investigated. The performance of each procedure was evaluated in terms of how well the estimated flow rates, when integrated over the flush duration, could reproduce known input flush volumes for each flush. The procedures considered are the following:

1. Application of Manning's equation assuming uniform flow, with variable roughness coefficient, n, and pipe slope computed using manhole elevations and segment length;

2. Application of Manning's equation assuming uniform flow, with variable η and a slope derived from a least square fit of the slope to field determinated steady state stage-discharge calibration points; and finally,

3. Establishment of stage discharge relationships through a mathematical optimization procedure that accounts for the unsteady nature of the flow regime of the flush wave while minimizing error between computed and measured flush volumes.

7.4.1 Definition of the Flush Input Volumes

Since the flush volumes were used to gage the relative predictive precision of the two initial approaches and directly in the third procedure for establishing the stage discharge relationships, it was therefore important to accurately determine delivered flush volumes. Input flush volumes were measured by a 4 inch turbine water meter located on the flush truck which was repeatedly calibrated during the project using a large vessel of known volume. For flushes where the upstream manhole was not blocked, the additional volume due to the upstream flow over the duration of the flush was added to the metered flush volume to yield the total flush volume. Background flow contributions were endogenously estimated in the flush flow rate computations which are discussed in Section 7.4.4. Except in the cases of unblocked manholes, metered flush truck volumes were finally used as the total quantity of flush water. This conclusion was reached after small and extraneous flow contributions and metering errors were considered in a sensitivity analysis to be negligible.

These considerations tended to either raise or lower the metered truck volumes. Additive sources of extraneous flow included hydrant leakage into gutters, catchbasins and eventually into the combined sewer test segments for Port Norfolk and Walnut Streets; ground water infiltration along the flushed segment; and, house connections along the flushed pipe segments. Factors tending to counter balance these additive effects were slight positive water meter bias and the fact that, for most of the flushes, the last flush wave stage level was on the average 20% higher than the initial background depth, indicating that the flush wave had not completely passed the sampling manhole by the time the last smaple was collected. Rough order of magnitude estimates for each factor and source and their effect on the metered flush volumes follow.

Hydrant Leakage. Hydrant leakage did not affect the flushes at Templeton and Shepton Streets since the sewers are separated. Leakage at Walnut Street was negligible, but could be as high as 2 cf at Port Norfolk Street, particularly during the high pressure flushes. This estimate resulted from computations involving gutter water depths, gutter shape and slope and duration of the time the hydrant was leaking.

Ground Water Infiltration. This contribution was small for the first phase flushes that were conducted from late August to mid-November. Infiltration was higher for the second phase program covering the period from January to the end of March. In the third phase of work, occurring between late September to mid-October, the infiltration was again low. It was estimated that ground water infiltration would range up to 1 cf during the 4 minutes of sampling. In terms of aerial infiltration rates, this figure corresponds to 2300 gallons/acre/day at Port Norfolk Street and 4000 gallons/acre/day at Walnut Street.

House Connections Along Flushed Segment. This contribution was the hardest to assess in terms of generalized average values because of the random nature of household inflows over the short periods of flushing. Using estimated population along each experimental segment and estimates of the number of per day uses of major household fixtures per capita and assuming that the number of occurances of such uses in a short interval of time is Poisson distributed (and independent of time of day), it was estimated that there is more than 95% chance that the number of discharges in a four-minute period is less than or equal to the values shown below:

MAXIMUM NUMBER OF ESTIMATED HOUSEHOLD DISCHARGES IN A 4 MINUTE PERIOD

STREET	NO. OF DISCHARGES
Templeton	3
Shepton	4
Port Norfolk	2
Walnut	1

Estimates of dry weather sewage contributions for a four minute interval using an average per capita waste rate of 85 gpd (4) are given below:

AVERAGE WASTE VOLUMES IN A 4 MINUTE PERIOD

STREET	POPULATION*	AVG. VOL. IN 4 MIN. (CF)
Templeton	50	1.58
Shepton	70	2.21
Port Norfolk	33	1.04
Walnut	14	0.44

*Contributions along the pipe segment flushed.

Using the expected maximum number of discharges and assuming a conservative figure of 1 cf per discharge, the probability is small that, if the contributions did occur during the flushing period, the estimated sewage volumes would be approximately 1, 2, 3 and 4 cf for Walnut, Port Norfolk, Templeton and Shepton Streets, respectively.

Water Meter Inaccuracies. Meter calibration was conducted by filling a vessel of known volume at different indicated rates. At the onset of phase one, the errors in the flow-rate range of 0.5 to 1.0 cfs were found to be random and within 2.0%. Near the end of the first phase flushing experiments, it was determined that between the flow rates of 0.5 and 1.0 cfs, the range of most of the flushes, the meter, on the average, tended to overestimate the delivered volumes by about 6% at 0.5 cfs to a maximum of 7.5% at 1.0 cfs. The average error at 0.1 cfs was of about 2%, 3.5% at 0.2 cfs, and 4.5% at 0.3 cfs. Later testing of a new replacement meter indicated errors averaging around 5% for flow rates between 0.5 and 1.0 cfs. Using the above results, it is estimated that the metered flush volumes during first and second phase programs were overestimated by about 3.5% to 7.5%. This bias amounts to a minimum absolute value of 1.23 cf for a 35 cf flush, and up to 3.75 cf for a 50 cf flush.

Wave Tail Considerations. It was mentioned earlier in this section that the last stage reading recorded during the flushing experiments was higher than the initial background depth for most flushes during the first and second phases. For the first phase flushes, the final depth was on the average 20% higher than the initial background depth of flow. This percentage was lower for the second phase flushes.

These observations are summarized in Table 5. Columns 3, 4 and 5 of Table 5 indicate, by phase and by street, the number of flushes in which the last depth of flow was respectively higher, equal to and smaller than the initial background depth of flow. The mean and standard deviation, in inches, of the differences between the last and first depth, computed for flushes for which the last flow depth exceeded the initial depth are given in Column 6. These results show that the average difference did not exceed 1 inch. The residual depth implies continued flush wave movement. What fraction of the original flush volume still remained in the flushed sewer pipe segment is extremely difficult to establish. It is believed that any residual volume still remaining in the flushed pipe at the time of the last measurement would be roughly several cubic feet. The following discussion is provided to support this conclusion.

TABLE 5. CHARACTERISTICS OF THE TAIL OF THE FLUSH WAVE AND FLUSH WAVE VELOCITY

(1) Phase	(2) Street	(3) Number of Flushes where: $H_{18} > H_1$*	(4) $H_{18} = H_1$	(5) $H_{18} < H_1$	(6) Mean/S.D. of H_{18} Above H_1 (inches)	(7) Mean/S.D. of Wave Velocity (fps)
1st	Templeton	17	4	-	0.94/0.42	1.92/0.52
	Shepton	20	2	-	0.84/0.58	1.77/0.46
	Port Norfolk	17	2	-	0.63/0.43	1.88/0.45
	Walnut	17	4	-	0.82/0.25	1.54/0.31
2nd	Port Norfolk 1 DMH**	11	6	1	0.35/0.17	1.85/0.32
	Port Norfolk 2 DMH	15	3	-	0.77/0.29	-
	Port Norfolk 3 DMH	14	-	1	0.96/0.65	-

*H_1 = background depth.

H_{18} = last depth measured.

**1 DMH = first downstream manhole.

The last column in Table 5 presents minimal estimates of the mean and standard deviation of wave velocities computed over all flushes for a particular street using the pipe segment length and the recorded time elapsed between the start of the flush and the time taken for the peak of the wave to reach the first downstream manhole. This procedure yields low wave velocity estimates because the travel times are referenced from the start of injection. An approximate estimate of the location of the wave peak downstream of the sampling manhole can be computed at the time of the last measurement using these velocities and the time between the passage of the peak of the wave and the last depth measurement taken.

The time intervals between the passage of the wave peak and the last depth measurement were, for the great majority of the flushes, between 190 to 200 seconds. On the basis of the wave velocities indicated in Table 5. it is estimated that the flush wave peak should be between 300 ft. (Walnut Street) to 385 ft. (Templeton Street) downstream from the sampling manhole at the time the last depth of flow was measured. These estimated distances represent more than one, sometimes two pipe segments downstream from the sampling manhole. These calculations indicate that by the time the last depth of flow was measured most, if not all, of the flush wave would have passed.

Additional evidence is provided by the dye injection flow monitoring experiments described in Section 5.7. Continuous recording of dye concentrations for a number of typical flush waves showed a considerable velocity decrease at the end of the wave. These observations indicate that the flush flow rate at the time of the last depth of flow measurement was slightly greater than background flow rates. These results together with the earlier crude time of passage calculations suggest that a nominal 5% estimate of the metered volume be assumed as the residual volume in the upstream pipe segment at the time the last sample was collected.

Assessment of the factors affecting the metered flush volumes are summarized in Table 6. The first three factors represent volumes that are additive to the nominal metered volumes while the last two factors represent negative corrections to the metered flush volumes. Subtotals of the ranges of both the positive and negative corrections are also presented and indicate that the nominal metered values should be slightly above the volume that could be computed from the depth measurements. In other words, the metered volumes are likely to slightly overestimate the volume of water discharged into the sampling manhole, that is, the estimated subtotal values differ by only a few cubic feet on each street.

TABLE 6. ESTIMATED RANGES OF POTENTIAL BIASES TO THE METERED FLUSH VOLUMES (cf)

	Street			
	Templeton	Shepton	Port Norfolk	Walnut
1. Hydrant leakage	-	-	+(0-2)	-
2. Ground water Infiltration	+(0-1)	+(0-1)	+(0-1)	+(0-1)
3. Household discharge	+(0-3)	+(0-4)	+(0-2)	+(0-1)
Subtotal	+(0-4)	+(0-5)	+(0-5)	+(0-2)
4. Meter bias	-(1.23-3.75)	-(1.23-3.75)	-(1.23-3.75)	-(1.23-3.75)
5. Wave tail considerations	-(0-2.5)	-(0-2.5)	-(0-2.5)	-(0-2.5)
Subtotal	-(1.23-6.25)	-(1.23-6.25)	-(1.23-6.25)	-(1.23-6.25)

It was assumed that all these factors would tend to balance, withstanding the apparent slight bias. The uncorrected metered flush volumes were taken as the yardsticks in judging the suitability of a particular stage-discharge relationship. A stage-discharge relationship was deemed desirable when the computed flow rates, integrated over the sampling period, yielded a volume approximating the metered flush volume for that flush.

7.4.2 Discharge Estimates: Manning's Equation with Pipe Slope

The use of Manning's equation to compute discharge in sewers from flow depth measurements is of universal practice in environmental engineering. In foot-second units, Manning's equation is given by

$$Q = \frac{1.49\ r^{2/3}\ S^{1/2}\ a}{n} \qquad (4)$$

where: Q = flow rate (cfs);
R = hydraulic radius (ft);
S = energy slope (ft/ft);
a = flow area (ft^2); and
n = roughness coefficient.

Application of Manning's equation usually assumes steady uniform flow by taking $S = S_o$, the pipe slope. It is also a common practice to assign a constant value for the roughness coefficient n according to pipe material, age and state of maintenance. Values of 0.013 and 0.015 are often used in sewer system computations.

As a first approximation Manning's equation was used for the first phase flushes, with two refinements:

1. The average of the sediment depths measured prior to and after the flush was taken into account since sediment depth changes the pipe shape, the hydraulic radius, and the area of flow; and,

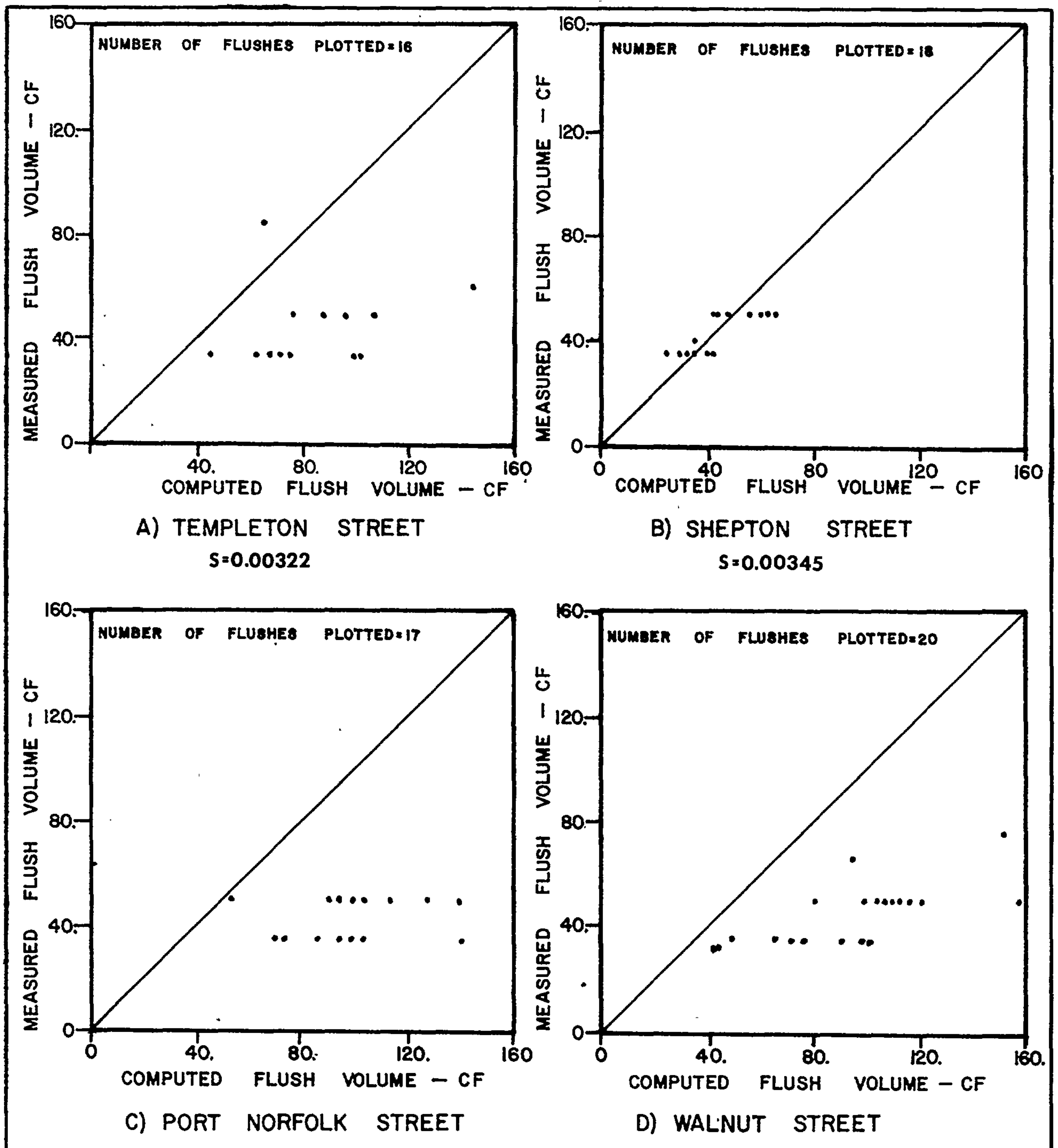
2. The Manning's roughness coefficient n was assumed variable with the flow depth.

The physical slopes of the test pipe segments, determined from construction drawings, at the four streets are:

Street	Slope(ft/ft)
Templeton	0.00322
Shepton	0.00345
Port Norfolk	0.00486
Walnut	0.00481

In all four cases the values of n varied from 0.015, at full pipe, to a maximum of 0.01935 at a depth around 1/3 of the pipe diameter.

Manning's equation was applied to the 87 flushes of the first phase to compute the flow rates from the flow depths measured in each flush. The total volume per flush was computed by a discrete integration of the flow rates over the flush duration. The computed flush volumes are plotted in Figure 28. versus the measured input flush volumes for each test segment. Except for the Shepton Street flushes and one flush in Templeton Street the plotted points fall all on one side of the 45° equivalence line. The rating curves would therefore substantially misestimate the flush volumes. Direct application of pipe slopes in Manning's equation for computing flow rates would have greatly misestimated flush pollutant masses.



Obs.: Several flushes may coincide in one plotted point

FIGURE 28 PLOTS OF MEASURED VERSUS COMPUTED FLUSH VOLUMES MANNING'S EQUATION (PLAN & PROFILE SLOPE) - FIRST PHASE

7.4.3 Discharge Estimates: Fitted Slope to Calibration Points Using Least Squares

One improvement to the straightforward application of Manning's equation is to adjust either the slope S and/or the roughness coefficient n. The second alternative procedure for establishing the required stage-discharge relationships was to utilize field stage-discharge calibration results in a least squares approach to determine an adjusted slope $\hat{S}$. Steady state stage-flow measurements were conducted using the flush truck and are described in Chapter 5. The roughness coefficient, n, was assumed variable in this analysis.

Extensive laboratory and field experiments (19, 20, 21 & 22) indicate that the value of n varies with the depth of flow in the pipe. A smooth curve relating the ratios n/n_f and h/D, where n_f is the value of n at full pipe depth was used (22). The value of n at full pipe (n_f) and its variation with depth was assumed known and the slope, $\hat{S}$, was taken as a surrogate for all the uncertainties with respect to S itself and the roughness coefficient n.

A number of the hydraulic parameters in Manning's equation can be expressed as a function of the ratio of flow depth to pipe diameter. Equation (4) is rewritten as:

$$Q_i = 1.49 \frac{r_i^{2/3} S^{1/2} a_i}{n_i} \qquad (5)$$

where:
- Q_i = $f(h_i/D)$, flow rate (cfs);
- r_i = $f(h_i/D)$, hydraulic radius (ft);
- S = constant, pipe slope (ft/ft);
- a_i = $f(h_i/D)$, wetted area (ft^2);
- n_i = $f(h_i/D)$, Manning's friction factor;
- h_i = water depth in the pipe, (ft); and
- D = pipe diameter, (ft).

Assuming that the roughness coefficient is known and allowing the slope to take up all the uncertainties with respect to the true hydraulic slope and roughness coefficient used, the objective function of a weighted least squares fitting is written as:

$$\min_S OF(S) = \sum_{i=1}^{m} \frac{1}{F(h_i/D)} [Q_i - S^{1/2} F(h_i/D)]^2 \qquad (6)$$

where: Q_i = observed values of flow from the calibration and

$$F(h_i/D) = \frac{1.49\, a_i\, r_i^{2/3}}{n_i} \quad ;$$

m = number of observed Q_i; and all other variables are as defined before.

Setting $dOF(S)/dS = 0$ and solving for $\hat{S}$ the expression for the "best-fit" value of $\hat{S}$ is given by:

$$\hat{S} = \left[\frac{1}{1.49} \frac{\sum_{i=1}^{m} Q_i}{\sum_{i=1}^{m} \frac{r_i^{2/3} a_i}{n_i}} \right]^{0.5} \tag{7}$$

Several other approaches could have been used to fit Manning's equation to the data points. One intuitive approach is to optimize the roughness coefficient, n, instead of the slope, $\hat{S}$. The variability of n with h/D can be expressed by an appropriate mathematical expression and the optimization of the variable n function by non-linear means would be possible. The procedure to fit slope $\hat{S}$, using least squares, is much simpler and was therefore adopted. Least squares results using equation 7 are presented for each test segment as follows:

Street	Number of Field Calibration Points	Best-Fit Slope	Correlation Coefficient
Templeton	6	0.00115	0.95
Shepton	9	0.00388	0.95
Port Norfolk	4	0.00141	0.91
Walnut	6	0.00221*	0.84

*Assuming a dead zone of 3.85" at the bottom of the pipe.

An illustration of the alternative stage-discharge curves discussed to this point for the Port Norfolk and Shepton Street test segments are plotted in Figures 29 and 30. The field calibration points are circled. Four alternative curves are shown in each figure representing:

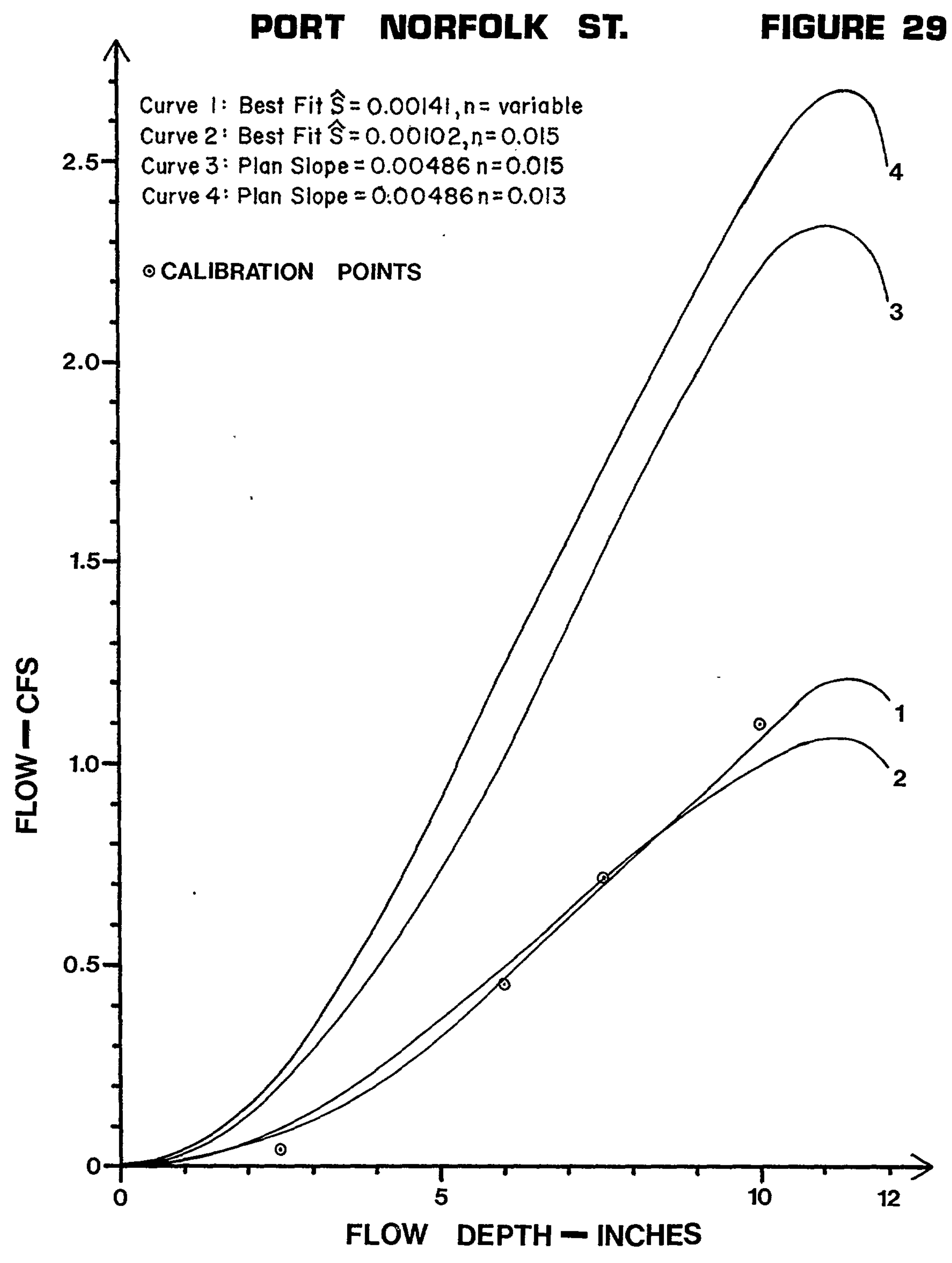
1) least squares fit of the slope assuming a variable roughness coefficient;

2) least squares fit of the slope assuming a constant roughness coefficient n = 0.015;

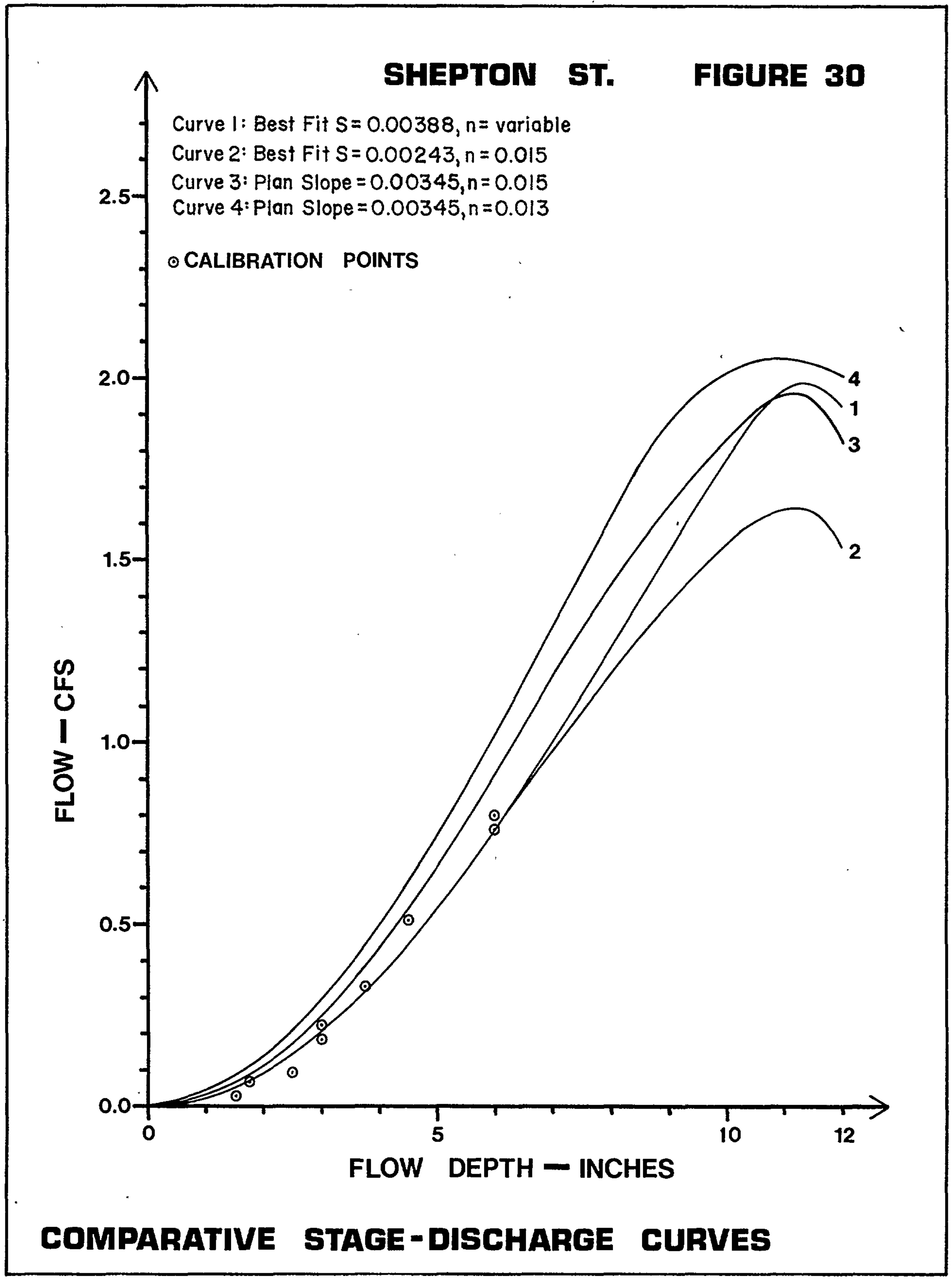
3) discharge curve given by Manning's equation using plan slope and n = 0.015; and,

4) discharge curve given by Manning's equation using the plan pipe slope and n = 0.013.

Curves labelled 1 and 2 in Figure 30 for the Shepton Street site differ only slightly in the range of the available calibration points. Only curve number 1 was plotted in that range for clarity. Curves labelled 3 and 4 derive from simple application of Manning's equation described in section 7.4.2.



COMPARATIVE STAGE-DISCHARGE CURVES


SHEPTON ST.
FIGURE 30
Curve 1: Best Fit S = 0.00388, n = variable
Curve 2: Best Fit S = 0.00243, n = 0.015
Curve 3: Plan Slope = 0.00345, n = 0.015
Curve 4: Plan Slope = 0.00345, n = 0.013
⊙ CALIBRATION POINTS
FLOW — CFS
0.0
0.5
1.0
1.5
2.0
2.5
0
5
10
12
FLOW DEPTH — INCHES
4
1
3
2
COMPARATIVE STAGE-DISCHARGE CURVES

Curve number 1 represents the proposed solution, against which the other three will be compared. Curve number 2 results from a least squares fit of the slope to the calibration points assuming roughness coefficient at a constant value of 0.015. Examination of curves 3 and 4 show that the least squares fit with the variable roughness formulation is superior to the curve with fixed roughness coefficient. Correlation coefficients computed over the pertinent range of observed h/D values for the Port Norfolk and Shepton Street segments support this conclusion and are given below:

Street	Range of Observed h/D	Variable n (n_f = 0.015)	Fixed n = 0.015
Port Norfolk	0.21 - 0.83	0.91	0.57
Shepton	0.13 - 0.50	0.95	0.93

The two examples also show that, although the two curves are not considerably different in the lower to mid range of depth of flow, the divergence increases at higher depths. Its implication is that the use of variable n becomes more important at the higher flow depths. The lack of a calibration point above the depth of 6 inches precludes a similar conclusion for the Shepton Street segment. Both curves 1 and 2 in Figure 30 represent extrapolation for depths beyond six inches. In that range, curve number 1 is likely to yield more precise flow estimates then curve number 2. The higher correlation coefficient for curve 1 does not warrent this conclusion but manipulation of the least squares analysis for Port Norfolk provides some evidence to support this assertion.

Three low to mid range depth of flow calibration points for Port Norfolk were used in a least squares fit of Manning's equation, using both variable (n_f = 0.015) and constant roughness coefficients (n = n_f = 0.015). The resulting slopes were then used to estimate the flow rate at the highest available calibration point, i.e., h = 10 inches. The calibration curve using constant n underestimated the flow rate by 18% whereas the variable n curve underestimated the same flow rate by about 7%. The correlation coefficients computed for the three calibration points are similar, 0.87 for n variable and 0.85 for n constant. When the fourth calibration point from the Port Norfolk Street data set, at h = 10 inches, was introduced in the regression, the correlation coefficients in the above table changed to 0.91 and 0.57, respectively. If similar behavior can be assumed for Shepton Street, introduction of a higher calibration point would imply a slight increase in the correlation coefficient for the variable n case, and a sharp drop on the correlation coefficient for the fixed n case. Therefore, for Shepton Street the extrapolation beyond six inches should be more precise using the stage-discharge curve incorporating the variable roughness formulation.

Curves 3 and 4 in Figures 29 and 30 were computed simply using pipe slopes and fixed n values of 0.015 and 0.013 in Manning's equation. A comparison of these curves with curve 1 in both figures illustrates the importance of calibrating Manning's equation in any flow measurement analysis in sewerage systems. The larger differences noted in Figure 29, for Port Norfolk Street, are typical of the results for the Templeton and Walnut Streets test segments.

The first phase flush volumes were computed for all four streets using least squares best-fit slope and the variable n roughness formulation. A comparison between the computed and input volumes is presented for the four streets in Figure 31. Comparison of Figures 28 and 31 illustrates the improvements achieved by using a calibrated slope in Manning's equation. In Figure 28 practically all plotted points fall to the right of the 45° line, indicating misestimation of the flush volumes, while in Figure 31 the plotted points are distributed in a relatively narrow band around the 45° line.

Remaining differences between predicted and measured flush volumes are attributed in part to the assumption of steady uniform flow which underlies the applications of Manning's equation. If the unsteady non-uniform nature of the flow is accounted for, the accuracy of the predicted flush volumes and the flow rates throughout the flushes will improve. The non-steady state stage-discharge result will more accurately estimate the flushed pollutant masses. A numerical procedure accounting for the flush wave unsteadiness was developed in this study and is described in the following section. It should be noted that the two alternative procedures described thus far were only used in the early stage of work, when the first phase flush results were available. The last formulation described in the next section was used to compute flush volumes for all three flushing phases.

7.4.4 Discharge Estimates: Loop-Rating Curve

The relationship between stage and discharge at a cross-section of a free surface channel flow is unique only if the flow is uniform. During the progress of a flood wave, energy slope terms neglected in Manning's equation, other than the bed slope, S_0, cease to be negligible as compared to S and the discharge is no longer a function of depth alone. For a given depth of flow, h, the discharge will be greater on the rising stage of the wave than on the falling stage, so that the stage-discharge curve will form a closed loop which is characteristic of conditions under which the wave was generated. Such a loop-rating curve is illustrated in Figure 32.

The solution of this unsteady non-uniform flow problem would be extremely complex under the conditions present in the flush experiments of this study using hydraulic theories of flow in open channels. In light of the many uncertainties regarding the conditions of the flush, too many simplifying assumptions would be necessary, casting serious doubt on the reliability of an approach based on pure non-steady state hydraulic considerations. Unknowns such as the behavior of the net a short distance from the nozzle, storage effects of the laterals and disturbances caused by flow contributions along the segment, would preclude an accurate hydraulic characterization. A simple numerical optimizing approach was used in which an arbitrary but reasonable loop-rating curve, developed around Manning's equation, was defined for each flush as to minimize the differences between the estimated and the measured flush volumes.

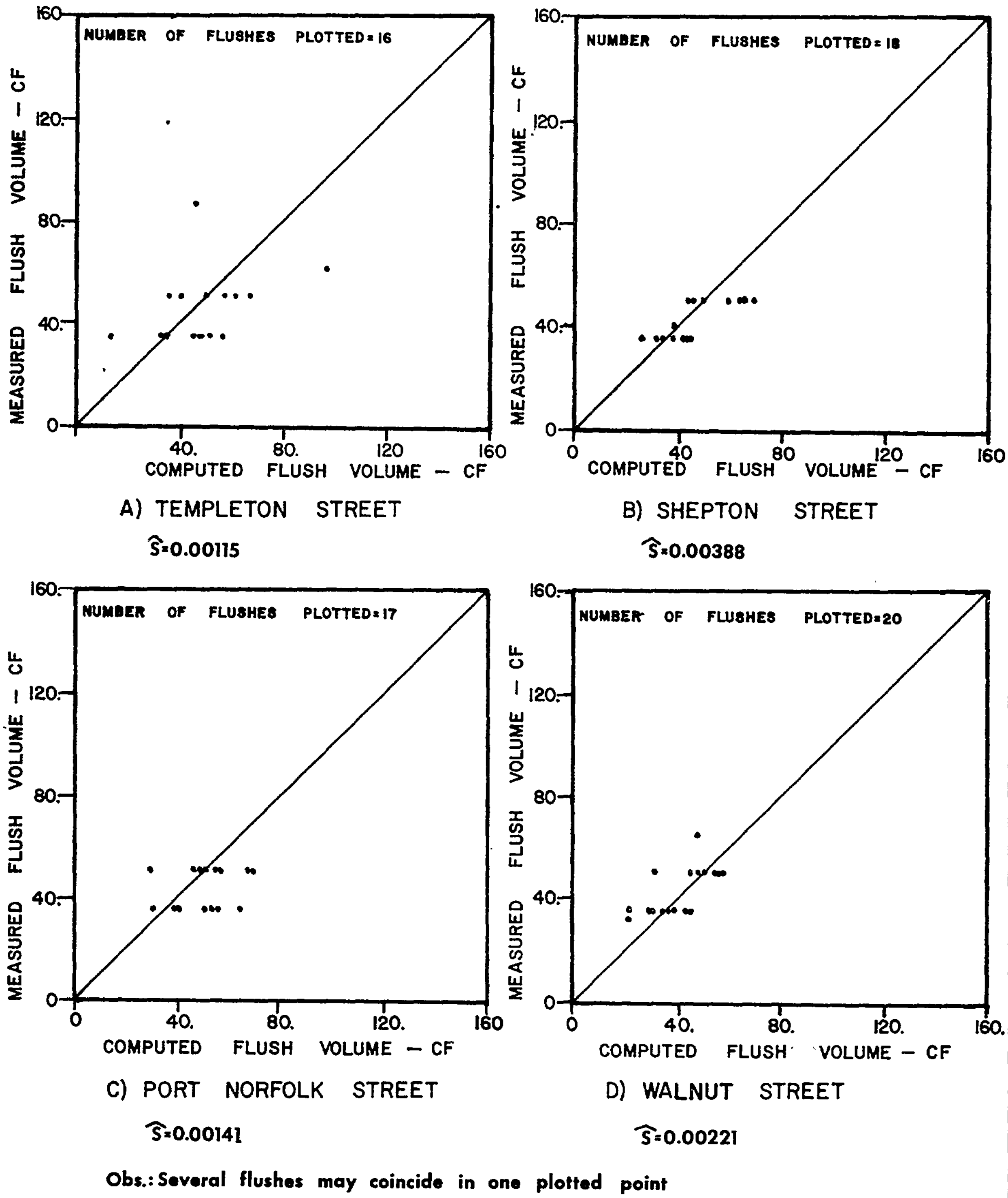


FIGURE 31 PLOTS OF MEASURED VERSUS COMPUTED FLUSH VOLUMES – MANNING'S EQUATION (LEAST SQUARES SLOPE) – FIRST PHASE

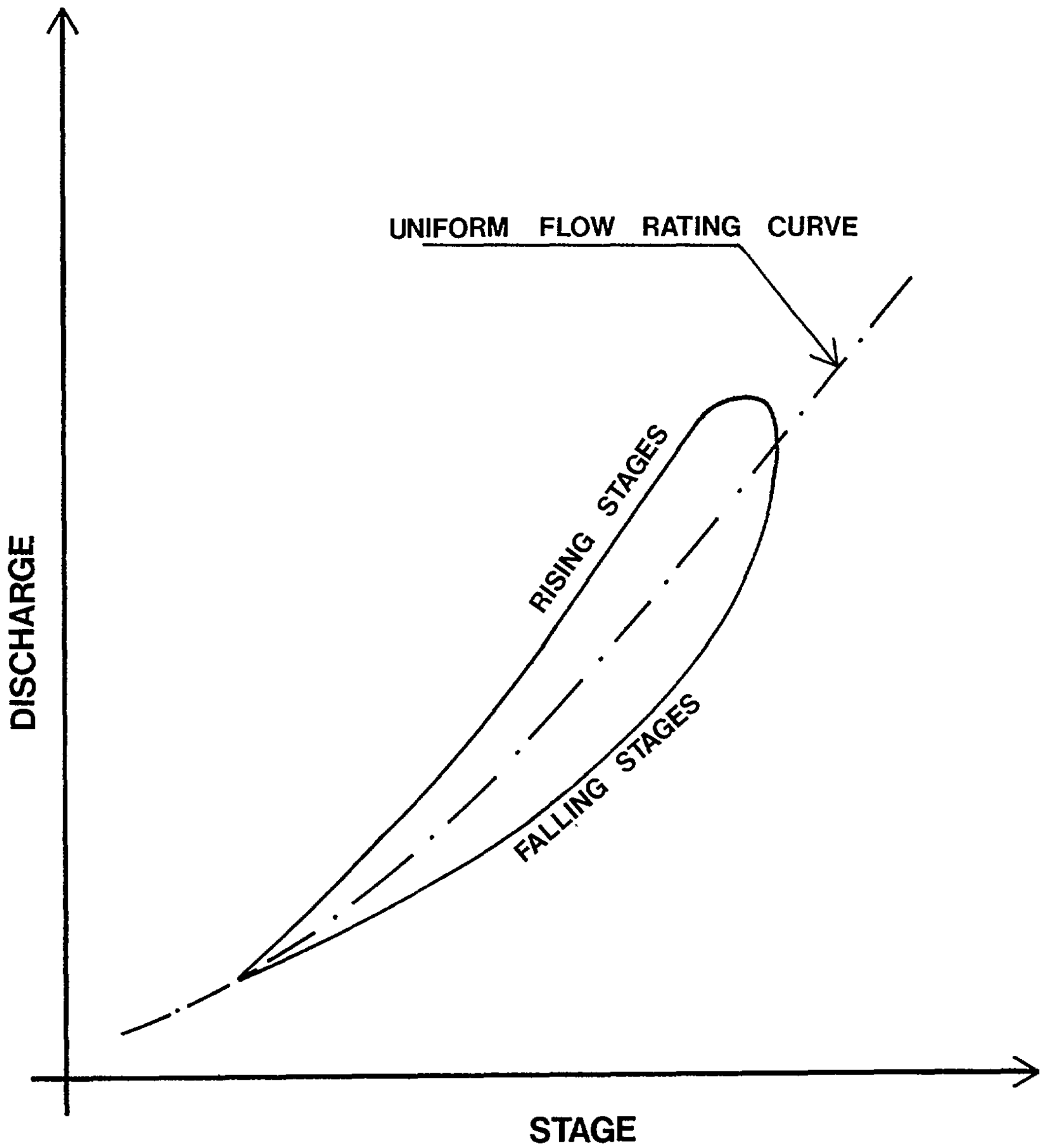


FIGURE 32 TYPICAL LOOP-RATING CURVE, SHOWING THE PROGRESS OF A FLOOD WAVE

7.4.4.1 Overview of the General Methodology. The methodology applied in the computation of the flow rates and ultimately the masses transported by all flushes performed in this study consisted of the following steps:

1. Define for each pipe segment and for each phase the coefficients of a complex stage-discharge function that minimizes the sum of the squares of the differences between the estimated flush volumes and their corresponding measured input volumes. These computations consider all flushing experiments conducted for a particular pipe segment. A mathematical programming optimization package was used to determine the coefficients of the stage-discharge function. Figure 33 outlines the approach.

2. The complex stage-discharge function containing the set of optimized coefficients defined in Setp 1 was used to compute the flow rates, mass rates, and ultimately the total flushed masses removed by each flush at a particular site and phase.

7.4.4.2 General Overview Details of Optimization Model. The overall optimization approach to find the coefficients of the stage-discharge function will be first described, leaving the details of the stage-discharge function to be described in section 7.4.4.4. Assume for the moment a general function $F_{i,j}$ relating stage to discharge, and define:

B = set of flushes performed at one pipe segment in one flush phase;

Vm_i = measured input volume of flush i, $i \supset B$;

$h_{i,j}$ = j^{th} flow depth reading during flush i (j=1, 18, $\forall$ flushes);

$Q_{i,j}$ = flow rate corresponding to depth $h_{i,j}$, given by

$$F(h_{i,1}, h_{i,j}, h_{max_{i,j}}, (Vm_i, \forall_i)) = F_{i,j} = \text{complex function}$$

relating stage to discharge for each flush and defined by a set of parameters;

$h_{i,1}$ = initial flow depth of flush i;

$h_{max_{i,j}}$ = maximum flow depth of flush i;

$$Ve_i = \sum_{j=1}^{18} (Q_{i,j} + Q_{i,j+1})/2 \bullet \Delta t_j, \text{ estimated flush volume i;}$$

Δt_j = time interval between depth readings $h_{i,j}$ and $h_{i,j+1}$ (j = 1, 18, $\forall$ flushes)

The objective function to be optimized is given by

$$\text{minimize } Z = \sum_{\text{All } i} (Vm_i - Ve_i)^2, \quad \forall_i \supset B$$

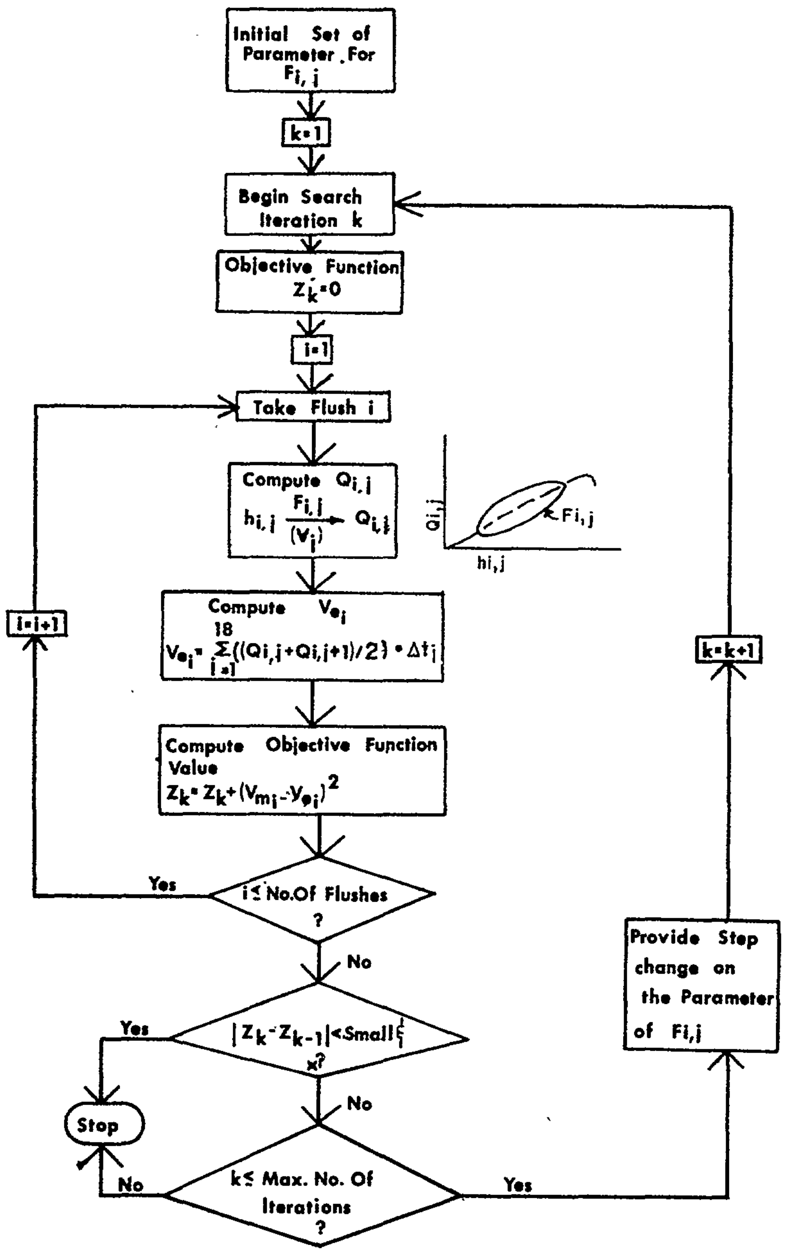


FIGURE 33 — SIMPLIFIED FLOW CHART OF THE OPTIMIZATION PROCESS

The minimization of equation 8 was performed using a direct search algorithm devised by Hooke and Jeeves (23) to provide step changes on the parameters of the function $F_{i,j}$ until the optimum is found. Figure 33 is a simplified flow chart of the optimization process. A set of parameter values is initially assumed for $F_{i,j}$. In the context of the optimization, these parameters define a point in the E^{η} space of the search, η being the number of parameters to optimize. The flushes of a given phase and at a particular pipe segment are taken one by one. Considering the first flush, its measured flow depths are converted into flow rates through $F_{i,j}$ and its initial parameters, yielding a discharge hydrograph for the flush. This hydrograph is then integrated by a discrete approach, as given by

$$Ve_i = \sum_{j=1}^{18} (Q_{i,j} + Q_{i,j+1})/2 \cdot \Delta t_j$$

The difference between the estimated flush volume Ve_i and the known measured value Ve_i is computed, raised to the second power and stored. All successive flushes are treated likewise, while accumulating the values of

$$(Vm_i - Ve_i)^2 \ , \ \forall i.$$

At the last flush of the data set, the value of the objective function is known.

The pattern search routine then initializes a local exploration about the initial point in E^{η} before a step is made. The parameters of $F_{i,j}$ are taken one at a time as the search variable. The first search variable (parameter) is given small positive and negative perturbations of a given size, tailored to the particular variable, and the objective function is evaluated as described before at those perturbation points. If improvement (reduction) with respect to the base point is found at any of the perturbation points, a temporary base will be moved to the lower perturbation point, while saving the initial one. Another parameter of $F_{i,j}$ is then taken as the search variable and the same process is repeated until all parameters have been searched. The first (base) and the last points of the local search are then connected by a straight line and a pattern move of a given step size is made along that line, defining a new base point around which local exploration will be resumed again.

When a point is reached where local exploration reveals no improvement, a minimum has been found or the exploration is on a ridge (inversed) in the surface of the objective function, at a point where the ridge is turning. The perturbation size is then reduced and the local search repeated. Reduction of the perturbation size is continued until: 1) The perturbation sizes are below a specified resolution of the parameters; 2) the difference between the values of the objective function at two successive bases is smaller than a prespecified target value ξ; or 3) a reduction of the objective function, greater than ξ, is obtained. In the first two cases an optimum has been found whereas in the third case, the search proceeds as before. The whole process is also limited by the prespecified maximum number of evaluations of the objective function. The optimization procedure assumes a unimodal objective function. Therefore, if the objective function is multimodal, a local rather than the global optimum may result.

The optimization package consists of a main program, that computes the objective function, and the search subroutine that commands the objective function surface exploration and provides the step changes on the parameter values.

7.4.4.3 Basic Concept of Looping Stage-Discharge Curve. The objective is to develop for each pipe segment flushed during each phase of work, a stage-discharge relationship that meets two requirements: 1) the flow is higher at the front of the wave than at the back of the wave for the same measured flow depth; and 2) the discharge hydrograph volumes should be such that the sum of the squares of the differences between estimated and measured input volumes is a minimum. The first requirement is important because the pollutant concentrations tend to peak in the early part of the flush wave, usually before the hydraulic peak. A uniform flow rating curve would therefore underestimate the masses transported in the front part of the wave and most likely the total transported masses. The second requirement insures that the resultant stage-discharge relationship closely estimate both flow rates and flush volumes. These two requirements were combined in a mathematical optimization problem that defines the stage-discharge function of each individual flush while meeting both requirements.

The conceptual details of the loop-rating curve are illustrated in Figure 34. The procedure consists of determining at each value of the flow depth h, indicated in the figure, a correction, $\pm \Delta h$, which is added to h before the uniform flow rating curve is used to compute the flow rate at flow depth h. In the rising limb of the wave, i.e., for all depths measured prior to the occurrance of the peak depth, the corrections Δh are all positive. At the maximum depth the correction Δh is zero, and for all depths following the maximum depth, i.e., in the falling limb of the wave, the correction Δh are all negative.

For example, suppose the indicated flow depth h was measured in the front of the wave. The uniform flow rating curve would indicate for h, a flow corresponding to point A in the figure. By computing at h a correction term, $+ \Delta h$, which is added to h, the same uniform flow rating curve will now indicate at $h + \Delta h$ a higher flow, corresponding to point B, which is then associated to h. This is equivalent to knowing point C on the loop-rating curve. Suppose now that h is a depth measured in the tail of the wave. If the correction term, $- \Delta h$, is computed at h, and added to its value, the uniform flow rating curve will indicate at $h - \Delta h$ the flow corresponding to point D. Again, it is as if point E, corresponding to h on the loop-rating curve, were known. By this simple artifice a virtual loop-rating curve is constructed for the particular flush. The unique feature of this approach is that the loop-rating curve is a function of the initial background flow depth, the maximum flow depth occurring during the flush and the sediment depth within the pipe segment. Although the uniform flow rating

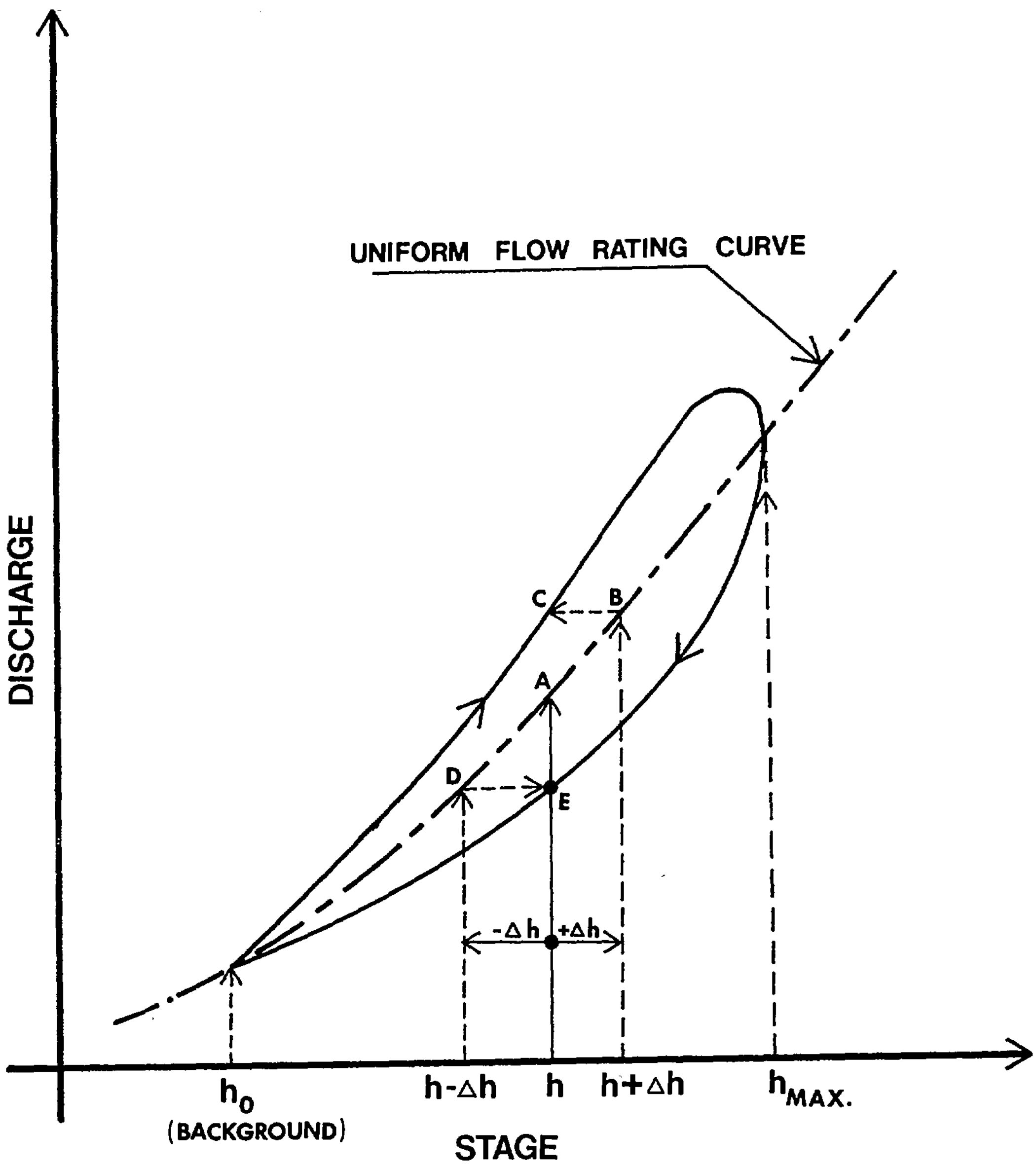


FIGURE 34 DERIVATION OF THE LOOP-RATING CURVE

curve remains the same for all flushes at a particular site, the resulting loop-rating curve represents a specific stage-discharge function for each flush.

7.4.4.4. Preliminary Looping Stage/Discharge Formulations

Several alternative mathematical expressions were initially investigated to represent the stage discharge function, $F_{i,j}$, with respect to the uniform flow rating curve and the depth correction, Δh. Partial summary results are presented here primarily to document the iterative "trial and error" development of the looping stage-discharge estimation model. Findings will be presented in terms of the optimized values of the objective function given by equation 8.

The optimization procedure described in 7.4.4.2 was originally devised to define for the uniform flow rating curve a polynomial function which, having more parameters than Manning's equation, would provide a better adjustment of the computed and measured volumes. Representation of the uniform rating curve by Manning's equation was also considered in this comparative analysis. Polynomials of the third degree were used. Both the complete polynomial given by

$$Q = ah^3 + bh^2 + ch + d, \tag{10}$$

where Q = flow, in cfs,
h = flow depth, in inches; and
a,b,c,d = coefficients to be determined by optimization.

and several incomplete polynomial forms, especially forms having one root equal to zero, were also considered. These incomplete polynomial forms are:

$$Q = ah^3 + bh^2 \tag{11}$$

and

$$Q = ah^3 + ch \tag{12}$$

which, for $a < 0$, have a concave shape much similar to Manning's equation in the first quadrant.

Associated with the polynomial expression for the uniform flow rating curve, a number of alternative functions were used for the correction term, Δh, which produces the hysterisis looping effect of the discharge curve. These functions added one or more parameters to the pattern search optimization. The various depth correction functions used in this analysis are as follows:

$$\Delta h_i = \pm\, e \left.\frac{dQ}{dh}\right|_{h=h_i} \tag{13}$$

$$\Delta h_i = \pm\, e \,(h_{max} - h_i) \left.\frac{dQ}{dh}\right|_{h=h_i} \tag{14}$$

$$\Delta h_i = \pm \frac{e\, f^2\, (h_i - h_{max})}{f^2 + (h_i - h_{max})^2}\ ; \tag{15}$$

$$\Delta h_i = \pm \frac{ef^2\, (h_i - h_{max})(h_i - h_o)}{[f^2 + (h_i - h_{max})^2](h_i - h_o)}\ ; \tag{16}$$

$$\Delta h_i = \pm\, [eh_i^2 - e(h_o + h_{max})\, h_i + eh_o\, h_{max}] \tag{17}$$

where all variables have been defined before, except for:

h_o = background flow depth, in inches;
h_{max} = maximum depth during the flush, in inches; and
e,f = coefficient to be optimized.

The optimization procedure was initiated using best-estimate values for the various parameters in the formulation. Initial values of the coefficients, a, b, c and d were determined by computing the coefficients of a polynomial about four stage-discharge points derived from the steady-state discharge curves described in section 7.4.2. The polynomial fitting was performed by a subroutine added to the optimization package. The particular form of the desired polynomial was specified in the input data. The initial values of the coefficients e and f were arbitrarily fixed. Especially with the complete polynomial, certain conditions with respect to the position of points of stationarity (dQ/dh = 0) had to be imposed to avoid degenerated solutions. Table 7 presents partial results obtained using the steady-state discharge curves described in section 7.4.1 and 7.4.2, various polynomial optimizations and finally, optimization using Manning's equation with Δh corrections given by equation 17. Objective function values shown under column 3 were computed using a complete polynomial and h corrections as given by equation 7.13, except for Walnut, where correction equation 14 was used. The value shown under column 4 for the second phase Port Norfolk flushes, was computed using the incomplete polynomial given by equation 11, and depth corrections, Δh, given by equation 16. The polynomial optimization results shown under column 5 were also computed using equation 11 for the uniform flow rating curve, but equation 17 was used for the correction term, Δh.

Column 6 of Table 7 presents results obtained with Manning's equation and the correction term Δh as given by equation 17. It should be noted that only two parameters, that is, the pipe slope and the coefficient e in equation 17 were optimized in this case. The advantage of Manning's equation, is that the number of search variables is reduced to the pipe slope (roughness n assumed known) and the coefficient or coefficients of the correction term, Δh, reducing the number of required iterations and consequently the computational time.

TABLE 7: COMPARATIVE OBJECTIVE FUNCTIONAL VALUES*
FOR ALTERNATIVE FLOW COMPUTATIONAL APPROACHES

PHASE	STREET	(1)	(2)	(3)	(4)	(5)	(6)
First	Templeton	34042.	6080.	-	-	-	7034.
First	Shepton	1884.	1226.	631.	-	-	691.
First	P. Norfolk	62649.	3177.	-	-	-	3604.
First	Walnut	57625.	2671.	2561.	-	-	-
Second	P. Norfolk-1DMH	-	-	994.	-	-	1398.
Second	P. Norfolk-2DMH	-	-	-	2883.	588.	522.
Second	P. Norfolk-3DMH	-	-	2027.	-	8.	1632.

*Objective Function:

$$\sum_{\text{all flushes}} (Vm_i - Ve_i)^2$$

(1) Results using stage-discharge curves derived from application of Manning's equation with pipe slope and fixed roughness coefficient - see section 7.4.1.

(2) Results using stage-discharge curves derived from least squares fit of observed field data to Manning's equation - see section 7.4.2.

(3) Polynominal optimization with Δh given by equation 13 except for Walnut Street where equation 14 applies.

(4) Polynominal optimization with Δh given by equation 16.

(5) Polynominal optimization with Δh given by equation 17.

(6) Manning's equation with Δh given by equation 17.

Obs.: Several other results were obtained from different combinations of polynomials and h corrections, but they included partial sets of flushes and were therefore omitted here.

It should be noted that the optimization procedure described in 7.4.4.2 is not guaranteed to yield the global optimum. If the objective function is not unimodal, as seems to be the case here, a local optimum can be obtained. This means that the numbers shown could be only local, rather than global optima. Nonetheless, they reveal substantial improvements with respect to the pipe slope and best-fit slope results. The total number of objective functional evaluation in the polynomial optimizations presented in Table 7 varied from 271 evaluations in the case of Shepton, column 3, to 889 evaluations in the case of Port Norfolk, column 5. Some of the preliminary optimization computer runs and all runs described in the following sections utilized a simple preoptimization procedure that led the initial point very close to the optimum.

7.4.4.5. <u>Preoptimization</u>

Attempts were made at starting the optimization procedure from a good initial point in order to save iteration steps. The uniform flow rating curve has the greatest influence in the value of the objective function. Starting with a good guess on the coefficients of that curve, considerable reduction of iteration steps were realized. As mentioned before, when a third degree polynomial was used, the initial coefficients of the polynomial were obtained by fitting the polynomial through h-Q points computed by Manning's equation with the best-fit slope defined for each segment. The procedure will be described only for the case when the uniform flow rating curve is expressed by Manning's equation. For the polynomial case the extension is immediate. Given the initial input values for the slope, the coefficient e of equation 17, and sediment depths, the flow rates and flush volumes are computed by the loop-rating curve approach for all flushes, yielding the estimated volumes, Ve_i, $\forall i$. A simple linear regression of the type:

$$Vm_i = \alpha Ve_i \tag{18}$$

is then performed on the values of Vm_i and Ve_i, described before where α is the slope of the regression line which can be obtained by:

$$\alpha = \frac{\sum_i Vm_i * Ve_i}{\sum Ve_i^2} \tag{19}$$

Clearly, α equal to unity corresponds to the best starting point. Disregarding for a moment the loop-rating curve and focusing on the uniform flow curve only, the values of Ve_i can be expressed by:

$$Ve_i = \sum_{j=1}^{18} \frac{1.49\, r_{i,j}^{2/3}\, S^{1/2}\, a_{i,j}\, \Delta t_j}{n_{i,j}} \tag{20}$$

where the j index is over the sampling times for flush i.

Combining equations 18 and 20 results in the following expression:

$$Vm_i = \alpha\, S^{1/2} \sum_j \frac{1.49\, r_{i,j}^{2/3}\, a_{i,j}\, \Delta t_j}{n_{i,j}} \tag{21}$$

By setting

$$S^* = \alpha^2\, S \tag{22}$$

the value S* will be a better initial value for the slope. Nevertheless, if S* is used instead of S and the same computation is repeated the value of the new α will not be exactly 1. This situation results because in the loop-rating curve computation the volumes, Ve_i, are not a linear function of the slope as assumed in equation 20. By repeating the adjustment given by equation 22 several times, α can approximate unity. In this procedure a maximum number of four successive revisions were performed in the preoptimization routine before the pattern search routine would be implemented.

7.4.4.6. Final Form of the Stage-Discharge Function $F_{i,j}$

Although the results obtained with the polynomial and Manning's equation optimization represented a substantial improvement over the uniform flow approaches given by the pipe slopes and the best-fit slope, large differences still remained between the estimated and measured volumes for several flushes, for which there was no clear explanation. Plots of flow depths for all flushes at all sites revealed cases where the measured depths for flushes of equal input volume and discharge rate were considerably different. This could be explained by different sediment depths. Although measurement of sediment depths were available for each flush, there is a concern as to whether or not the measurements at the ends of the pipe are representative of the average sediment depth along the entire segment and throughout the flush. Whatever the true causes of these discrepancies in flow depth might have been, it was felt that some improvement would be gained by introducing the average sediment depth in the pipe, for each flush, as a variable magnitude to be optimized by the procedure.

The final stage-discharge function, $F_{i,j}$, entailed optimizing:

1) Manning's n, which is applicable for all flushes at one particular pipe segment and in a given phase;

2) The coefficient, e, of the correction factor, Δh, equation 17, which although applicable to all flushes at one site and phase, implies different correction values at different flushes; and

3) The average sediment depth in the pipe segment at the time of the flush. The measured sediment depths were used as the starting point for the optimization.

The optimized sediment depths represent a manageable way of lumping all the unknown and complex effects that the sediments have on the flow regime. Comparative results using Manning's equation from column 6,

Table 7 and the final results obtained by this procedure are as follows:

RESULTANT OBJECTIVE FUNCTIONAL VALUES

PHASE	STREET	A	B
First	Templeton	7034	1085
First	Shepton	691	353
First	Port Norfolk	3604	108
First	Walnut	-	21
Second	Port Norfolk 1-DMH	1398	606
Second	Port Norfolk 2-DMH	522	23
Second	Port Norfolk 3-DMH	1632	1008

A = Manning's equation with Δh correction given by equation 17.
B = Manning's equation with Δh correction given by equation 17 and optimized sediment depth.

The final values of the objective function reveal a considerable improvement in all cases. It is believed that the flow rates generated by $F_{i,j}$ are very close to their actual values. Figures 35 through 38 illustrate typical rating curves obtained by the methodology.

Plots of estimated versus measured flush volumes, similar to those described in sections 7.4.1 and 7.4.2, were also prepared for the first and second phase values computed by this methodology. The first phase results are presented in Figure 39 whereas the second phase values are presented in Figure 40. A summary overview of test segment, the plan and profile pipe slopes, the measured pipe slopes, the least squares fitted slopes and the final optimized slopes resulting from the pattern search procedure are shown in Table 8. The actual pipe slopes were determined using surveying equipment.

TABLE 8: OVERVIEW OF ESTIMATED SEGMENT PIPE SLOPES

PHASE	STREET	PLAN&PROFILE PIPE SLOPE	MEASURED PIPE SLOPE	BEST-FIT SLOPE	FINAL SLOPE
First	Templeton	0.00322	0.0029	0.00115	0.00191
First	Shepton	0.00345	0.0026	0.00388	0.00296
First	Port Norfolk	0.00486	0.0055	0.00141	0.00185
First	Walnut	0.00481	0.0010	0.00221	0.00134
Second	Port Norfolk-1DMH	0.00701	0.0059	-	0.00221
Second	Port Norfolk-2DMH	0.00141	0.0055	0.00141	0.00246
Second	Port Norfolk-3DMH	0.00400	0.0046	-	0.00242

The third phase automated flushes performed in Shepton Street were not subjected to the optimization of this section. All of these flushes were performed by back-up and quick release of stored flush waters. The parameters of the stage discharge function, $F_{i,j}$, determined from the first phase computations, were considered applicable to compute the third phase flush rates and volumes.

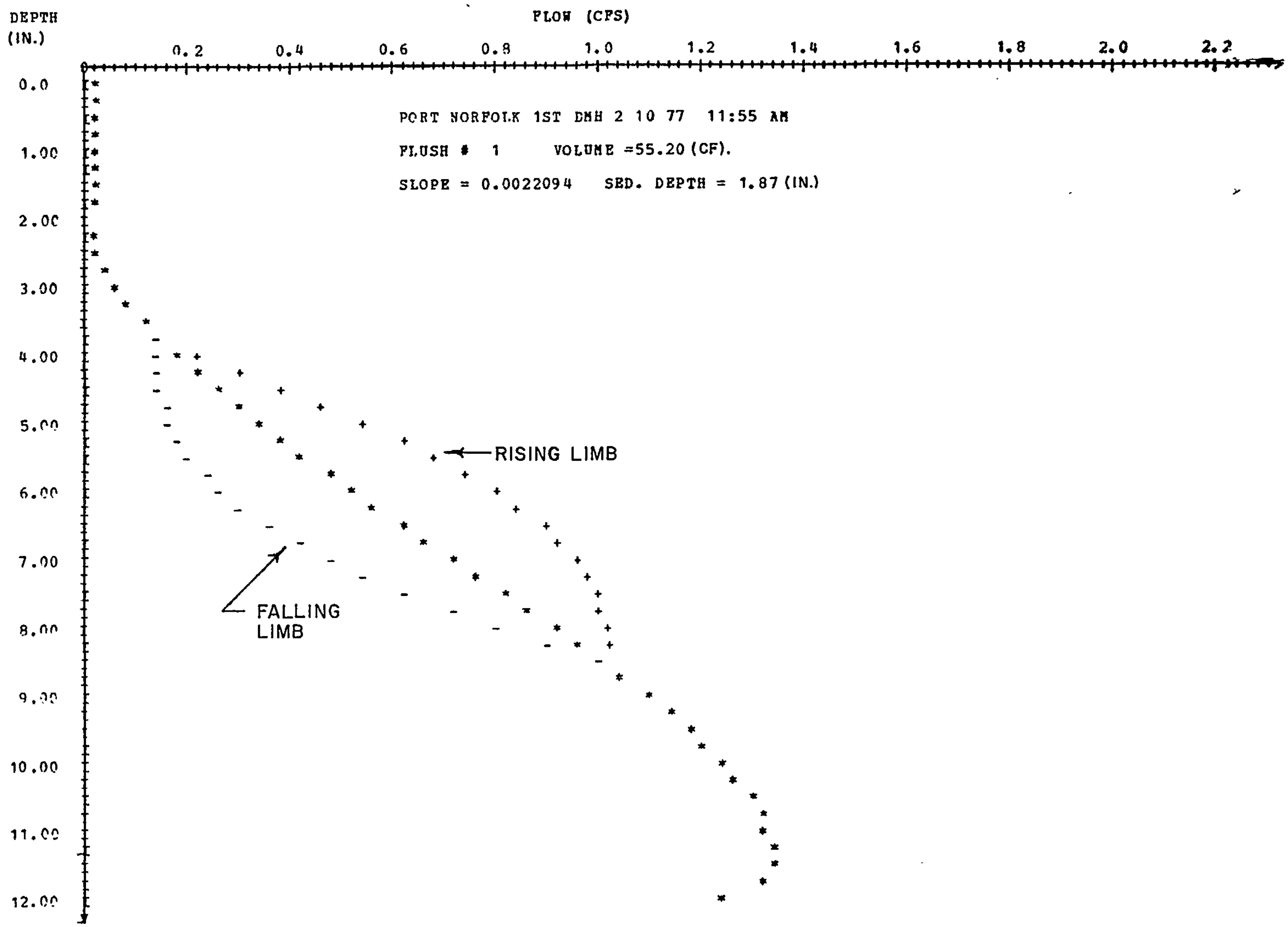


FIGURE 35 TYPICAL LOOP RATING CURVE PORT NORFOLK STREET, 2/10/77

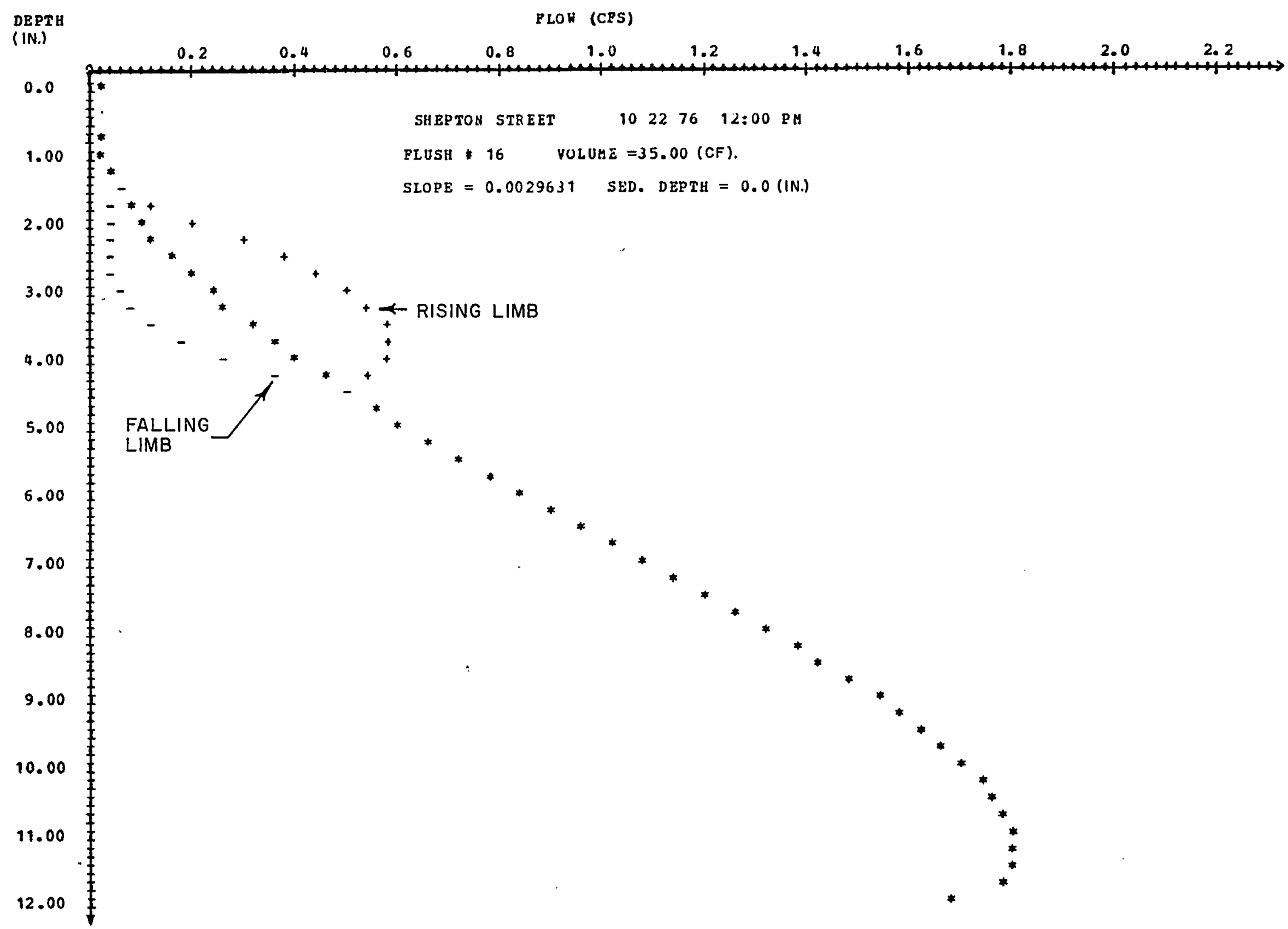


FIGURE 36 TYPICAL LOOP RATING CURVE SHEPTON STREET, 10/22/76

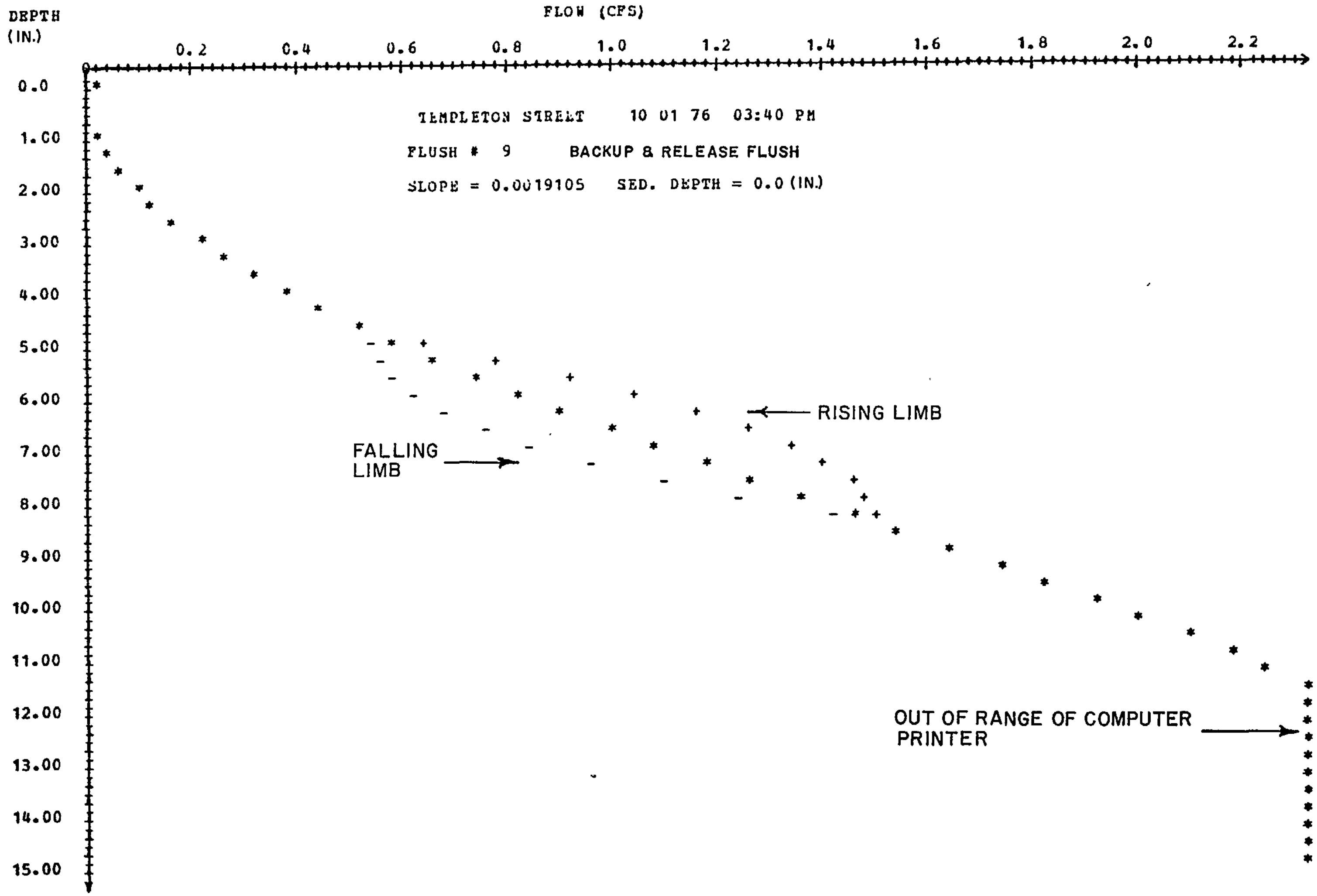


FIGURE 37 TYPICAL LOOP RATING CURVE TEMPLETON STREET, 10/01/76

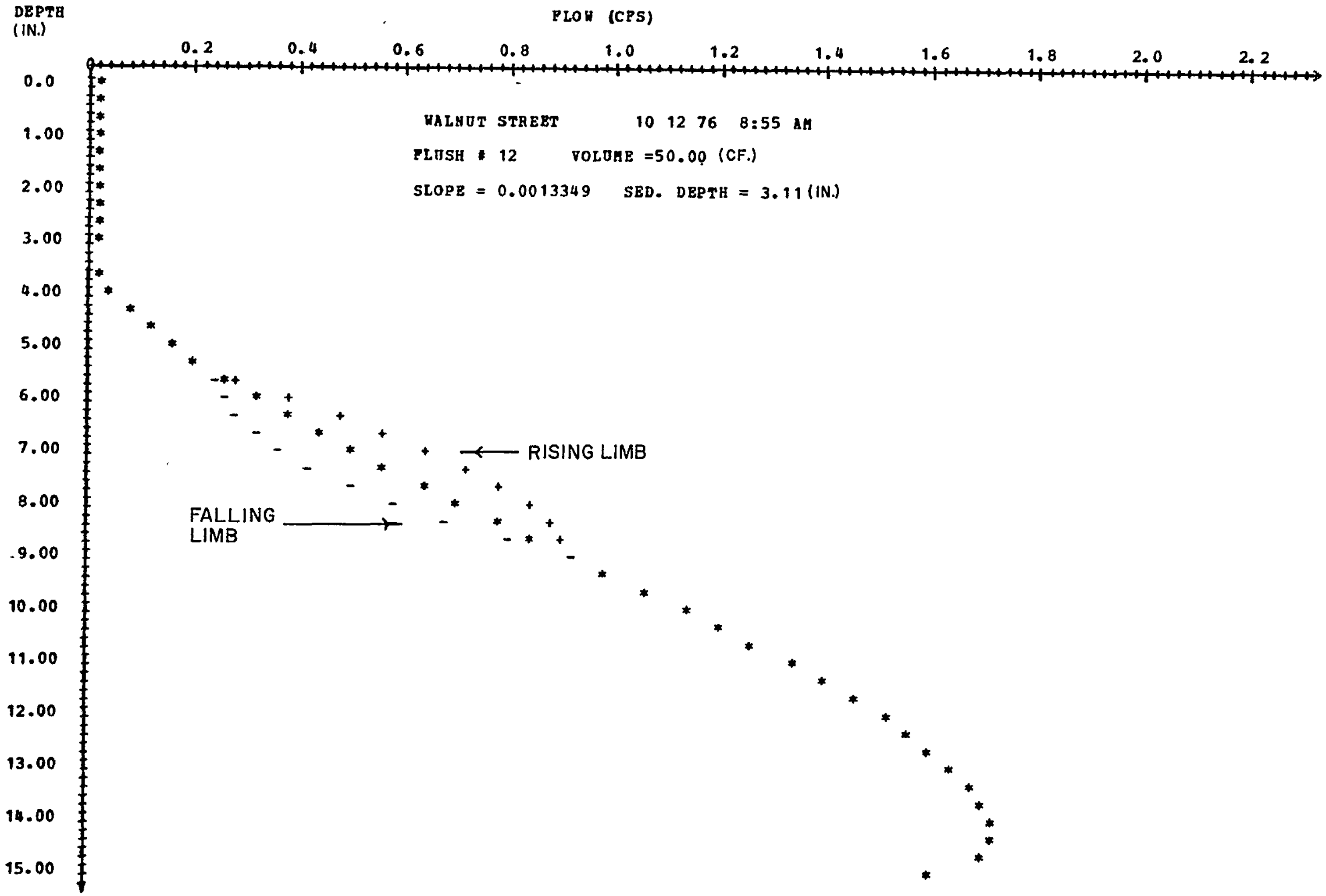
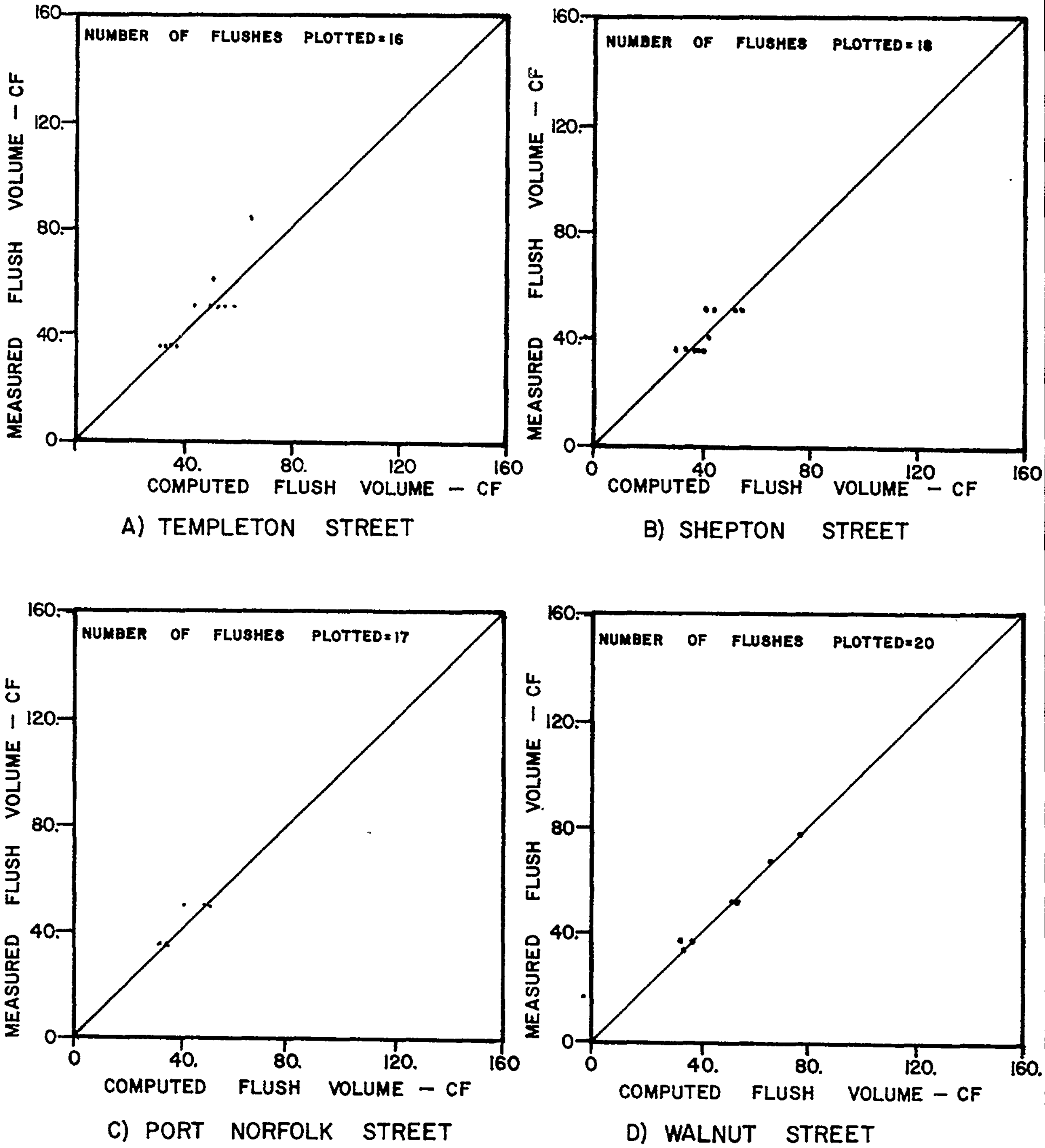
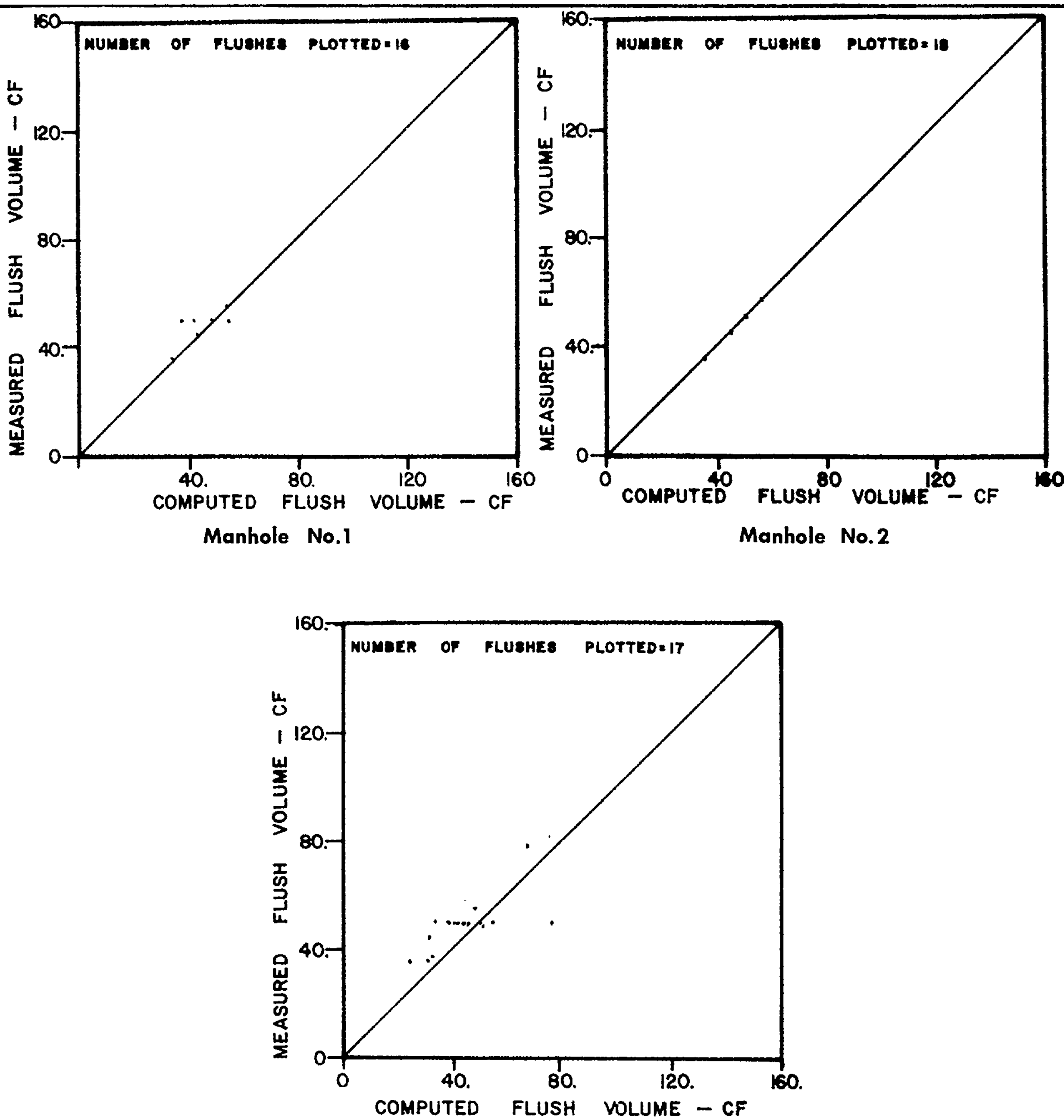


FIGURE 38 TYPICAL LOOP RATING CURVE WALNUT STREET, 10/12/76



OBS: SEVERAL FLUSHES MAY COINCIDE IN ONE PLOTTED POINT.

FIGURE 39 PLOTS OF MEASURED VERSUS COMPUTED FLUSH VOLUMES - SEARCH OPTIMIZATION (FIRST PHASE)



Manhole No.1 Manhole No.2 Manhole No.3

OBS: SEVERAL FLUSHES MAY COINCIDE IN ONE PLOTTED POINT.

Port Norfolk St.

FIGURE 40 PLOTS OF MEASURED VERSUS COMPUTED FLUSH VOLUMES - SEARCH OPTIMIZATION - SECOND PHASE

7.4.4.7 Comparison of Computer versus Measure Flush Wave Rates

An independent verification of the methodology was provided by the dye injection experiments described in Section 5.7. Using the flush volumes measured for 10 experiments and their corresponding measured flow depths, this methodology was applied to optimize the discharge function, $F_{i,j}$. The flow rates given by $F_{i,j}$ compared reasonably well with the flow rates measured by the dye injection. A comparison of measured flush wave hydraulic characteristics using dye injection methods versus the optimized results are presented in Table 9 for four of the experiments. The volume measured by the meter on the flush truck is given in the top row of Table 9. Next, the volumes computed by integrating the discharge curves generated by the dye injection measurement procedure and the numerical optimization methodology are given. Finally, the peak flush wave rates derived from both procedures are given.

TABLE 9. COMPARISON OF MEASURED VERSUS OPTIMIZED FLUSH WAVE HYDRAULIC CHARACTERISTICS, DYE INJECTION EXPERIMENTS

	EXPERIMENT			
	1	2	3	4
DELIVERED FLUSH VOLUME (cf)	35.	35.	35.	35.
MEASURED FLUSH VOLUME DYE INJECTION (cf)	37.5	34.9	38.0	36.0
OPTIMIZED FLUSH VOLUME (cf)	31.5	33.6	35.9	37.9
MEASURED PEAK FLUSH RATE DYE INJECTION (cfs)	.71	.68	.57	.50
ESTIMATED PEAK FLUSH RATE OPTIMIZATION (cfs)	.52	.52	.46	.38

Plots of measured flush flow rates versus estimated discharge rates using the optimization procedure for two of the field experiments are shown in Figure 41. The ad hoc dye injection field procedure developed for the project worked remarkably well in view of the extreme difficulties in accurately monitoring non-steady state flush wave discharge lasting about two minutes in duration. The optimization procedure reasonably reproduced the flush wave characteristics. The degree of adherence between the predicted and measured flush flow rates was not perfect, but reasonably close considering the complexities in numerically trying to reproduce the dynamics of flush waves. Application of the loop rating curve concept estimated flush wave characteristics that more closely resembled actual measured conditions than did the two prior steady state rating curve procedures described in Sections 7.4.2 and 7.4.3. The accuracy in estimating the flush wave hydraulic profiles and subsequently, the masses of pollutant flushes over the course of the project increased by an order of magnitude. A great deal of the project resources were expended in both the dye injection calibrations and in the "cut and try" development and application of the optimization flow

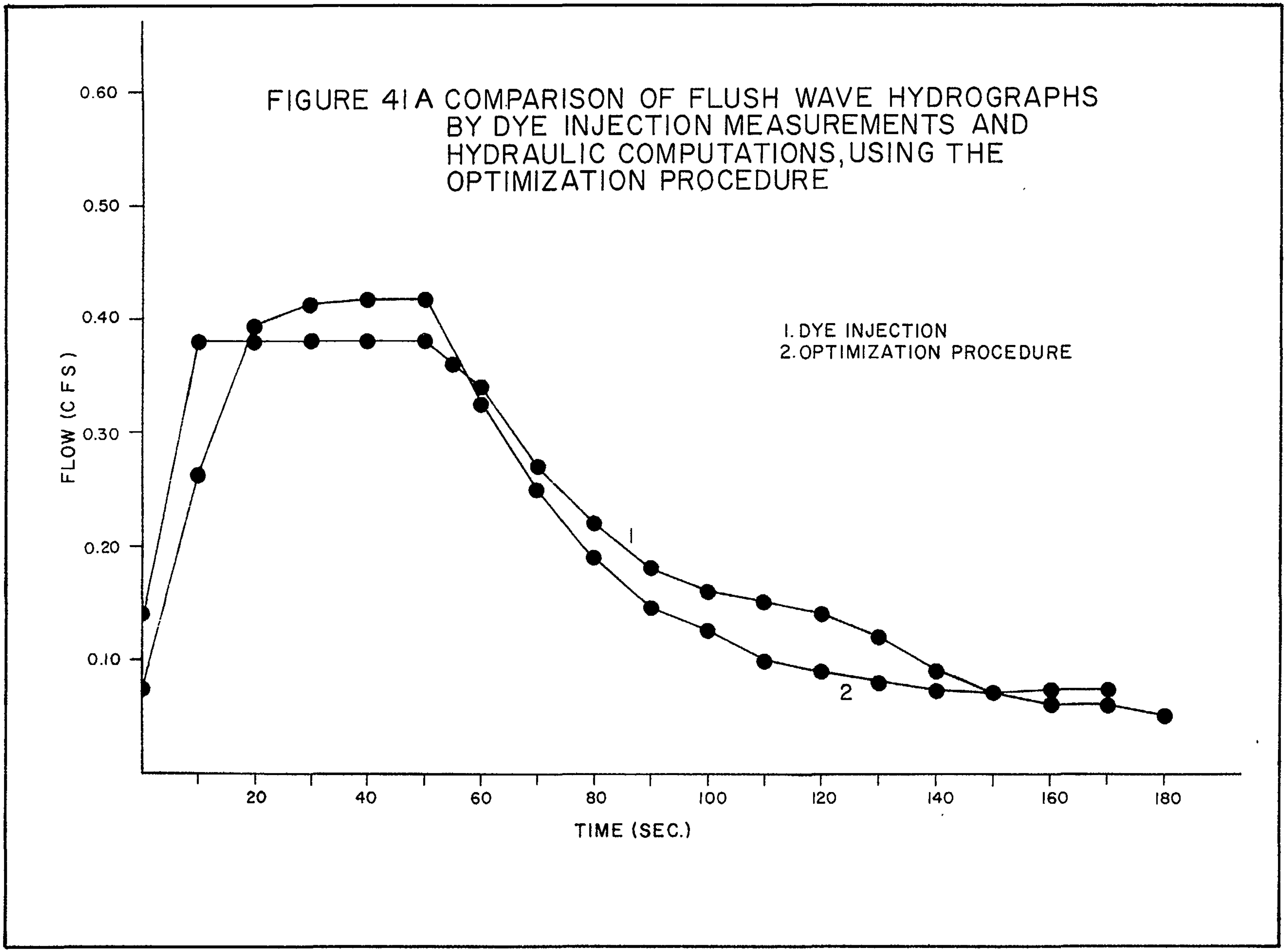
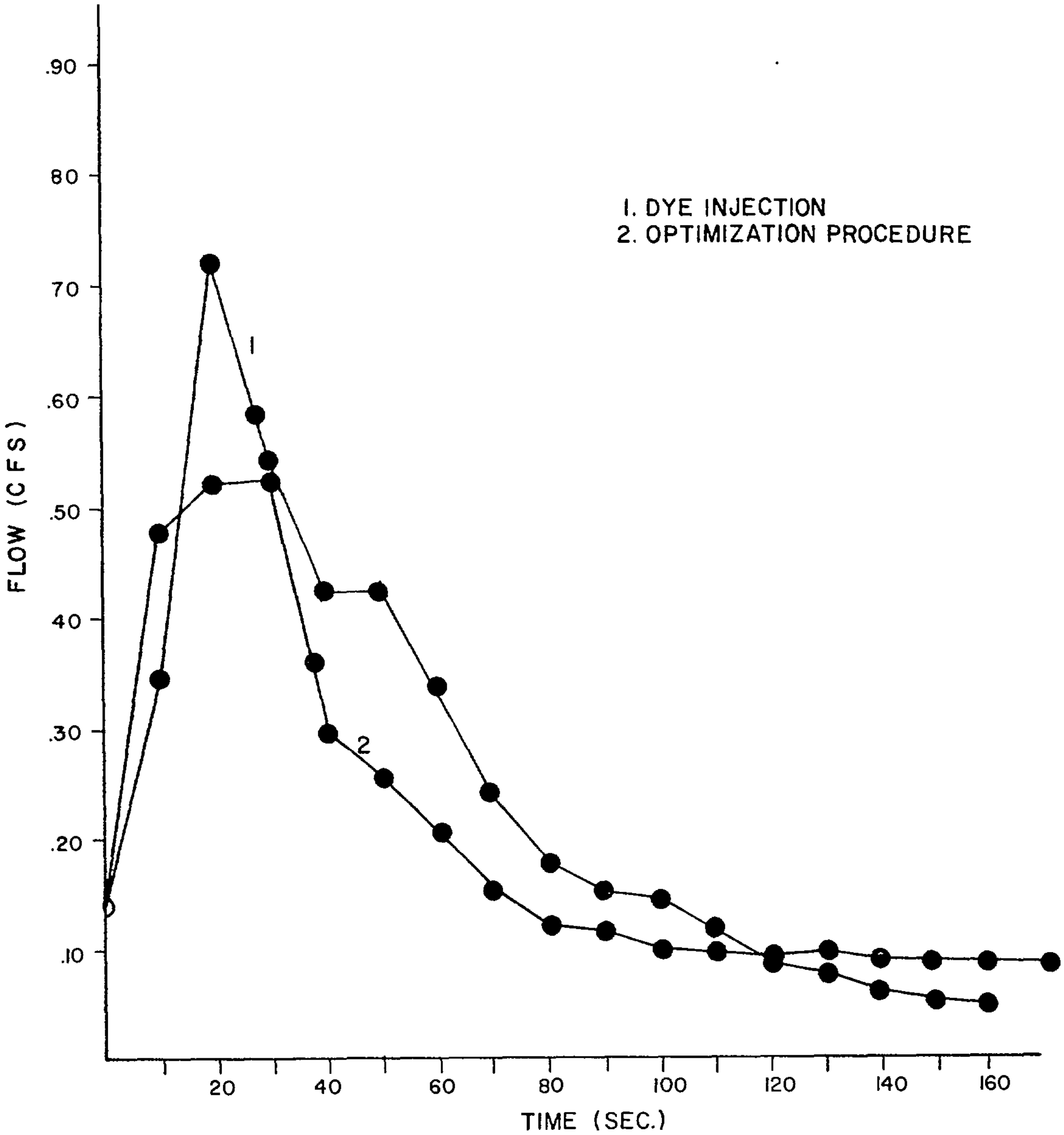


FIGURE 41 A COMPARISON OF FLUSH WAVE HYDROGRAPHS BY DYE INJECTION MEASUREMENTS AND HYDRAULIC COMPUTATIONS, USING THE OPTIMIZATION PROCEDURE

FIGURE 41B COMPARISON OF FLUSH WAVE HYDROGRAPHS BY DYE INJECTION MEASUREMENTS AND HYDRAULIC COMPUTATIONS, USING THE OPTIMIZATION PROCEDURE



estimation methodology. These efforts and expenditures were not forseen in the conceptual development of the study. Since the field flushing experiments conducted in this project constituted the first major data collection research effort of this type ever performed, these additional efforts to more accurately estimate flushed masses were deemed necessary.

7.5 Masses of Pollutants Removed by the Flush Wave

Utilizing the flow predictive methodology and equations described in section 7.4, the next step was to compute, from the analytical concentrations of each pollutant, at discrete time intervals, the estimated mass rates of the pollutants carried by the flush wave. A pollutograph of each pollutant analyzed was obtained for the duration of the sampled flush wave. A numerical integration of the pollutographs yielded the total estimated masses of the pollutants transported by the wave. A computer program was prepared to perform all the computations. The runs were done for flushes grouped by site or pipe segment, and by phase. These groups are shown below:

GROUPING OF FLUSHES FOR THE MASS COMPUTATION RUNS

GROUP NO.	PHASE	SITE	NO. OF FLUSHES
1	First	Templeton	21
2	First	Shepton	22
3	First	Port Norfolk	19
4	First	Walnut	21
5	Second	Port Norfolk-1DMH	18
6	Second	Port Norfolk-2DMH	18
7	Second	Port Norfolk-3DMH	15
8	Third	Shepton	7

The same mechanisms for computing flow rate from flow depth developed in the prior section were implemented in the program. By reading in for each group the appropriate adjusted parameters from the optimization the same flow rates and flush volumes associated with the final iteration of the optimization process were re-estimated and were used to compute the mass rates for each flush of the group. The data files described in section 7.2, provided all the information on flow depths, times of the sampling and pollutant concentrations necessary for the computation of the pollutant mass rates and total flushed loadings mass. The procedures for filling-in missing data, described in section 7.3, were also incorporated into this program.

The complete output from the program consists of four tables printed out for each flush of a group containing: a) analytical results, b) mass calculation, c) cumulative masses of pollutant carried by the flush wave, and d) cumulative percentile masses of pollutants carried by the flush wave. A sample of these tables is presented in this section as Tables 10, 11, 12 and 13 respectively. These tables are well labelled and are self-explanatory.

TABLE 10. ANALYTICAL RESULTS - WALNUT STREET TEST SEGMENT - 8/30/76

* * * * * STORM AND COMBINED SEWER RESEARCH: ANALYTICAL RESULTS * * * * *

STREET: WALNUT STREET

DATE: 08 30 76

FLUSH TECHNIQUE: GRAVITY FLUSH WITH 3 INCH NOZZLE AND HOSE

HOUR OF FLUSH: 12:35 PM

DYE USED: NO

PRESSURE IN TANK: -- PSIG

SIZE OF FLUSH: 50.0 CUBIC FEET

TIME TO COMPLETE FLUSH: 15.0 SECONDS

TIME FOR FLUSH TO REACH DOWNSTREAM: 48.0 SECONDS

SAMPLE NO.	TIME SEC.	DEPTH IN.	COD MG/L	BOD MG/L	TKN MG/L	NH3 MG/L	TOT. P MG/L	ORTH.P MG/L	SS MG/L	VSS MG/L
BCKGND	0	6.00	440	163	36	34	9.6	7.4	114	78
10	0	7.00	520	210	44	37	11.6	8.8	309	175
20	10	7.00							900	423
30	20	7.50	1640	10	72	38	17.6	13.0	2367	1155
40	30	8.00							3379	1625
50	40	8.50	3120	900	113	41	20.0	17.6	5178	2548
60	50	8.50								
70	60	8.50	4200	680	166	42	34.0	18.0	8114	4067
80	70	8.75							2692	1350
90	80	8.75	7800	380	155	44	23.8	17.4	8292	4300
100	100	8.75							5306	2857
110	120	8.00	2840	360	127	27	27.2	13.4	7184	3797
120	140	7.50							4771	2530
130	160	7.50	2840	320	67	14	16.0	7.6	3622	1927
140	180	7.50							2398	1298
150	200	7.50	1480	600	48	8	11.6	5.6	2423	1328
160	220	7.25							2648	1468
170	240	7.25	1680	340	31	4	8.8	3.6	1791	1040

TABLE 11. POLLUTANT MASSES FLUSHED - WALNUT STREET TEST SEGMENT 8/30/76

* * * * * STORM AND COMBINED SEWER RESEARCH: MASS CALCULATIONS * * * * *

STREET: WALNUT STREET

DATE: 08 30 76

FLUSH TECHNIQUE: GRAVITY FLUSH WITH 3 INCH NOZZLE AND HOSE

HOUR OF FLUSH: 12:35 PM

DYE USED: NO

PRESSURE IN TANK: -- PSIG

SIZE OF FLUSH: 50.0 CUBIC FEET

TIME TO COMPLETE FLUSH: 15.0 SECONDS

TIME FOR FLUSH TO REACH DOWNSTREAM: 48.0 SECONDS

SAMPLE NO.	TIME SEC.	FLOW CFS.	COD GRAMS	BOD GRAMS	TKN GRAMS	NH3 GRAMS	TOT. P GRAMS	ORTH.P GRAMS	SS GRAMS	VSS GRAMS
BCKGND	0	0.016	1.9	0.7	0.2	0.2	0.0	0.0	0.5	0.3
1	0	0.170	25.1	10.1	2.1	1.8	0.6	0.4	14.9	8.4
2	10	0.170	52.1	5.3	2.8	1.8	0.7	0.5	43.4	20.4
3	20	0.252	116.9	0.7	5.1	2.7	1.3	0.9	168.6	82.3
4	30	0.320	215.5	41.2	8.4	3.6	1.7	1.4	306.0	147.2
5	40	0.370	327.1	94.3	11.8	4.3	2.1	1.8	542.8	267.1
6	50	0.370	383.7	82.8	14.6	4.4	2.8	1.9	696.7	346.7
7	60	0.370	440.3	71.3	17.4	4.4	3.6	1.9	850.6	426.3
8	70	0.388	659.1	58.2	17.6	4.7	3.2	1.9	295.7	148.3
9	80	0.388	1713.7	83.5	34.1	9.7	5.2	3.8	1821.7	944.7
10	100	0.388	1168.8	81.3	31.0	7.8	5.6	3.4	1165.7	627.7
11	120	0.203	326.6	41.4	14.6	3.1	3.1	1.5	826.0	436.6
12	140	0.120	192.8	23.1	6.6	1.4	1.5	0.7	323.8	171.7
13	160	0.120	192.8	21.7	4.5	1.0	1.1	0.5	245.9	130.8
14	180	0.120	146.6	31.2	3.9	0.7	0.9	0.4	162.8	88.1
15	200	0.120	100.5	40.7	3.3	0.5	0.8	0.4	164.5	90.1
16	220	0.089	79.7	23.7	2.0	0.3	0.5	0.2	133.6	74.1
17	240	0.089	84.8	17.2	1.6	0.2	0.4	0.2	90.4	52.5
TOTAL MASS	VOL=	50.2	6227.9	728.5	181.6	52.5	35.1	22.1	7853.7	4063.4

TABLE 12. CUMULATIVE POLLUTANT MASSES FLUSHED - WALNUT STREET TEST SEGMENT - 8/30/76

***** CUMULATIVE MASSES OF POLLUTANTS CARRIED AWAY BY THE FLUSH WAVE *****

COD:	1.9	27.1	79.2	196.0	411.6	738.6	1122.3	1562.6	2221.7
	3935.4	5104.2	5430.8	5623.5	5816.3	5962.9	6063.4	6143.1	6227.9
BOD:	0.7	10.9	16.2	16.9	58.1	152.4	235.2	306.5	364.7
	448.2	529.5	570.9	594.0	615.7	646.9	687.7	711.4	728.5
TKN:	0.2	2.3	5.1	10.2	18.6	30.4	45.1	62.5	80.1
	114.1	145.1	159.7	166.3	170.9	174.8	178.0	180.0	181.6
AMM:	0.2	1.9	3.7	6.5	10.0	14.3	18.7	23.1	27.8
	37.5	45.3	48.4	49.8	50.7	51.5	52.0	52.3	52.5
T.PH:	0.0	0.6	1.3	2.6	4.3	6.4	9.2	12.8	15.9
	21.2	26.8	29.9	31.4	32.4	33.4	34.2	34.7	35.1
O.PH:	0.0	0.5	1.0	1.9	3.3	5.1	7.0	8.9	10.8
	14.7	18.0	19.6	20.3	20.8	21.3	21.6	21.9	22.1
SS:	0.5	15.4	58.9	227.5	533.5	1076.3	1773.0	2623.6	2919.3
	4741.0	5906.7	6732.8	7056.6	7302.5	7465.2	7629.7	7763.3	7853.7
VS:	0.3	8.8	29.2	111.5	258.7	525.8	872.5	1298.8	1447.1
	2391.8	3019.5	3456.1	3627.8	3758.6	3846.7	3936.9	4011.0	4063.4
VOL.:	0.0	0.8	2.3	4.3	7.0	10.3	13.8	17.4	21.0
	24.7	32.2	37.8	40.7	42.8	44.8	46.9	48.7	50.2

Each row represents cumulative pollutant mass transported in grams, past sampling manhole at each instant of sampling, including background sewage contribution. Last row is cumulative flush volume in cubic feet.

TABLE 13. CUMULATIVE PERCENTILES OF POLLUTANTS FLUSHED - WALNUT STREET TEST SEGMENT
8/30/76

* * * * * PERCENTUAL MASSES OF POLLUTANTS CARRIED AWAY BY THE FLUSH WAVE * * * * *

COD:		0.4	1.3	3.1	6.6	11.9	18.0	25.1	35.7
	63.2	82.0	87.2	90.3	93.4	95.7	97.4	98.6	100.0
BOD:		1.5	2.2	2.3	8.0	20.9	32.3	42.1	50.1
	61.5	72.7	78.4	81.5	84.5	88.8	94.4	97.6	100.0
TKN:		1.3	2.8	5.6	10.2	16.8	24.8	34.4	44.1
	62.9	79.9	88.0	91.6	94.1	96.2	98.0	99.1	100.0
AMM:		3.7	7.1	12.3	19.1	27.3	35.6	44.0	53.0
	71.4	86.2	92.1	94.8	96.6	98.0	99.0	99.6	100.0
T.PH:		1.7	3.7	7.3	12.1	18.1	26.2	36.3	45.4
	60.2	76.2	85.1	89.3	92.4	95.0	97.3	98.7	100.0
O.PH:		2.1	4.5	8.7	14.9	23.3	31.8	40.3	49.1
	66.5	81.8	88.8	92.0	94.4	96.4	98.1	99.2	100.0
SS:		C.2	0.7	2.9	6.8	13.7	22.6	33.4	37.2
	60.4	75.2	85.7	89.9	93.0	95.1	97.1	98.8	100.0
VS:		0.2	0.7	2.7	6.4	12.9	21.5	32.0	35.6
	58.9	74.3	85.1	89.3	92.5	94.7	96.9	98.7	100.0

Each row represents cumulative percentage of pollutant mass transported past sampling manhole at each instant of sampling, starting at first occurrence of flush wave.

SECTION 8

SINGLE SEGMENT FLUSHING RESULTS

8.1 Foreword

All field flushing results conducted during the first phase of experimentation are presented in this Chapter. Descriptions of the test segments, field sampling procedures and equipment, analytical laboratory techniques and computational methods used to process field and laboratory information have all been presented in Chapter 4, 5, 6 and 7, respectively. Preliminary flushing results conducted early in the program on Shepton Street are presented in Section 8.2 along with typical results for flushes conducted at other test segments over the course of the program. The solids, organics and nutrient flushed loadings for 86 flushing experiments conducted during the fall of 1976 are presented in Section 8.3. Statistical results per flushing site are presented for the total flushed pollutant masses, pollutant masses normalized by antecedent periods between flushes and pollutant masses normalized by both antecedent periods and contributary population. Similar results are presented in Section 8.4 for the masses of heavy metals flushed per test segment. The relative pollutant removal effectiveness of different flush methods considered in this phase of work are examined in Section 8.5. Section 8.6 presents various visual observations and analytical results pertaining to sediment characteristics encountered during the flushing program.

8.2 Typical First Phase Flushing Results

The field flushing program was initated during the middle of August, 1976. Pre-cleaning of segments was accomplished during the latter part of that month. Figure 42 shows two photographs of heavily deposited sewers in the study area. Figure 42A shows a photograph taken at the intersection of Florida and Templeton Street. A photograph of the Templeton Street sewer segment is shown in Figure 42B. These photographs of heavy deposits were typical of the flat segments in the study area. The photographs show that dry weather deposition in upstream collection system laterals means rags, sticks, large globs of organic material, toilet paper along with fine silt, sand and rocks. A contrast of several sewer segments on fairly steep streets in the area where light to moderate deposition was present is shown in Figure 43.

Several flushing experments were conducted on each test segment during the pre-cleaning period primarily to develop field procedures and to uncover any mechanical difficulties with the flush truck. Two flushes at the Shepton Street test segment were conducted on August 18, 1976 prior to pre-cleaning. About 4 inches of thick black sediments and heavy sanitary

A. Florida and Templeton Streets

B. Templeton Street

FIGURE 42. PHOTOGRAPHS OF COLLECTION SEWERS IN THE STUDY AREA WITH HEAVY DEPOSITION RATES

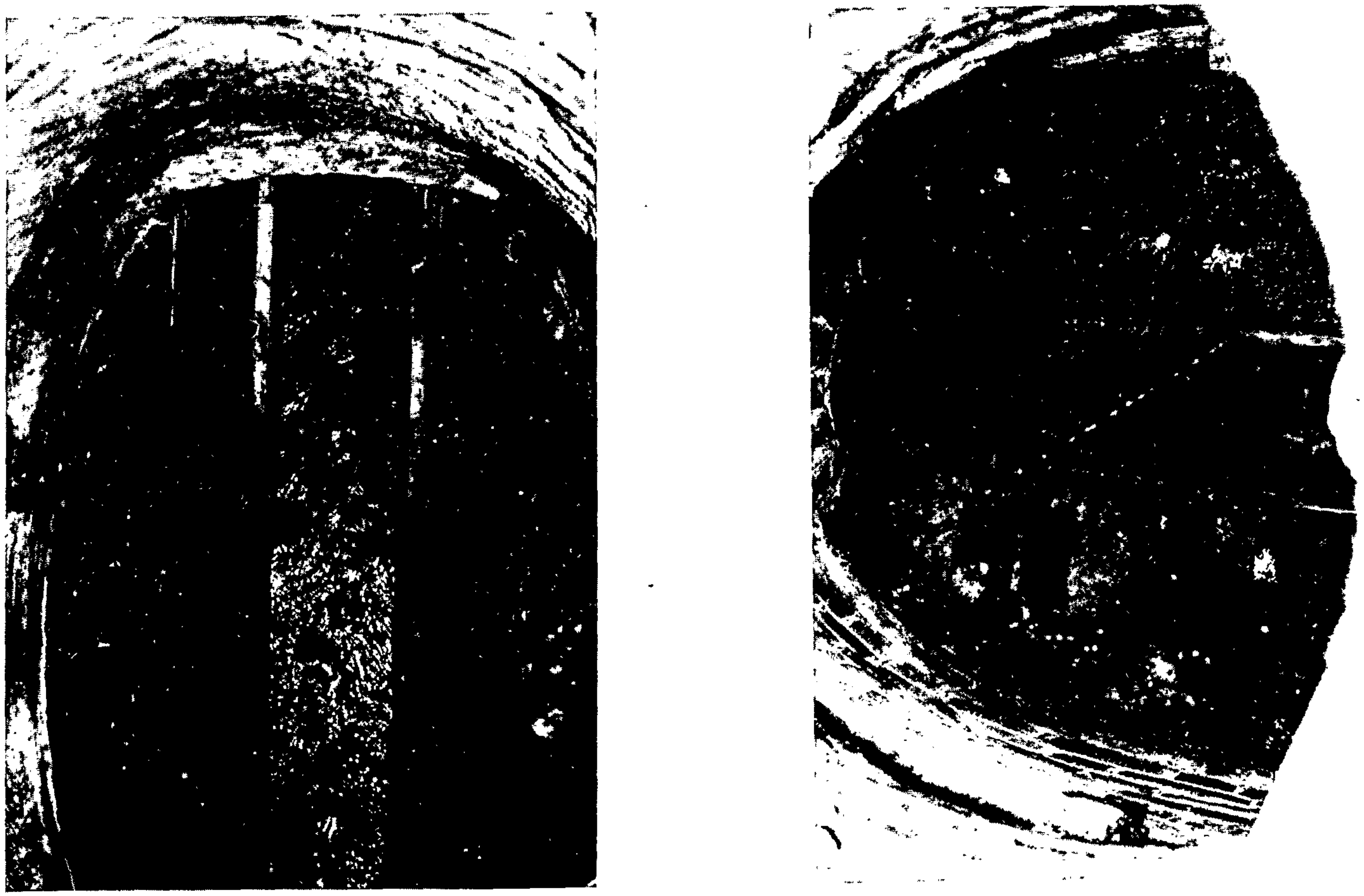


FIGURE 43. PHOTOGRAPHS OF COLLECTION SEWERS IN THE STUDY AREA WITH LOW DEPOSITION RATES

deposit levels were present in the segment prior to the flushes.

The second flush was conducted ten minutes after the first flush experiment was performed and flush wave sampling completed. The volume and rate of both flushes were 50 cubic feet at 0.5 cfs. Plots of TSS, VSS, BOD and COD flush wave concentrations for both flushes are shown in Figure 44. Similar plots for TKN, TP, NH_3 and OP are shown in Figure 45. With the exception of TSS, the second flush wave concentrations were significantly less in magnitude to the first flush. Peak COD concentrations shown in Figure 44 were 5100 and 750 mg/l for the two flushes, respectively. Peak TKN concentrations shown in Figure 45 were 135 and 23 mg/l for the two flushes, respectively. These results were extremely encouraging in that the first flush seemed to transport most of the organic and nutrient related pollutants without pre-cleaning the segments. The mass loadings flushed for these two experiments were not computed, since the wave heights were not recorded during these two flushes.

Typical flush wave pollutant concentration plots for the four test segments are shown in Figures 46 and 47. VSS peak concentrations ranged between 6000 mg/l for the Walnut Street flush conducted on 11/06/76 to 7600 mg/l for the Templeton Street flush conducted on 9/13/76. These plots were typical of the first phase experimentation program. Background sewage concentrations are also indicated on the plots. The TSS and VSS background concentration levels shown in Figure 46 for Port Norfolk and Walnut Street typified background sampling during the project. Test segment background sewage levels determined during the project greatly exceeded nominal concentrations found further downstream at sewage facilities. Summarized background sampling results are given in Chapter 11.

An example showing the mass rate of pollutants transported during a given flush is shown in Figure 48 for the experiment conducted at Port Norfolk Street on 11/01/76. Pollutographs of BOD, TSS and VSS are shown in the left hand plot and pollutographs of NH_3, TP and TKN are shown in the right hand figure. The flushed mass rates peaked within one minute for the solids and organics and within a half a minute for the nutrients. This phenomena was typical of flushes throughout the program where the peak mass wave of the lighter pollutant fraction occurred sooner than the heavier solids loadings. The plots also show the importance of accurate definition of the flush wave hydraulic characteristics. The rigorous flush wave discharge computational analysis described in Chapter 7 was initiated since it was apparent earlier in the project that any significant error in estimation of the hydraulic profiles of the flush wave would result in erroneous estimation of pollutant masses transported.

8.3 Solids, Organics and Nutrient Flush Removal Results

The pollutant mass removals, in kilograms, for the flushing experiments conducted between August 30 through November 12, 1976 on Port Norfolk, Shepton, Templeton and Walnut Street test segments are presented in Tables 14 through 17, respectively. Following the date of flush on the left hand side of these tables, the information under column headings are presented as follows: A) truck flush volume (cf); B) truck flush

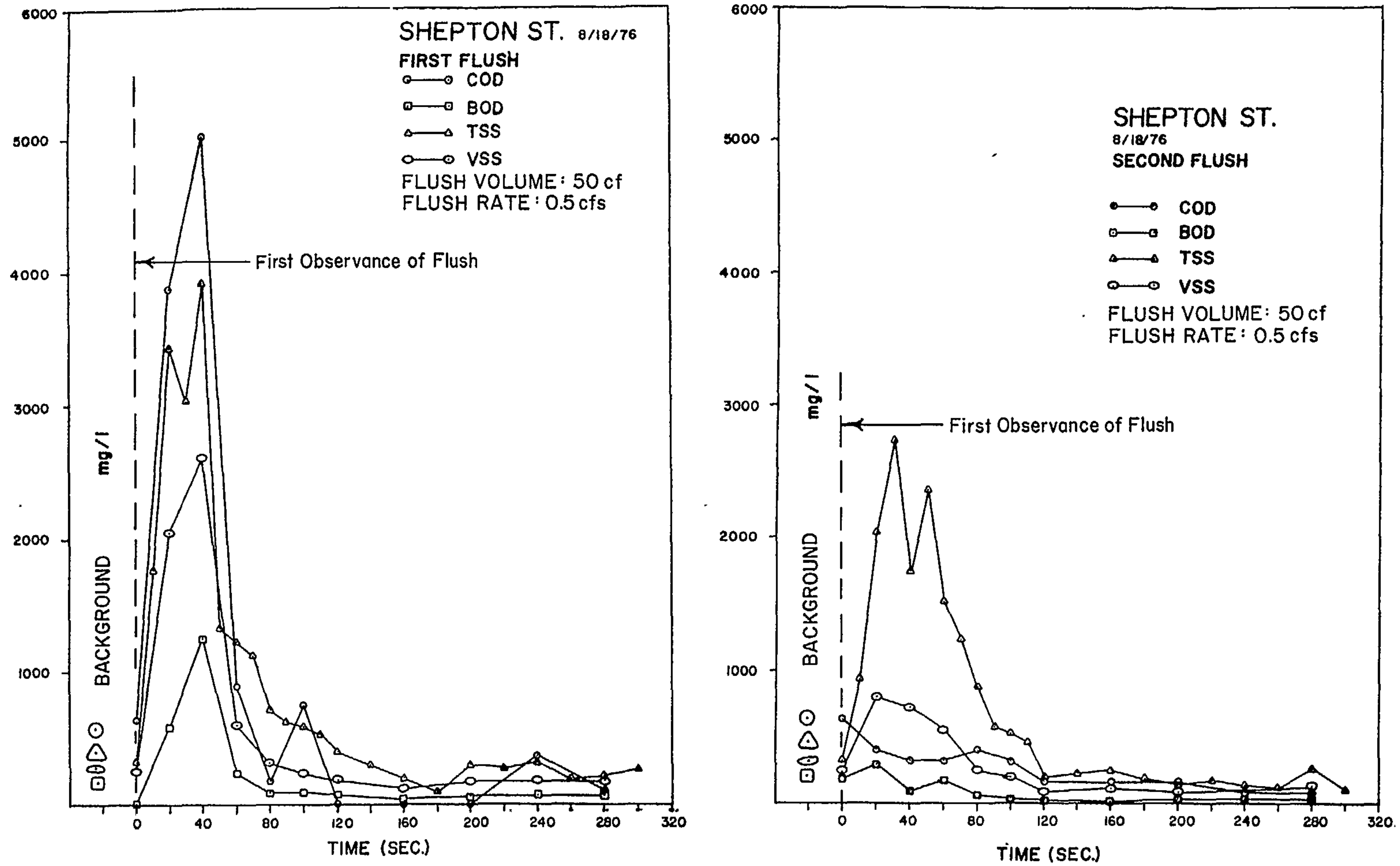


FIGURE 44. PLOTS OF FLUSH WAVE SOLIDS AND ORGANIC POLLUTANT CONCENTRATIONS - SHEPTON STREET 8/18/76

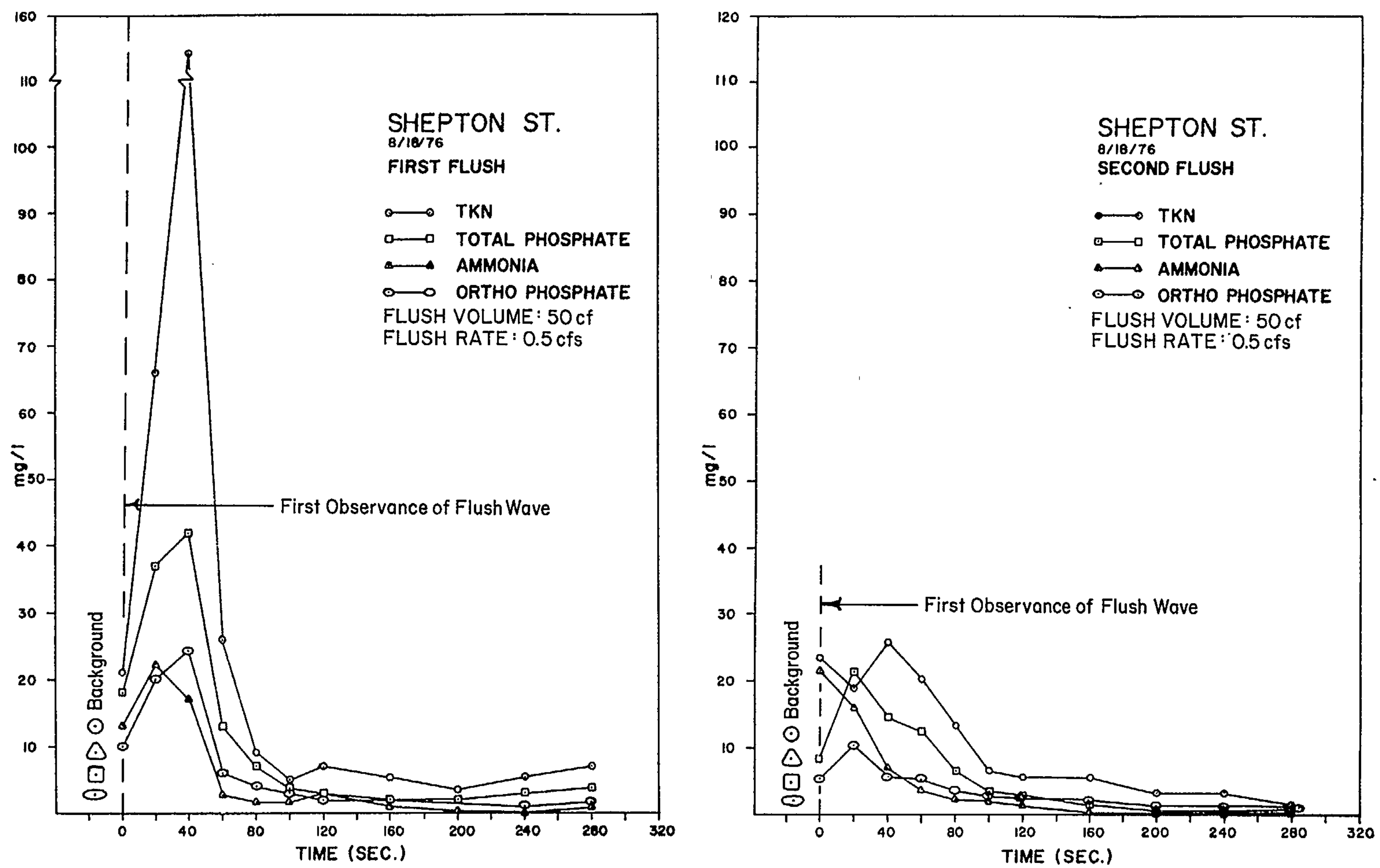


FIGURE 45. PLOTS OF FLUSH WAVE NUTRIENT POLLUTANT CONCENTRATIONS - SHEPTON STREET 8/18/76

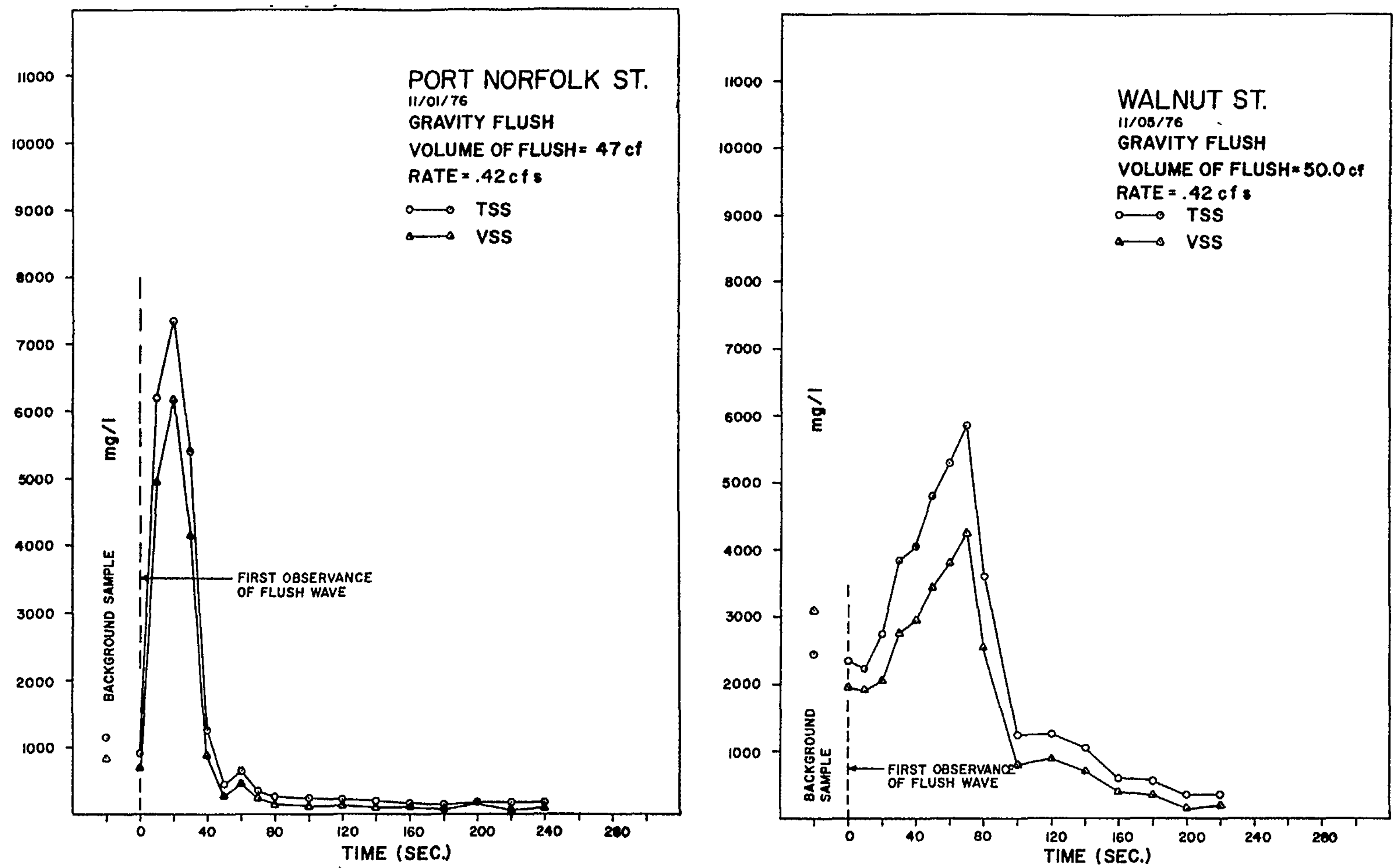


FIGURE 46. FLUSH WAVE TSS AND VSS CONCENTRATIONS PLOTS FOR PORT NORFOLK STREET (11/1/76) AND WALNUT STREET (11/6/76)

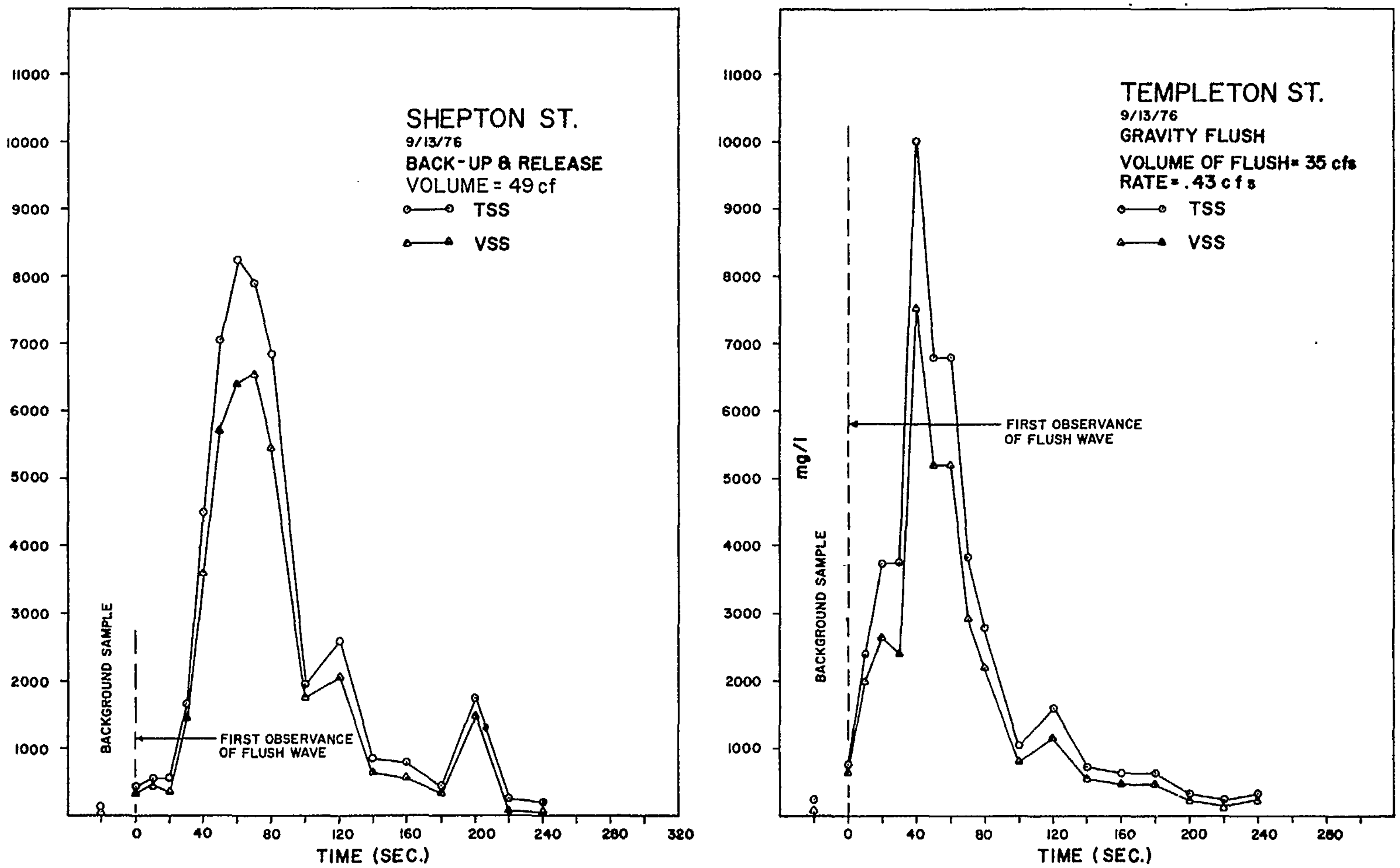


FIGURE 47. FLUSH WAVE CONCENTRATIONS PLOTS - SHEPTON STREET (9/13/76) AND TEMPLETON STREET (9/13/76)

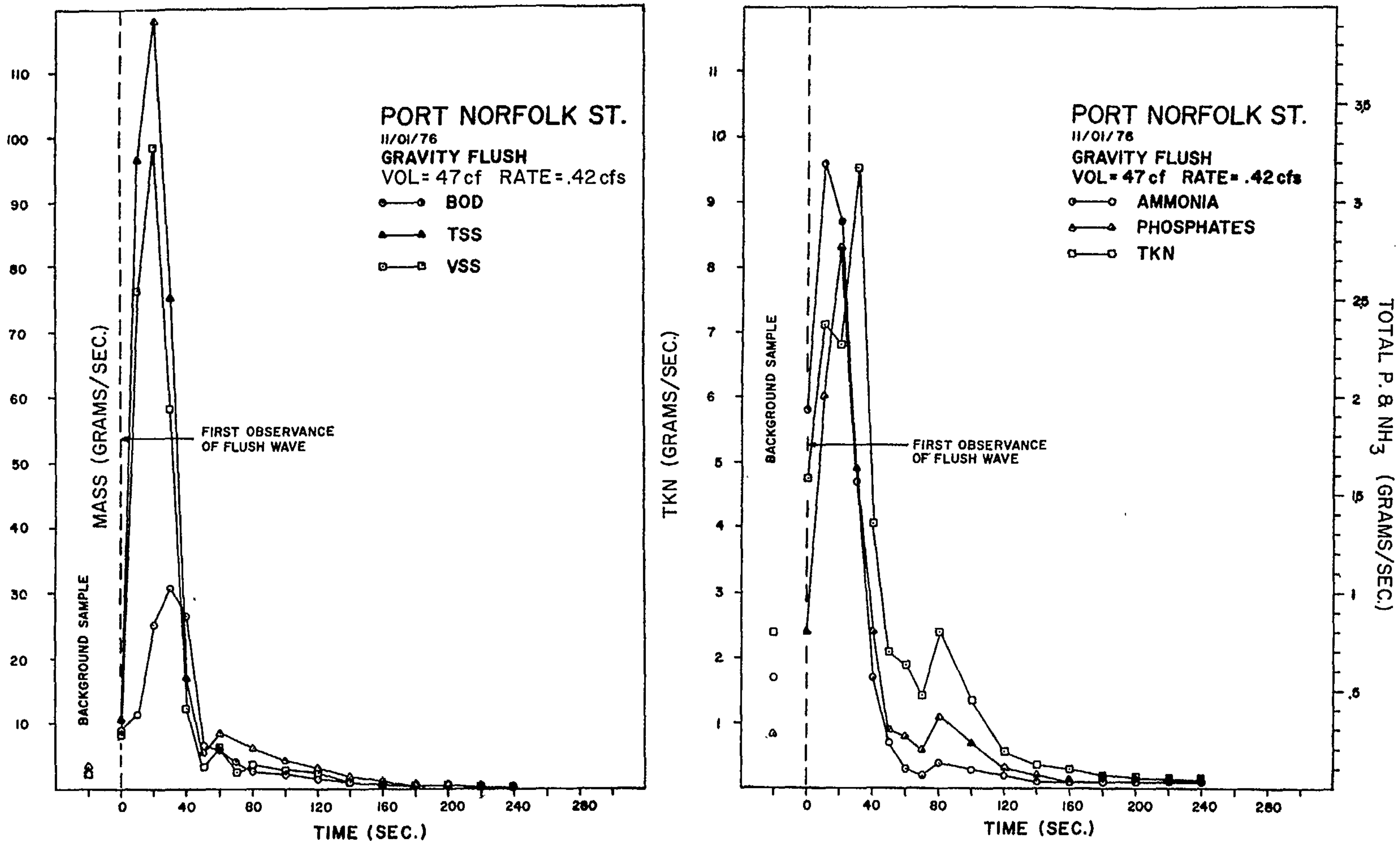


FIGURE 48. FLUSH WAVE POLLUTANT MASS VERSUS TIME PLOTS - PORT NORFOLK STREET 11/01/76

TABLE 14. PHASE I - FIELD FLUSHING POLLUTANT REMOVALS (kg)
PORT NORFOLK STREET

DATE	A	B	C	D	E	F	G	TOTAL MASS (kg) COD	BOD	TKN	NH_3	TP	TSS	VSS
8/30/76	32.9	.47	1	x	20	x	x	0.74	0.28	0.020	0.003	0.007	1.74	0.54
9/2	46.5	.92	3	x	2	x	x						16.61	10.18
9/7	32.8	.48	5			x	x	1.97	0.87	0.085	0.016	0.018	2.35	1.64
9/16	47.0	.44	9			x							2.14	1.46
9/21	32.6	.82	5	x	82	x	x	2.33	0.46	0.091	0.019	0.031	3.29	1.58
9/24	46.6	.79	3			x	x						3.26	2.12
10/1	33.7	.19	7			x	x	3.80	1.28	0.125	0.015	0.017	4.46	3.69
10/4	B&R***		3	x	41	x	x	10.71	4.12	0.426	0.084	0.082	11.42	8.72
10/8	32.9	.46	4	x	64	x	x						2.70	1.79
10/12	33.4	.27	4	x	72	x	x	1.78	0.71	0.068	0.008	0.019	2.65	1.96
10/15	33.1	.33	3			x	x						1.87	1.51
10/18	47.0	.42	3			x	x	4.69	1.58	0.142	0.028	0.001	3.96	3.32
10/25	32.7	.63	3			x	x						2.19	1.61
10/22	46.6	.71	4	x	37	x	x						2.01	1.55
10/29	32.9	.46	4			x	x						2.68	2.22
11/1	47.0	.42	3				x	4.32	1.32	0.152	0.035	0.030	3.59	2.82
11/5	46.6	.85	4				x						3.16	2.91
11/8	32.9	.47	3				x	1.30	4.84	0.042	0.012	0.013	1.34	1.06
11/12	B&R***		4				x						7.69	7.08

Legend
A - Truck Flush Volume (cf)
B - Truck Flush Rate (cfs)
C - Total Antecedent Days Between Flushes
D - Rainfall Impacted Event
E - Hours After Rainfall Event to Flush*
F - Segment Cleaned After Flush**
G - Good Flush

*Number of hours between rainfall exceeding 0.15 inch/hour and flush event.

**Segment flushed after experiment using maximum truck discharge for five minutes.

***Backup and release.

TABLE 15. PHASE I - FIELD FLUSHING POLLUTANT REMOVALS (kg)
SHEPTON STREET

DATE	A	B	C	D	E	F	G	TOTAL MASS (kg) COD	BOD	TKN	NH_3	TP	TSS	VSS
8/23/76	47.8	.26	5			x	x	5.73	1.22	.097	.028	.027	3.91	2.82
8/26	35.4	.76	3			x	x		... A N A L Y T I C A L P R O B L E M S ...					
8/30	37.3	.65	4	x	23	x	x						2.94	1.79
9/2	B&R***		3	x	4	x	x		2.02				7.57	5.55
9/7	20.0		5			x							11.54	9.03
9/10	32.6	.69	3			x	x		1.07				4.52	3.60
9/13	B&R***		3	x	66	x	x						2.81	2.24
9/16	46.6	.67	3			x	x		1.97				6.76	5.53
9/21	32.9	.42	5	x	86	x	x						4.34	2.88
9/24	33.1	.37	3			x	x		.73				2.76	2.01
10/1	46.5	.93	7			x	x						2.19	1.51
10/4	32.9	.47	3	x	45	x	x						5.76	4.58
10/8	32.9	.46	4	x	60	x	x		1.97				5.16	4.10
10/12	B&R***		4	x	75	x	x						4.30	3.12
10/15	46.6	.71	3			x	x		2.62				4.66	3.77
10/18	33.0	.47	3			x	x						6.67	5.36
10/22	32.9	.43	4	x	35	x	x		1.21				2.52	1.98
10/25	47.2	.37	3			x	x	7.28	2.02	.189	.047	.051	4.31	3.63
10/29	46.6	.72	4			x	x		.93				3.44	2.79
11/1	32.7	.61	3				x						2.70	2.17
11/5	33.0	.40	4				x		2.20				5.07	4.21
11/8	47.1	.41	3				x						5.29	4.21
11/12	B&R***		4										1.08	.94

Legend

A - Truck Flush Volume (cf)
B - Truck Flush Rate (cfs)
C - Total Antecedent Days Between Flushes
D - Rainfall Impacted Event
E - Hours After Rainfall Event to Flush*
F - Segment Cleaned After Flush**
G - Good Flush

*Number of hours between rainfall exceeding 0.15 inch/hour and flush event.
**Segment flushed after experiment using maximum truck discharge for five minutes.
***Backup and release.

TABLE 16. PHASE I - FIELD FLUSHING POLLUTANT REMOVALS (kg)
TEMPLETON STREET

DATE	A	B	C	D	E	F	G	TOTAL MASS (kg)						
								COD	BOD	TKN	NH_3	TP	TSS	VSS
8/30/76	46.5	.98		x	25	x							7.01	2.71
9/2	32.8	.50	3	x	5	x	x		7.15				18.32	11.75
9/7	B&R***		5			x							4.48	3.84
9/10	57.4	.83	3			x	x		4.27				11.88	9.08
9/13	32.9	.43	3	x	65	x		3.55	.65	.097	.010	.019	3.07	2.29
9/16	B&R***		3			x			2.78				8.44	6.04
9/21	B&R***		5	x	87	x	x						7.85	4.83
9/24	33.1	.36	3			x	x						3.45	2.897
10/1	B*R***		7			x	x						13.18	9.47
10/4	46.6	.87	3	x	43	x	x						7.43	6.01
10/8	32.9	.46	4	x	62	x	x		.75				3.39	2.92
10/12	32.9	.43	4	x	73	x	x						33.21	26.66
10/15	47.1	.41	3			x	x		.91				4.44	3.04
10/18	46.6	.76	4			x	x						12.52	9.96
10/22	32.7	.59	3	x	34	x	x		1.87				2.32	1.53
10/25	32.9	.46	4			x	x						3.41	2.78
10/29	46.9	.49	3			x	x		2.92				6.55	4.83
11/1	46.7	.75	3				x						9.49	6.39
11/5	32.7	.56	4				x		3.47				8.43	6.96
11/8	32.9	.43	3				x						2.88	2.12
11/12	B&R***		4				x		6.56					

Legend
A - Truck Flush Volume (cf)
B - Truck Flush Rate (cfs)
C - Total Antecedent Days Between Flushes
D - Rainfall Impacted Event
E - Hours After Rainfall Event to Flush*
F - Segment Cleaned After Flush**
G - Good Flush

*Number of hours between rainfall exceeding 0.15 inch/hour and flush event.
**Segment flushed after experiment using maximum truck discharge for five minutes.
***Backup and release.

TABLE 17. PHASE I - FIELD FLUSHING POLLUTANT REMOVALS (kg)
WALNUT STREET

DATE	A	B	C	D	E	F	G	TOTAL MASS (kg)						
								COD	BOD	TKN	NH_3	TP	TSS	VSS
8/23/76	69.8	.80	3			x	x	7.40	2.11	.183	.088	.063	5.64	2.60
8/26	46.6	.86	3			x	x		1.79				1.42	.65
8/30	47.1	.41	4	x	21	x	x	6.23	7.28	.181	.052	.035	7.85	4.06
9/2	60.6	.65	3	x	1	x							6.32	3.38
9/7	B&R***		3			x	x	2.76	1.02	.144	.078	.049	2.42	1.09
9/10	32.0	.47	3			x	x						4.15	3.59
9/13	46.5	.96	3	x	62	x	x	9.50	2.03	.245	.202	.049	6.33	3.84
9/16	32.0	1.39	3			x							2.17	1.76
9/21	46.6	.89	5	x	84	x	x	6.69	1.21	.147	.040	.075	13.33	3.67
9/24	B&R***													
10/1	33.0*	.39*	7			x	x	5.10	1.81	.138	.033	.043	4.19	2.86
10/4	32.9	.46	3	x	40	x	x	8.92	3.42	.323	.088	.091	9.31	6.55
10/8	B&R***		4	x										
10/12	47.0	.98	4	x	71	x	x	31.45	7.68	.526	.109	.091	29.34	16.69
10/15	32.8	.50	3			x	x						1.06	.58
10/18	33.0	.42	3			x	x	2.59	1.24	.139	.062	.001	2.43	1.84
10/22	47.3	.35	4	x	38	x	x						7.74	6.99
10/25	46.5	.95	3			x	x	4.51	1.22	.165	.072	.045	3.14	1.91
10/29	32.9	.47	4				x						1.71	1.29
11/1	32.9	.43	3				x	4.21	1.65	.141	.041	.030	2.53	1.88
11/5	47.0	.42	4				x						4.39	3.17
11/8	46.6	.88	3				x	9.12	3.34	.235	.127	.052	6.39	4.23
11/12	B&R***		4										4.30	3.57

Legend

A - Truck Flush Volume (cf)
B - Truck Flush Rate (cfs)
C - Total Antecedent Days Between Flushes
D - Rainfall Impacted Event
E - Hours After Rainfall Event to Flush*
F - Segmented Cleaned After Flush**
G - Good Flush

*Number of hours between rainfall exceeding 0.15 inch/hour and flush event.
**Segment flushed after experiment using maximum truck discharge for five minutes.
***Backup and release.

rate (cubic feet per second); C) total number of antecedent days respective of rainfall occurrences between flushes; D) an indication noted by "X: if the event was impacted by rainfall; E) Number of antecedent hours between rainfall exceeding 0.15 inch/hour and flush event; F) an indication noted by "X" if the post flushing of the flushing experiment was conducted; and G) an indication noted by "X" if the flushing experiment was completed without any major mechanical or procedual problem such as inoperative water meter, burst hoses and structural failure of manhole table. The next set of columns present the computed pollutant masses, in kilograms, of flushed load for the following parameters: COD, BOD, TKN, NH_3, TP, TSS and VSS. These estimates were prepared using the loop stage rating curves developed by optimization procedures, described in Chapter 7.

Hourly rainfall information measured at the Blue Hills Observatory located about 5 miles from the study area were collected and compared with strip charts from the automatic liquid level sensing devices installed at each of the test segments. These devices were operated and maintained during most of the experimentation period. Typical results of dry weather flow gaging are discussed in Chapter 11. It appeared upon inspection of the strip charts that rainfall intensities less than 0.10 inches/hour at the two combined sewer segments, that is, on Port Norfolk and Walnut Streets, would not result in any marked increase in depth of flow while intensities of less than 0.15 inches/hour would not result in depth of flow increases in the two separate sewer segments on Templeton and Shepton Streets. The two separate sewer segments receive clear water inflow from roof drain connections. These empirical criteria were then used to determine whether the antecedent period prior to flushing events were impacted by rainfall of sufficient magnitude that would change dry weather accumulations. The number of hours between the last rainfall occurrence and the flushing event was arbitrarily defined at a rainfall intensity of 0.15 inches/hour.

The rationale for terminating the post flushing cleaning operation in November was to determine, in an indirect way, whether substantial residual materials remained after the routine experimental flushes. The nature of these experiences would provide indirect evidence of the flushing pollutant removal efficiencies.

In total, there were 86 flushes performed during this period. Table 18 summarizes for each test segment the number of flushes, the number of flushes free from operational problems, the number of good flushes with dry antecedent periods and the number of good flushes impacted by rainfall events occurring during periods between flushes. Table 18 shows that 64% of all good flushes were conducted under the conditions of dry antecedent periods and that 86% of all the flushes were operationally acceptable.

The mean and standard deviation of total solids, organic and nutrient flushed mass removals for Port Norfolk, Shepton, Templeton and Walnut Street test segments are given in Tables 19 through 22. Typically, each table contains pollutant mass removal statistics including the mean, standard deviation and number of experiments used in the computations for the following flush groupings: a) all flushes; b) all

TABLE 18. SUMMARY OF FIRST PHASE FLUSHING EVENT CHARACTERISTICS

Flush Event Characteristics	Port Norfolk	Shepton	Templeton	Walnut	Total
Number of Flushes	19	23	21	23	86
Number of Good Flushes	18	21	17	18	74
Number of Good Flushes with Dry Antecedent Periods	11	13	11	12	47
Number of Good Flushes Impacted by Rainfall Events	7	8	6	6	27

good flushes free from operational problems; c) all good flushes with dry antecedent dry periods; and d) all flushes impacted by rainfall. Although these results are not normalized for antecedent periods prior to flushing, several observations can be drawn from inspection of the tables. First of all, the coefficients of variation, that is, the standard deviation divided by the mean, for the mass removals are all less than unity with the exception of the rainfall impacted mass removals for the two combined sewer segments on Port Norfolk and Walnut Streets. This implies that the flushing removal rates and also the rates of deposition are reasonably stable statistics which is important from a prediction and control standpoint. Large coefficients of variation, on the order of 2 to 3, were expected. Secondly, the ratios of VSS/TSS for the good non-rainfall impacted flushes were remarkably consistent, ranging from 0.65 to 0.75 with an average of 0.71. Thus, nearly three quarters of the suspended solids transported were volatile in nature.

The total flushed pollutant masses given in Tables 14 through 18 for each of the test segments were divided by the total antecedent dry periods between flushing events given under column C of each table. Figure 49 through 52 show the time series of flushed masses for various pollutants normalized by the antecedent dry periods for Port Norfolk, Shepton, Templeton and Walnut Streets, respectively. Plots of daily rainfall collected at the Blue Hills Observatory are presented at the top of each figure. Plots of total phosphate are followed by TSS with VSS. The final two plots per figure include TKN with NH_3, and COD with BOD. The remarkable feature of the plots for any given pollutant is the degree of flushing removal consistency for a given test segment.

Statistical summarys of the pollutant mass removals normalized by antecedent days between flushes are presented in Tables 23 through 26 for the Port Norfolk, Shepton, Templeton and Walnut test segments, respectively. Means, standard deviations and the number of flushes used in the computations are presented for the seven pollutants for eight different groupings of the

TABLE 19. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF TOTAL MASS REMOVALS
PORT NORFOLK STREET TEST SEGMENT

TYPE	POLLUTANT REMOVALS (kg/flush)						
	COD	BOD	TKN	NH_3	TP	TSS	VSS
A: ALL FLUSHES	3.51 3.03 (9)	1.60 1.58 (10)	0.128 0.120 (9)	0.024 0.024 (9)	0.024 0.024 (9)	4.16 3.83 (19)	3.04 2.66 (19)
B. ALL GOOD * FLUSHES	3.51 3.03 (9)	1.00 1.58 (10)	0.128 0.120 (9)	0.024 0.024 (9)	0.024 0.024 (9)	4.28 3.91 (18)	3.13 2.71 (18)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	3.22 1.50 (5)	1.12 0.41 (5)	0.109 0.045 (5)	0.021 0.010 (5)	0.016 0.010 (5)	3.32 1.72 (11)	2.72 1.66 (11)
D: ALL RAINFALL IMPACTED FLUSHES	3.89 4.59 (4)	1.39 1.83 (4)	0.151 0.186 (4)	0.029 0.038 (4)	0.035 0.033 (4)	5.78 5.84 (7)	3.76 3.93 (7)

* See definition in text, page 129.

KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 20. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF TOTAL MASS REMOVALS
SHEPTON STREET TEST SEGMENT

TYPE	POLLUTANT REMOVALS (kg/flush)						
	COD	BOD	TKN	NH_3	TP	TSS	VSS
A: ALL FLUSHES	6.50 1.10 (2)	1.63 0.62 (11)	0.143 0.065 (2)	0.0375 0.0134 (2)	0.039 0.017 (2)	4.56 2.24 (22)	3.54 1.79 (22)
B. ALL GOOD * FLUSHES	6.50 1.10 (2)	1.63 0.62 (11)	0.143 0.065 (2)	0.0375 0.0134 (2)	0.039 0.017 (2)	4.38 1.53 (20)	3.39 1.27 (22)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	6.50 1.10 (2)	1.60 0.69 (8)	0.143 0.065 (2)	0.0375 0.0134 (2)	0.039 0.017 (2)	4.36 1.47 (12)	3.47 1.27 (20)
D: ALL RAINFALL IMPACTED FLUSHES	-	1.73 0.45 (3)	-	-	-	4.43 1.72 (8)	3.28 1.35 (8)

* See definition in text, page 129.

KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 21. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATIONS OF TOTAL MASS REMOVALS
TEMPLETON STREET TEST SEGMENT

	POLLUTANT REMOVALS (kg/flush)						
TYPE	COD	BOD	TKN	NH_3	TP	TSS	VSS
A: ALL FLUSHES	3.55 - (1)	3.13 2.31 (10)	0.097 - (1)	0.010 - (1)	0.019 - (1)	8.59 7.16 (20)	6.31 5.63 (20)
B: ALL GOOD FLUSHES *	-	3.49 2.41 (8)	-	-	-	9.30 7.82 (16)	6.95 6.11 (16)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	-	3.63 2.06 (5)	-	-	-	7.62 4.03 (10)	5.75 3.03 (10)
D: ALL RAINFALL IMPACTED FLUSHES	-	2.61 3.08 (4)	0.097 (1)	0.010 - (1)	0.019 - (1)	10.32 10.54 (8)	7.34 8.46 (8)

* See definition in text, page 129.

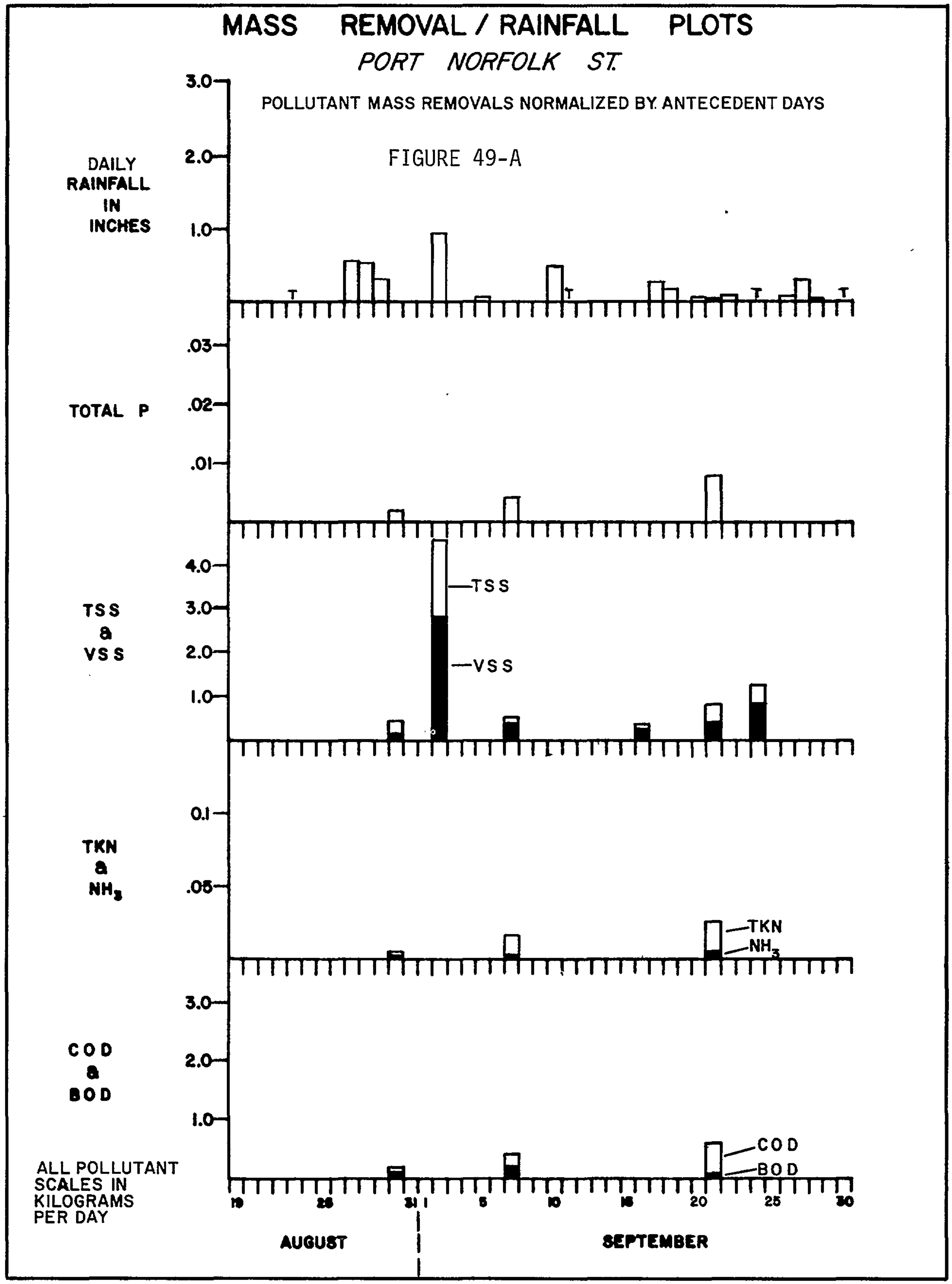
KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 22. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF TOTAL MASS REMOVALS
WALNUT STREET TEST SEGMENT

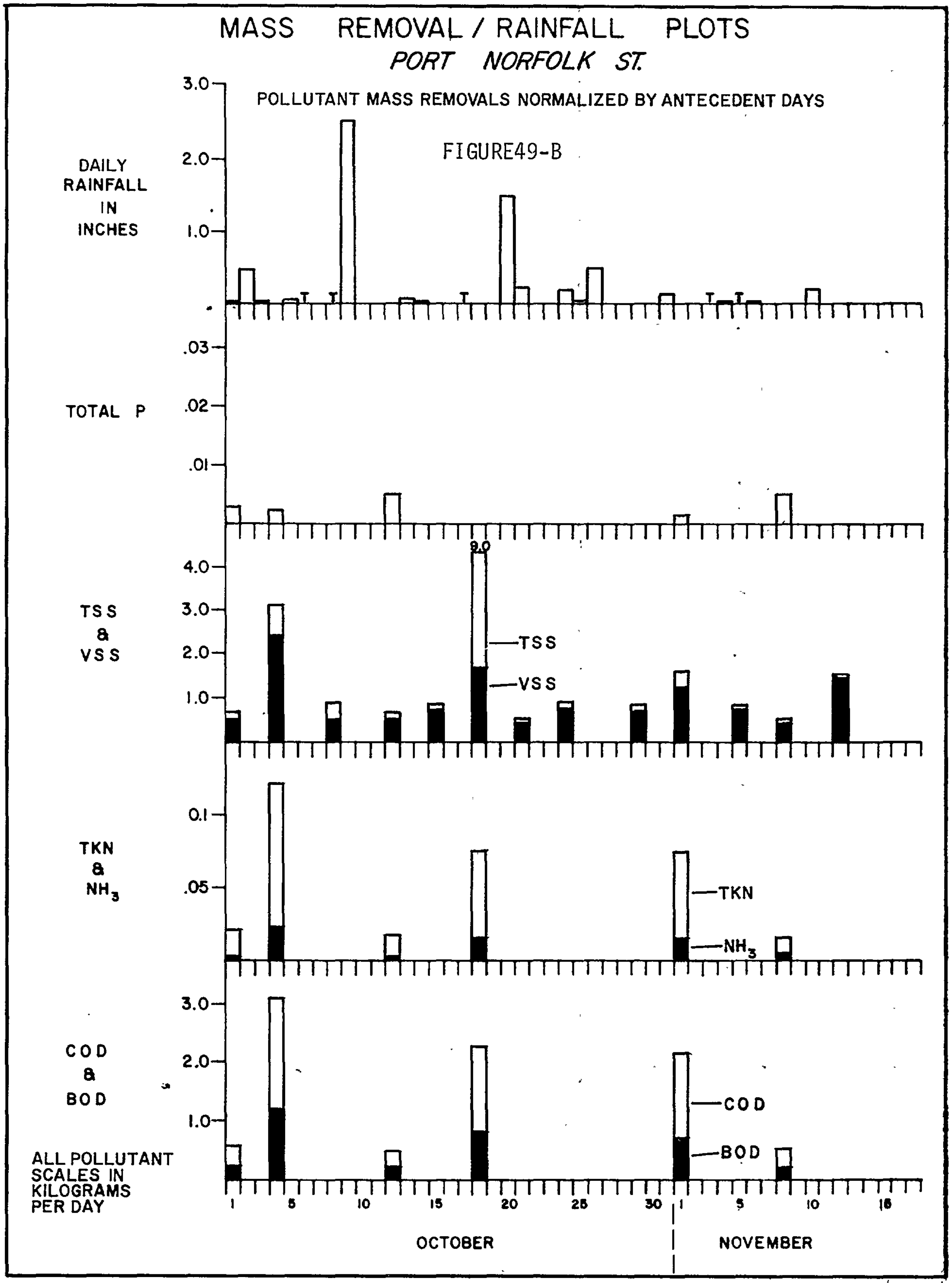
TYPE	POLLUTANT REMOVALS (kg/flush)						
	COD	BOD	TKN	NH_3	TP	TSS	VSS
ALL FLUSHES	8.21 7.69 (12)	2.75 2.23 (13)	0.214 0.113 (12)	0.083 0.047 (12)	0.052 0.026 (12)	6.01 6.14 (21)	3.63 3.43 (21)
ALL GOOD * FLUSHES	8.21 7.69 (12)	2.75 2.23 (13)	0.214 0.113 (12)	0.083 0.047 (12)	0.052 0.026 (12)	6.30 6.57 (18)	3.75 3.69 (18)
ALL GOOD NON-RAINFALL IMPACTED FLUSHES	5.10 2.39 (7)	1.77 0.73 (8)	0.164 0.036 (7)	0.072 0.031 (7)	0.040 0.020 (7)	3.29 1.68 (12)	2.14 1.16 (12)
ALL RAINFALL IMPACTED FLUSHES	12.56 10.65 (5)	4.32 2.99 (5)	0.284 0.151 (5)	0.098 0.064 (5)	0.068 0.025 (5)	11.46 8.24 (7)	6.45 4.74 (7)

* See definition in text, page 129.

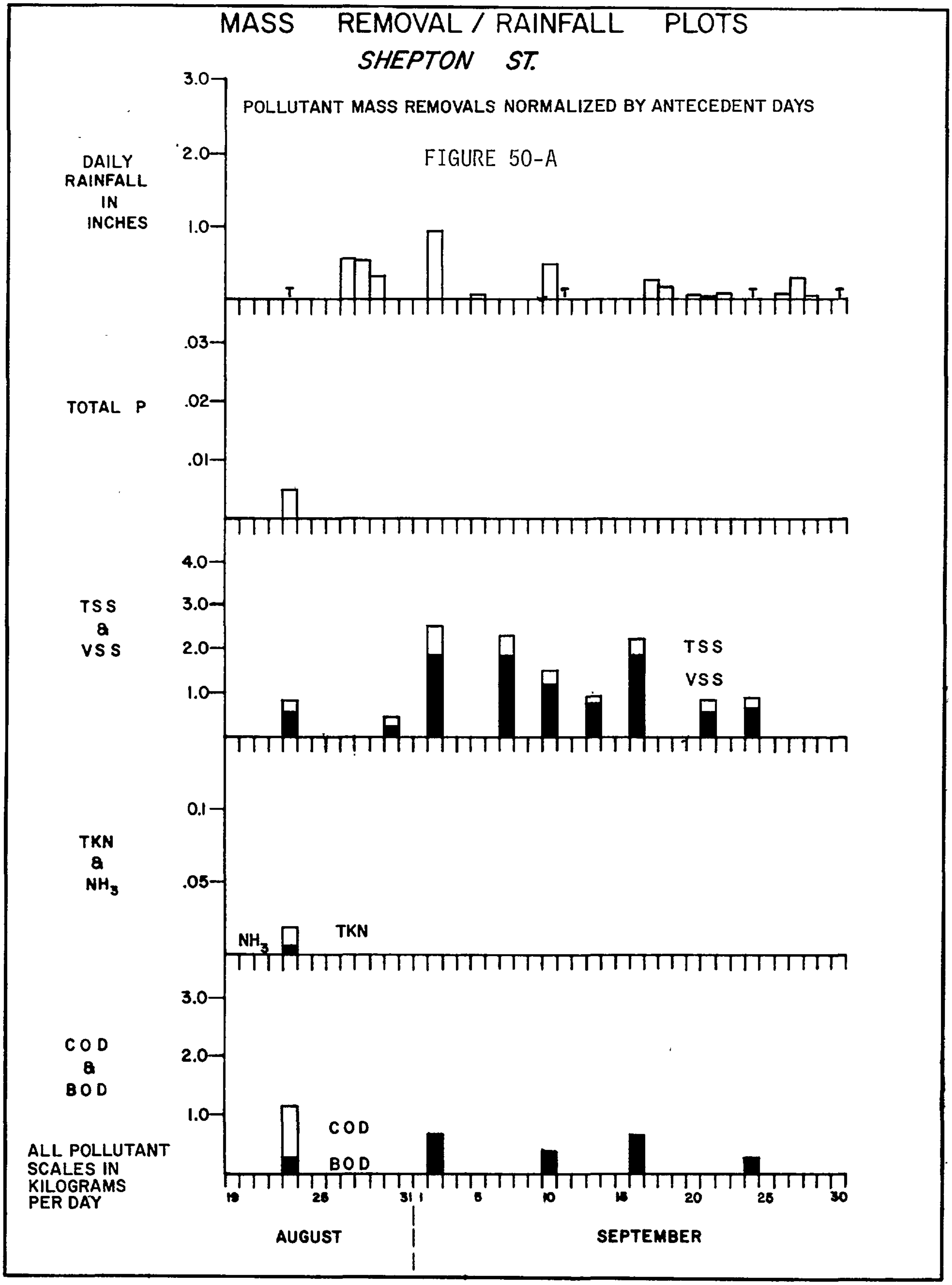
KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

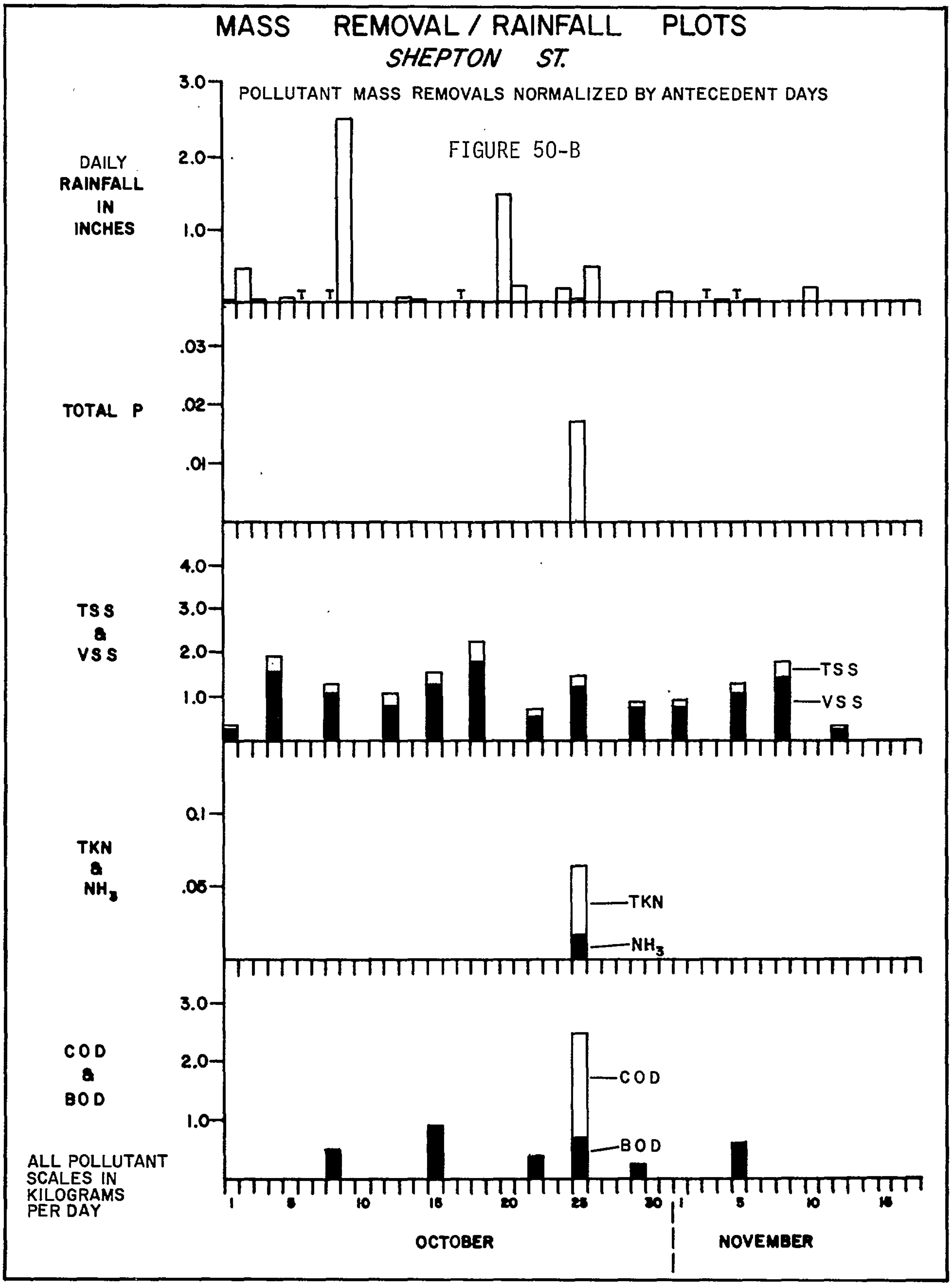


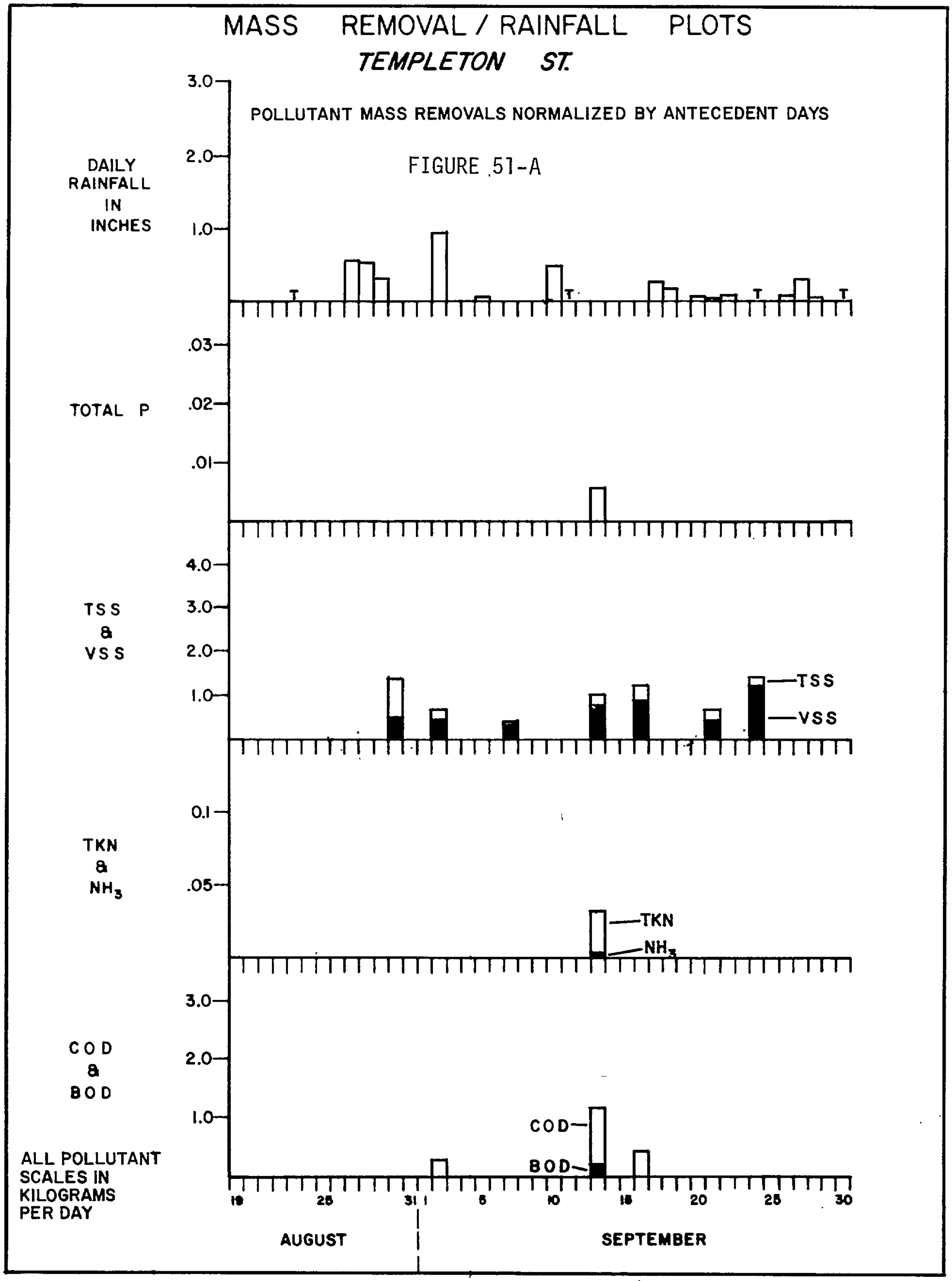
MASS REMOVAL / RAINFALL PLOTS
PORT NORFOLK ST.
POLLUTANT MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS
FIGURE 49-A
DAILY RAINFALL IN INCHES
3.0
2.0
1.0
T
TOTAL P
.03
.02
.01
TSS & VSS
4.0
3.0
2.0
1.0
TSS
VSS
TKN & NH3
0.1
.05
TKN
NH3
COD & BOD
3.0
2.0
1.0
COD
BOD
ALL POLLUTANT SCALES IN KILOGRAMS PER DAY
19
25
31
1
5
10
15
20
25
30
AUGUST
SEPTEMBER



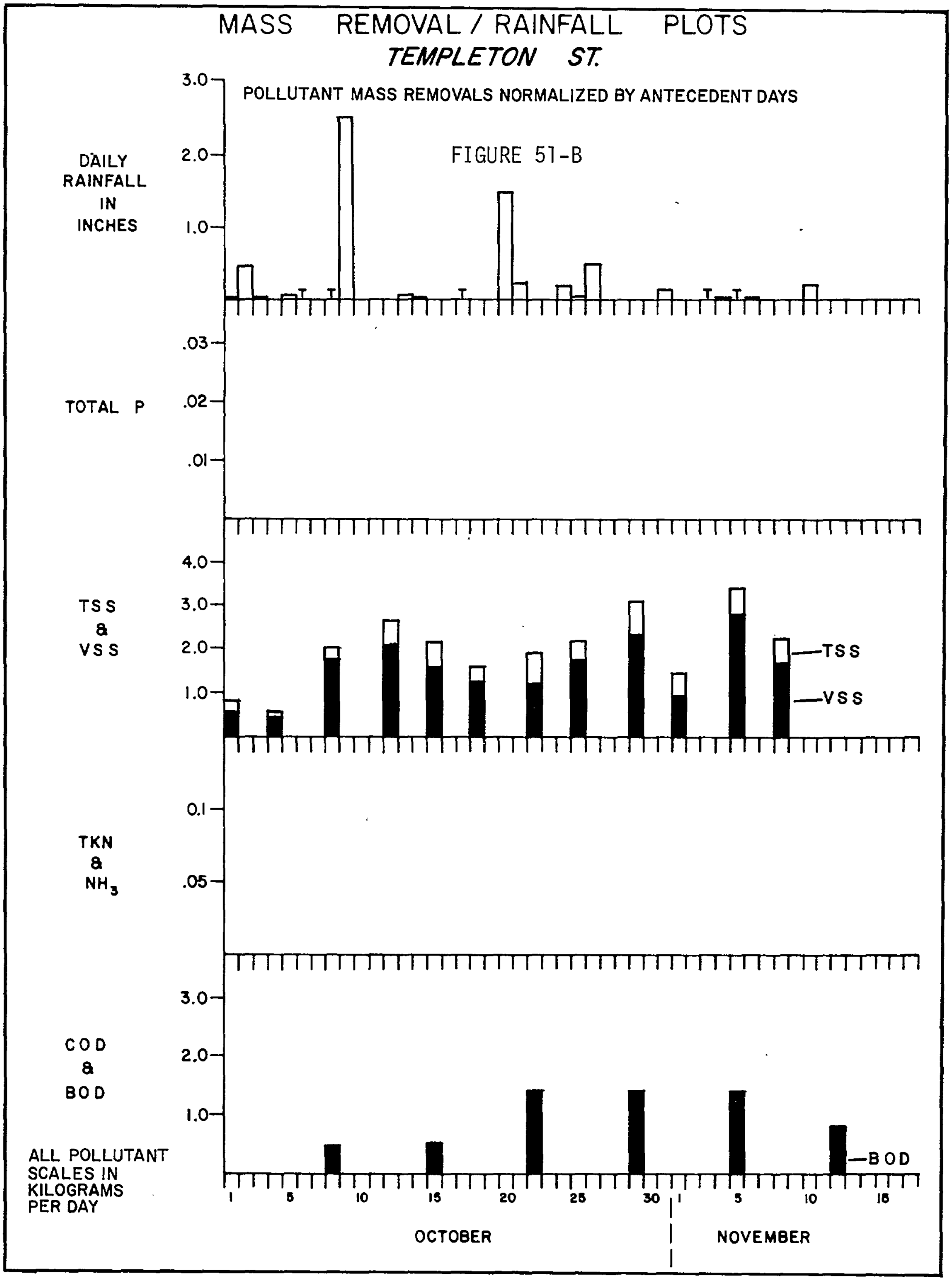
MASS REMOVAL / RAINFALL PLOTS
PORT NORFOLK ST.
POLLUTANT MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS
FIGURE49-B
DAILY RAINFALL IN INCHES
3.0
2.0
1.0
T
TOTAL P
.03
.02
.01
TSS & VSS
9.0
4.0
3.0
2.0
1.0
TSS
VSS
TKN & NH3
0.1
.05
TKN
NH3
COD & BOD
3.0
2.0
1.0
COD
BOD
ALL POLLUTANT SCALES IN KILOGRAMS PER DAY
1
5
10
15
20
25
30
1
5
10
15
OCTOBER
NOVEMBER

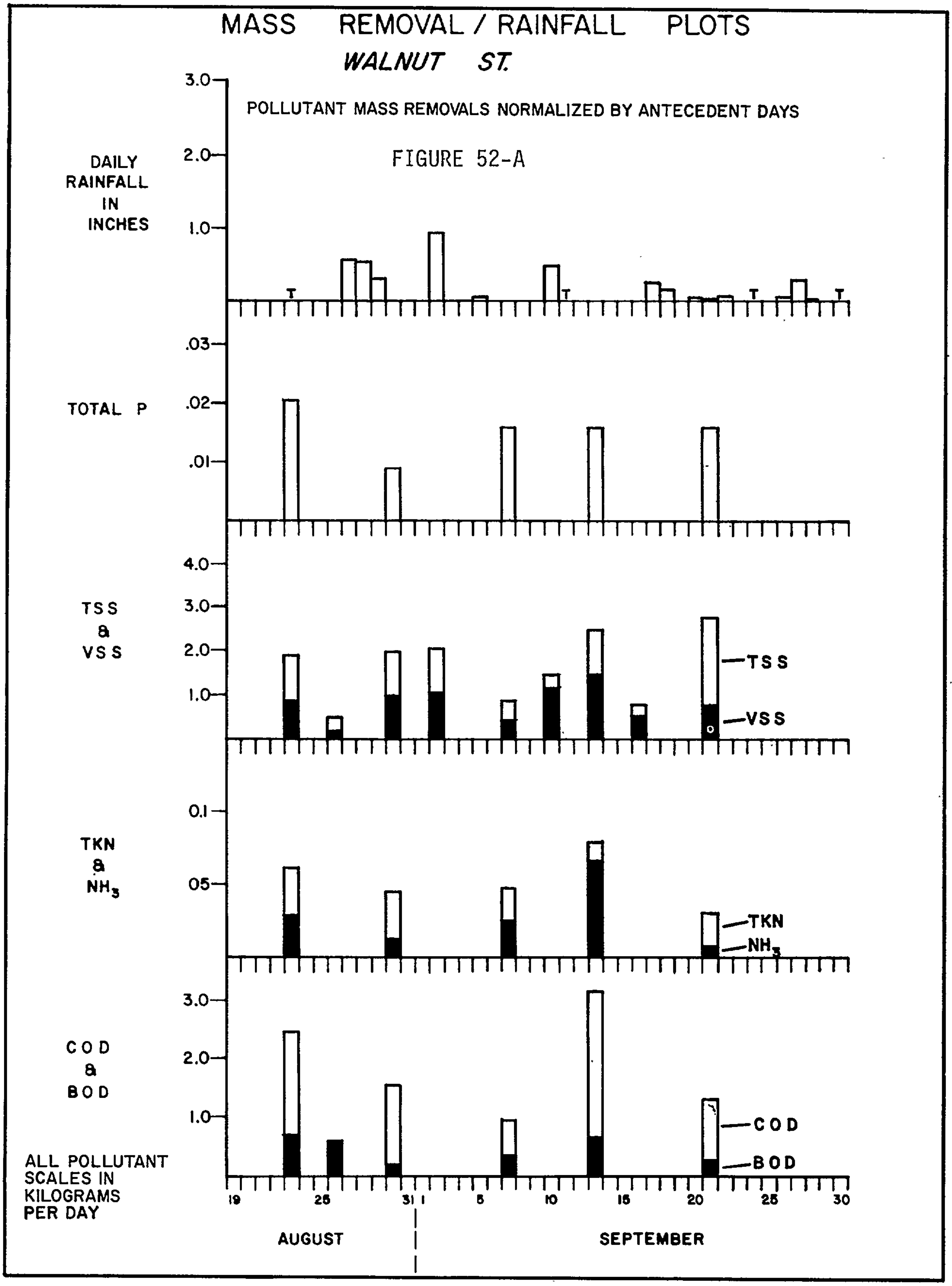

MASS REMOVAL / RAINFALL PLOTS
SHEPTON ST.
POLLUTANT MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS
FIGURE 50-A
DAILY RAINFALL IN INCHES
3.0
2.0
1.0
T
TOTAL P
.03
.02
.01
TSS & VSS
4.0
3.0
2.0
1.0
TSS
VSS
TKN & NH_3
0.1
.05
NH_3
TKN
COD & BOD
3.0
2.0
1.0
COD
BOD
ALL POLLUTANT SCALES IN KILOGRAMS PER DAY
19
25
31
1
5
10
15
20
25
30
AUGUST
SEPTEMBER

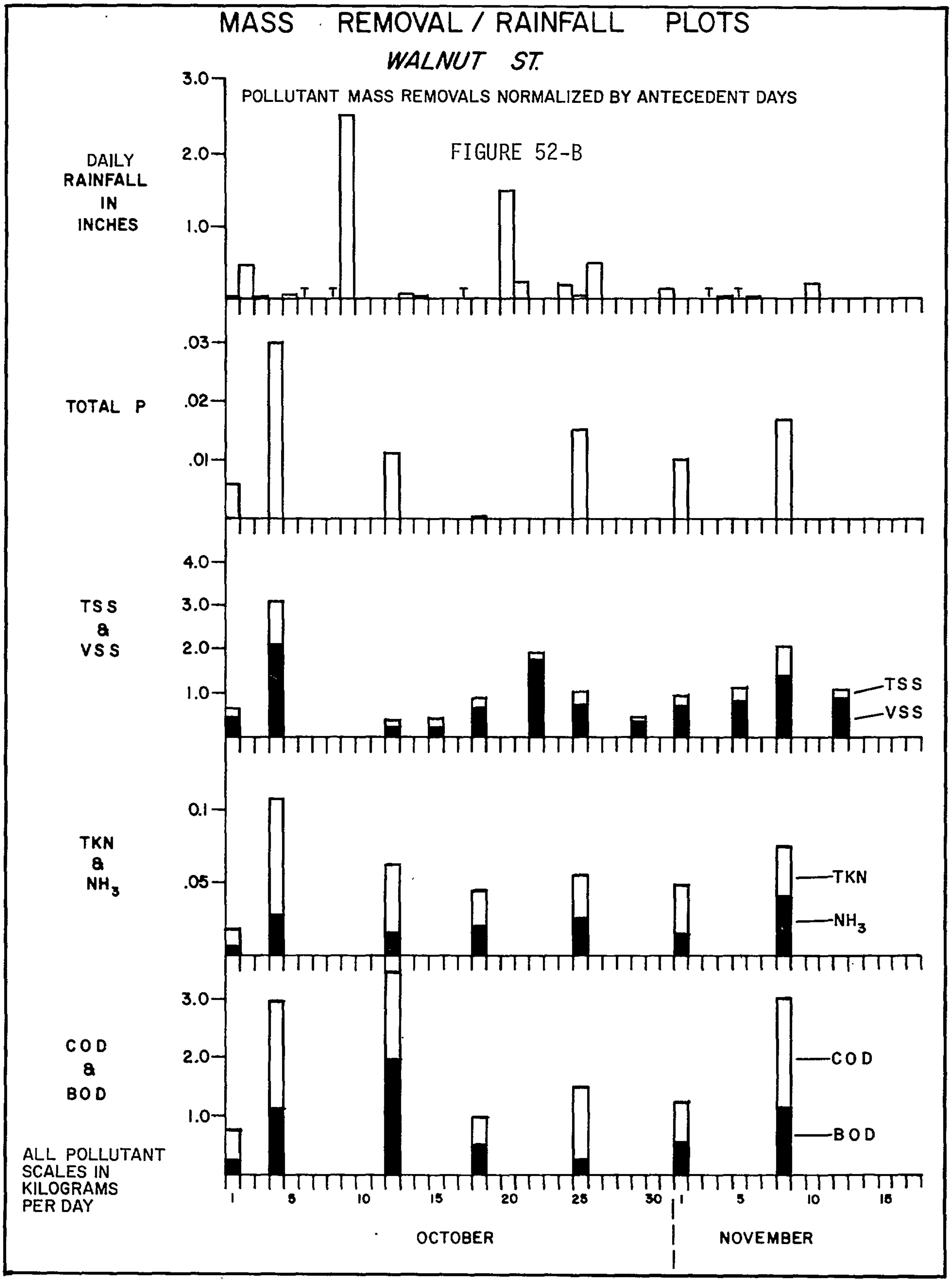

MASS REMOVAL / RAINFALL PLOTS
SHEPTON ST.
POLLUTANT MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS
FIGURE 50-B
DAILY RAINFALL IN INCHES
3.0
2.0
1.0
T
TOTAL P
.03
.02
.01
TSS & VSS
4.0
3.0
2.0
1.0
TSS
VSS
TKN & NH3
0.1
.05
TKN
NH3
COD & BOD
3.0
2.0
1.0
COD
BOD
ALL POLLUTANT SCALES IN KILOGRAMS PER DAY
1
5
10
15
20
25
30
1
5
10
15
OCTOBER
NOVEMBER



MASS REMOVAL / RAINFALL PLOTS
TEMPLETON ST.
POLLUTANT MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS
FIGURE 51-A
DAILY RAINFALL IN INCHES
3.0
2.0
1.0
T
TOTAL P
.03
.02
.01
TSS & VSS
4.0
3.0
2.0
1.0
TSS
VSS
TKN & NH3
0.1
.05
TKN
NH3
COD & BOD
3.0
2.0
1.0
COD
BOD
ALL POLLUTANT SCALES IN KILOGRAMS PER DAY
19
25
31
1
5
10
15
20
25
30
AUGUST
SEPTEMBER


MASS REMOVAL / RAINFALL PLOTS
TEMPLETON ST.
POLLUTANT MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS
FIGURE 51-B
DAILY RAINFALL IN INCHES
3.0
2.0
1.0
TOTAL P
.03
.02
.01
TSS & VSS
4.0
3.0
2.0
1.0
TSS
VSS
TKN & NH3
0.1
.05
COD & BOD
3.0
2.0
1.0
BOD
ALL POLLUTANT SCALES IN KILOGRAMS PER DAY
1
5
10
15
20
25
30
1
5
10
15
OCTOBER
NOVEMBER


MASS REMOVAL / RAINFALL PLOTS
WALNUT ST.
POLLUTANT MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS
FIGURE 52-A
DAILY RAINFALL IN INCHES
3.0
2.0
1.0
T
TOTAL P
.03
.02
.01
TSS & VSS
4.0
3.0
2.0
1.0
TSS
VSS
TKN & NH_3
0.1
05
TKN
NH_3
COD & BOD
3.0
2.0
1.0
COD
BOD
ALL POLLUTANT SCALES IN KILOGRAMS PER DAY
19
25
31 1
5
10
15
20
25
30
AUGUST
SEPTEMBER



MASS REMOVAL / RAINFALL PLOTS
WALNUT ST.
POLLUTANT MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS
FIGURE 52-B
DAILY RAINFALL IN INCHES
TOTAL P
TSS & VSS
TKN & NH_3
COD & BOD
ALL POLLUTANT SCALES IN KILOGRAMS PER DAY
OCTOBER
NOVEMBER

TABLE 23. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED
BY ANTECEDENT DAYS BETWEEN FLUSHES - PORT NORFOLK STREET TEST SEGMENT

TYPE	POLLUTANT REMOVALS (kg/day/flush)						
	COD	BOD	TKN	NH_3	TP	TSS	VSS
A: ALL FLUSHES	1.07 1.04 (9)	0.46 0.46 (10)	0.038 0.041 (9)	0.008 0.008 (9)	0.007 0.008 (9)	1.25 1.31 (19)	0.88 0.88 (19)
B: ALL GOOD * FLUSHES	1.07 1.04 (9)	0.46 0.46 (10)	0.038 0.041 (9)	0.008 0.008 (9)	0.007 0.008 (9)	1.30 1.33 (18)	0.92 0.89 (18)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	0.88 0.58 (5)	0.45 0.40 (6)	0.029 0.018 (5)	0.006 0.004 (5)	0.004 0.004 (5)	0.90 0.44 (11)	0.73 0.42 (11)
D: ALL RAINFALL IMPACTED FLUSHES	1.30 1.52 (4)	0.48 0.60 (4)	0.049 0.062 (4)	0.009 0.012 (4)	0.011 0.011 (4)	1.94 1.97 (7)	1.21 1.33 (7)
E: ALL GOOD NON-RAINFALL IMPACTED FLUSHES UP TO 11/1	0.83 0.64 (3)	0.33 0.18 (4)	0.033 0.018 (4)	0.007 0.005 (4)	0.004 0.004 (4)	0.84 0.31 (8)	0.65 0.26 (8)

(continued)

* See definition in text, page 129.

TABLE 23. Cont. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED
BY ANTECEDENT DAYS BETWEEN FLUSHES - PORT NORFOLK STREET TEST SEGMENT

	POLLUTANT REMOVALS (kg/day/flush)						
TYPE	COD	BOD	TKN	NH_3	TP	TSS	VSS
F:** ALL GOOD NON-RAINFALL IMPACTED FLUSHES (FLUSH RATE COMPARISON)	0.88 0.58 (5)	0.30 0.17 (5)	0.029 0.018 (5)	0.006 0.004 (5)	0.004 0.004 (5)	0.77 0.35 (7)	0.62 0.30 (7)
F:*** ALL GOOD NON-RAINFALL IMPACTED FLUSHES (FLUSH RATE COMPARISON)	-	-	-	-	-	0.87 0.19 (3)	0.66 0.11 (3)
G: ALL GOOD B&R FLUSHES	3.57 - (1)	1.29 0.12 (2)	0.142 - (1)	0.028 - (1)	0.027 - (1)	2.86 1.33 (2)	2.34 0.80 (2)

**Excluding B&R flushes, all good non-rainfall impacted experiments divided into two sets using median of truck delivery rates. Experiments where flush rate is less than or equals median flush rate. (.19 - .48 cfs).

***Excluding B&R flushes, all good non-rainfall impacted experiments divided into two sets using median of truck delivery rates. Experiments where flush rate exceeds median flush rate (.63 -.85 cfs).

KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 24. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED
BY ANTECEDENT DAYS BETWEEN FLUSHES - SHEPTON STREET TEST SEGMENT

TYPE	POLLUTANT REMOVALS (kg/day/flush)						
	COD	BOD	TKN	NH_3	TP	TSS	VSS
A: ALL FLUSHES	1.79 0.91 (2)	0.48 0.22 (11)	0.041 0.031 (2)	0.011 0.007 (2)	0.011 0.008 (2)	1.29 0.65 (22)	1.00 0.53 (22)
B: ALL GOOD* FLUSHES	1.79 0.91 (2)	0.48 0.22 (11)	0.041 0.031 (2)	0.011 0.007 (2)	0.011 0.008 (2)	1.29 0.60 (20)	1.00 0.49 (20)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	1.79 0.91 (2)	0.48 0.24 (8)	0.041 0.031 (2)	0.011 0.007 (2)	0.011 0.008 (2)	1.32 0.59 (12)	1.05 0.50 (12)
D: ALL RAINFALL IMPACTED FLUSHES	-	0.47 0.19 (3)	-	-	-	1.25 0.65 (8)	0.93 0.51 (8)
E: ALL GOOD NON-RAINFALL IMPACTED FLUSHES UP TO 11/1	1.79 0.91 (2)	0.47 0.26 (7)	0.041 0.031 (2)	0.011 0.007 (2)	0.011 0.008 (2)	1.28 0.63 (10)	1.02 0.53 (10)

(continued)

* See definition in text, page 129.

TABLE 24. Cont. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED
BY ANTECEDENT DAYS BETWEEN FLUSHES - SHEPTON STREET TEST SEGMENT

TYPE	POLLUTANT REMOVALS (kg/ day/flush)						
	COD	BOD	TKN	NH_3	TP	TSS	VSS
F:** ALL GOOD NON-RAINFALL IMPACTED FLUSHES (FLUSH RATE COMPARISON)	1.79 0.91 (2)	0.42 0.22 (4)	0.041 0.031 (2)	0.011 0.007 (2)	0.011 0.008 (2)	1.40 0.54 (6)	1.11 0.46 (6)
F:*** ALL GOOD NON-RAINFALL IMPACTED FLUSHES (FLUSH RATE COMPARISON)	-	0.53 0.29 (4)	-	-	-	1.23 0.68 (6)	0.99 0.57 (6)
G: ALL GOOD B&R FLUSHES	-	0.67 - (1)	-	-	-	1.80 1.02 (2)	1.32 0.76 (2)

**Excluding B&R flushes, all good non-rainfall impacted experiments divided into two sets using median of truck delivery rates. Experiments where flush rate is less than or equals median flush rate. (.26 - .47 cfs)

***Excluding B&R flushes, all good non-rainfall impacted experiments divided into two sets using median of truck delivery rates. Experiments where flush rate exceeds median flush rate (.67 - .93 cfs)

KEY: Top Row - Mean

Middle Row - Standard Deviation

Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 25. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED
BY ANTECEDENT DAYS BETWEEN FLUSHES - TEMPLETON STREET TEST SEGMENT

	POLLUTANT REMOVALS (kg/ day/flush)						
TYPE	COD	BOD	TKN	NH_3	T P	TSS	VSS
A: ALL FLUSHES	1.18 - (1)	0.95 0.70 (10)	0.032 - (1)	0.003 - (1)	0.006 - (1)	2.41 1.96 (19)	1.79 1.49 (19)
B. ALL GOOD* FLUSHES	-	1.05 0.74 (8)	-	-	-	2.56 2.08 (16)	1.91 1.58 (16)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	-	1.04 0.52 (5)	-	-	-	2.09 1.04 (10)	1.57 0.79 (10)
D: ALL RAINFALL IMPACTED FLUSHES	1.18 - (1)	0.85 1.04 (4)	0.032 - (1)	0.003 - (1)	0.006 - (1)	3.01 2.99 (7)	2.22 2.29 (7)
E: ALL GOOD NON-RAINFALL IMPACTED FLUSHES UP TO 11/1	-	0.90 0.57 (3)	-	-	-	2.23 1.10 (8)	1.66 0.82 (8)

(continued)

* See definition in text, page 129.

TABLE 25. Cont. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED
BY ANTECEDENT DAYS BETWEEN FLUSHES - TEMPLETON STREET TEST SEGMENT

TYPE	POLLUTANT REMOVALS (kg/ day/flush)						
	COD	BOD	TKN	NH_3	TP	TSS	VSS
F: ** ALL GOOD NON-RAINFALL IMPACTED FLUSHES(FLUSH RATE COMPARISON)	-	0.64 0.47 (2)	-	-	-	1.33 0.53 (5)	1.00 0.37 (5)
F:*** ALL GOOD NON-RAINFALL IMPACTED FLUSHES (FLUSH RATE COMPARISON)	-	1.15 0.39 (2)	-	-	-	3.09 0.76 (4)	2.35 0.55 (4)
G: ALL GOOD B&R FLUSHES	-	1.64 - (1)	-	-	-	1.73 0.22 (2)	1.16 0.27 (2)

** Excluding B&R flushes, all good non-rainfall impacted experiments divided into two sets using median of truck delivery rates. Experiments where flush rate is less than or equals median flush rate. (.36 - .49 cfs).

*** Excluding B&R flushes, all good non-rainfall impacted experiments divided into two sets using median of truck delivery rates. Experiments where flush rate exceeds median flush rate (.56 - .83 cfs)

KEY: Top Row - Mean

Middle Row - Standard Deviation

Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 26. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED
BY ANTECEDENT DAYS BETWEEN FLUSHES - WALNUT STREET TEST SEGMENT

	POLLUTANT REMOVALS (kg/ day/flush)						
TYPE	COD	BOD	TKN	NH_3	TP	TSS	VSS
A: ALL FLUSHES	2.32 1.95 (12)	0.78 0.56 (13)	0.062 0.032 (12)	0.025 0.017 (12)	0.015 0.008 (12)	1.66 1.52 (21)	1.02 0.88 (21)
B: ALL GOOD * FLUSHES	2.32 1.95 (12)	0.78 0.56 (13)	0.062 0.032 (12)	0.025 0.017 (12)	0.015 0.008 (12)	1.72 1.62 (18)	1.04 0.95 (18)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	1.56 0.87 (7)	0.55 0.27 (8)	0.051 0.018 (7)	0.023 0.012 (7)	0.012 0.007 (7)	0.99 0.56 (12)	0.64 0.38 (12)
D: ALL RAINFALL IMPACTED FLUSHES	3.38 2.64 (5)	1.16 0.72 (5)	0.079 0.042 (5)	0.029 0.023 (5)	0.019 0.008 (5)	3.03 1.95 (7)	1.75 1.17 (7)
E: ALL GOOD NON-RAINFALL IMPACTED FLUSHES UP TO 10/29	1.30 0.72 (5)	0.45 0.17 (6)	0.046 0.068 (5)	0.021 0.010 (5)	0.012 0.008 (5)	0.86 0.50 (9)	0.54 0.33 (9)

(continued)

* See definition in text, page 129.

TABLE 26. Cont. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED
BY ANTECEDENT DAYS BETWEEN FLUSHES - WALNUT STREET TEST SEGMENT

TYPE	POLLUTANT REMOVALS (kg/day/flush)						
	COD	BOD	TKN	NH_3	TP	TSS	VSS
F:** ALL GOOD NON-RAINFALL IMPACTED FLUSHES (FLUSH RATE COMPARISON)	1.00 0.36 (3)	0.41 0.15 (3)	0.038 0.016 (3)	0.013 0.008 (3)	0.006 0.005 (3)	0.79 0.37 (7)	0.59 0.34 (7)
F:*** ALL GOOD NON-RAINFALL IMPACTED FLUSHES (FLUSH RATE COMPARISON)	2.33 0.78 (3)	0.71 0.30 (4)	0.065 0.012 (3)	0.032 0.010 (3)	0.018 0.003 (3)	1.38 0.76 (4)	0.78 0.50 (4)
G: ALL GOOD B&R FLUSHES	0.09 - (1)	0.34 - (1)	0.048 - (1)	0.026 - (1)	0.016 - (1)	0.81 - (1)	0.36 - (1)

**Excluding B&R flushes, all good non-rainfall impacted experiments divided into two sets using median of truck delivery rates. Experiments where flush rate is less than or equals median flush rate. (.39 - .50 cfs).

***Excluding B&R flushes, all good non-rainfall impacted experiments divided into two sets using median of truck delivery rates. Experiments where flush rate exceeds median flush rate (.80 - .95 cfs).

KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

flush removal data. Statistics for each of the groupings in these tables are presented as follows: A) all flushes; B) all good operational flushes; C) all good non-rainfall impacted flushes; D) all rainfall impacted flushes; E) all good non-rainfall impacted flushes with post cleaning performed; F) all good non-rainfall impacted flushes where the experimental flush rates are first less than the median flush rate of the experiments and then those flushes where the flush rates exceed the median flush rate; and G) all backup and release flushes. Comparison of groupings C and D show the impact of rainfall within the periods between flushes. Comparison of groupings C and E show the net effects of post cleaning the segments, or equivalently, the gross overall degree of flushing effectiveness assuming that the post cleaning operation removes all residual pollutants after flushing. Close agreement between these two groupings would suggest high flushing removal efficiency. The comparison within groupings F roughly show the impact of increased flush rate on pollutant removals. Partitioning the sample set on the basis of the median flush rate was arbitrary.

Several general observations can be drawn from inspection of the normalized pollutant removal statistics shown in Tables 23 through 26. First of all, the coefficients of variation are all between 0.5 to 1.0 indicating that the variation of removal about the average is small. Secondly, comparison of the categories C and D statistics for the combined sewer laterals on Port Norfolk and Walnut Streets show that the flushed loadings for events impacted by rainfall events exceed the non-rainfall impacted flushed masses by 50 to 100 %.

Similar comparison on the separate sewer streets shows mixed results. The normalized rainfall impacted loadings for Shepton Street generally decreased in comparison to the non-rainfall impacted events. The results reversed for Templeton Street in that the normalized TSS and VSS wet weather impacted flush loadings increased. One explanation for these results is the following. Both streets receive clear water inflow from roof drains connected into the sewers. The segments on both streets are flat but are at the foot of a hill. Materials could either be washed into the segment, thereby increasing the deposition loading or, washed out of the segment, depending on the relative intensity of prior storm events. Visual inspections of both segments during wet weather indicated that the flow in Templeton Street was sluggish and under slight backwater conditions from the main trunk sewer on Florida Street while the flow in Shepton Street discharged freely. Materials would "wash-out" during storm events at Shepton Street and settle more rapidly at Templeton Street. There was no backwater effect at Templeton Street during dry weather conditions.

The third general observation of groupings C and E in Tables 23 through 26 is that the post flush cleaning operation with the exception of Templeton Street reduced the average pollutant removals suggesting that the flushing experiments were in general extremely effective. The issue of flushing effectiveness is addressed more rigoriously in the phase two serial flushing program described in Chapter 9.

The fourth observation of the data is that the average flushing removal rates were greater for the higher flush rates. An analysis of flushing method effectiveness is presented in Section 8.5. The final observation of the data is that the flushed loadings for the backup and release experiments were comparable to the other removal rates. The backup and release flush experiments at Shepton Street were extremely favorable and were much higher than the average removals for good non-rainfall impacted experiments. This result led to the placement of the automatic sewer flushing module on this segment. Operational results of the automatic module is described in Chapter 11.

The average normalized flushed pollutant loadings presented in Tables 23 through 26 for each segment were again normalized by estimates of the total upstream tributary population including population contributions along the segment. Two estimates of population for the Port Norfolk Street segment were used. The census information indicated a population of 94 people while dry weather flow results described in Chapter 11 indicated that a population estimate of 61 people would be more reasonable. The average normalized results per segment by antecedent period and population are given in Table 27 for all good flushes and for all good operational non-rainfall impacted events. The results for the two separated and the two combined sewer streets were averaged and are presented in the last two rows of Table 27. The average flushed loads for the combined sewer streets are generally two to three times the loads for the separate streets.

8.4 Heavy Metals Flush Removals

Heavy metals were determined for the solids fraction of settled flush wave composites including: cadmium, chromium, copper, lead, nickel, zinc and mercury. Sample handling procedures for the composited flush wave samples are discussed in Chapter 6. Preliminary heavy metal analyses for both the supernatant and settled solids fractions showed that the heavy metals within the flush wave supernatant were extremely low, on the order of less than half a part per billion range. Flush wave composites were settled for four hours under ideal quiescent conditions. The purpose of the heavy metals tests conducted in this phase of work was aimed at assessing dissolved and settleable fractions under ideal conditions. The heavy metals analyses performed as part of the settling column experiments in the second phase program portray a more realistic assessment of heavy metals settleability characteristics. Those experiments showed that roughly half of the flushed heavy metals would rapidly settle therefore, associated with heavier solids particles and that the remaining fraction would not settle within the time period of the settling column test and therefore would be transported to a treatment facility. Results of those experiments are presented and discussed in Chapter 10.

Results of the heavy metals analyses for the settled solids fraction of the flush wave composites are presented in Tables 28 through 31 for each of the test segments. Heavy metal results are presented in terms of micrograms per kilogram of total dry suspended solids. Table 32 summarizes the results given in Tables 28 through 31 in terms of the minimum, average and maximum flushed metals per solid rates for each test segment. High zinc and copper levels may be attributable to the copper and brass piping used in many of the older residences in the area. Statistical

TABLE 27. SUMMARY OF AVERAGE PHASE I FLUSHING POLLUTANT REMOVALS NORMALIZED BY ANTECEDENT DAYS BETWEEN FLUSHES AND BY ESTIMATED TRIBUTARY POPULATION

	POLLUTANT REMOVALS (grams/day/capita/flush)						
SHEPTON STREET (Separated) (Estimated Tributary Population = 230)*							
TYPE	COD	BOD	TKN	NH_3	TP	TSS	VSS
ALL GOOD FLUSHES	7.78 (2)	2.10 (11)	0.178 (2)	0.048 (2)	0.048 (2)	5.60 (20)	4.35 (20)
ALL GOOD NON-RAINFALL IMPACTED FLUSHES	7.78 (2)	2.08 (8)	0.178 (2)	0.048 (2)	0.048 (2)	5.72 (12)	4.57 (12)
TEMPLETON STREET (Separated) (Estimated Tributary Population = 221)*							
ALL GOOD FLUSHES	-	4.75 (8)	-	-	-	11.58 (16)	8.64 (16)
ALL GOOD NON-RAINFALL IMPACTED FLUSHES	-	4.71 (5)	-	-	-	9.46 (10)	7.10 (10)
WALNUT STREET (Combined) (Estimated Tributary Population = 71)*							
ALL GOOD FLUSHES	32.64 (12)	11.02 (13)	0.873 (12)	0.352 (12)	0.211 (12)	24.23 (18)	14.65 (18)
ALL GOOD NON-RAINFALL IMPACTED FLUSHES	21.97 (7)	7.72 (8)	0.718 (7)	0.324 (7)	0.169 (7)	13.90 (12)	8.97 (12)

(continued)

TABLE 27. Cont. SUMMARY OF AVERAGE PHASE I FLUSHING POLLUTANT REMOVALS NORMALIZED BY ANTECEDENT DAYS BETWEEN FLUSHES AND BY ESTIMATED TRIBUTARY POPULATION

PORT NORFOLK STREET (Combined) (Estimated Tributary Population = 94*; Estimated Population using DWF considerations = 61)	POLLUTANT REMOVALS (grams/day/capita/flush)						
TYPE	COD	BOD	TKN	NH_3	TP	TSS	VSS
ALL GOOD FLUSHES POP.=94	11.38 (9)	4.94 (10)	0.404 (9)	0.079 (9)	0.074 (9)	13.83 (18)	9.77 (18)
ALL GOOD FLUSHES POP.=61	17.54 (9)	7.61 (10)	0.623 (9)	0.123 (9)	0.115 (9)	21.3 (18)	15.06 (18)
ALL GOOD NON-RAINFALL IMPACTED FLUSHES POP.=94	9.30 (5)	4.819 (6)	0.308 (5)	0.064 (5)	0.043 (5)	9.57 (11)	7.79 (11)
ALL GOOD NON-RAINFALL IMPACTED FLUSHES POP.=61	14.34 (5)	7.43 (6)	0.475 (5)	0.098 (5)	0.067 (5)	14.75 (11)	12.00 (11)
SUMMARY							
SEPARATED STREETS ALL GOOD FLUSHES (AVERAGE)	7.78	3.43	0.178	0.048	0.048	8.59	6.50
COMBINED STREETS ALL GOOD FLUSHES (AVERAGE)**	22.01 - 25.09	7.98 - 9.32	0.639 - 0.748	0.215 - 0.238	0.143 - 0.163	19.03 - 22.77	12.21 - 14.86

*Population from census information.

**Range reflects various tributary population estimates for Port Norfolk Street.

TABLE 28. PHASE I FIELD FLUSHING HEAVY METALS REMOVALS PER UNIT MASS OF SOLIDS FLUSHED - PORT NORFOLK STREET TEST SEGMENT

								MASS (µg OF METALS/kg TOTAL SUSPENDED SOLIDS)						
DATE	A	B	C	D	E	F	G	Cd	Cr	Cu	Pb	Ni	Zi	Hg
8/30/76	32.9	.47	1	x	20	x	x							
9/2	46.5	.92	3	x	2	x	x	6.00	5.5	1517	42.4	28.8	1261	10.70
9/7	32.8	.48	5			x	x	6.28	34.8	1093	137.0	28.6	1155	-
9/16	47.0	.44	9			x		-	-	-	-	-	-	-
9/21	32.6	.82	5	x	82	x	x	6.73	-	334	244.9	48.9	1510	-
9/24	46.6	.79	3			x	x	4.44	41.7	375	269.7	54.7	1293	3.16
10/1	33.7	.19	7			x	x	2.09	40.2	322	138.9	12.3	689	1.08
10/4	B&R***		3	x	41	x	x	7.72	9.7	807	8.3	27.8	1533	-
10/8	32.9	.46	4	x	64	x	x	8.31	60.1	134	559.0	5.8	2042	3.50
10/12	33.4	.27	4	x	72	x	x	2.72	37.0	308	454.0	21.0	282	0.50
10/15	33.1	.33	3			x	x	2.26	39.7	255	60.4	12.7	892	-
10/18	47.0	.42	3			x	x	8.17	54.8	411	624.2	44.6	1180	1.71
10/25	32.7	.63	3			x	x	-	-	-	-	-	-	-
10/22	46.6	.71	4	x	37	x	x	-	-	-	-	-	-	-
10/29	32.9	.46	4			x	x	6.17	-	252	209.7	22.2	855	1.72
11/1	47.0	.42	3				x	2.37	2.9	47	43.3	5.3	129	2.11
11/5	46.6	.85	4				x	2.69	33.7	203	126.9	42.4	817	2.08
11/8	32.9	.47	3				x	3.78	31.5	366	275.6	28.4	948	-
11/12	B&R***		4				x	6.97	31.7	412	338.0	19.9	1172	-

Legend
A - Truck Flush Volume (cf)
B - Truck Flush Rate (cfs)
C - Total Antecedent Days Between Flushes
D - Rainfall Impacted Event
E - Hours After Rainfall Event to Flush*
F - Segment Cleaned After Flush**
G - Good Flush

*Number of hours between rainfall exceeding 0.15 inch/hour and flush event.
**Segment flushed after experiment using maximum truck discharge for five minutes.
***Backup and release.

TABLE 29: PHASE I FIELD FLUSHING HEAVY METALS REMOVALS PER UNIT MASS OF SOLIDS FLUSHED - SHEPTON STREET TEST SEGMENT

								MASS (μg OF METALS/kg TOTAL SUSPENDED SOLIDS)						
DATE	A	B	C	D	E	F	G	Cd	Cr	Cu	Pb	Ni	Zi	Hg
8/23/76	47.8	.26	5			x	x							
8/26	35.4	.76	3			x	x							
8/30	37.3	.65	4	x	23	x	x							
9/2	B&R***		3	x	4	x	x	1.71	33.4	97	61.7	6.9	20	-
9/7	20.0		5			x		-	-	-	-	-	-	-
9/10	32.6	.69	3			x	x	7.57	56.8	353	298.4	25.5	1310	-
9/13	B&R***		3	x	66	x	x	4.48	37.4	221	232.2	26.9	1210	-
9/16	46.6	.67	3			x	x	4.08	31.9	539	98.3	19.7	1058	3.65
9/21	32.9	.42	5	x	86	x	x	5.34	40.1	348	570.7	3260.0	1306	-
9/24	33.1	.37	3			x	x	-	-	-	-	-	-	-
10/1	46.5	.93	7			x	x	4.10	27.5	214	44.4	23.1	13	3.47
10/4	32.9	.47	3	x	45	x	x	3.88	34.7	242	129.0	12.0	722	-
10/8	32.9	.46	4	x	60	x	x	2.72	18.3	252	269.0	21.0	770	-
10/12	B&R***		4	x	75	x	x	6.01	52.6	196	524.2	37.8	938	4.13
10/15	46.6	.71	3			x	x	2.25	29.2	188	45.0	11.9	549	-
10/18	33.0	.47	3			x	x	3.72	31.0	280	300.8	16.7	1074	3.90
10/22	32.9	.43	4	x	35	x	x	4.85	53.0	216	235.5	10.9	911	4.40
10/25	47.2	.37	3			x	x	2.84	15.4	175	15.9	13.1	407	3.50
10/29	46.6	.72	4			x	x	5.56	52.8	216	161.7	20.0	899	2.48
11/1	32.7	.61	3				x	-	-	-	-	-	-	-
11/5	33.0	.40	4				x	3.93	32.7	239	190.7	20.6	907	1.67
11/8	47.1	.41	3				x	5.80	13.8	357	70.4	26.1	550	1.70
11/12	B&R***		4					2.50	23.4	161	121.4	11.2	975	1.70

Legend

A - Truck Flush Volume (cf)
B - Truck Flush Rate (cfs)
C - Total Antecedent Days Between Flushes
D - Rainfall Impacted Event
E - Hours After Rainfall Event to Flush*
F - Segment Cleaned After Flush**
G - Good Flush

*Number of hours between rainfall exceeding 0.15 inch/hour and flush event.

**Segment flushed after experiment using maximum truck discharge for five minutes.

***Backup and release.

TABLE 30. PHASE I FIELD FLUSHING HEAVY METALS REMOVALS PER UNIT MASS OF SOLIDS FLUSHED - TEMPLETON STREET TEST SEGMENT

								MASS (μg OF METALS/kg TOTAL SUSPENDED SOLIDS)						
DATE	A	B	C	D	E	F	G	Cd	Cr	Cu	Pb	Ni	Zi	Hg
9/2	32.8	.50	3	x	5	x	x	4.11	29.5	1565	169.0	21.6	1572	2.62
9/7	B&R***		5			x		6.72	-	246	128.0	20.4	31	-
9/10	57.4	.83	3			x	x	5.77	45.5	318	233.6	28.1	2230	-
9/13	32.9	.43	3	x	65	x		8.63	47.9	385	372.5	25.9	1110	-
9/16	B&R***		3			x		8.93	51.0	1172	52.1	46.4	1438	-
9/21	B&R***		5	x	87	x	x	7.65	35.8	370	684.2	34.4	1280	-
9/24	33.1	.36	3			x	x	3.12	19.5	120	75.7	14.0	361	5.24
10/1	B&R***		7			x	x	7.51	40.2	354	598.9	55.8	1116	-
10/4	46.6	.87	3	x	43	x	x	5.05	98.4	643	315.0	28.4	352	13.1
10/8	32.9	.46	4	x	62	x	x	2.56	33.0	259	238.5	18.1	845	14.0
10/12	32.9	.43	4	x	73	x	x	2.04	30.8	233	280.0	8.9	525	-
10/15	47.1	.41	3			x	x	8.40	58.1	295	410.2	25.2	607	3.75
10/18	46.6	.76	4			x	x	3.56	35.6	823	62.9	18.8	854	-
10/22	32.7	.59	3	x	34	x	x	3.13	43.0	211	68.3	22.5	975	2.62
10/25	32.9	.46	4			x	x	-	-	-	-	-	-	-
10/29	46.9	.49	3			x	x	3.75	39.1	254	242.8	22.5	578	1.71
11/1	46.7	.75	3				x	7.90	14.1	141	548.1	45.7	314	2.67
11/5	32.7	.56	4				x	3.02	25.2	339	244.5	22.7	786	-
11/8	32.9	.43	3				x	4.14	31.0	274	355.2	12.4	718	-
11/12	B&R***		4				x	18.90	35.7	195	277.4	28.6	601	-

Legend
A - Truck Flush Volume (cf)
B - Truck Flush Rate (cfs)
C - Total Antecedent Days Between Flushes
D - Rainfall Impacted Event
E - Hours After Rainfall Event to Flush*
F - Segment Cleaned After Flush**
G - Good Flush

*Number of hours between rainfall exceeding 0.15 inch/hour and flush event.
**Segment flushed after experiment using maximum truck discharge for five minutes.
***Backup and release.

TABLE 31. PHASE I FIELD FLUSHING HEAVY METALS REMOVALS PER UNIT MASS OF SOLIDS FLUSHED - WALNUT STREET TEST SEGMENT

								MASS (µg OF METALS/kg TOTAL SUSPENDED SOLIDS)						
DATE	A	B	C	D	E	F	G	Cd	Cr	Cu	Pb	Ni	Zi	Hg
8/23/76	69.8	.80	3			x	x							
8/26	46.6	.86	3			x	x							
8/30	47.1	.41	4	x	21	x	x	5.82	30.3	1883	394.0	34.8	1295	5.29
9/2	60.6	.65	3	x	1	x		9.08	69.9	2670	711.0	62.4	2262	-
9/7	B&R***		3			x	x	7.58	79.6	1063	246.0	76.2	2145	-
9/10	32.0	.47	3			x	x	7.70	52.5	456	186.9	50.4	1540	-
9/13	46.5	.96	3	x	62	x	x	8.16	27.7	1722	874.0	63.0	1629	6.33
9/16	32.0	1.39	3			x		9.03	98.9	863	226.0	20.3	1680	8.99
9/21	46.6	.89	5	x	84	x	x							
9/24	B&R***													
10/1	33.0	.39*	7			x	x	5.81	52.8	395	615.5	47.3	1710	5.44
10/4	32.9	.46	3	x	40	x	x	5.67	56.9	801	50.6	35.5	904	18.8
10/8	B&R***		4	x										
10/12	47.0	.98	4	x	71	x	x	8.16	56.6	343	176.0	48.7	1360	3.19
10/15	32.8	.50	3			x	x	4.40	14.7	87	99.7	9.9	371	5.0
10/18	33.0	.42	3			x	x	7.63	35.6	83	144.0	35.3	1419	-
10/22	47.3	.35	4	x	38	x	x	5.80	45.3	284	387.4	39.9	1130	18.89
10/25	46.5	.95	3			x	x	5.03	26.6	74	108.0	45.2	1105	-
10/29	32.9	.47	4				x	4.67	87.5	151	70.9	15.7	10009	11.44
11/1	32.9	.43	3				x	3.94	20.7	180	191.6	70.7	1750	5.29
11/5	47.0	.42	4				x	8.56	45.9	437	296.5	84.9	1550	3.83
11/8	46.6	.88	3				x	7.20	32.7	249	285.9	58.5	993	3.33
11/12	B&R***		4					-	-	-	-	-	-	-

Legend
A - Truck Flush Volume (cf)
B - Truck Flush Rate (cfs)
C - Total Antecedent Days Between Flushes
D - Rainfall Impacted Event
E - Hours After Rainfall Event to Flush*
F - Segment Cleaned After Flush**
G - Good Flush

*Number of hours between rainfall exceeding 0.15 inch/hour and flush event.
**Segment flushed after experiment using maxium truck discharge for five minutes.
***Backup and release.

TABLE 32. SUMMARY OF PHASE I HEAVY METALS MASS LOADINGS
PER UNIT MASS OF SOLIDS FLUSHED
HEAVY METALS RATES (μg/kg OF SOLIDS)

	SHEPTON			TEMPLETON			PORT NORFOLK			WALNUT		
	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.
CADMIUM	1.7	4.2	7.5	2.0	6.0	18.9	2.1	5.1	8.3	3.9	6.7	9.1
CHROMIUM	13.8	34.4	56.8	14.1	39.6	98.4	2.9	32.6	60.1	14.7	49.1	98.9
COPPER	175.0	252.6	539.0	120.0	431.4	1565.0	47.0	455.7	1517.0	73.6	690.6	2670.0
LEAD	15.9	198.2	570.7	52.1	281.9	684.2	8.3	235.5	624.2	50.6	297.9	615.0
NICKEL	10.9	18.9*	3260.0	12.4	26.3	55.8	5.3	26.9	54.7	9.9	47.0	84.9
ZINC	13.3	810.0	1360.0	31.0	857.5	1577.0	129.0	1050.5	1533.0	371.0	1932.5	2262.0
MERCURY	1.7	3.1	3.65	1.7	5.7	14.0	0.5	2.9	10.7	3.2	8.0	18.9

*Excludes high nickel concentration flush - 9/21 - (3260 mg/kg) summaries of the total heavy metals mass removals, in micrograms per flush, are presented in Tables 33 through 36 for each of the test segments. The mean, standard deviation and number of flushing experiments used in the computations are given for each pollutant. The data set is again partitioned into four groupings including all flushes, all good flushes, all good non-rainfall impacted flushes and rainfall impacted flushes. These results were computed using the solids mass removals given in Tables 19 through 22 and the heavy metals solids concentration given in Tables 28 through 31 for each test segment. The coefficients of variation range from about 0.5 to 1.5 which are low considering the unknown origin of the pollutant sources.

Heavy metals results normalized by antecedent periods between flushes for each pollutant and data groups are given in Tables 37 through 40 for each test segment. Inspection of the results shows that the heavy metals loadings for the combined sewer streets are generally much higher than for the separated sewer segments. Table 41 presents a summary of the heavy metals removals both normalized by antecedent days between flushes and estimates of the tributary population. Again these results show significant differences between the combined and separated sewer segments. The results are otherwise scattered to make any other general observations.

8.5 Comparison of Flush Methods

One aim of the first phase flushing operation was to assess pollutant removal effectiveness of different flushing methods for small diameter laterals. Plots are presented in Figure 53 depicting flushed TSS and VSS loadings normalized by antecedent periods between experiments versus flush discharge rate for all good operational flushes for the Shepton and Templeton Streets test segments. Similar plots for Port Norfolk and Walnut Streets are shown in Figure 54. Flush volumes are grouped into 35 and 50 cubic feet categories and differences between nonrainfall and rainfall impacted flushes* are also noted. Finally, results for backup and release flushes are also noted in the plots for both nonrainfall and rainfall impacted flushes.

Visual inspection of the plots indicates that the TSS and VSS removals are roughly invariant of the method used but tend to increase with both the higher flush rates and the higher flush volumes. This observation was derived in the following manner. The TSS and VSS flushed masses normalized by antecedent periods for all good nonrainfall impacted flushes shown in Figures 53 and 54 were compared with their respective averages for each street. Backup and release flushes were excluded from the comparison. The flushing experiments were arbitrarily divided into a four-way categorization by flush volume and flush rate, that is, flush rates less than 0.5 cfs and flush rates exceeding 0.5 cfs, and flush volumes approximately equal to 35 and 50 cubic feet, respectively. These categories are shown below and are labeled as low volume/low rate, low volume/high rate, high volume/low rate, and high volume/high rate. The total number of experiments where the normalized TSS and VSS flushed masses exceeded the good nonrainfall impacted TSS and VSS averages per segment over all four segments were tallied and are noted below. The total number of events used in this comparison is 41 flushes with about an equal division between the four categories. Inspection

* See definition on page 129.

of the table below shows that the greater preponderance of flush masses exceeding the mean TSS and VSS mass removals per street occurs within the high volume/high rate flush category.This implies that the experiments with flushing volumes of about 50 cubic feet and rates exceeding 0.5 cfs were more effective in comparison to the average removals than were, for example, low volume/low rate flushes. This flush category typified both high flush energy and momentum. Another observation that can be drawn from the table is that pollutant removals exceeded the average more frequently for the higher flush volume experiments (independent of rate) than for the higher flush rate experiments (independent of volume). The optimal condition, however, is for both high rates and high volumes.

NUMBER OF GOOD NONRAINFALL IMPACTED TSS AND VSS NORMALIZED FLUSH MASSES EXCEEDING AVERAGE REMOVALS PER STREET FOR DIFFERENT FLUSHING CONDITIONS	
Low Volume/Low Rate	Low Volume/High Rate
TSS: 2	TSS: 2
VSS: 4	VSS: 2
High Volume/Low Rate	High Volume/High Rate
TSS: 6	TSS: 10
VSS: 6	VSS: 10

Comparison of the backup and release flushes for the four test segments indicate that the flush at Port Norfolk exceeds the average dry weather TSS and VSS mass removals, whereas the flushed pollutant masses for experiments on Walnut and Templeton Streets are less than the average. Backup and release flushes on Shepton Street were all conducted following a rainfall event. The average TSS and VSS removals for the three backup and release flushes on Shepton Street exceed the averages for the rainfall impacted events. These results agree well with field experiemce since backup and release flushes were difficult to accomplish on Templeton and Walnut Streets.

The above discussion deals with the analysis of the optimal flush method with respect to flush pollutant removal efficiency. All methods resulted in comparable removal effectiveness. Another issue worth investigating is over-flushing. Figure 55 and 56 show plots of cumulative VSS mass removals versus the cumulative fraction of flush volume passing the sampling manhole at Shepton Street for delivered flush truck volumes of 35 and 50 cubic feet, respectively. Flushing rates used in the experiments are indicated on the two figures. Both plots show that ninety percent mass removal occur at roughly the "knee-of-the-curve" points. The average percent volume corresponding to the "knee-of-the-curve" point is roughly 80% in Figure 55 for the 35 cubic foot flushes and 70% for the 50 cubic foot flushes in Figure 56. This means that ninety percent of the load would be moved or transported by flush volumes of about 30 cubic feet. Flush volumes of 30 cubic feet are recommended for single segment flushing.

TABLE 33. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING HEAVY METALS RESULTS
MEAN AND STANDARD DEVIATION OF TOTAL MASS REMOVALS (FLUSH SOLIDS FRACTION)
PORT NORFOLK STREET TEST SEGMENT

	HEAVY METAL REMOVALS (μg/flush)						
TYPE	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
A: ALL FLUSHES	27.14 30.01 (15)	119.5 66.7 (13)	3278. 6464. (15)	854.8 789.6 (15)	128.9 127.5 (15)	5443. 6039. (15)	25.51 57.28 (9)
B: ALL GOOD FLUSHES	27.14 30.01 (15)	119.5 66.7 (13)	3278. 6464. (15)	854.8 789.6 (15)	128.9 127.5 (15)	5443. 6039. (15)	25.51 57.28 (9)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	16.73 15.25 (10)	121.2 79.2 (9)	1247. 979. (10)	849.2 916.9 (10)	90.5 63.3 (10)	3196. 2403. (10)	6.78 2.09 (6)
D: ALL RAINFALL IMPACTED FLUSHES	47.94 42.61 (5)	115.7 31.9 (4)	7341. 10637. (5)	866.1 539.1 (5)	205.7 191.9 (5)	9939. 8763. (5)	62.96 99.79 (3)

KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 34. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING HEAVY METALS RESULTS
MEAN AND STANDARD DEVIATION OF TOTAL MASS REMOVALS (FLUSH SOLIDS FRACTION)
SHEPTON STREET TEST SEGMENT

TYPE	HEAVY METAL REMOVALS (μg/flush)						
	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
A: ALL FLUSHES	18.50	151.4	1186.	882.6	145.4	3551.	12.50
	8.60	71.5	805.	759.4	256.9	2170.	7.20
	(17)	(17)	(17)	(17)	(17)	(17)	(17)
B: ALL GOOD FLUSHES	18.25	150.2	968.	912.5	150.8	3321.	11.38
	8.42	70.9	557.	779.3	264.6	2026.	6.08
	(15)	(15)	(15)	(15)	(15)	(15)	(11)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	17.62	142.4	1031.	622.6	76.5	2945.	10.70
	9.52	79.1	557.	604.2	36.9	2254.	6.34
	(10)	(10)	(10)	(10)	(10)	(10)	(9)
D: ALL RAINFALL IMPACTED FLUSHES	15.50	146.7	846.	707.2	191.3	2598.	10.80
	5.30	67.5	389.	757.8	379.4	1989.	3.00
	(6)	(6)	(6)	(6)	(6)	(6)	(6)

KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 35. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING HEAVY METALS RESULTS
MEAN AND STANDARD DEVIATION OF TOTAL MASS REMOVALS (FLUSH SOLIDS FRACTION)
TEMPLETON STREET TEST SEGMENT

TYPE	HEAVY METAL REMOVALS (μg/flush)						
	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
A: ALL FLUSHES	42.18 27.39 (18)	346.6 263.7 (18)	4920. 6888. (18)	5657.4 9489.0 (18)	228.9 176.7 (18)	8873. 8357. (17)	33.81 28.18 (8)
B: ALL GOOD FLUSHES	43.57 29.18 (15)	354.3 275.8 (15)	4837. 7172. (15)	6306.1 9952.6 (15)	237.1 181.3 (15)	8770. 8939. (15)	33.81 28.18 (8)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	44.12 30.58 (9)	281.0 181.8 (9)	3013. 3077. (9)	8169.4 12508.1 (9)	252.5 209.7 (9)	7921. 7803. (9)	17.82 5.04 (4)
D: ALL RAINFALL IMPACTED FLUSHES	40.43 27.86 (7)	419.0 357.2 (7)	6661. 10038. (7)	3173.0 3210.0 (7)	194.8 124.0 (7)	9631. 9383. (7)	49.80 32.42 (4)

KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 36. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING HEAVY METALS RESULTS
MEAN AND STANDARD DEVIATION OF TOTAL MASS REMOVALS (FLUSH SOLIDS FRACTION)
WALNUT STREET TEST SEGMENT

	HEAVY METAL REMOVALS (μg/flush)						
TYPE	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
A: ALL FLUSHES	41.90 52.00 (18)	291.6 364.7 (18)	4265. 5328. (18)	1826.0 1785.9 (18)	282.70 313.80 (18)	8963. 8709. (18)	51.30 56.30 (12)
B: ALL GOOD FLUSHES	29.30 17.00 (14)	194.5 129.6 (14)	3300. 4514. (14)	1469.6 1576.0 (14)	221.17 127.64 (14)	7027. 3998. (14)	50.24 59.71 (10)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	21.52 13.44 (10)	142.9 76.8 (10)	1086. 925. (10)	847.9 812.9 (10)	178.27 124.09 (10)	6074. 4161. (10)	16.52 5.87 (6)
D: ALL RAINFALL IMPACTED FLUSHES	81.98 77.29 (6)	566.0 551.7 (6)	10380. 5244. (6)	3625.6 1864.8 (6)	522.85 446.73 (6)	15307. 11166. (6)	99.37 54.54 (5)

KEY: Top Row - Mean
Middle Row - Standard Deviation
Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 37. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING HEAVY METALS RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED BY
ANTECEDENT DAYS BETWEEN FLUSHES (FLUSH SOLIDS FRACTION) - PORT NORFOLK STREET TEST SEGMENT

TYPE	HEAVY METAL REMOVALS (μg/day/flush)						
	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
A: ALL FLUSHES	8.00 9.75 (15)	32.4 18.1 (13)	1010. 2102. (15)	232.1 224.7 (15)	37.82 42.12 (15)	1929. 2187. (15)	8.28 18.02 (9)
B: ALL GOOD FLUSHES	8.00 9.75 (15)	32.4 18.1 (13)	1010. 2102. (15)	232.1 224.7 (15)	37.86 42.12 (15)	1929. 2187. (15)	8.28 18.08 (9)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	4.54 3.96 (10)	32.1 21.4 (9)	316. 223. (10)	237.3 261.8 (10)	25.24 19.62 (10)	1355. 1532. (10)	2.08 0.76 (6)
D: ALL RAINFALL IMPACTED FLUSHES	14.90 13.51 (5)	33.1 6.0 (4)	2397. 3204. (5)	221.8 119.2 (5)	63.10 59.97 (5)	3076. 2772. (5)	20.69 27.36 (3)

KEY: Top Row - Mean

Middle Row - Standard Deviation

Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 38. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING HEAVY METALS RESULTS MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS BETWEEN FLUSHES (FLUSH SOLIDS FRACTION) - SHEPTON STREET TEST SEGMENT

TYPE	HEAVY METAL REMOVALS (μg/day/flush)						
	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
A. ALL FLUSHES	5.41	44.3	354.	244.7	61.04	1108.	3.71
	3.01	23.8	272.	196.8	114.62	667.	2.56
	(17)	(17)	(17)	(17)	(17)	(16)	(12)
B: ALL GOOD FLUSHES	5.70	46.7	374.	258.0	64.67	1165.	4.01
	2.84	122.5	268.	195.3	117.37	650.	2.46
	(16)	(16)	(16)	(16)	(16)	(15)	(11)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	5.84	45.9	455.	216.8	74.26	1393.	4.09
	3.73	24.1	325.	216.3	145.32	737.	2.71
	(9)	(9)	(9)	(9)	(9)	(8)	(9)
D: ALL RAINFALL IMPACTED FLUSHES	4.80	47.7	270.	310.9	52.33	904.	3.61
	1.47	20.3	98.	152.6	71.50	417.	
	(7)	(7)	(7)	(7)	(7)	(7)	(2)

KEY: Top Row - Mean

Middle Row - Standard Deviation

Bottom Row (in parentheses) - Number of flushes used in computation

TABLE 39. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING HEAVY METALS RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED BY
ANTECEDENT DAYS BETWEEN FLUSHES (FLUSH SOLIDS FRACTION) - TEMPLETON STREET TEST SEGMENT

TYPE	HEAVY METAL REMOVALS (μg/day/flush)						
	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
A: ALL FLUSHES	12.20 8.34 (18)	90.4 65.5 (17)	1305. 2247. (18)	651.3 619.6 (18)	60.00 45.24 (18)	2500. 2784. (18)	8.45 5.02 (8)
B: ALL GOOD FLUSHES	11.98 8.31 (15)	89.6 67.7 (15)	1305. 2388. (15)	738.8 643.5 (15)	60.30 43.87 (15)	2488. 2925. (15)	8.45 5.02 (8)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	11.96 7.69 (9)	76.3 48.1 (9)	717. 782. (9)	673.7 515.4 (9)	64.69 45.63 (9)	2168. 2597. (9)	5.94 1.99 (4)
D: ALL RAINFALL IMPACTED FLUSHES	11.54 9.14 (7)	100.9 86.4 (7)	1932. 3414. (7)	771.4 732.4 (7)	49.84 38.71 (7)	2707. 3079. (7)	10.92 5.36 (4)

KEY: Top Row - Mean

Middle Row - Standard Deviation

Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 40. STATISTICAL SUMMARY OF PHASE I SEWER FLUSHING HEAVY METALS RESULTS
MEAN AND STANDARD DEVIATION OF MASS REMOVALS NORMALIZED BY
ANTECEDENT DAYS BETWEEN FLUSHES (FLUSH SOLIDS FRACTION) - WALNUT STREET TEST SEGMENT

TYPE	HEAVY METAL REMOVALS (μg/day/flush)						
	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
A: ALL FLUSHES	11.45	81.8	1260.	512.7	80.41	2505.	14.53
	11.69	93.3	1627.	534.2	80.77	2201.	17.12
	(18)	(18)	(18)	(18)	(18)	(18)	(12)
B: ALL GOOD FLUSHES	11.57	80.1	1067.	468.7	84.68	2522.	15.26
	12.59	101.0	1318.	516.1	85.05	2300.	17.76
	(15)	(15)	(15)	(15)	(15)	(15)	(11)
C: ALL GOOD NON-RAINFALL IMPACTED FLUSHES	6.23	40.5	312.	221.6	52.30	1689.	4.28
	4.33	22.8	290.	176.9	37.98	1083.	1.77
	(10)	(10)	(10)	(10)	(10)	(10)	(6)
D: ALL RAINFALL IMPACTED FLUSHES	21.58	157.4	3084.	1052.2	126.74	4285.	28.40
	15.69	135.0	1687.	608.9	119.96	2931.	19.65
	(6)	(6)	(6)	(6)	(6)	(6)	(9)

KEY: Top Row - Mean

Middle Row - Standard Deviation

Bottom Row (in parentheses) - Number of flushes used in computations

TABLE 41. SUMMARY OF AVERAGE PHASE I HEAVY METAL MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS BETWEEN FLUSHES AND BY ESTIMATED TRIBUTARY POPULATION

	HEAVY METAL MASS REMOVALS (nanograms/day/capita/flush)						
SHEPTON STREET (Separated) (Estimated Tributary Population = 230.)*							
TYPE	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
ALL GOOD FLUSHES	24.82	203.3	1627.	1121.8	281.16	5065.	17.42
ALL GOOD NON-RAINFALL IMPACTED FLUSHES	25.39	199.8	1979.	943.0	322.89	6058.	17.80
TEMPLETON STREET (Separated) (Estimated Tributary Population = 221.)*							
ALL GOOD FLUSHES	55.21	409.2	5908.	2947.4	271.49	11315.	38.24
ALL GOOD NON-RAINFALL IMPACTED FLUSHES	54.13	345.5	3245.	3048.4	292.71	9813.	26.89
WALNUT STREET (Combined) (Estimated Tributary Population = 71.)*							
ALL GOOD FLUSHES	161.34	1152.6	17757.	7221.7	1132.58	35287.	204.63
ALL GOOD NON-RAINFALL IMPACTED FLUSHES	89.14	570.6	4402.	3121.7	736.62	23789.	60.27

(continued)

TABLE 41. Cont. SUMMARY OF AVERAGE PHASE I HEAVY METAL MASS REMOVALS NORMALIZED BY ANTECEDENT DAYS BETWEEN FLUSHES AND BY ESTIMATED TRIBUTARY POPULATION

HEAVY METAL MASS REMOVALS (nanograms/day/capita/flush)

PORT NORFOLK STREET (Combined)
(Estimated Tributary Population = 94*; Estimated Population using DWF considerations = 61)

TYPE	CADMIUM	CHROMIUM	COPPER	LEAD	NICKEL	ZINC	MERCURY
ALL GOOD FLUSHES POP.=94	85.07	345.4	10750.	2469.6	402.79	20523.	88.12
ALL GOOD FLUSHES POP.=61	131.09	532.3	16566.	3805.7	620.69	31625.	135.79
ALL GOOD NON-RAINFALL IMPACTED FLUSHES POP.=94	48.33	342.2	3370.	2524.7	268.54	14422.	22.15
ALL GOOD NON-RAINFALL IMPACTED FLUSHES POP.=61	74.48	527.3	5194.	3890.5	413.82	22224.	34.13
SUMMARY							
SEPARATED STREETS ALL GOOD FLUSHES (AVERAGE)	40.02	306.3	3767.	2034.4	276.33	8190	27.83
COMBINED STREETS ALL GOOD FLUSHES (AVERAGE)**	123.21-146.22	749.0-842.5	14254.-17162	4845.7-5513.7	767.69-876.64	27905.-33456.	146.38-170.21

*Population from census information.

**Range reflects various tributary population estimates for Port Norfolk Street.

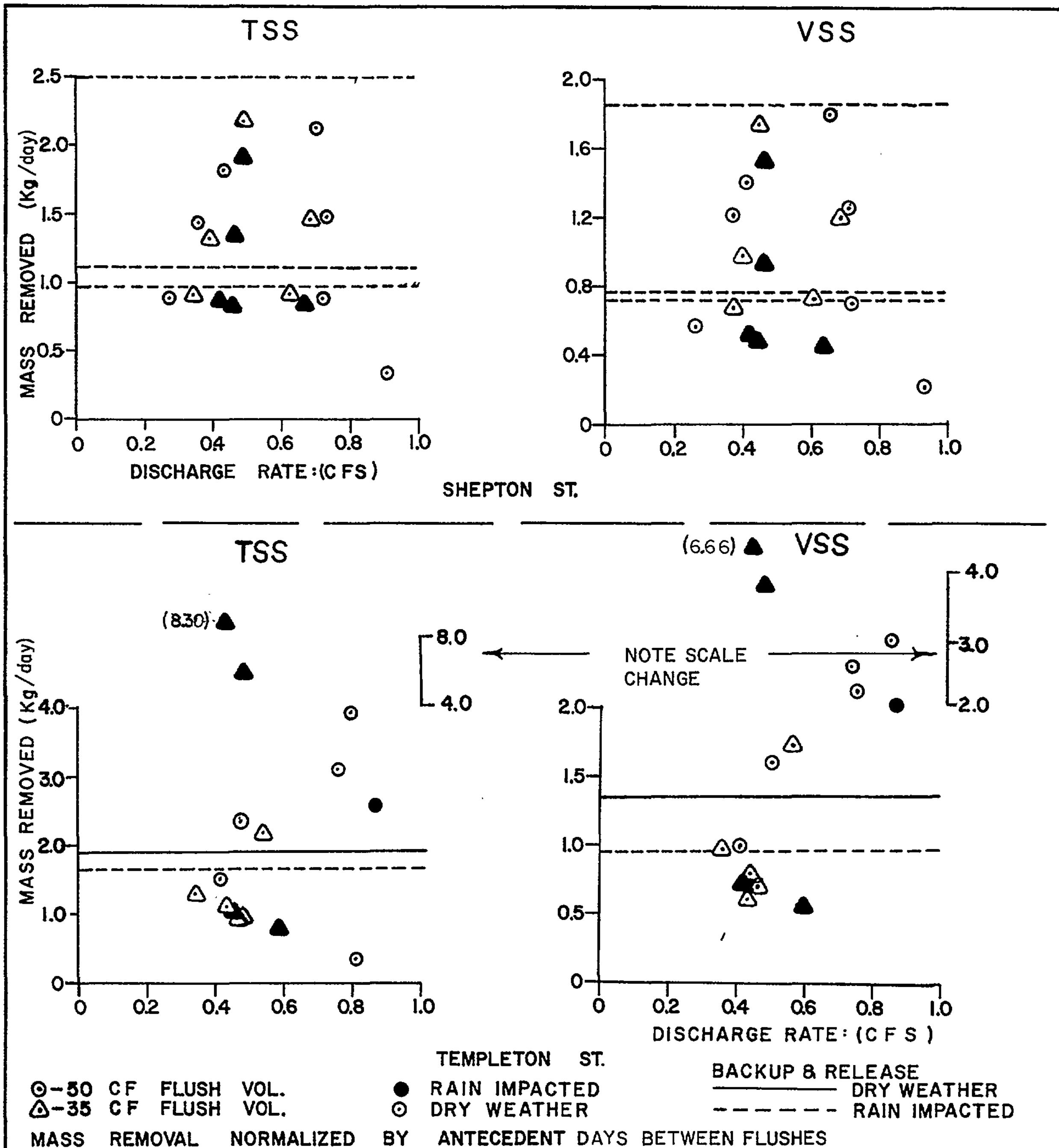


FIGURE 53

COMPARATIVE FLUSH EFFICIENCY PLOTS

SHEPTON AND TEMPLETON STREETS

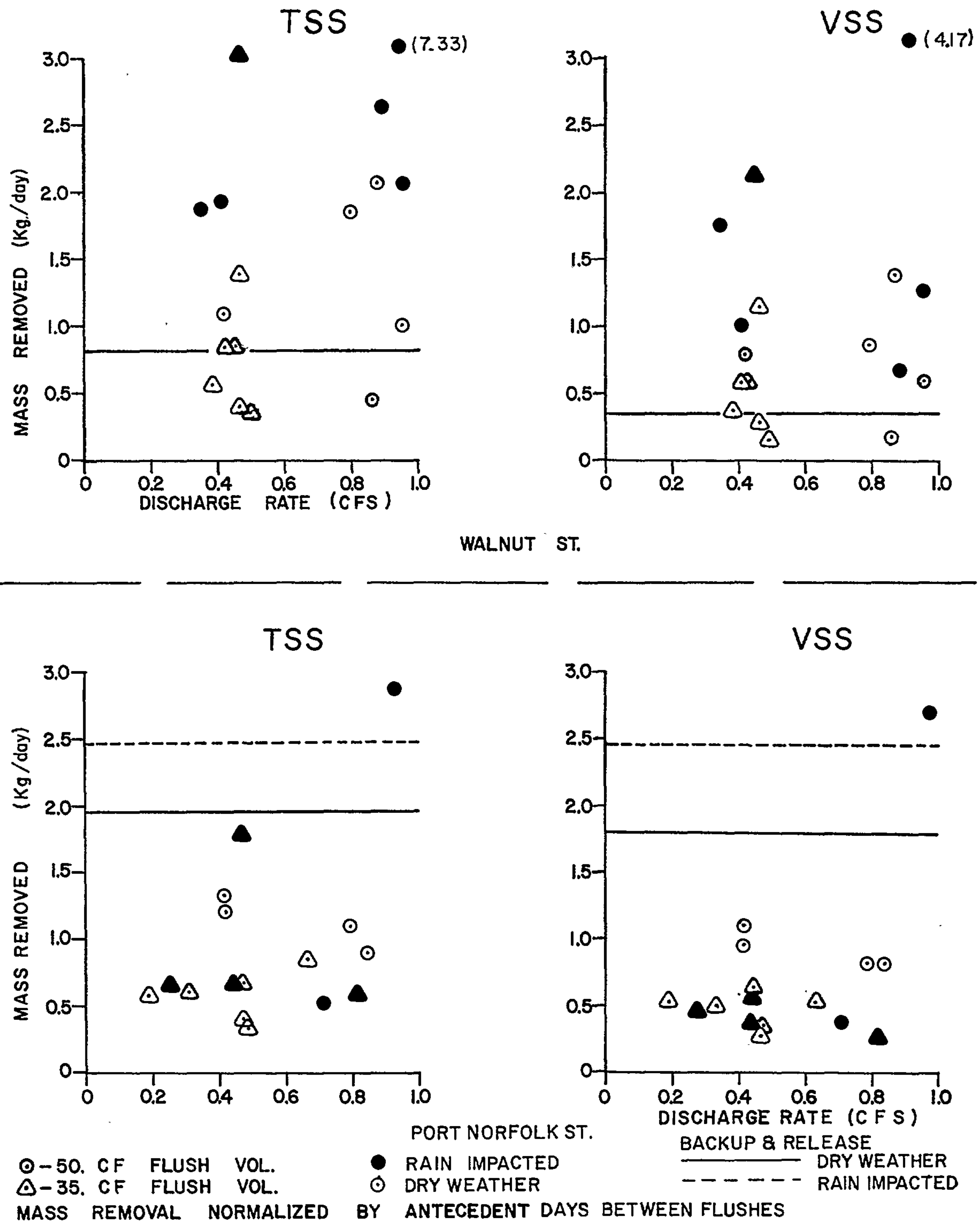


FIGURE 54 COMPARATIVE FLUSH EFFICIENCY PLOTS WALNUT AND PORT NORFOLK STREETS

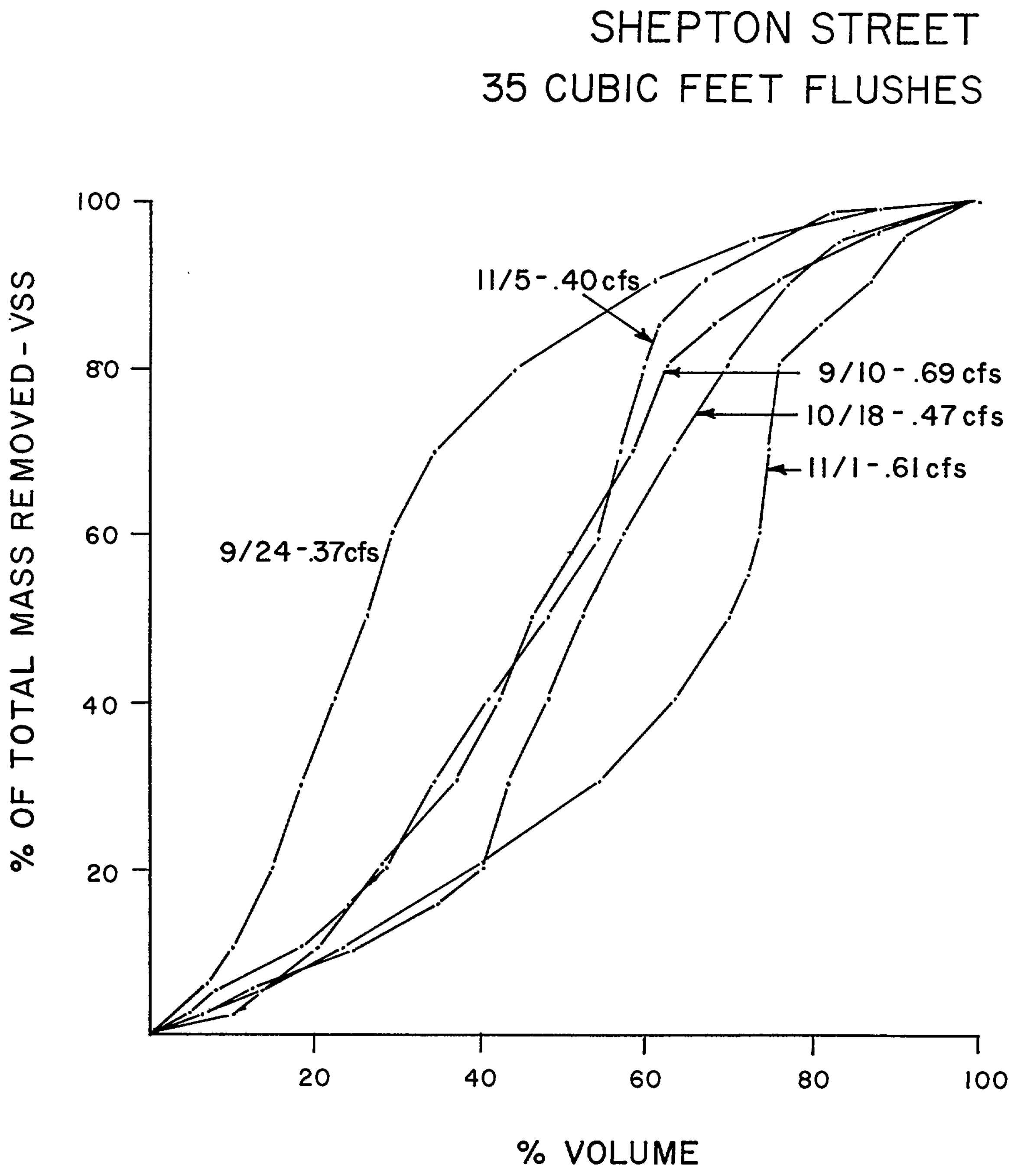


FIGURE 55 CUMULATIVE MASS VERSUS VOLUME OF FLUSH SHEPTON STREET FLUSHES (35 cf)

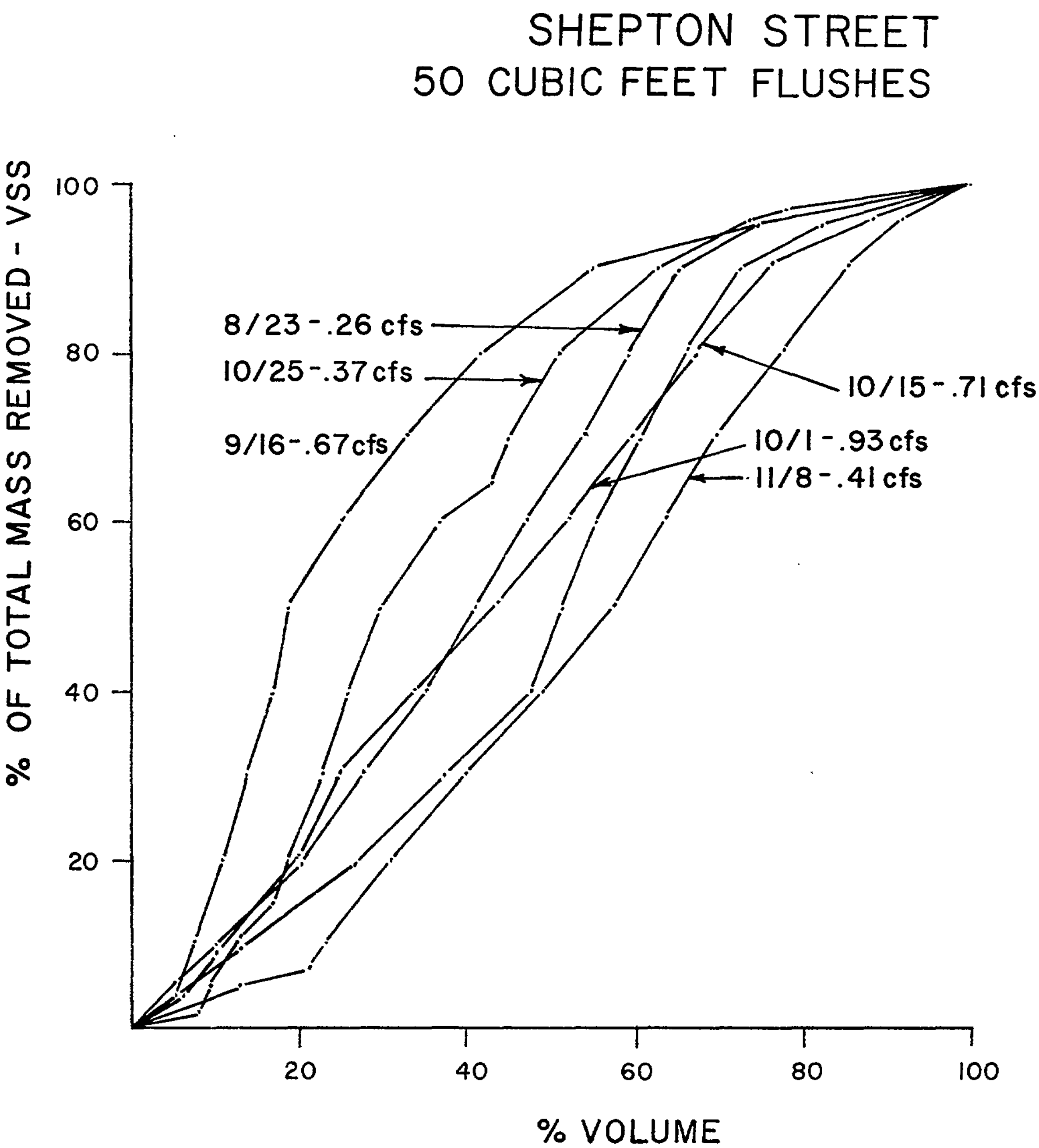


FIGURE 56 CUMULATIVE MASS VERSUS VOLUME OF FLUSH SHEPTON STREET EXPERIMENTS (50cf)

8.6 Phase I Sediment Characteristics

Sediment characteristics within each of the test segments were observed before and after each flushing experiment. Visual observations of sewer segment sediment characteristics for each street are given in Tables 42 through 45. Field observations are often noted for both the upstream flushing and downstream sampling manholes. Visual observations given in Tables 42 through 45 indicate that in general the organic sanitary deposits were removed by the flushing, and that sand, grit and gravel remained. The amount of organic deposits found on any given day before flushing was fairly constant over the three month flushing period. The quantity of inorganic sediments decreased with time during the flushing program. This phenomena was also observed during the second and third phase programs.

Representative sediment scraping analyses are shown in Table 46 for each of the test segments. After the name of the location and date, the percent volatile content of scrapings taken upstream prior to flushing and downstream after flushing are given in Table 46. Sieve analyses of total dry solids and of the volatile fraction for downstream sediment scraping samples remaining after flushing are then given. Solids handling procedures and analysis techniques are discussed in Chapter 6. Difficulties encountered performing sieve analyses on sediment scrapings taken prior to flushing are also described in Chapter 6.

The volatile content of upstream scrapings taken prior to flushing showed a high fraction of organic matter in the sediments. The volatile content of downstream sediments scraped subsequent to the flushing operation were low. The sieve analyses of those samples indicated that roughly 80% by mass of the dry solids sieved were greater than 0.2 mm or greater than fine sand. The dry weight sieve analyses for Shepton and Port Norfolk Streets scrapings indicated that most of the material was in the range of grit, that is, between 0.2 mm to 2 mm. The dry weight sieve analyses for Templeton and Walnut Streets scrapings indicated that a sizeable fraction of the material, ranging from 18% for 11/12/76 sample on Walnut up to 48% for the 10/15/76 sample on Templeton Street, exceeded the 2 mm particle size range. Sieve analyses of the volatile fraction of the scraping samples indicated that little of the organic material remaining was smaller than fine sand. Little fine organic material remained after flushing.

Dry solids determinations for a number of scrapings taken over a unit length of pipe at the upstream flushing manhole prior to flushing are presented in Table 47. These scrapings provide a good indication of dry weather deposition rates since the upper end of the segment was always cleaned bare by the flushing experiment. Accumulated materials during the period to next flush would therefore start from a zero base-line condition. The total dry solids weights per lineal foot of scrapings were normalized by the antecedent period between flushes, and are also shown in Table 47. Averages are presented for each street. Scrapings were not performed on Walnut Street, due to the "soupy" deposits in suspension in the segment. These primary estimates of deposition rates are used in Chapter 12 in a comparative analysis of predicted versus measured deposition loading rates.

TABLE 42: VISUAL OBSERVATIONS OF DEPOSITION CHARACTERISTICS
FIRST PHASE FLUSHES - PORT NORFOLK STREET

Date	Prior Flush	Post Flush
8/18/76	Heavy sanitary deposition	Partial removal
8/30	Heavy gravel and organic sediment	Slight removal
9/2	Heavy sand	Reduced
9/7	Heavy sand buildup, light sanitary deposits	Cleaned sanitary deposits, sand remained
9/16	Light-medium sanitary deposition	Clean
9/21	Light sanitary deposits	Clean
9/24	Heavy sanitary deposition	Cleaned sanitary deposition
10/1	Heavy sanitary suspension	Clean
10/4	Heavy sanitary suspension	Clean
10/8	Moderate sanitary solids	Upstream: some sanitary deposits remaining Downstream: grit
10/12	Moderate sanitary suspension, no silt-sand	Clean
10/15	Heavy sanitary suspension, light deposits	Clean
10/18	Heavy sanitary suspension	Removal of suspension/deposition of silt remained
10/22	Upstream: heavy sanitary suspension Downstream: heavy sanitary with sand deposits	Removed sanitary suspension & deposition, ≈2" sand remained

(continued)

TABLE 42: VISUAL OBSERVATIONS OF DEPOSITION CHARACTERISTICS
FIRST PHASE FLUSHES - PORT NORFOLK STREET (Cont'd)

Date	Prior Flush	Post Flush
10/25	Upstream: heavy sanitary suspension Downstream: 4" sediment, very fine sand	Removed sanitary deposition, 1" sand removed
10/29	Upstream: heavy sanitary deposition Downstream: 3" sediment, fine sand	Sanitary deposits removed, sand remained
11/1	Heavy sanitary suspension & sediment, some sand	Removed sanitary solids, sand left but moved a little
11/5	Upstream: heavy sanitary suspension	Removed sanitary suspension; sand remained
11/8	Upstream: heavy sanitary suspension Downstream: medium sanitary deposition & grit	Upstream: clean Downstream: heavy sanitary in suspension
11/12	Upstream: moderate sanitary suspension Downstream: light sanitary suspension & 2" of grit	Most of sanitary suspension removed, grit remained

TABLE 43: VISUAL OBSERVATIONS OF DEPOSITION CHARACTERISTICS
FIRST PHASE FLUSHING PROGRAM - SHEPTON STREET

Date	Prior Flush	Post Flush
8/23/76	Heavy sanitary deposits	Reduced
8/26	2"-3" sanitary deposition prior to flush	Clean
8/30	Medium sanitary deposit	Clean
9/2	High sanitary deposition - difficult to observe	Difficult to observe
9/7	Medium sanitary deposits	Reduced
9/10	Medium-heavy sanitary deposits	Considerable reduction
9/13	Heavy sanitary deposits	Clean
9/16	Moderate sanitary deposits	Floating debris in flush
9/21	Heavy sanitary deposits	Clean
9/24	Heavy sanitary deposition (mostly toilet paper)	Removed sanitary deposits but heavy particles remain
10/1	Heavy sanitary suspension	Removal of suspension formation of toilet paper scum
10/4	Moderate sanitary suspension	Removal of suspension
10/8	Upstream: heavy sanitary suspension Downstream: fairly clean	Upstream: clean Downstream: light gritty deposition
10/12	Heavy sanitary deposition	Clean
10/15	Heavy sanitary deposition	Clean
10/18	Heavy sanitary suspension & deposition	Clean

(continued)

TABLE 43: VISUAL OBSERVATIONS OF DEPOSITION CHARACTERISTICS
FIRST PHASE FLUSHING PROGRAM - SHEPTON STREET (Cont'd)

Date	Prior Flush	Post Flush
10/22	Very heavy sanitary suspension. Approx. 1" sediment downstream.	Removed suspension & part deposition, very silty
10/25	Light sanitary suspension	Removed
10/29	Heavy sanitary suspension	Suspension removed, light gritty deposits near downstream manhole
11/1	Moderate sanitary suspension & light deposition	Clean
11/5	Heavy sanitary suspension & light deposition	Clean
11/8	Upstream:heavy sanitary suspension Downstream: Medium-light sanitary suspension	Still heavy sanitary suspension remaining
11/12	Upstream: heavy sanitary deposits Downstream: light grit	Upstream: heavy sanitary suspension Downstream: light sanitary suspension

TABLE 44: VISUAL OBSERVATIONS OF DEPOSITION CHARACTERISTICS
FIRST PHASE FLUSHING PROGRAM - TEMPLETON STREET

Date	Prior Flush	Post Flush
8/30/76	4" sanitary deposit	Some change
9/2	High sanitary deposition	Reduced
9/7	Heavy sanitary deposits & silt	Reduced
9/10	Heavy sanitary deposits	Clean
9/13	Heavy sanitary deposits	Clean
9/16	Heavy sanitary deposits/gravel	Sanitary deposition removed, not gravel
9/21	Heavy sanitary deposits & silt	Cleaned, silt remained
9/24	Heavy sanitary deposition (50% toilet paper)	Heavy deposition remained in line
10/1	Heavy sanitary suspension & silt	Partial removal
10/4	Heavy sanitary suspension & sand deposits	Removal of sanitary suspension
10/8	Upstream: heavy deposition of solids & toilet paper Downstream: moderate deposition of sanitary & gritty solids	Upstream: clean Downstream: some toilet paper & small rocks left
10/12	Heavy sanitary suspension, little sand/silt	Removal of suspension
10/15	Heavy sanitary suspension, light deposits	Removal of suspension
10/18	Heavy sanitary suspension upstream Heavy sanitary, sand, silt downstream	Removal of sanitary suspension
10/22	Very heavy sanitary suspension. Approx. 1" sediment downstream	Removed suspension & part deposition, very silty

(continued)

TABLE 44: VISUAL OBSERVATIONS OF DEPOSITION CHARACTERISTICS
FIRST PHASE FLUSHING PROGRAM - TEMPLETON STREET (Cont'd)

Date	Prior Flush	Post Flush
10/25	Heavy sanitary suspension, ≈2" sand downstream	Removed sanitary suspension, sand remained
10/29	Heavy sanitary suspension upstream Heavy suspension & deposition downstream	Suspension removed, gritty deposition remained downstream
11/1	Heavy sanitary suspension & 1" in downstream	Upstream: clean Downstream: left sand & some sediment
11/5	Upstream: heavy sanitay suspension Downstream: heavy sanitary suspension & light sediment	Sanitary suspension removed, sediment left
11/8	Upstream: heavy sanitary suspension Downstream: ≈2" sanitary & gritty deposits	Removed most of suspension & deposition
11/12	Upstream: heavy sanitary deposition Downstream: grit & sanitary deposition	Deposition removed and a suspension formed in upstream & downstream manholes

TABLE 45: VISUAL OBSERVATIONS OF DEPOSITION CHARACTERISTICS
FIRST PHASE FLUSHING PROGRAM - WALNUT STREET

Date	Prior Flush	Post Flush
8/23/76	Heavy sanitary deposits	Reduced
8/26	Heavy sanitary deposits	Cleaned
8/30	No sign of deposits	No change
9/2	4" sanitary deposition	Reduced
9/7	Heavy sanitary deposits & sand	Cleaned sanitary deposits Sand remained
9/10	Mid-heavy sanitary deposits & sand	Cleaned sanitary deposits Sand remained
9/13	Heavy deposits Difficult to observe	Difficult to observe
9/15	Light deposits	Cleaned
9/21	Light deposits 4"-6" gravel	Cleaned deposits, not gravel
9/24	Heavy deposition	Heavy deposition remained
10/1	Heavy sanitary suspension & heavy silt	Removal of suspension
10/4	Sanitary suspension	Removal of suspension
10/8	Heavy sanitary deposits	Heavy sanitary deposits
10/12	Moderate silt/sand/gravel & light sanitary deposition	Sanitary removed
10/15	Heavy sanitary suspension, light deposits	Removed
10/18	Heavy sanitary suspension, light silt & gravel	Removal of sanitary suspension & some silt

(continued)

TABLE 45: VISUAL OBSERVATIONS OF DEPOSITION CHARACTERISTICS
FIRST PHASE FLUSHING PROGRAM - WALNUT STREET (Con't)

Date	Prior Flush	Post Flush
10/22	Heavy sanitary suspension & deposition	Removed, some silt remained ≃1"
10/25	Upstream: no deposition Downstream: 1" mud & medium sanitary deposits	Removal of sanitary deposition
10/29	Clean upstream Heavy sanitary suspension & 1/2" mud in downstream manhole	Clean
11/1	Upstream: light black suspension Downstream: moderate sanitary suspension	Clean
11/5	Medium sanitary suspension, no deposition	Clean
11/8	Upstream: heavy sanitary suspension Downstream: medium sanitary deposits & grit	Unchanged
11/12	Upstream: fairly clean Downstream: 1" of grit	Upstream: light sanitary suspension Downstream: light grit

TABLE 46: REPRESENTATIVE SEDIMENT SCRAPING ANALYSES FIRST PHASE FLUSHING PROGRAM

LOCATION	DATE	% VOLATILE UPSTREAM SCRAPING PRIOR TO FLUSH	% VOLATILE DOWNSTREAM SCRAPING AFTER FLUSH	DOWNSTREAM SCRAPING DRY SIEVE RESULTS % WEIGHT OF MATERIAL BETWEEN STATED PARTICLE SIZE INTERVALS: (>2mm	2-.6mm	.6-.2mm	.2-.06mm	<.06mm)*	DOWNSTREAM SCRAPING VOLATILE SIEVE RESULTS % WEIGHT OF METERIAL BETWEEN STATED PARTICLE SIZE INTERVALS: (>2mm	2-.6mm	.6-.2mm	.2-.06mm	<.06mm)*
Shepton	10/15/76	89.1	2.2	4	12	64	19	1	16	34	38	11	1
Port Norfolk	10/18	84.0	2.0	4	17	61	17	1	5	27	55	13	0
	10/22	57.3	1.1	2	33	61	4	0	4	36	57	3	0
	10/25	88.0	1.1	12	39	45	4	0	17	38	43	2	0
	11/1		3.2	9	61	27	3	0	25	55	19	1	0
	11/8		4.3	14	72	13	1	0	20	73	6	1	0
Templeton	10/15	80.9	5.6	48	25	18	8	1	41	47	9	2	1
	10/18	79.1	6.2	35	29	25	10	1	39	47	10	3	1
	11/5		9.7	40	26	23	10	1					
Walnut	10/15		5.4	28	20	30	20	2	42	26	20	10	2
	10/18	72.3	3.2	20	24	35	19	2	33	26	27	12	2
	10/25		4.2	44	30	17	9	0	36	34	22	8	0
	11/12		6.2	18	24	38	20	0	28	29	31	12	0

* M.I.T. Classification Scheme Used (see Section 6.4.1):

>2mm: larger than coarse sand
2-.6mm: coarse sand
.6-.2mm: medium sand
.2-.06mm: fine sand - coarse silt
<.06mm: smaller than coarse silt

TABLE 47: RESULTS OF UPSTREAM SCRAPINGS PRIOR TO FLUSHING
PHASE I

DATE	ANTECEDENT DAYS	DRY GRAMS/FOOT	NORMALIZED FOR ANTECEDENT DAYS	AVERAGE GRAM/FT/DAY
		SHEPTON STREET		
9/24	3	55	18.3	
10/8	4	45	11.3	
10/15	3	15	5.0	
10/18	3	77	25.6	
10/25	3	15	5.9	
10/29	4	20	5.9	
				12.0
		TEMPLETON STREET		
9/24	3	134	44.6	
10/8	4	100	25.0	
10/12	4	40	10.0	
10/15	3	59	19.7	
10/18	3	50	16.7	
10/22	4	25	6.3	
10/25	3	6	2.0	
10/29	4	18	4.5	
				16.1
		PORT NORFOLK STREET		
10/8	4	30	7.3	
10/12	4	65	16.3	
10/15	3	33	11.0	
10/18	3	67	22.3	
10/22	4	79	19.8	
10/25	3	53	17.7	
10/29	4	54	13.5	
				15.4

SECTION 9

SERIAL FLUSHING RESULTS

9.1 Foreword

Flushed pollutant removal results for experiments conducted during the first half of the second phase flushing program on Port Norfolk Street during the winter/spring of 1977 are presented in this Chapter. Settleability experiments performed on flush samples gathered during the latter portion of Phase two in the summer of 1977 are discussed in Chapter 10. Six experiments consisting of three flushes per experiment were conducted. Details of the field procedures are given in Chapter 5. In Section 9.2 preliminary flushing experiments are described that were conducted to assess flush wave hydraulic characteristics for the multiple flush segment experimentation. Flushed solids and organic loadings for the six experiments are described in Section 9.3. Characteristics of sediments noted during this period of flushing are presented in Section 9.4 as well as for the period of sampling for settleability testing.

9.2 Preliminary Flush Wave Hydraulic Experiments

The first phase flushing program entailed a single manhole to manhole flushing operation. It appeared from that program that those experiments conducted using high flush volume (50 cf) together with high injection rates, that is, in excess of 0.5 cfs, were the most effective in removing pollutants from the segments. The hydraulic characteristics of flush waves travelling in excess of 276 feet were not known at that point in time. A series of hydraulic experiments were conducted during December of 1976 on Port Norfolk Street to assess flush wave characteristics for the envisioned serial flushing program. Flushes were injected into the upstream injection manhole and wave heights as a function of time were noted at the three downstream manholes. Various flush volumes and rates were investigated and it appeared that flush rates in excess of 0.45 cfs would be adequate to ensure a noticeable wave roughly 675 feet from the point of injection. In addition, the larger volume flushes (50 cf) maintained slightly better wave form at the furtherest downstream manhole, presumably from increased momentum. Flush waves with water depth of 4 to 5 inches were noted at the end of the street, roughly 1000 feet downstream for flush volumes of 50 cubic feet injected at rates exceeding one cfs. These experiments together with first phase experience suggested that flush volumes of around 50 cubic feet injected at rates of 0.4 cfs or better would be adequate for the serial flushing program aimed at assessing multiple segment flushing effectiveness.

9.3 Pollutant Removal Results

Six flushing experiments on Port Norfolk Street were conducted during the winter and spring period of 1977. Three flushes were conducted per experiment with the flush injected into an upstream manhole and flush wave samples noted at three downstream manholes. Typical flush wave concentration levels for this phase of work are shown in Figure 57 for the experiments conducted on March 3, 1977. Nine plots of TSS and VSS concentrations versus time of flush passage at each manhole are presented. Each row of plots present flush wave concentration as a function of time for the three sampling manholes per flush. The top three plots refer to the first flush while the last three plots are associated with the third flush. Computed flushed loadings of TSS and VSS in kilograms, are indicated on each plot. These loadings were estimated using flush wave discharge rates computed using the loop stage rating curves described in Chapter 7 and the measured flush wave concentrations at each manhole. The scale change in flush wave TSS and VSS concentration between the first and subsequent flushes should be noted. The peak flush wave concentrations for the first flush ranged from 2500 and 1800 mg/l for TSS and VSS, respectively, at the first manhole, up to 7200 and 6000 mg/l at the second and third manholes. The peak TSS and VSS concentrations noted at all three sampling manholes for the second flush are all less than 800 mg/l. With the exception of the first sample at the first sampling manhole for the third flush, the peak TSS and VSS concentrations are again less than 800 mg/l. Similar plots of BOD, COD and TKN showed the same dramatic concentration levels for the initial flush and much lower levels for the second two flushes.

The flushed pollutant masses, in kilograms, for the 18 flushes conducted during this phase of work are given in Table 48. After the date of the flushing experiments in Table 48, the following information under the labelled columns are: a) the flush number; b) the order of the sampling manhole, that is, in a downstream sequence; c) the delivered flush volume, in cubic feet; and d) the flush rate, in cubic feet per second. The next set of columns present the estimated flushed masses for the following pollutants: COD, BOD, TKN, TSS and VSS. The antecedent periods between experiments were 11, 35, 21, 5 and 9 days, respectively for the experiments taken in chronlogical order. These flushed pollutant masses were then normalized by antecedent periods between experiments and sums of pollutant masses transported by each sampling manhole computed. Percentages of mass transported per flush relative to the total mass transported during an experiment for each of the pollutants were then computed. These percentages for the six flushing experiments are presented in Table 49. Information in Table 49 under the labelled headings is as follows: a) flush number; b) order of sampling manhole, that is, in a downstream sequence; c) delivered flush volume in cubic feet; and d) flush rate, in cubic feet per second. The next set of columns presents the percentages of pollutant mass transported per flush. For example, the relative percentages of BOD mass transported past the first sampling manhole for the experiment on 1/04/77 are 88, 13 and 12 percent, respectively. Inspection of the results indicate that high removal or transport effectiveness is accomplished with the first flush and the effectiveness decreases as the distance from the point of flush injection increases. The results appear to be invariant to both the volume of

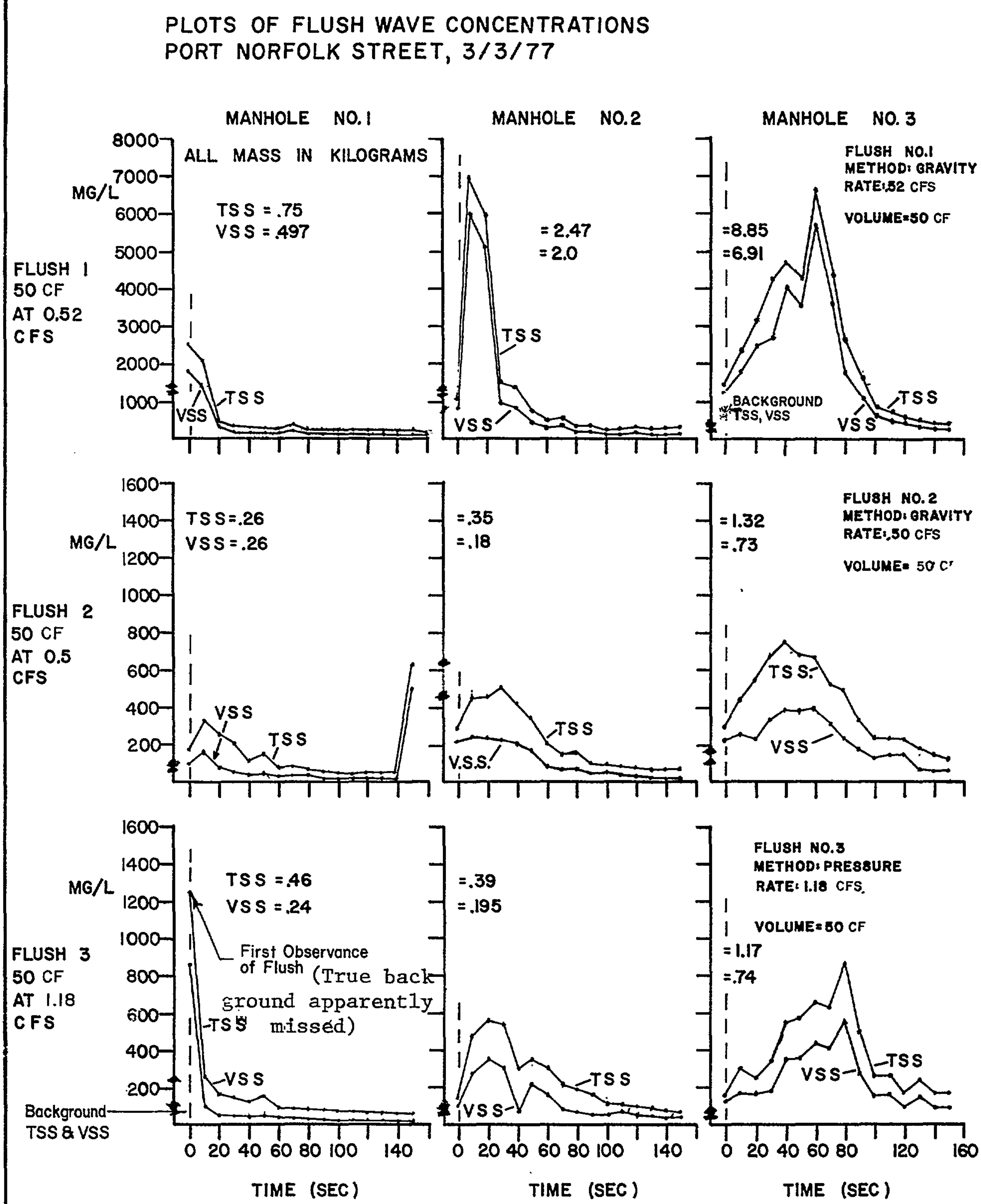


FIGURE 57. TYPICAL PHASE 2 RESULTS

TABLE 48. PHASE II - FIELD FLUSHING POLLUTANT REMOVALS (kg)
PORT NORFOLK STREET

DATE	A	B	C	D	TOTAL MASS (kg/flush) COD	BOD	TKN	TSS	VSS
1/04/77	1	1	50.0	0.64	5.03	0.93		4.78	3.72
		2			8.30	1.57		7.75	5.93
		3*							
	2	1	50.0	0.65	1.57			0.54	0.28
		2			1.51	0.25		0.90	0.44
		3*							
	3	1	50.0	0.65	0.52	0.14		0.42	0.24
		2			0.82	0.20		0.72	0.34
		3*							
2/10/77	1	1	55.2	1.10	3.50	0.93		4.39	3.07
		2			8.07	1.03		3.33	2.71
		3			12.15	1.64		7.09	3.72
	2	1	50.0	0.70	1.38			1.17	0.70
		2			2.61			2.03	1.32
		3			5.40			5.20	3.45
	3	1	50.0	0.98	0.92			0.71	0.37
		2			1.72			1.57	0.78
		3			3.21			2.45	1.37
3/03/77	1	1	50.0	0.52	1.19	0.37	0.033	0.75	0.58
		2			3.03	0.40	0.079	2.47	2.00
		3			8.83	2.45	0.231	8.85	6.91
	2	1	50.0	0.50	0.45	0.06	0.027	0.26	0.10
		2			0.46	0.13	0.048	0.35	0.18
		3			3.50	0.46	0.041	1.32	0.73
	3	1	50.0	1.18	0.48	0.14	0.012	0.46	0.24
		2			0.59	0.13	0.014	0.39	0.20
		3			1.33	0.45	0.018	1.17	0.74
3/08/77	1	1	50.0	0.40	2.74			1.56	1.20
		2			4.24			3.28	2.61
		3			7.36			6.04	4.89
	2	1	50.0	0.50	0.90			0.31	0.14
		2			1.52			0.53	0.21
		3			2.08			1.14	0.70
	3	1	50.0	1.0	0.45			0.43	0.17
		2			0.79			0.61	0.27
		3			0.91			0.93	0.51
3/17/77	1	1	35.0	0.51	3.31			3.67	3.25
		2			3.53			3.68	3.22
		3			14.88			10.38	10.64
	2	1	35.0	0.48	0.59			0.36	0.28
		2			0.97			0.43	0.31
		3			3.05			1.55	1.17
	3	1	50.0	0.85	0.34			0.23	0.16
		2			0.48			0.35	0.20
		3			1.43			1.19	0.73

(continued)

TABLE 48 (CONT.). PHASE II FIELD FLUSHING POLLUTANT REMOVALS
PORT NORFOLK STREET

DATE	A	B	C	D	TOTAL MASS (kg/flush) COD	BOD	TKN	TSS	VSS
3/24/77	1	1	35.5	0.47	2.70			1.97	1.61
		2			2.84			3.68	3.27
		3			5.21			4.58	3.84
	2	1	35.0	0.44	0.50			0.23	0.15
		2			0.96			0.51	0.34
		3			2.57			1.99	1.40
	3	1	50.0	0.94	0.70			0.27	0.16
		2			1.24			0.83	0.44
		3			1.84			1.65	1.08

Legend

A - Flush Number

B - Downstream Sampling Manhole

C - Flush Volume (cf)

D - Flush Rate (cfs)

*Samples not taken at third downstream manhole.

TABLE 49. PHASE II - FIELD FLUSHING PROGRAM PERCENTAGES OF TOTAL MASS TRANSPORTED PER FLUSH AT EACH SAMPLING MANHOLE

DATE	A	B	C	D	COD	BOD	TKN	TSS	VSS
					PERCENTAGES OF MASS TRANSPORTED PER FLUSH				
1/04/77	1	1	50.0	0.64	71	88		83	88
		2	50.0	0.65	22	13		9	6
		3	50.0	0.65	7	12		8	6
	2	1			78	77		83	88
		2			14	13		9	6
		3			8	10		8	6
	3	*							
2/10/77	1	1	55.2	1.10	60	**		70	74
		2	50.0	0.70	24	**		19	17
		3	50.0	0.98	16	**		11	9
	2	1			65	**		48	56
		2			21	**		29	27
		3			14	**		23	17
	3	1			59	**		48	43
		2			26	**		35	40
		3			15	**		17	17
3/03/77	1	1	50.0	0.52	56	63	46	51	59
		2	50.0	0.50	22	11	37	18	12
		3	50.0	1.18	22	26	17	31	29
	2	1			74	62	56	77	84
		2			11	19	34	11	7
		3			15	19	10	12	9
	3	1			64	72	80	78	82
		2			26	14	14	12	9
		3			10	14	6	10	9
3/08/77	1	1	50.0	0.40	67			68	79
		2	50.0	0.50	22			13	9
		3	50.0	1.0	11			19	12
	2	1			65			74	82
		2			23			12	10
		3			12			14	8
	3	1			71			74	80
		2			20			14	12
		3			9			12	8
3/17/77	1	1	35.0	0.51	78			86	88
		2	35.0	0.48	14			8	8
		3	50.0	0.85	8			6	4
	2	1			71			82	87
		2			20			10	8
		3			9			8	5
	3	1			77			79	85
		2			16			12	9
		3			7			9	6

(continued)

TABLE 49 (CONT.). PHASE II - FIELD FLUSHING PORT NORFOLK STREET. PERCENTAGES OF TOTAL MASS TRANSPORTED PER FLUSH AT EACH SAMPLING MANHOLE

					PERCENTAGES OF MASS TRANSPORTED PER FLUSH				
DATE	A	B	C	D	COD	BOD	TKN	TSS	VSS
3/24/77	1	1	35.5	.47	69			80	84
		2	35.0	.44	13			9	8
		3	50.0	.94	18			11	8
	2	1			56			73	81
		2			19			10	8
		3			25			17	11
	3	1			54			56	61
		2			27			24	22
		3			19			20	17

Legend

A - Sampling Manhole

B - Flush Number

C - Flush Volume (cf)

D - Flush Rate (cfs)

*Samples were not taken at third sampling manhole.

**BOD determined only for first sampling manhole.

flush used and the flush rate.

The fractional removal percentages in Table 49 were averaged and are summarized in Table 50, showing the average percentage per flush of the total load removed for each of the three segments downstream of the flush injection manhole. These averages were computed using the loads computed per manhole for the six sets of flushing experiments. The results indicate that most of the loads for all three segments were removed during the first flush. For example, 81.7% of the volatile suspended solids load was removed from the first flush. The second and third flushes removed an additional 18.3% of the total. No appreciable gain is achieved by repeated flushing. Furthermore, the experiments indicate that a single flush at the upper end of the street was reasonably effective in removing most of the deposited load along the 675 feet stretch of 12-inch combined sewer lateral.

TABLE 50. AVERAGE PERCENTAGES OF POLLUTANT LOADS REMOVED PER FLUSH FOR EACH PIPE SEGMENT

	TSS	VSS	COD	BOD
First Sampling Manhole				
Flush 1	76.1	81.7	67.7	87.
Flush 2	12.7	10.1	19.8	2.3
Flush 3	11.2	8.2	12.5	10.7
Second Sampling Manhole				
Flush 1	72.4	79.5	68.7	81.
Flush 2	14.2	11.6	18.4	10.2
Flush 3	13.4	8.9	12.9	8.8
Third Sampling Manhole				
Flush 1	66.5	71.6	65.7	81.8
Flush 2	20.2	17.8	22.5	9.2
Flush 3	13.3	10.6	11.8	9.0

9.4 Discussion of Sediment Characteristics

Sediment within the three combined sewer segments on Port Norfolk Street was mostly sand, grit and with some septic organic sanitary waste deposits. The level of depositions had substantially increased during the winter snow period when the first phase operation terminated (11/12/76) and the preliminary hydraulic experiments described in Section 9.2 began (12/20/76). Considerable sand from de-icing operations over the winter had washed into the segments. Visual observations taken before and after the flushing operations indicated that the sanitary deposits were generally washed away leaving sand and grit accumulations. Volatile solids were determined for 32 pre and post flushing scrappings and ranged from 1.4 to 50.2% with an average of about 5.1%. Little difference in volatile content was noted in pre/post flushing, presumably the result of a sand and silt layer. After the end of the snow period sand, grit and gravel layers were maintained at constant levels.

Sediment levels were also noted along the Port Norfolk Street test segments during the spring and summer of 1977. During this period the latter portion of the second phase program was conducted in which flush wave samples were taken for the settleability analyses described in Chapter 10. In addition, the special dye-injection experiments meant to verify the loop stage discharge methodology described in Chapter 7 were conducted during this period. In total, roughly 50 flushes were conducted over a four month period. The grit and sand accumulations were gradually reduced to minimal layers toward the end of the summer of 1977.

SECTION 10

SETTLEABILITY TESTING RESULTS

10.1 Foreword

To characterize the settleability of the solids in the flush waves a series of flush waves were analyzed as the waves traveled from manhole to manhole in a three segment series. Samples were taken from each of the three manholes in the Port Norfolk Street sewer section described in Chapter 5, using the sampling procedures described in Chapter 5. Samples taken at each location for each flush were then composited based on the hydraulics of the wave and analyzed according to the procedures described in Chapter 6.

The settling column testing subphase of the second phase flushing program, phase IIB, was conducted during the period of July 27 - September 7, 1977. During that period a total of 18 flushes were sampled at each of the three manholes described in Chapter 4 of this report. As indicated in Chapter 5, on each flushing day three successive flush waves were injected along the sewer line and were sampled at each of the three downstream sampling locations. The locations were the same as those used during the serial flushing, phase IIA program, that had been completed earlier in the year. Specialized sampling techniques using special devices were used to ensure the collection of "undisturbed" flush wave samples as described in Chapter 5.

In addition to the settling column tests, Imhoff cone tests were conducted to determine settleability and general character of the supernatants of the flush samples. Procedures used for conducting the settling column and Imhoff cone tests were described in Chapter 6.

All samples taken from the settling columns for settleability analysis were analyzed to determine pollutant concentrations associated with the various settling velocities. Pollutants analyzed included, TSS, VSS, BOD_5, COD, ammonia nitrogen, TKN, orthophosphate, total phosphate cadmium, chromium, copper, lead, zinc, and nickel.

Attempts were made to analyze mercury levels in the various settling column samples, but the concentrations proved to be so low, on the order of 1.0 ppb, as to make the reliability and meaning of the determination unsuitable for further use.

Classical settling theory considers that the sedimentation may be one of four types: Type I - Discrete Particle Settling, Type II - Flocculent Settling, Type III - Zone or Hindered Settling, and Type IV - Compression Settling. A more detailed description of the four types of sedimentation may be found in Metcalf and Eddy (24). The nature of the particles in suspension determines whether Type I - Discrete Particle Settling or Type II - Flocculent Settling will occur for suspensions with a low concentration of solids. As the concentration of solids in suspension becomes progressively higher Type III - Hindered Settling and Type IV - Compression Settling will be the case for either discrete or flocculant particles. The suspension produced by sewer flushing activities contains a mixture of discrete and flocculant particles. Most of the larger particles in suspension such as grit settle as discrete particles uninfluenced by surrounding flocculent particles. Smaller particles generally undergo flocculent settling. Thus in this study, laboratory analyses were carried out for both types of settling.

10.2 Assessment of Initial Concentration Data

As previously indicated the composite flush wave samples were analyzed for a number of parameters. Table 51 presents a summary of the initial concentration or concentration of the composited flush wave samples for all three manholes during the 18 column testing flushes on each of the six dates. Data is presented for the primary parameters of the analysis, TSS and percent volatile. Careful analysis of the data presented on the table leads to several conclusions which agree favorably with the results of the phase IIA program, as well as with the settling column and Imhoff cone test results presented later in this chapter. Although the induced turbulent energy of the flush wave decreases as the wave proceeds downstream along the three sewer segments, the concentration of solids and percent volatile tend to increase sharply. This phenomenon is indicative of the cumulative effects of the flush wave scouring a progressively increasing deposited load along the pipe. Although the scour or grit removal energy dissipates sharply along segment 1-2 as indicated by the large change in percent volatile generally exhibited, the pollutant removal still remains high. This fact is most important in assessing the performance of sewer flushing as a pollution control measure.

The second distinct trend indicated in the data is the relatively high removal efficiency of the first flush. Again the results are consistent with those found in phase IIA of the program. After the first flush there is generally a large drop off in TSS concentration especially with respect to manholes 2 and 3. Similarly the percent volatile or percent organic level of the samples also decreases indicating that the readily available surface pollutants have been carried away in segments 1 and 2 by the first flush and to a large degree in segment 3.

Since the bulk of the deposit was scoured during the first flush as indicated in Table 51, settling column testing was only performed on the first flush of each flushing day. Imhoff cone tests were performed on all flushes.

TABLE 51. INITIAL CONCENTRATION*AND PERCENT VOLATILE OF INITIAL CONCENTRATION FOR ALL FLUSHES AND MANHOLES

		MANHOLE #1		MANHOLE #2		MANHOLE #3	
DATE	FLUSH	INITIAL TSS mg/ℓ	% VOL.	INITIAL TSS mg/ℓ	% VOL.	INITIAL TSS mg/ℓ	% VOL.
7/27	1	1029	59	1856	68	3958	85
	2	501	40	1285	68	478	54
	3	226	52	475	51	2593	41
8/4	1	505	58	1610	74	2214	82
	2	134	54	596	61	285	52
	3	52	52	369	44	264	55
8/22	1	528	73	3316	84	7055	86
	2	110	51	665	68	482	95
	3	103	45	717	70	748	70
8/25	1	221	68	2226	84	1880	92
	2	58	57	438	71	579	80
	3	40	55	118	60	312	67
8/29	1	126	60	2669	80	2254	82
	2	77	51	-	-	812	74
	3	-	-	200	64	423	62
9/7	1	1130	76	5927	80	5542	83
	2	102	60	763	63	613	89
	3	57	53	315	49	783	57
Average	1	589	65.7	2934	78.3	3817	85.0
	2	164	52.2	749	66.2	542	74.0
	3	96	51.4	366	56.3	854	58.7

* Initial Concentration of Settling Column Experiments

10.3 Presentation of Results

The composite flush sample suspensions are a mixture of particles which exhibit discrete settling behavior and particles which exhibit flocculent settling behavior. Total suspended solids concentrations in the samples were found to vary between 500 and 8,000 mg/l. Utilization of a large sedimentation column is a good method for determining the settling characteristics of such suspensions. Column test results in this study were graphically analyzed. Several graphical approaches were used. The first were plots of percent of particles removed versus the settling velocity, V. This type of plot is extremely useful in evaluating settling characteristics of pollutants as associated to various settling velocities. A second approach plotting concentration versus settling velocity, V, was used to differentiate characteristics of one manhole versus another.

Assessment of sewer flushing related settling column data requires a different philosophical viewpoint than that commonly used with this type of information. Typically, settleability testing is done to design clarifiers or sedimentation tanks where high removals are the aim. On the other hand, sewer flushing is aimed at resuspending deposited solids and keeping them in suspension until they reach a treatment or removal point. This difference is most important in reviewing the data generated, in that poor settleability characteristics equate with increased overall flushing efficiency.

Tables 52 and 53 are the data summaries of typical settling column tests for flush 1, manhole 1 for 8/25 and 9/7, respectively. From this data it is possible to construct curves of percentage removal of a specific pollutant versus measured settling velocity. Careful analysis of the data presented shows that sewer flushing is far more efficient with respect to long-term pollutant removal than grit and other readily settleable inorganics. Data presented in Tables 52 and 53 illustrates this fact by indicating that for the particular sample, removals for TSS averaged roughly 80 % for 30-minute quiescent periods, where BOD, TKN, Total Phosphorous averaged from 50-55%. This is most significant when the converse or percent remaining is viewed. In this case, after 30 minutes of settling only 20 % of the solids remain in suspension, while between 45-50 % of the significant organic pollutants remain. Extended settling periods, up to 120 minutes as illustrated in Table 52, showed little additional change beyond the 30-minute increment. Surprisingly, sewer flushing COD removal was much lower than other pollutants,with approximately 80 % removal, or 20 % remaining after 30 minutes.

The marked difference between COD and BOD is most probably due to the COD being primarily representative of grease and other long-term decomposing matter complexed in the heavy sediments. This type of sediment primarily composed of materials that are difficult to move, would probably not be moved by storm events, and on a pollution control basis, is of lesser significance than the more readily available BOD and nutrients.

Although the corresponding removals vary from flush to flush and manhole to manhole, the general pattern remains the same. As this pattern

TABLE 52. COLUMN TEST RESULTS 8/25 FL1 - MH1

SAMPLE NO.	TIME MIN.	PORT NO.	DEPTH FT.	S.O.R.* gpd/sf	VERT. VEL. fps x 10^3	COD mg/ℓ	COD % Rem.**	ORTHO-P mg/ℓ	ORTHO-P % Rem.**	TOTAL-P mg/ℓ	TOTAL-P % Rem.**	TSS mg/ℓ	TSS % Rem.**	VSS mg/ℓ	VSS % Rem.**
D13	Back***	-	-	-	-	353	-	1.58	-	1.85	-	221	-	151	-
F1	5	9	0.71	1555	2.40	102	71.1	0.36	77.2	0.97	47.6	51	76.9	39	74.2
F2		3	2.98	6437	9.93	110	68.8	0.47	70.3	1.17	36.8	68	69.2	47	68.9
F3		1	4.98	10757	16.60	145	58.9	0.53	66.4	1.26	31.9	90	59.3	54	64.2
F4	10	7	0.93	1004	1.55	96	72.8	0.44	72.2	1.02	44.9	61	72.4	42	72.2
F5		3	2.64	2851	4.40	91	74.2	0.40	74.7	0.99	46.5	49	77.8	35	76.2
F6		1	4.64	5011	7.73	79	77.6	0.47	70.3	0.99	46.5	50	77.4	36	76.2
F7	20	5	1.05	567	0.89	64	81.9	0.41	74.1	0.99	46.5	52	76.5	36	76.2
F8		3	2.34	1264	1.95	71	79.9	0.24	84.8	1.00	45.9	47	78.7	33	78.1
F9		1	4.34	2344	3.62	79	77.6	0.27	82.9	2.45	34.2 +	47	78.7	39	74.2
F10	30	3	2.06	742	1.14	52	85.3	0.32	79.7	1.26	31.9	43	80.5	34	77.5
F11		1	4.06	1462	2.26	64	81.9	0.31	80.4	0.71	61.6	46	79.3	31	79.5
F12	60	5	0.56	100	0.16	59	83.3	0.47	70.3	0.93	49.7	43	80.5	33	78.1
F13		3	1.85	333	0.51	52	85.3	0.35	77.8	0.85	54.1	32	85.5	24	84.1
F14		1	3.85	693	1.07	91	74.2	0.32	79.7	1.03	44.3	40	81.9	32	78.8
F15	90	3	1.61	193	0.30	60	83.0	0.27	82.9	0.88	52.4	27	87.8	20	86.6
F16		1	3.61	433	0.67	133	62.3	0.28	82.3	0.80	56.8	26	88.2	20	86.6
F17	120	3	1.41	126	0.20	48	86.4	0.13	91.8	0.71	61.6	29	86.9	27	82.1
F18		1	3.41	307	0.47	56	84.1	0.14	91.1	0.71	61.6	22	90.9	24	84.1

* S.O.R. - Surface Overflow Rate
** % Rem. - Percent Removal
*** Back - Initial Concentration
\+ Greater than initial concentration

TABLE 53. COLUMN TEST RESULTS 9/7 FL1 - MH 1

SAMPLE NO.	TIME MIN.	PORT NO.	DEPTH FT.	S.O.R.* gpd/sf	VERT. VEL. fps x 10^3	COD mg/ℓ	COD % Rem.**	BOD mg/ℓ	BOD % Rem.**	TKN mg/ℓ	TKN % Rem.**	SUSPENDED SOLIDS TSS	SUSPENDED SOLIDS % Rem.**	SUSPENDED SOLIDS VSS	SUSPENDED SOLIDS % Rem.**
P1	Back***	-	-	-	-	1682	-	345	-	42.6	-	1130	-	859	-
P3	2	9	0.71	3825	5.92	311	81.5	160	-	26.3	38.3	378	66.5	290	66.2
P4		3	2.98	16092	24.83	1008	40.1	308	10.7	29.1	31.7	631	44.2	478	44.4
P5		1	4.98	26892	41.50	1492	11.3	435	26.1 +	42.0	1.4	1021	9.6	744	13.4
P6	4	8	0.73	1958	3.04	692	58.9	204	40.9	26.6	37.6	296	73.8	231	73.1
P7		3	2.71	7317	11.29	621	63.1	216	37.4	25.2	40.8	297	73.7	232	73.0
P8		1	4.71	12717	19.63	803	52.3	294	14.8	32.2	24.4	511	54.8	351	59.1
P9	6	7	0.72	1290	2.00	545	67.6	198	42.8	24.6	42.3	237	79.0	196	77.2
P10		3	2.43	4374	6.75	477	71.6	210	39.1	23.8	44.1	236	79.1	176	79.5
P13		1	4.43	7974	12.31	523	68.9	150	56.5	16.8	60.6	320	71.7	199	76.8
P14	8	5	0.87	1175	1.81	477	71.6	174	49.6	27.2	36.2	190	83.2	179	79.2
P15		3	2.16	2916	4.50	508	69.8	164	52.5	22.7	46.7	198	82.5	159	81.5
P16		1	4.16	5616	8.67	470	72.1	153	55.7	22.4	47.4	192	83.0	154	82.1
P17	10	5	0.56	605	0.93	485	71.2	168	51.3	22.7	46.7	192	83.0	160	81.4
P18		3	1.85	1998	3.08	470	72.1	157	54.5	21.8	48.8	200	82.3	170	80.2
P19		1	3.85	4158	6.42	447	73.4	192	44.3	25.2	40.8	204	81.9	173	79.9
P20	20	3	1.56	842	1.30	333	80.3	162	53.0	21.3	50.0	155	86.3	135	84.3
P21		1	3.56	1922	2.97	439	73.9	153	55.7	21.8	48.8	182	83.9	156	81.8
P22	30	3	1.35	486	0.75	394	76.6	153	55.7	20.2	52.6	156	86.2	135	84.3
P23		1	3.35	1206	1.86	374	77.8	153	55.7	20.7	51.4	153	86.5	129	85.0

* S.O.R. - Surface Overflow Rate

** % Rem. - Percent Removal

*** Back - Initial Concentration

\+ Greater than initial concentration

became evident the initial sampling time intervals for the column tests were decreased in an attempt to better define the shape of the curve, as evidenced in the sampling times presented in the two tables. The sampling intervals per each experiment are given in Chapter 6.

Due to the difficulty in mixing the initial sample suspension, very short initial sampling times (less than 60 seconds) could not produce meaningful data. In all cases, the particles with settling velocities associated with grit and sand settled too quickly* for distribution analyses to be carried out.

It is also evident from the data in Tables 52 and 53 that a certain portion of the particles and associated pollutants will have settling velocities so low that this fraction will be transported long distances** by the flush wave. By examining the sampling results for the 30-minute time interval which represent settling velocities of .002 fps or less, it becomes evident that 15 to 30 % of the total suspended solids and volatile suspended solids will remain in suspension.*** In terms of organic materials or nutrients associated with the solids in suspension, the samples ranged from 20% to 50%. Such percentages are representative of the fraction of the flush which will at minimum be carried to a treatment plant. The Imhoff cone data typified by Table 54 provides an additional example of the material which will remain in suspension after an hour of quiescent settling. Table 54 shows the removal percentages after a one-hour settling period for all flushes and all manholes sampled on August 25, 1977. The percentage removals for total suspended solids ranged from 65 to 85 %, while the removals for volatile suspended solids ranged from 59 to 87% on the date. In the case of such data the first flush samples represent the bulk of the sewer deposit flushed. On that basis it is apparent that 85% of the TSS and 80% of the VSS are removed by the one-hour quiescent settling or that 15% TSS and 20% VSS will definitely remain in suspension.

Figures 58 and 59 present the results of flushes conducted on 8/25 and 9/7 for TSS and COD respectively. Figures 58 and 59 provide interesting insights into many factors of the settling column tests. First of all, the comparison of data from the short time frame sampling on 9/7 with the long time frame sampling on 8/25 shows amazing consistency. Although not presented, the comparison of the results of the test conducted on 8/22 and 8/25 yield similar consistency. The second important item to note is the relatively rapid settling velocities of a large percentage of the flushed pollutants, especially solids reaching a plateau at roughly 18% remaining. Comparison of Figures 58 and 59 reiterates the prior discussion pertaining to lesser removals or lower degrees of settleability of the organic pollutants. Although not shown, a similar trend was found to be present for all other pollutants with percent remaining ranging up to 50% for BOD and nutrients.

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* Settled out in 30-60 seconds

** A minimum of 1550 feet (457.2 m)

*** A more formal analysis of transport is presented in Chapter 12.

TABLE 54. IMHOFF CONE TEST RESULTS 8/25

FL-MH	SAMPLE	COD mg/ℓ	COD %Rem*	OP mg/ℓ	OP % Rem*	TP mg/ℓ	TP % Rem*	TSS mg/ℓ	TSS % Rem*	VSS mg/ℓ	VSS % Rem*	ml Settled	Settleable Matter mg/ℓ	Conc. of Deposited Solids mg/ℓ
1-1	D13	353		1.58		1.85		221		151				
	**D14	25	92.9	0.23	85.4	0.74	60.0	32	85.5	29	80.8	8.5	189	22,235
1-2	D15	4131		10.55		25.61		2226		1876				
	D16	822	80.1	8.21	22.2	14.78	42.3	332	85.1	325	82.7	150.0	1894	12,627
1-3	M1	4723		9.88		24.53		1880		1724				
	M2	1320	72.1	10.35		18.03	26.5	374	80.1	318	81.6	165.0	1506	9,127
2-1	D11	129		0.92		0.64		59		33				
	D12	64	50.4	0.77	16.3	0.46	28.1	18	67.2	16	51.5	2.0	41	20,500
2-2	D7	749		2.32		2.02		438		309				
	D8	210	72.0	1.30	44.0	0.92	54.5	42	90.4	39	87.4	25.0	396	15,840
2-3	D3	1166		2.59		2.91		579		462				
	D4	317	72.8	1.95	42.7	1.75	39.9	108	81.3	99	78.6	39.0	471	12,077
3-1	D9	64		0.32		3.02		40		22				
	D10	37	42.2	0.24	25.0	1.64	45.7	13	67.5	11	50.0	0.7	27	38,571
3-2	D5	313		0.80		5.18		118		71				
	D6	68	78.7	0.32	60.0	3.08	40.5	41	65.3	34	52.1	4.5	77	17,111
3-3	D1	498		0.76		1.57		312		208				
	D2	191	61.6	0.75	1.3	1.46	7.0	68	78.2	60	71.2	16.0	244	15,250

* % Rem - Percent Removal

** Typical: sample (D13) is initial and sample (D14) is final.

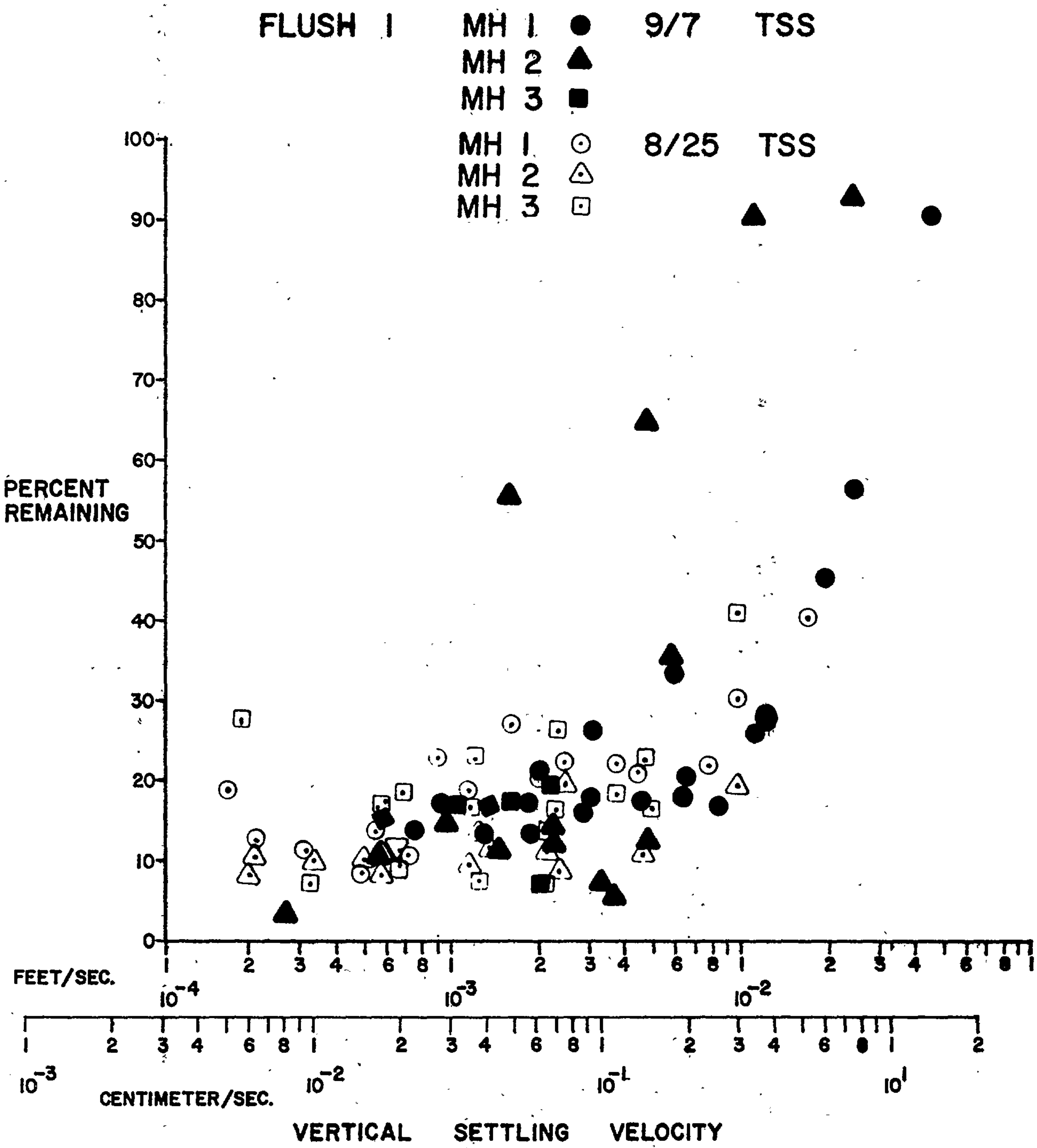


FIGURE 58. PLOT OF TSS REMAINING VS SETTLING VELOCITY

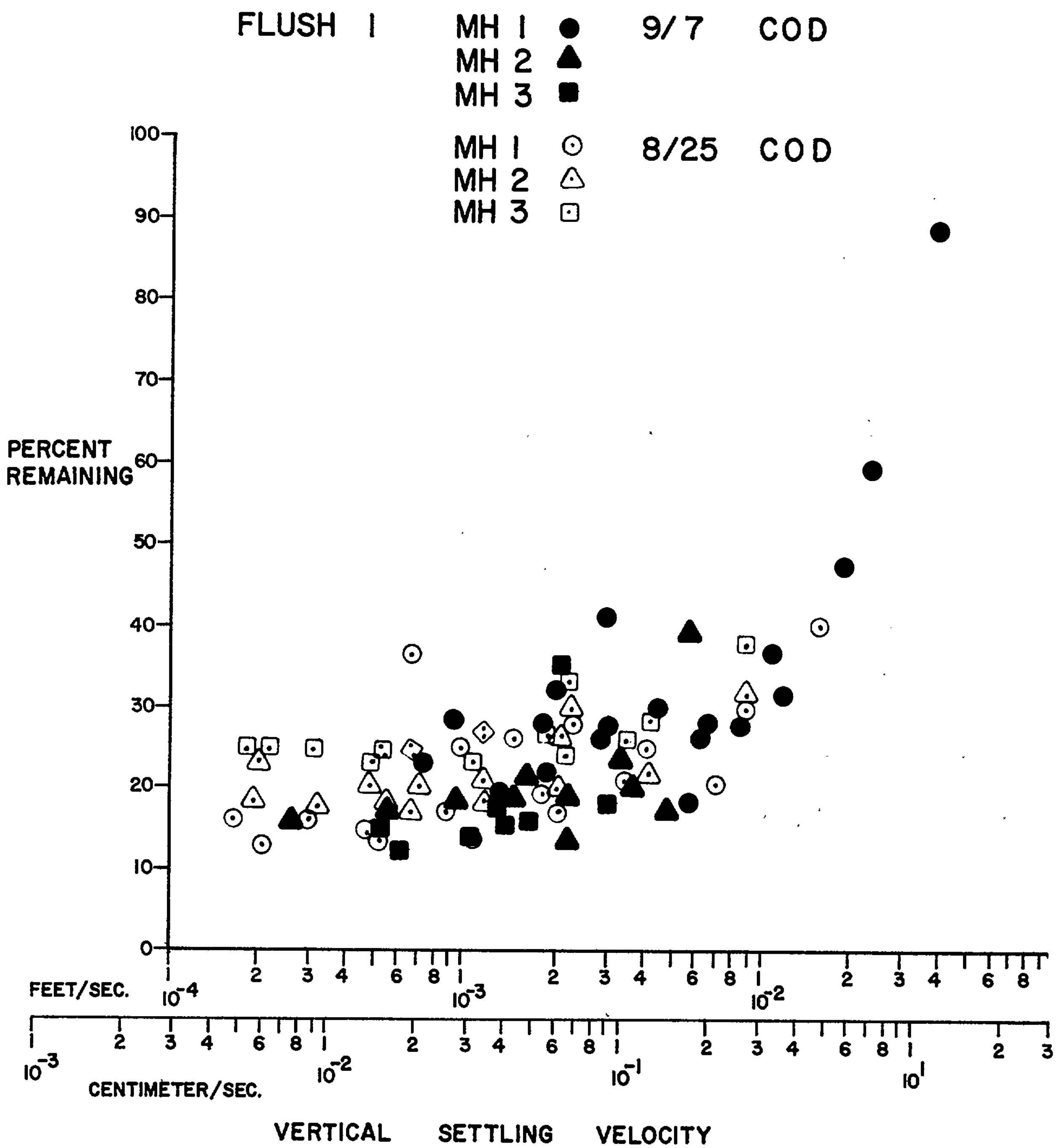


FIGURE 59. PLOT OF COD REMAINING VS SETTLING VELOCITY

Figures 60 and 61 are plots of actual concentration vs settling velocity for 8/22, 9/7 and 8/29, respectively, for TSS. The plots are particularly useful in comparing removals from the 3 manholes. What is clearly indicated is the tendency for the flush wave to scour progressively lighter materials as it flows downstream. It should be remembered that as the flush wave progressed further down the line from one manhole to another, the wave energy decreased. The result of this was that heavier particles dropped out of suspension while the lighter particles remained.

10.3.1 Heavy Metals Analysis

An extensive program was conducted as part of the column settleability testing program to assess the placement of heavy metals in relation to particle settling velocities. As the problem of heavy metals discharges and subsequent accumulations in bottom deposits and aquatic life is addressed, it becomes increasingly important to attempt to quantify the sources of such metals. One possible source of significant metals discharges are combined sewer overflows. Data generated in the first phase of this study, which are reported in Chapter 8, indicated that the largest percentage of heavy metals in the combined sewer system are located in fractions, which tend to settle after extended periods. The settling column effort was aimed at assessing to which fractions the metals were attached. Analyses were conducted for a total of six metals, including: copper, zinc, chromium, cadmium, lead and nickel.

Table 55 is a comparison of the heavy metals data generated per manhole per flush. The table represents a summary of each flush, indicating initial concentration, concentration and corresponding percent remaining of the first sample of each settling column test, and the average concentration and corresponding percent remaining of the metals after reaching a maximum removal plateau. Analysis of the data presented in the table clearly indicates several facts. First of all, metals exhibited a very high removal rate for the initial few minutes of the test, as shown by the rapid drop in concentration between the initial and first sample of each column test. Second, no significant settling then occurs after the initial period, as indicated by the average plateau concentrations and percent remaining. This is very significant in terms of metals accumulation and movement. The metals exhibited relatively rapid settling characteristics, amounting to 50% or more in metals concentration between the background and the first settling sample, for both the short and long interval sampling tests, and thereafter generally had a very slight positive tendency toward settling. The metals located in deposited sewer solids are split into two distinct fractions: roughly 50% that are entrained on the heavy grit and sand, and which are not readily transported; and the remaining 50% which are entrained in extremely light near-colloidal fractions with settling velocities so low as to negate any further removal once suspended.

The significance of this finding is especially clear when the removals due to storm events are assessed as discussed in Chapter 11. Low to moderate intensity storms, which are of highest incidence in the northeast, tend to move only light fractions, which is to say that these storm, although

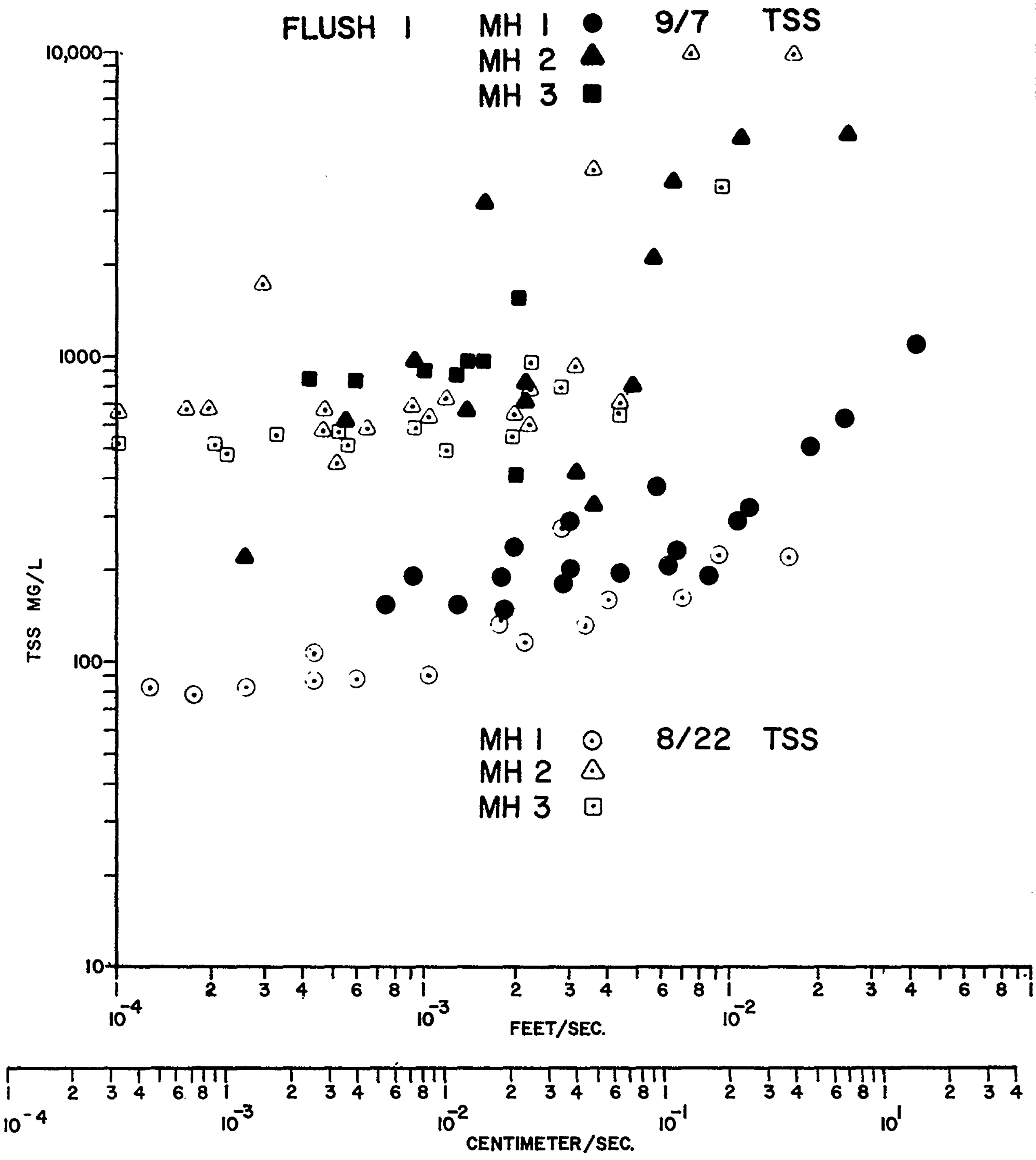


FIGURE 60. PLOT OF TSS CONCENTRATION VS SETTLING VELOCITY

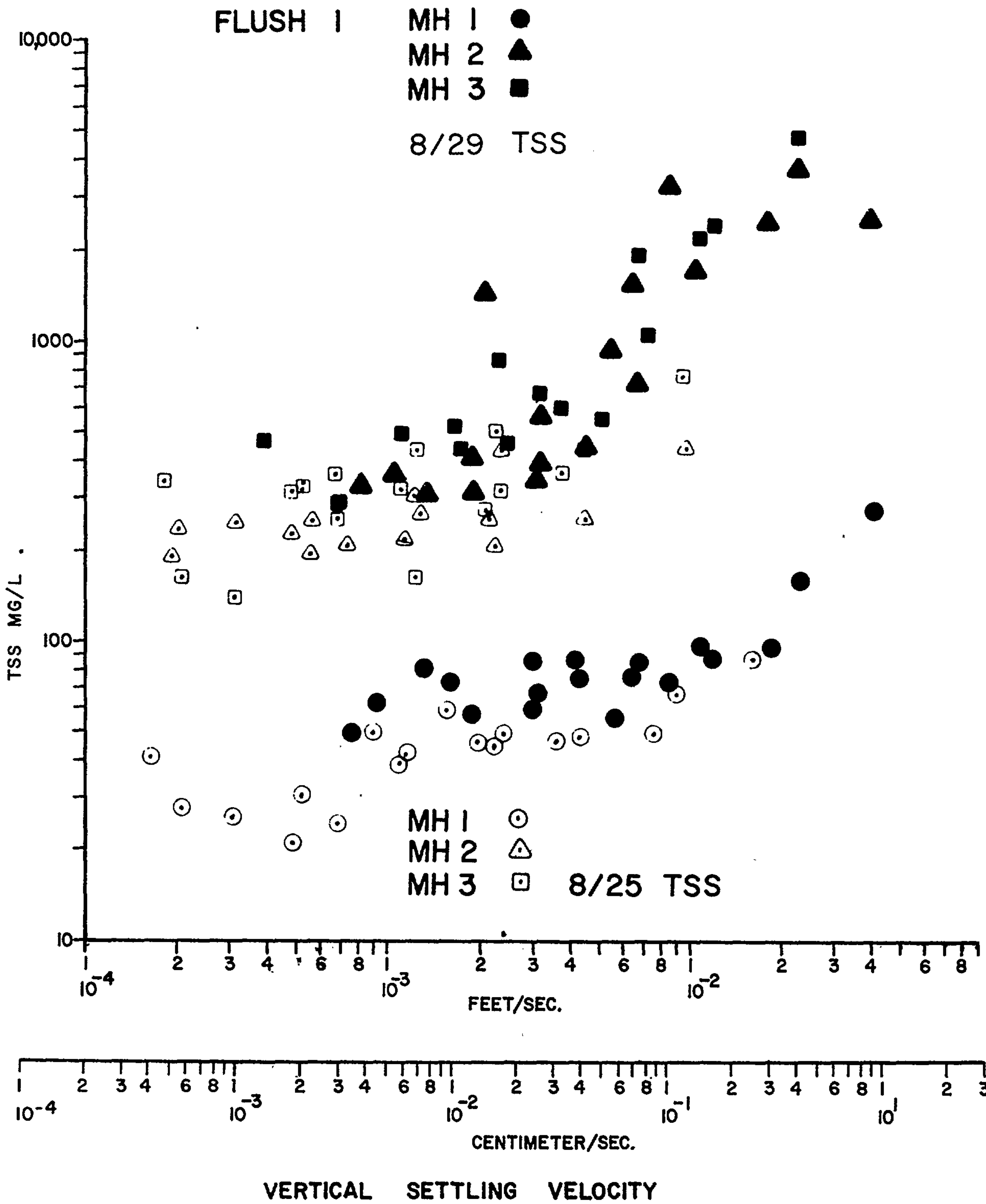


FIGURE 61. PLOT OF TSS CONCENTRATION VS SETTLING VELOCITY

TABLE 55. COMPARISON OF HEAVY METALS CONCENTRATIONS FROM SETTLING COLUMN TESTS

DATE	M.H. #	CADMIUM					COPPER					CHROMIUM				
		BK **	1st SAMP.	% REM.*	AVG.	% REM. AVG.	BK	1st SAMP.	% REM.	AVG.	% REM. AVG.	BK	1st SAMP.	% REM.	AVG.	% REM. AVG.
7/27	1	.0180	.0050	27.7	.0046	25.6	.6000	.1800	30.0	.1600	26.7	-	-	-	-	-
7/27	2	.0090	.0039	43.3	.0022	24.4	.9827	.5078	51.7	.3015	30.7	-	-	-	-	-
7/27	3	.0860	.0023	2.7	.0020	2.3	.9400	.1938	20.6	.1837	19.5	-	-	-	-	-
8/4	1	.0063	.0026	41.3	.0044	69.8	.7857	.1210	15.4	.2731	34.8	-	-	-	-	-
8/4	2	.0086	.0088	-	.0050	58.1	.6550	.6250	95.4	.3288	50.2	-	-	-	-	-
8/4	3	.0050	-	-	.0042	84.0	.4750	.5851	-	.3368	70.9	-	-	-	-	-
8/22	1	-	-	-	-	-	.2250	.4300	-	.3576	-	-	-	-	-	-
8/22	2	-	-	-	-	-	.7850	.3850	49.0	.1403	17.9	-	-	-	-	-
8/22	3	-	-	-	-	-	5.8500	.1900	3.2	2.76	47.2	-	-	-	-	-
8/25	1	-	-	-	-	-	.2750	.1150	41.8	.2673	97.2	-	-	-	-	-
8/25	2	-	-	-	-	-	.6250	.2950	47.2	.3197	51.2	-	-	-	-	-
8/25	3	-	-	-	-	-	.5550	-	-	.7363	-	-	-	-	-	-
8/29	1	.0020	.0022	-	.0028	-	.1800	.1544	85.8	.4386	-	.028	.063	-	.033	-
8/29	2	.0065	.0058	89.2	.0028	43.1	.5850	.8500	-	.3928	67.1	.120	.021	17.5	.038	31.7
8/29	3	.0006	.0021	-	.0030	-	.4100	.7500	-	1.600	-	.006	.011	-	.028	-
9/7	1	.0047	.0026	55.3	.0023	48.9	.4091	.2354	57.0	.2644	64.6	.019	.026	-	.017	89.5
9.7	2	.0062	.0050	80.6	.0032	51.6	6.1000	.8000	13.0	2.009	32.9	-	.013	-	.018	-
9/7	3	.0065	.0042	64.6	.0038	58.5	15.0000	.1900		3.632	24.2	.034	.030	88.3	.024	70.6

(continued)

- Indicates negative removals

* % Remaining

** BK - Background (initial) concentrations (mg/l)

TABLE 55 (Cont'd). COMPARISON OF HEAVY METALS CONCENTRATIONS FROM SETTLING COLUMN TESTS

		ZINC					LEAD					NICKEL				
		BK **	1st SAMP.	% * REM.	AVG.	% REM. AVG.	BK	1st SAMP.	% REM.	AVG.	% REM. AVG.	BK	1st SAMP.	% REM.	AVG.	% REM. AVG.
7/27	1	2.0800	.4080	19.6	.8176	39.3	-	-	-	-	-	-	-	-	-	-
7/27	2	2.0755	.9188	44.2	.5738	27.6	-	-	-	-	-	-	-	-	-	-
7/27	3	2.3520	.6500	27.6	.6323	26.9	-	-	-	-	-	-	-	-	-	-
8/4	1	1.2751	.4516	35.4	.9895	77.6	-	-	-	-	-	-	-	-	-	-
8/4	2	1.8600	1.6800	90.3	1.1700	62.9	-	-	-	-	-	-	-	-	-	-
8/4	3	1.8600	1.2000	64.5	.9693	52.1	-	-	-	-	-	-	-	-	-	-
8/22	1	.1400	.0640	44.4	.0883	61.3	-	-	-	-	-	-	-	-	-	-
8/22	2	.4040	.1320	32.6	.3564	88.2	-	-	-	-	-	-	-	-	-	-
8/22	3	.7080	.2200	31.1	.1992	28.1	-	-	-	-	-	-	-	-	-	-
8/25	1	.7040	.3680	52.2	.5125	72.8	-	-	-	-	-	-	-	-	-	-
8/25	2	1.6400	.6400	39	.5857	35.7	-	-	-	-	-	-	-	-	-	-
8/25	3	.2480	-	-	.7738	-	-	-	-	-	-	-	-	-	-	-
8/29	1	.5200	.5588	-	.7190	-	.0057	.0027	47.4	.002	35.	.026	.038	-	.035	-
8/29	2	2.0000	1.8400	92.0	2.0500	-	.0107	.0089	83.2	.035	-	.125	.117	93.6	.054	43.2
8/29	3	.7200	1.3320	-	1.2900	-	.0003	.0030	-	.002	-	-	.025	-	.056	-
9/7	1	1.3900	.5246	37.7	.5961	42.9	.0010	.0033	-	.002	-	.085	.052	61.1	1.120	-
9/7	2	4.8000	.2600	5.4	1.2700	26.5	.0012	.0080	-	.006	-	.092	.058	63.1	.040	43.5
9/7	3	5.2000	1.5200	28.7	1.8500	35.0	.0016	.0038	-	.003	-	.102	.111	-	.066	64.7

- Indicates negative removals

* % Remaining

** BK - Background (initial) concentrations (mg/l) they tend to move little of the total accumulated solids load, will wash out significant masses of heavy metals. Conversely, the data indicates that sewer flushing is very effective with respect to metals removal, and therefore minimizes storm entrainment and potential metals in overflows.

Figures 62 and 63 are plots of settling velocity vs concentration for copper (8/22) and nickel (8/29), respectively. The figures clearly show the wide scatter of the metals data which allowed only for assessment of settling trends, but again show the very high percent remaining plateau effect of the metals. Of all the metals, copper and zinc were the only metals to exhibit any real settling tendency as exhibited in Figure 62. Figure 63 is quite representative of the other metals where no real settling tendency existed.

The results of the metals testing program definitely showed that a significant portion of the heavy metals were associated with large particles with very high settling velocities, as shown by the rather large drop in concentration from the background to sample. The remaining fraction of fifty percent or more tended not to settle at any significant rate, and would be carried downstream for long distances.

10.3.2 Correlation Analysis

A correlation analysis was conducted between the various monitored pollutant parameters and settling velocity, TSS and VSS, in a linear and logarithmic fashion for various combinations of flushes. The results are shown in Table 56. High correlations were shown to exist between all parameters except metals. Significant correlations existed between cadmium, copper and zinc, with settling velocity, TSS and VSS. The relationship between these metals and settling velocity tends to reiterate the slightly positive settling velocity exhibited. The correlation analysis reiterated all of the prior findings.

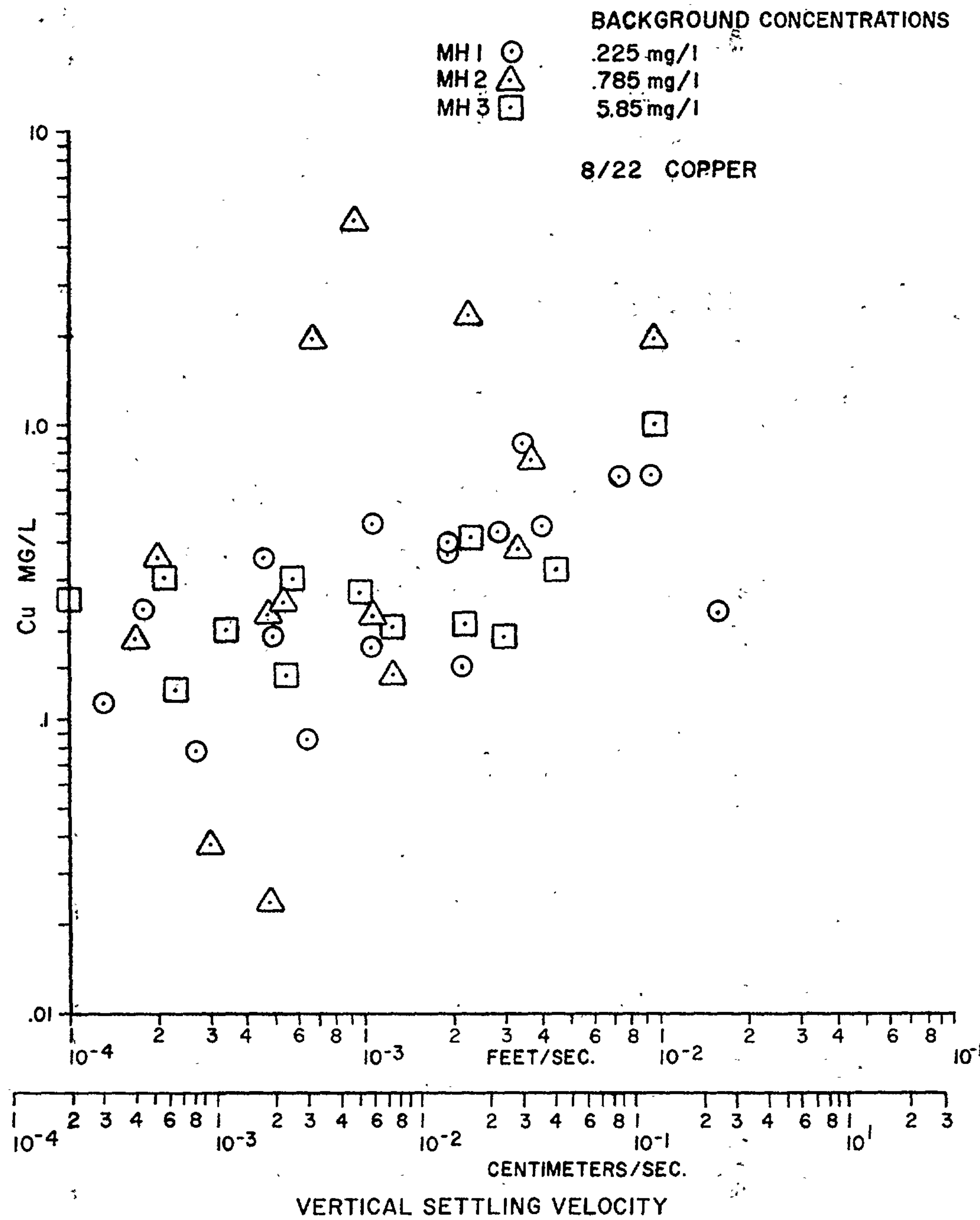


FIGURE 62 PLOT OF Cu CONCENTRATION VS SETTLING VELOCITY

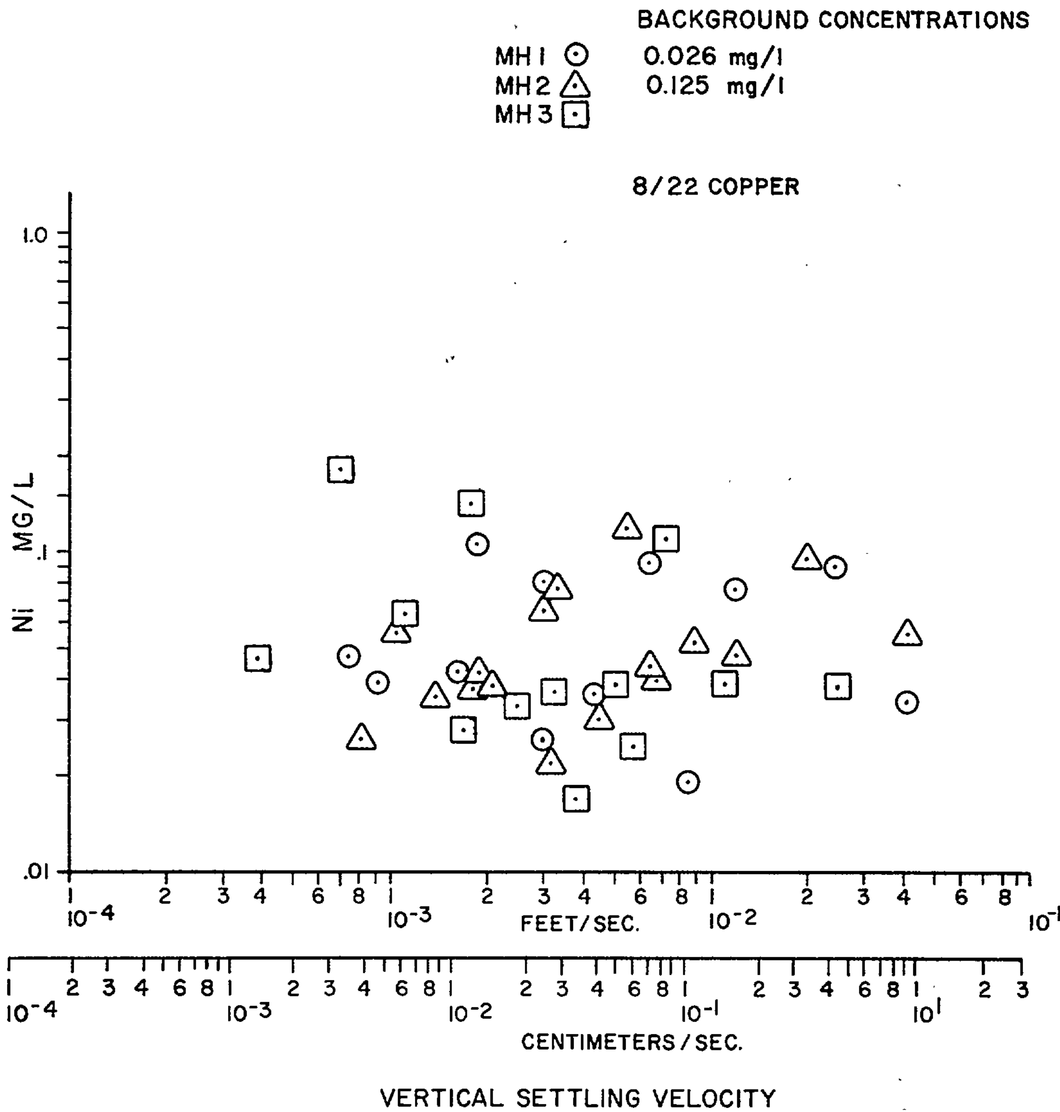


FIGURE 63 PLOT OF Ni CONCENTRATION VS SETTLING VELOCITY

TABLE 56. SETTLING COLUMN RESULTS CORRELATION MATRIX
COLUMN TESTS 8/22, 8/25, 8/29, 9/7+

IND. VARIABLE	MANHOLE 1 DEPENDENT VARIABLE										
	COD	BOD	TKN	TSS	VSS	Cd	Cr	Cu	Pb	Zn	Ni
VEL-LOG	.34	.68	.51	.54	.44	.29	.31	.34	.31	.38	.20
VEL-LINEAR	.46	.88	.75	.61	.55	.15	.27	.11	.12	.21	-.03
TSS-LOG	.91	.85	.65	1	.97	.34	-.08	.31	.26	.02	.14
TSS-LINEAR	.94	.87	.58	1	.98	.18	-.07	.07	.03	.21	-.03
VSS-LOG	.95	.86	.67	.98	1	.31	-.12	.29	.27	-.06	.10
VSS-LINEAR	.96	.86	.57	.98	1	.16	-.08	.06	.02	.19	-.05

IND. VARIABLE	MANHOLE 2 DEPENDENT VARIABLE										
	COD	BOD	TKN	TSS	VSS	Cd	Cr	Cu	Pb	Zn	Ni
VEL-LOG	.59	.61	.73	.62	.60	.41	.56	.34	.37	.49	.01
VEL-LINEAR	.43	.74	.65	.50	.50	.09	.39	.13	.39	.62	-.10
TSS-LOG	.90	.68	.71	1	.98	.28	.38	.51	.28	.41	.17
TSS-LINEAR	.90	.84	.79	1	.98	.14	.35	.95	.14	.46	-.09
VSS-LOG	.89	.68	.70	.98	1	.31	.38	.53	.28	.42	.18
VSS-LINEAR	.90	.83	.78	.98	1	.14	.35	.44	.14	.45	-.09

IND. VARIABLE	MANHOLE 3 DEPENDENT VARIABLE										
	COD	BOD	TKN	TSS	VSS	Cd	Cr	Cu	Pb	Zn	Ni
VEL-LOG	.39	.29*+	.59*+	.56	.56	.37	.36	.36	.38	.54	-.27
VEL-LINEAR	.22	.43	.61*+	.83	.83	.36	.23	.59	.27	.51	-.20
TSS-LOG	.78	.79*+	.67*+	1	.98	.32	.18	.54	.04	.25	-.14
TSS-LINEAR	.54	.85*+	.76*+	1	.98	.36	.10	.61	.05	.42	-.05
VSS-LOG	.78	.70*+	.60*+	.98	1	.27	.17	.54	.01	.25	-.15
VSS-LINEAR	.53	.84*+	.74*+	.98	1	.34	.10	.60	.05	.41	-.06

+ - Date set includes both short and long interval experiments.

*Less than ten observations.

TABLE 56. SETTLING COLUMN RESULTS CORRELATION MATRIX
COLUMN TESTS 8/29 and 9.7 (Cont'd) ++

MANHOLE 1

IND. VARIABLE	DEPENDENT VARIABLE COD	BOD	TKN	TSS	VSS	Cd	Cr	Cu	Pb	Zn	Ni
VEL-LOG	.22	.68	.51	.40	.31	.29	.31	.30	.31	.22	.20
VEL-LINEAR	.42	.88	.75	.57	.52	.15	.27	.05	.12	.10	-.03
TSS-LOG	.92	.85	.65	1	.97	.34	-.08	.15	.26	.19	.14
TSS-LINEAR	.93	.87	.58	1	.97	.18	-.07	-.01	.03	.20	-.03
VSS-LOG	.94	.86	.67	.97	1	.31	-.12	.14	.27	.16	.10
VSS-LINEAR	.95	.86	.57	.97	1	.16	-.08	-.02	.02	.19	-.05

MANHOLE 2

IND. VARIABLE	DEPENDENT VARIABLE COD	BOD	TKN	TSS	VSS	Cd	Cr	Cu	Pb	Zn	Ni
VEL-LOG	.64	.61	.73	.70	.69	.65	.56	.48	.37	.67	.01
VEL-LINEAR	.50	.74	.65	.61	.61	.47	.39	.11	.39	.60	-.10
TSS-LOG	.84	.68	.71	1	.96	.50	.38	.54	.28	.59	.17
TSS-LINEAR	.88	.84	.79	1	.97	.60	.35	.50	.14	.82	-.09
VSS-LOG	.84	.68	.70	.96	1	.52	.38	.56	.28	.59	.18
VSS-LINEAR	.88	.83	.78	.97	1	.60	.35	.50	.14	.81	-.09

MANHOLE 3

IND. VARIABLE	DEPENDENT VARIABLE COD	BOD	TKN	TSS	VSS	Cd	Cr	Cu	Pb	Zn	Ni
VEL-LOG	.42	.29	.59	.62	.60	.37	.36	.32	.38	.33	-.27
VEL-LINEAR	.61	.43	.61	.89	.88	.36	.23	.57	.27	.40	-.20
TSS-LOG	.79	.74	.67	1	.94	.32	.18	.54	.04	.43	-.14
TSS-LINEAR	.70	.85	.76	1	.96	.36	.10	.71	.05	.63	-.05
VSS-LOG	.77	.70	.60	.94	1	.27	.17	.52	.01	.42	-.15
VSS-LINEAR	.69	.84	.74	.96	1	.34	.10	.70	.05	.62	-.06

++ - Date set includes only long interval experiments.

SECTION 11

AUTOMATED SEWER FLUSHING AND RELATED TOPICS

11.1 Foreword

The results of the third phase field flushing program are described in this chapter. Several ancillary topics are also presented, covering joint wet weather sampling followed by flushing, and summary results of all background sewage sampling conducted during the project. Operational results for the automated flushing module installed on Shepton Street during the fall of 1977 are described in section 11.2. In the summer of 1977 the pollutant levels of several storm events were monitored on Shepton Street, using automatic samplers. Flushing experiments were immediately conducted on the segments to compare the relative fraction of material transported from the segment by the storm event and by sewer flushing. These results are described in section 11.3. The results of all the background sewage flow and quality sampling activities conducted during the three phases of field operations are summarized in section 11.4.

11.2 Results of Automated Sewer Flushing Module

The third phase of the sewer flushing research project dealt with the development and operation of a simplistic automated sewer flushing module. The device developed was an air-operated gate capable of backing up sewage flows to a predetermined level and then suddenly retracting, inducing a flushing wave. Mechanical and operational details of the device are presented in Chapter 5. This type of backup and release device was well suited to the situation as found on Templeton and Shepton Streets in Dorchester. The sewer lines in the Templeton/Shepton Street area were previously described in Chapter 4. Upstream of the flushing manhole is a hill allowing development of sufficient static head for a clean discharge. The module so developed was installed on 8/30/77 and operated on a daily basis from 8/31 - 10/31/77. During this period the module was checked at least 3 times per week to ensure performance as well as conduct sampling runs. Automated flushes were sampled seven times during the period of 9/22 - 10/13/77. Module operation was continued after 10/31/77 until mid January 1978 on a periodic inspection basis to assess long-term operation and serviceability, as well as visual performance with respect to flushing of both the upstream or reservoir segment and the downstream flushed segments.

Typical flush wave pollutant concentrations are shown in Figure 64 for the automated flushing module sampling experiment conducted on 10/3/77. The plots of COD, TSS and VSS shown in the figure are for samples collected

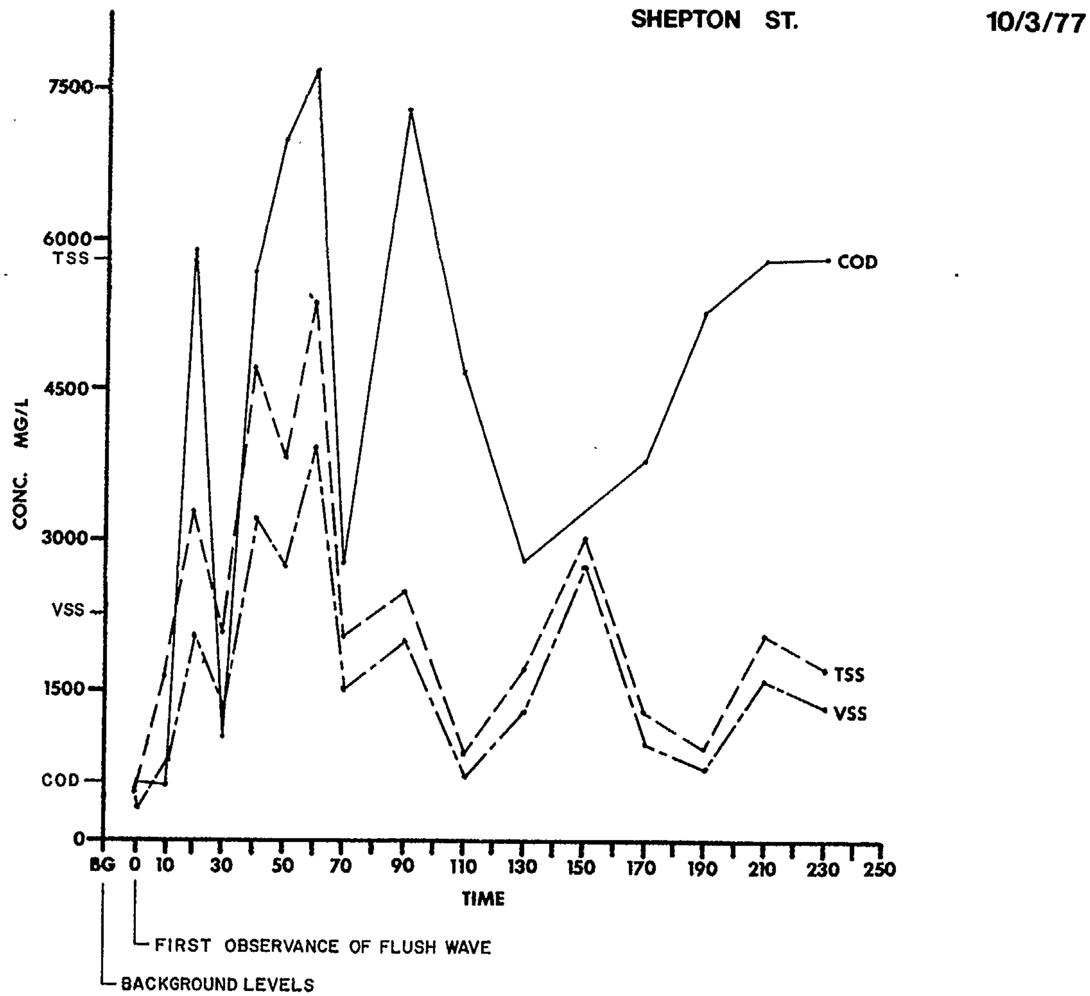


FIGURE 64 FLUSH WAVE CONCENTRATIONS FOR AUTOMATED SEWER FLUSHING

when the module released stored sewage, thereby inducing a flush wave. The high TSS and VSS background sewage levels for this experiment were atypical of sewage characteristics generally found on Shepton Street. Time plots of mass transported for COD, TSS and VSS are shown in Figure 65 for the 10/3/77 flush. Total masses removals of 15.76, 9.02 and 6.48 kilograms for COD, TSS and VSS are also indicated in Figure 65. Flush wave concentrations for the third phase experiments were similar to the first phase flushing experiments conducted at this location.

Flush volume details of the third phase flushing module operation are shown in Figure 66. A profile sketch of the two pipe segments upstream of the flushing module location is given in Figure 66. The first phase flushing manhole is 226 feet upstream of the module. The flushing module used in the third phase operation is located in the first phase sampling manhole. Profiles of backed-up sewage are depicted starting from 10 inches and ranging up to 16 inches of sewage referenced to the invert within the flushing module manhole. It should be noted from Figure 66 that the profile of backed-up sewage extends up to the first phase flushing manhole for a depth of about 10 inches. Depth of sewage backup greater than this height would extend beyond the first phase flushing manhole and well up into the second upstream segment. Accordingly, any flushing experiment where sewage backup is greater than 10 inches would actually be flushing portions of the second upstream segment from the flushing module. This distinction is important in interpreting the pollutant flushing removals.

A table of estimated volumes versus depth of sewage is also presented in Figure 66. Two estimates of volume are given for a given level of backup. The first estimates, labeled A in Figure 66, assume that only pipe segment volume is used whereas the second estimates, labeled B, account for manhole and house connection laterals volumes. These estimates were graphically determined from profile plots of the segments. Actual volume occupied during any experiment is not known accurately but probably fall somewhere in the range cited in the table.

The flushing mass removal results for the sewer sampling events are presented in Table 57. After the date of the experiment the depth of sewage backed up in the flushing manhole is given followed by estimates of the total mass removals of COD, TSS and VSS. The total mass removals are also the mass removals normalized by antecedent days between flushes since the module was programmed to flush daily.

TABLE 57: PHASE III - AUTOMATED SEWER FLUSHING POLLUTANT REMOVAL RESULTS

DATE	Depth (inches) of Backup at Flushing Module	Total Mass Removals (kg/Flush) COD	TSS	VSS
9/26/77	15.0	11.30	9.75	6.93
10/3/77	16.0	15.74	9.02	6.48
10/6/77	9.5	1.85	1.22	.89
10/7/77	9.0	1.17	.86	.53
10/11/78	11.9	2.33	1.53	1.09
10/13/78	14.0	4.08	2.62	1.72

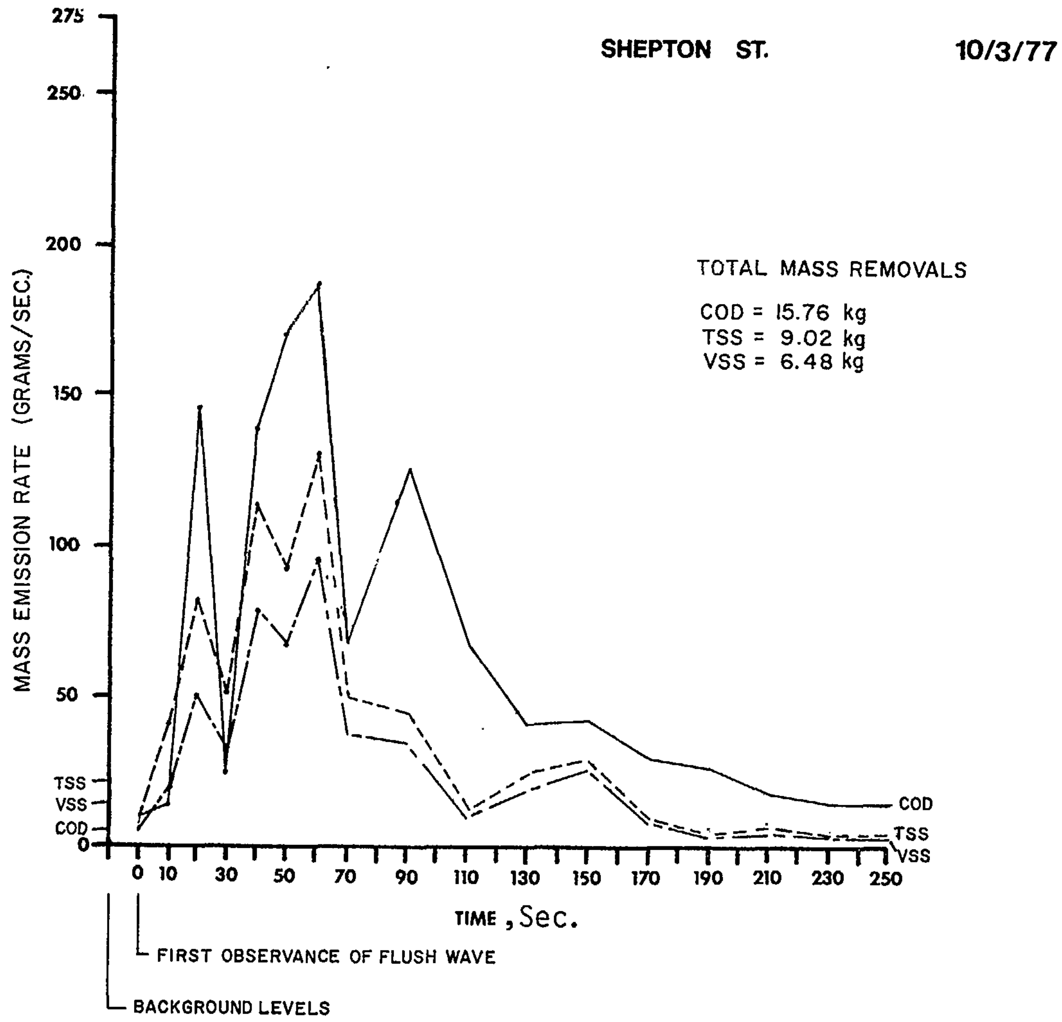


FIGURE 65 FLUSH WAVE MASS REMOVALS

DEPTH OF BACKUP (INCHES)	VOLUME (cf) A	B
8	54	67
10	82	100
12	108	132
14	151	180
16	194	230

A = PIPE VOLUME
B = PIPE VOLUME PLUS MANHOLE VOLUME PLUS LATERAL VOLUME

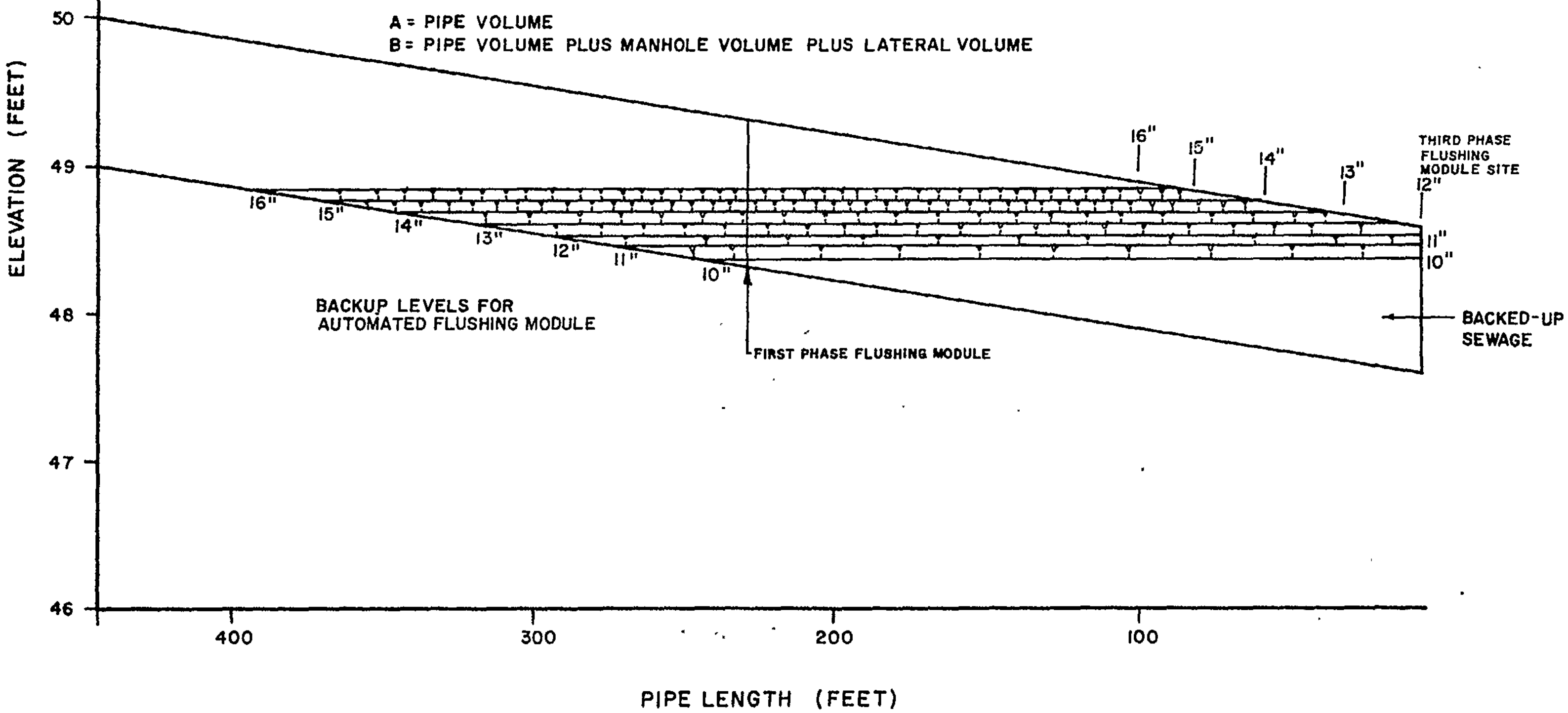


FIGURE 66 FLUSH VOLUME DETAILS OF THIRD PHASE PROGRAM

The pollutant removal results for the 10/6/77 and 10/7/77 experiments approximate the removal rates during the first phase flushing program on Shepton Street. Average normalized mass removals of COD, TSS and VSS for all good experiments conducted in the first phase are 1.79, 1.29 and 1.0 kg/day/flush. The depth of sewage backed up during these two experiments was less than 10 inches. These two experiments were flushing the same pipe length as in the first phase program. The depths of backed-up sewage exceeded 10 inches for experiments conducted on 9/26/77, 10/3/77, 10/11/77 and 10/13/77. For the experiment conducted on 10/13/77 the sewage profile extended 175 feet up into the second segment, or 400 feet from the flushing module. Inspection of the mass removal results for these dates in Table 57 indicate that the flushed loadings were substantially higher for these experiments in comparison to the two flushing experiments on 10/3/77 and 10/6/77. The last time the second segment upstream from the flushing module was flushed was during the precleaning work conducted in September of 1976, in preparation for the first phase program.

The mass removal results shown in Table 57 indicate that the automated flushing removal effectiveness was comparable to first phase flushing results. Sediment levels in both upstream segments were noted during the flushing events and during the maintenance checks. No accumulation of sanitary deposits was present in the pipes, indicating that the flushing waves adequately scoured and entrained any materials that deposited during the backup period. All pipe segments including the downstream sampling manhole were free of deposits during this period.

The module was left in operation at the end of the sampling phase until the middle of January, 1978 when the site was inspected. Review of the chart from the liquid level sensory devices, which recorded the depth of sewage backed up during a flushing operation, indicated that the module continued to perform daily unattended until the end of the year. The car batteries that provided electrical power to the device were inoperative. Temporary batteries were installed and the module went through a normal sequence. The module was again checked in late April of 1978. Visual inspection of the sediments indicated that sanitary deposition, sand and grit had again accumulated to pre-project levels.

11.3 Storm Event Monitoring

During the summer of 1977 an automated sampling was installed in the sampling manhole on Shepton Street for the purpose of monitoring the impact of stormwater flows on sewage deposits. The Shepton Street test segment is separated but receives substantial clear water inflow from roof drains connected into the sewer. The idea of the experiments was to monitor the pollutant masses transported during a storm, and then immediately flush the segment to determine residual deposited masses. Visual wet weather flow in small diameter pipe segments over the course of the project indicated relatively smooth flowline turbulent flow conditions. Flush waves at comparable depths of flow were always extremely turbulent with many random erratic flow eddies. It was of interest to determine just what effect stormwater conditions would have on suspending and transporting deposited organic pollutants. The concept of the experiment was simple. In practice, the logistics of field flushing just after a rainstorm was almost insurmountable.

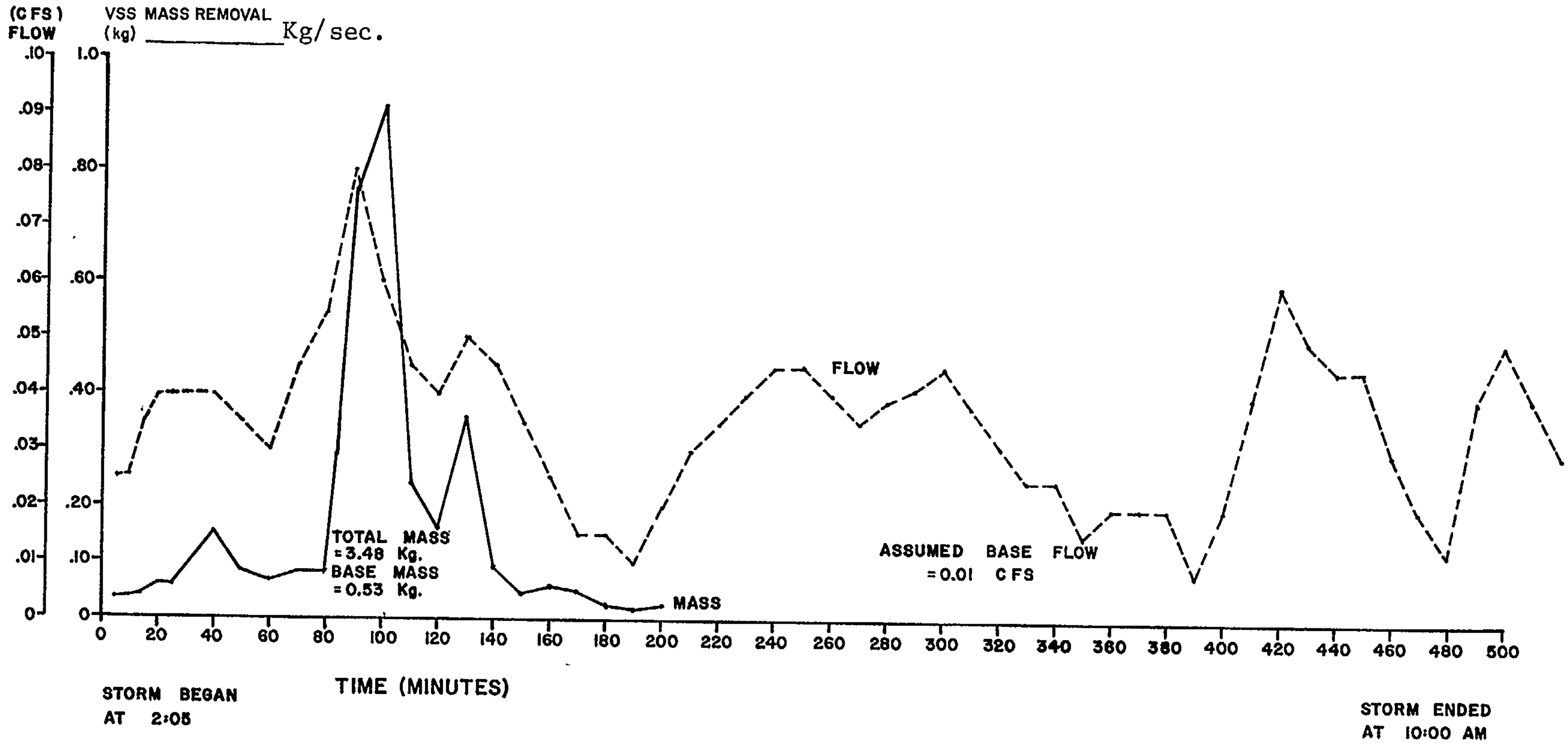
The first rainstorm monitored occurred on May 5, 1977. Very little rainfall was recorded at the Blue Hill Observatory. Heavy rainfall was noted at the site for about 1 1/2 hours during the storm event. The automatic sampler went into sequence and withdrew 24 1-liter samples at 10-minute intervals. Concentrations of TSS, VSS and BOD ranged from 150 to 250 mg/l over the four-hour period. BOD concentrations correlated with VSS levels. The liquid level sensing device was inoperative so that depths of flow were not recorded for the entire event. An average flow of .05 cfs was estimated for the storm event. The amount of BOD transported during the storm event, less the base level dry weather contribution, was roughly estimated at 3.53 kg. Heavy sanitary deposits were noted in the segment. The segment was immediately flushed within 2 hours using 50 cubic feet of water at a rate of about 0.5 cfs. Peak VSS and BOD levels during the flush event equaled 8000 and 1800 mg/l. The flushed masses of VSS and BOD were estimated to be 10.86 and 6.16 kg, respectively. The segment was flushed clean by the experiment.

A second storm event was monitored on June 7, 1976 on Shepton Street. The storm began at 2 a.m. and stopped at 10 a.m. Rainfall intensities of 0.17 and 0.14 inches per hour for the first two hours of the storm were recorded at the Blue Hill Observatory, and thereafter varied from trace to 0.10 inches per hour at 9 a.m. The automatic sampling began its sequence at 2:05 a.m. and continued for four hours, taking samples at 10-minute intervals. The liquid level sensing device monitored flow levels throughout the storm event. Figure 67 shows the storm hydrograph and the VSS pollutograph for the first four hours of the event. The peak VSS concentration monitored during the storm event was 1077 mg/l. The total mass under the measured VSS pollutograph is 3.48 kg, and the estimated base-line dry weather contribution is 0.53 kg. Although the rainfall information shows only significant rainfall falling within the last hour of the storm, the hydrograph during the final 5 hours shows several minor broad peaks. A flushing experiment was conducted at 10 a.m. on that morning. Peak flush wave VSS concentrations reached 8100 mg/l and the estimated flushed mass is 7.37 kg. The segment contained sanitary deposits prior to flushing and was clean after the flushing event.

Although the results of these two storms are meager, several suggestive inferences can be drawn from the data. First of all, sanitary deposits are not easily dislodged and transported during storm events. Extreme "first flush" phenomena may be the results of an extreme storm event flushing the accumulated deposits of a long sequence of dry and alternatively light rainfall periods. Secondly, flushing does cause the required turbulence to to suspend, entrain and transport sanitary deposits. Thirdly, the time interval required for flushing can be, on the average, longer than the average return period for storm events, since minor to moderate intensity storms may not flush clean the pipe segments.

FIGURE 67

SHEPTON STREET– TYPICAL STORM
HYDROGRAPH / POLLUTOGRAPH
6/7/77



11.4 Background Sewage Characteristics

Summary statistics of background sewage characteristics in mg/l for the four test segments are given in Table 58. The mean and standard deviation shown for each pollutant was computed using data collected over the entire field flushing program. The results are generally atypical of sewage strength levels normally encountered at treatment plants for strictly domestic sewage. Both the mean levels and coefficients of variation are markedly higher, especially for Port Norfolk and Walnut Streets, than comparable treatment plant levels. Great care was taken in sample collection to ensure that sediments were not scraped into sample bottles.

Dry weather sewage flows were measured at different times of the year over the course of the project. Procedure measurement details are given in Chapter 5. Measurements were primarily performed at Shepton Street and Port Norfolk Street test segments since discharge levels at Walnut and Templeton Streets were extremely sluggish and difficult to measure. Instantaneous flow rate determinations at Shepton Street ranged from 5800 to 14,200 gpd with an average of 12,900 gpd. The per capita waste rate for the Shepton Street test segment is 56 gpcd using the average discharge and the census population of 230 people. Dry weather sewage flow gaged at the first phase sampling manhole at Port Norfolk Street ranged from 3000 to 4000 gpd with an average value of 3600 gpd. A special flume described in Chapter 5 was installed in the first phase flush injection manhole. The flume was flow calibrated and then used to determine dry weather flow levels. Discharge rates at this location averaged about 0.0025 cfs corresponding to a per capita waste rate of 53 gpcd. The estimated population tributary to the first phase manhole down to the sampling manhole was adjusted downward from 94 to 60 people based on an assumed rate of 60 gpcd for the entire street. Per capita waste rates of 60 gpcd were assumed for the Templeton and Walnut Street segments based on the flow measurements at Shepton and Walnut Streets. The per capita waste rates are low but agree with previous measurement work performed in the Dorchester area (4).

TABLE 58. SUMMARIES OF BACKGROUND SEWAGE CHARACTERISTICS

BACKGROUND CONCENTRATIONS (mg/l)

POLLUTANT	PORT NORFOLK		WALNUT		TEMPLETON		SHEPTON	
	Mean/Std.Dev.*	No. Samples	Mean/Std.Dev.*	No. Samples	Mean/Std.Dev.*	No. Samples	Mean/Std.Dev.*	No. Samples
COD	2065/2066	21	2314/4238	17	546/ -	1	405/ -	2
BOD	774/714	14	1069/1468	13	407/273	10	256/49	5
TKN	117/103	12	91/69	12	17/ -	1	54/ -	1
NH_3	29/17	9	43/15.2	12	8/ -	1	33/ -	1
TP	17/11.2	9	20/13.2	12	4.8/ -	1	11.5/ -	1
OP	8/5.2	9	3.5/7.5	12	1.8/ -	1	10.0/ -	1
SS	1116/1829	50	1792/2879	27	1112/1323	26	392/971	36
VSS	1332/1813	35	1523/2074	26	1021/1226	20	265/204	12

*Std. Dev. - Standard deviation

SECTION 12

PREDICTIVE TOOLS

12.1 Foreword

In this chapter two predictive approaches are presented that are useful in the general assessment of potential sewer flushing programs. The first model deals with estimating the transported fraction of flushed pollutant loading as a function of distance downstream from the point of flush. This approach was empirically developed from the second phase field flushing program data. A comparison is presented of results using the prediction procedure with other second phase data. The second model discussed is a generalized procedure for estimating daily dry weather sewage solids deposition loadings within each manhole to manhole segment of an entire collection system network. A comparison of predicted pollutant deposition loadings for the phase one test segments with phase one field flushing results is also presented.

Estimates of flushed solids transported downstream using the first of the two aforementioned modes are presented in Section 12.2. This procedure was expanded to provide estimates of organics and nutrients transported downstream by flushing and these results are presented in Section 12.3. Details of a generalized sewer system deposition model are given in Section 12.4. Verification of the approach using field flushing results are given in Section 12.5.

12.2 Downstream Transport of Solids in Suspension

12.2.1 Overview of Methodology

Solids suspended by a flush wave will tend to redeposit some distance downstream from the point of suspension due to the action of gravity. The distance downstream at which particles of a given size will settle is a function of the particle characteristics, concentrations and the residual energy and shear force of the flush wave or flow at those distances.

The question as to how far the suspended materials will go is fundamental in establishing the spatial frequency of flushing to guarantee that a specified fraction of the flushed materials will ultimately reach a treatment plant. The objective here is to provide a crude and preliminary answer to that question. A more accurate analysis of the flush wave effectiveness in carrying the scoured solids long distances downstream would require characterization of the settleability characteristics of the solids picked up by the wave and the development of a predictive tool for the propagation of the wave downstream.

Knowledge of the evolution of the wave or flow in space and time, associated with some criteria of shear stress, would probably provide the best assessment of its downstream carrying capacity. Nevertheless, it should be recognized that regardless of the level of effort and sophistication used, the answers that can be derived from such an approach are still based on ideal conditions and crude approximations to the problem. This operational conclusion was reached due to the impossibility of actually predicting the behavior of the wave and the materials in suspension under the effects of manhole drops, sharp flow turns, and the downstream interference of household connections and contributions from merging portions of the collection system.

Ackers and Harrison (25), Mitchel (26), and Martin and deFazio (27) dealt successfully with the specific problem of the attenuation of flood waves in straight, circular pipes, using numerical methods for the solution of the continuity and momentum differential equations involved. Sonnen (28) modelled the transport of sediments in sewers by considering three distinctive transport mechanisms recognized in the literature, namely: bed-load, suspended load and washload transport. Each of the processes was modelled separately, with sediment mass balances performed to guarantee that the predictions do not exceed the amounts of materials available for each type of transport. Although the application of these concepts to the problem of sediment transport in collection system laterals is encouraged, many questions regarding the whole methodology still remain unresolved and require considerably more research. It is believed that unless the hydraulic problem is solved with greater accuracy, especially in the case of the small volume flushes occurring over short time intervals considered in this study, sophisticated approaches to the sediment transport process itself are not justified.

A much simpler approach employed here used estimates of forward velocities together with discrete settling and indirectly accounting for flocculance and hindrance. Figure 68 depicts the methodology used and the following steps summarize the computations performed:

1. The travel times of flush waves to the three downstream manholes measured during the Port Norfolk second phase flushes were used to define an exponentially decaying function* relating velocity with downstream distance travelled. Assuming a straight pipe of uniform slope and neglecting downstream flow contributions a residual wave forward velocity, V_L, was estimated at a downstream distance, L, as follows:

$$V_L = e^{a+bL} \tag{23}$$

2. An average velocity over the distance, L, was defined as the average of the velocities V_0, at L = 0, and V_L computed at L, as follows:

$$V = 0.5\ (V_0 + V_L) = 0.5\ e^a(1+e^{bL}) \tag{24}$$

3. With the estimated average forward velocity V, an iterative procedure (method of secants) was used to solve Manning's equation for the depth of flow h;

* Empirical function with parameters determined by least squares.

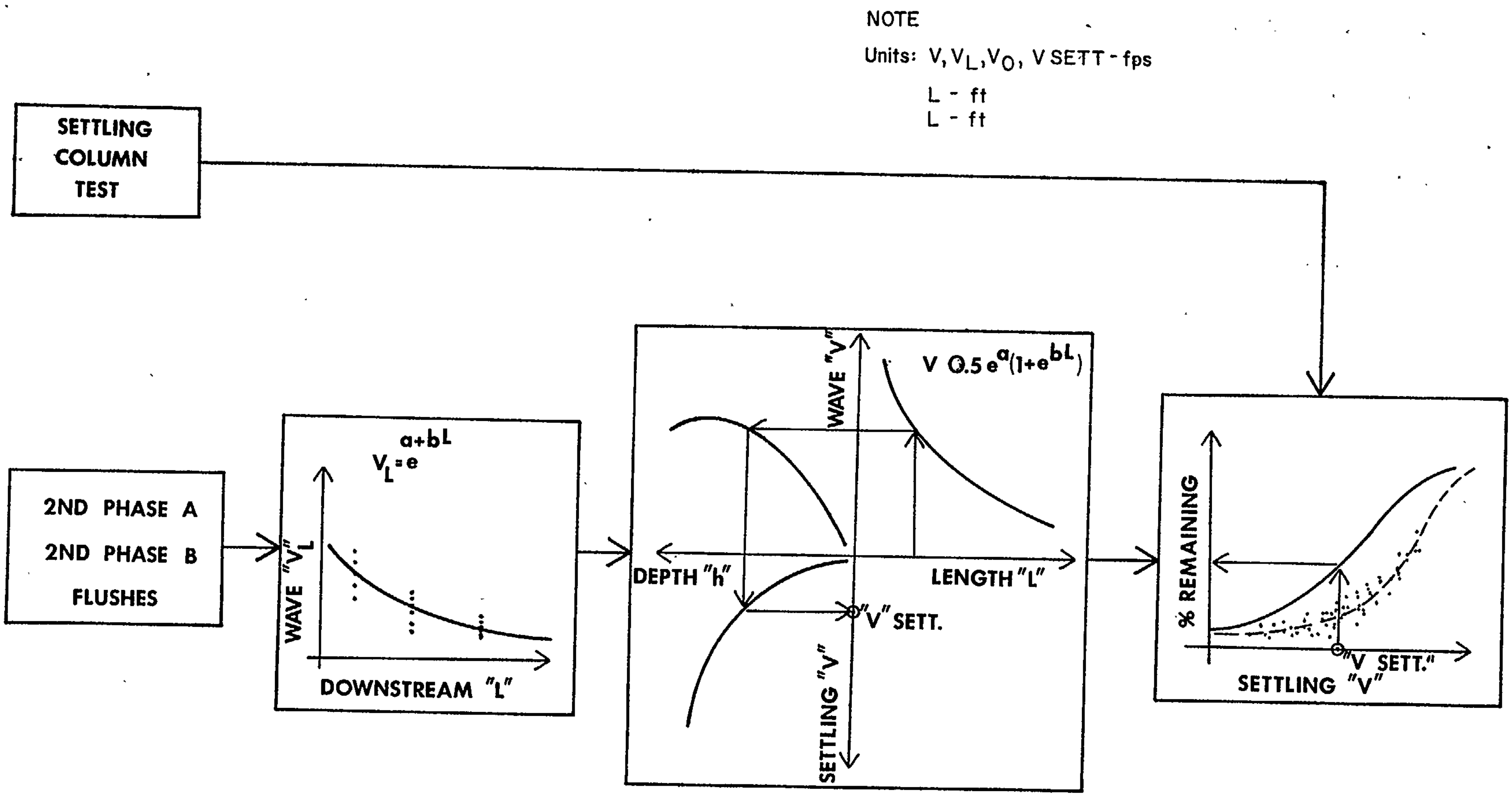


FIGURE 68. OVERVIEW OF PROCEDURE FOR ESTIMATION OF FLUSHED SOLIDS TRANSPORTED DOWNSTREAM

4. Assuming the flow depth to be uniform and equal to h from 0 to L, the geometry of the pipe and the flow depth, h, defined the cross-sectional flow area, A_C (ft^2);

5. The flow surface area, A_s (ft^2), was computed by:

$$A_s = \frac{\text{Volume}}{\text{Depth}} = \frac{A_c \cdot L}{h} ; \tag{25}$$

6. The overflow rate defining the cut-off settling velocity of the particle that just passed length L in suspension is given by:

$$\text{O.R.} = V_{\text{Sett.}} = \frac{\text{Flow}}{\text{Surface Area}} = \frac{A_c \cdot V}{A_s} \tag{26}$$

Equation 26 can be rewritten as:

$$V_{\text{Sett.}} = \frac{h \cdot V}{L} \tag{27}$$

which could have been derived directly by equating the time of fall of the particle to the bottom of the pipe with its time of horizontal travel to the distance L;

7. The settling column test results from all three downstream manholes in Port Norfolk Street were used to define a curve relating the percentage of solids remaining in suspension with settling velocity and also accounting for flocculent and hindered settling; and

8. Finally, the fraction of the suspended solids that remain in suspension throughout length, L, can be defined using the settling velocity from equation 27 in the percent remaining versus settling velocity curve.

12.2.2 Definition of the Flush Wave Forward Velocity

The travel times of the flush wave, measured at three downstream manholes from the flush injection manhole, in Port Norfolk Street, were used to define an exponentially decaying function relating the flush wave velocity with downstream pipe length. The travel times used in each flush and at each manhole were those corresponding to the arrival of the peak of the flush wave, identified by the highest depth measured. Whenever the maximum depth was measured at more than one successive time step, indicating a flat peak, the average of the times to the first and last highest depth measurements was used. A diagram of the flushing and sampling manholes along Port Norfolk Street is presented in Figure 20, Chapter 5.

The total distances traveled by the wave to the successive downstream manholes are approximately equal to:

First Downstream Manhole (DMH)	162 ft.
Second " "	409 ft.
Third " "	659 ft.

These distances, combined with the travel times defined above, provided estimates of the flush wave velocities. There were a total of 101 velocity determinations for all three downstream manholes from the second phase field program.

A linear regression of the logarithm of the velocities on the corresponding manhole distances, for all 101 points, yielded a correlation coefficient about equal to -0.4. The mean and standard deviation of the velocities at each manhole were then computed and any velocity measurement at each manhole falling outside its corresponding range of the computed mean $\pm$ 2 standard deviations was excluded in a new regression run. An improved correlation coefficient was obtained. The resulting equation for velocity, V_L, at a downstream distance L is:

$$V_L = e^{0.789-0.000634L} \tag{28}$$

where $e = 2.71828$ $(R = -0.63)$

A total of 10 velocity points were eliminated from the original set leaving 91 points, almost equally distributed among the three manholes, for the regression. It should be noted that the data used in the regression encompassed different flush volumes and injection rates, which explains in part the correlation coefficient obtained.

A partition of the velocity data by flush volumes and injection rates could have been adopted. The percentages of solids remaining in suspension at downstream lengths, L, could have been associated with ranges of flush volumes and injection rates. However, it was felt that the precision of the whole approach would not warrant such refinements.

12.2.3 Overflow Rates or Settling Velocities

For a fixed downstream distance L, the wave average velocity, V, can be estimated using equations 23 and 28. Computation of the overflow rate at the distance, L, requires that a flow depth be associated to that velocity. Manning's equation was used to determine depth of flow. The slope used in Manning's equation derived from the results of the loop-rating curve optimization described in Chapter 7. Although the three pipe segments flushed in Port Norfolk Street have different slopes, the optimized slope of the intermediate pipe segment was assumed constant throughout the range of valid values of L. Programming logic was prepared to solve Manning's equation for h given the velocity V, by a trial and error approach of drawing successive secants to the velocity versus depth curve. Precise convergence was obtained in a few iteration steps.

Using this flow depth together with the distance L and corresponding forward velocity V in equation 27 the overflow rate or settling velocity can be computed.

12.2.4 Percent Solids Remaining versus Settling Velocity Curve

The computed values of percent solids remaining in suspension and their associated vertical velocities, in fps, derived from the settling columns tests described in Chapter 10, were plotted and are shown in Figure 69 for all three downstream manholes. A smooth average curve was then drawn through those points labelled, "average settling column curve." This curve was constructed in the following manner. The settling velocities were broken into small ranges and all remaining fractions falling within a given range were averaged out and the average values were plotted against the mid-range value of the settling velocity. The "average settling column curve" was then drawn through those averaged points with a high degree of adherence. This curve was considered to be representative of the ideal settling column.

Another curve was defined to approximate the turbulent settling conditions occurring in the sewer lines, labelled "conservative turbulent settling curve," Five plotted points falling exactly on or very near the average settling column curve, and well spaced in the range of 1×10^{-4} to 3×10^{-3} fps, were used to construct that curve. The settling column data referring to those points were then reviewed by: a) doubling the settling or detention time while keeping the percentage remaining unchanged; and b) assuming the overflow rate to be only 65% of the revised overflow rate. A smooth curve was drawn connecting the plotted revised settling velocities corresponding to the five selected points as indicated above. The criteria used in drawing the revised curve is usually adopted in the design of settling basins. Conditions in sewer lines are certainly more turbulent implying that the resulting curve should be considered conservative in regard to the fractions remaining in suspension.

The turbulent settling curve was fitted by a polynomial in the range of settling velocities of 1×10^{-4} to 1×10^{-2} fps. This polynomial is given by:

$$P = 424.\text{x}10^5 V^3_{Sett.} - 960.\text{x}10^3 V^2_{Sett.} + 123.\text{x}10^2 V_{Sett.} + 10.8 \qquad (29)$$

where P = Percent TSS remaining in suspension
$V_{Sett.}$ = Settling velocity, in fps.

It should be noted that a fraction of particles, i, with settling velocities less than that given by equation 27 defined as V_i, will also settle along the distance, L. In the design of settling tanks it is usually assumed that the fraction is given by $V_i/V_{Sett.}$ for a particular i. That fact was neglected here under the assumption that the conservative turbulent settling curve already accounts for this fraction.

12.2.5 Final Results

The procedure described above was used to compute final percentages of solids remaining for various downstream distances, L. A plot of the computed values is shown in Figure 70 indicating the percentage of TSS remaining in suspension, P, as a function of downstream distances, L, in feet. The results indicate that the

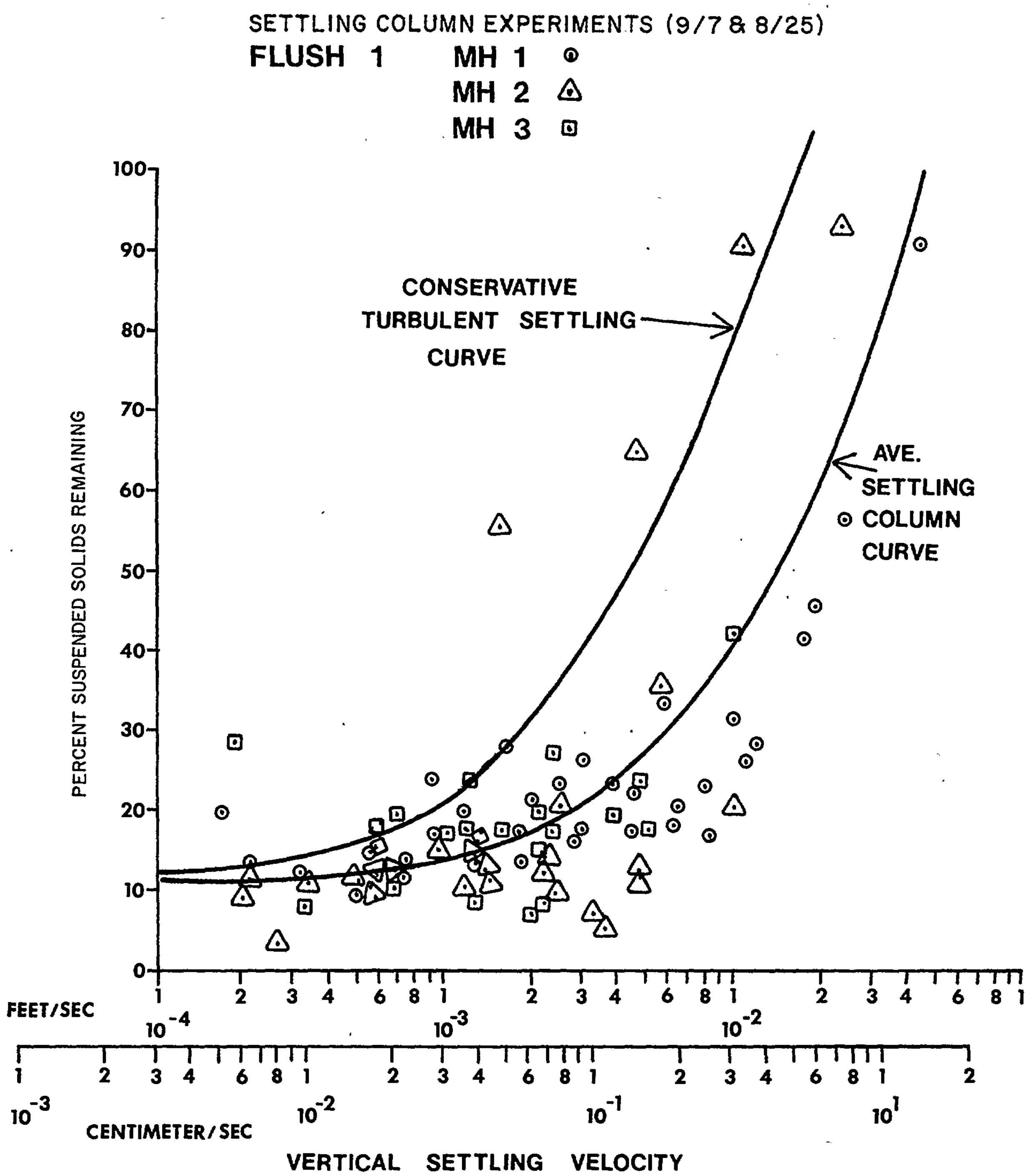


FIGURE 69. PLOT OF PERCENT SUSPENDED SOLIDS REMAINING VERSUS SETTLING VELOCITY

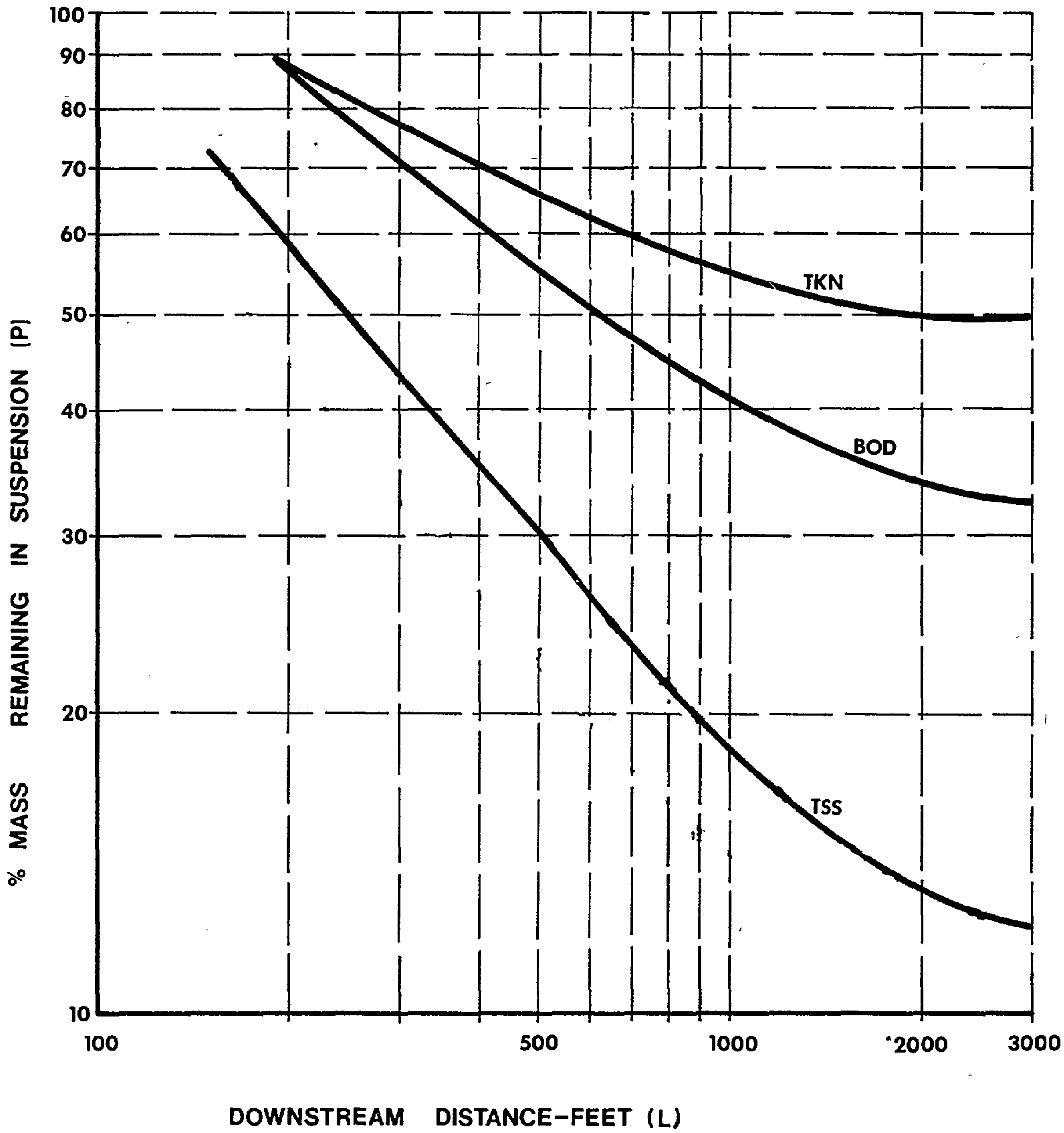


FIGURE 70 PERCENT TSS, BOD AND TKN REMAINING IN SUSPENSION VERSUS DOWNSTREAM DISTANCE FROM MANHOLE FLUSHED. PORT NORFOLK DATA

percentages of solids suspended from the first segment remaining in suspension at the successive downstream manholes in Port Norfolk are:

First Downstream Manhole (DMH)			68%
Second	"	"	34%
Third	"	"	24%

The computations also indicate that roughly 12% of the solids suspended from the first segment would not resettle again. This estimate agrees favorably with the Imhoff cone results for the first flush at the first sampling manhome described in Chapter 10. The percentage of solids remaining in suspension is 16.9% which was computed by averaging out the results from all Imhoff cone tests performed on samples taken from the first flush for this location.

The smooth curve drawn in Figure 70 is mathematically expressed by:

$$P(\%) = -\,1.33\,(\ln L)^3 + 33.86\,(\ln L)^2 - 288.57\,(\ln L) + 834.32 \qquad (30)$$

$$(160 \leq L \leq 3000)$$

where: P = percentage of solids remaining in suspension;
ln = indicates Napierian logarithms; and
L = downstream distances, in feet

The percentages given by equation 30 should apply only to deposits over a single segment downstream from the flushing manhole. A convolution of equation 30 would be necessary to account for masses removed from more than one pipe segment.

Examples of convolution computations over multiple segments involving the application of equation 30 are given in the next section and in section 12.3.1. Furthermore, detailed examples are also presented in Chapter 16 of this report.

12.2.6 Verification

The methodology just described was verified using average TSS values measured in the Port Norfolk second phase serial flushing program, described in Chapter 9. The average masses removed by the first, second and third flushes of the day in each of the manholes were computed and are presented in Table 59. This table has been previously prepared in Chapter 9 but is repeated for ease in reference. The percent removals at each manhole, by flush, were also computed and are indicated in Table 59 assuming that after the third successive flush all deposits had been flushed out.

If the assumption is made that the first pipe segment was actually clean after the third flush, then the percentages indicated in Table 59 for manhole 1 can be taken as average percentages of the total mass deposited in the pipe which were removed by each successive flush. The same interpretation cannot be applied for manholes number 2 and 3, because it is unclear how much of the measured masses originated in the pipe segment immediately

TABLE 59. AVERAGE TSS MASS AND PERCENT REMOVAL BY FLUSH AND BY MANHOLE - SECOND PHASE-SERIAL FLUSHES

Flush No.	Manhole Number 1 Mass(kg)	1 %	2 Mass(kg)	2 %	3 Mass(kg)	3 %
1	2.85	76.05	4.03	72.39	7.39	66.52
2	0.48	12.74	0.79	14.20	2.24	20.17
3	0.42	11.21	0.75	13.41	1.48	13.31
Totals	3.75	100.00	5.57	100.00	11.11	100.00

upstream from the manhole, and how much came as suspended load from upstream segments.

Using the first segment as a reference, the settling velocity methodology just described was applied successively to the three segments and the average total TSS masses transported past each manhole were estimated and compared to the average measured values for the first flush. The following computations were performed in terms of the average values given in Table 59. The average removal for flush 1, manhole 1 is 76%. The settling velocity criteria estimated that percentage as 67.5%, implying that the total mass in the first pipe segment, before the flushes, was actually 4.22 kg, as opposed to 3.75 kg. This means that after the third flush there was still 0.47 kg of TSS remaining in the pipe segment. In order to estimate, from the percentage removals, the actual masses flushed out of each pipe, it is necessary to have an estimate of the total masses deposited in each pipe segment prior to the flush. Starting with the value of 4.22 kg in the first segment, it is a reasonable approximation to assume that the deposits in the other pipe segments are inversely proportional to their slopes. Total deposit estimates of 6.09 kg in the second downstream segment and 7.40 kg in the third using that assumption. These values are indicated in Table 60.

Percent removals from the successive downstream segments were computed using exactly the same procedure described for the removals from the first segment, only that the overflow rates or settling velocities were computed from equation 27 modified to:

$$V_{Sett.} = \frac{h \cdot V}{L - L_o} \tag{31}$$

where: L_o = 0 ft for the materials removed from the first segment;

L_o = 162 ft for the materials moved from the second downstream segment;

L_o = 409 ft for the materials moved from the third downstream segment;

and, all other variables have exactly the same meaning as defined for equation 27.

The percent removals computed by the methodology using equation 31 instead of 27 are presented in Table 60. Direct use of equation 30 in the convolution computations would have slightly overestimated the percentages indicated for segments 2 and 3 because the average wave velocities corresponding to those segments would have been slightly overestimated. The percentages indicated in Table 60 can be applied to the appropriate total masses deposited in the pipe segments to yield the estimated total masses flushed out from each manhole. Assuming that all the deposits in each segment are at some point in time suspended by the flush wave to redeposit again (reasonable assumption for a few pipe segments), and assuming independence in the process of scouring and deposition of the materials from each pipe segment. The resulting values are shown in Table 60 as Item 3 - estimated mass flushed out. Comparison of the estimated and measured average values shown in the last two lines of Table 60 indicate a satisfactory agreement for the masses flushed out from manholes 2 and 3. The value for manhole 1 was fixed as a starting point.

The proceeding result supports the credibility of the crude approach used in this analysis to define TSS masses remaining in suspension as a function of downstream distance from the point of the flush injection. It also encouraged using the TSS formulation as a basis for assessing flush wave efficiency in transporting organics, typified by BOD, and nutrients, typified by TKN.

TABLE 60. COMPARISON OF MEASURED VERSUS ESTIMATED AVERAGE TSS MASS REMOVALS - PORT NORFOLK ST. (SECOND PHASE)

Item	Manhole Number		
	1	2	3
1. Estimated mass deposited (kg)	4.22	6.09	7.40
2. % remaining in suspension from:			
Segment 1	67.50	34.20	24.20
Segment 2	-	46.80	27.90
Segment 3	-	-	41.20
3. Estimated mass flushed out (kg)	2.85	4.29	5.77*
4. Measured mass flushed out (kg)	2.85	4.03	7.39

*$0.242 \times 4.22 + 0.279 \times 6.09 + 0.412 \times 7.40 = 5.77$.

12.3 Downstream Transport of Organics and Nutrients

As described in Chapter 10, the samples collected from the settling column tests were analyzed for VSS, COD, BOD, TKN, NH_3, OP and TP besides TSS. The settling column determinations of BOD and TKN were used to estimate flushing efficiency in transporting suspended organic matter and nutrients downstream.

The following steps were performed:

a) All settling column analytical results including results of samples from all flushes and all sampling ports were used to define regression equations relating the fractions of BOD and TKN removed from suspension to the fractions of TSS removed from suspension. The regression equations computed in this step are given by:

$$BOD_{Sett.} = 0.861 \times TSS_{Sett.}^{2.01} \qquad (R = 0.74) \qquad (32)$$

$$TKN_{Sett.} = 0.601 \times TSS_{Sett.}^{1.465} \qquad (R = 0.80) \qquad (33)$$

Where $BOD_{Sett.}$, $TKN_{Sett.}$ and $TSS_{Sett.}$ above refer to the fractions of BOD, TKN and TSS removed during the settling analysis. The correlation coefficients of 0.74 and 0.80 for BOD and TKN indicate that fractions of settleable BOD and TKN can be reasonably estimated on the basis of the fractions of TSS removed. The TSS fraction settled values used in the BOD regression covered the range of 0.44 to 0.96, with the great majority of the values falling above 0.7. In the TKN regression the TSS values ranged from 0.10 to 0.96, again with the great majority of the values above 0.7.

b) Equation 30 was used with the above regression equations to determine the percentages of BOD and TKN remaining in suspension as a function of downstream distance from the point of the flush injection. Given equations 30 , 31 and 32 the following calculations were performed:

- For a fixed, L, equation 27 was used to estimate the percent of TSS remaining in suspension, P;
- The value (1. - P/100) represents the fraction of TSS removed from suspension, $TSS_{Sett.}$;
- $TSS_{Sett.}$ = (1. - P/100) was used in equations 32 and 33 to estimate $BOD_{Sett.}$ and $TKN_{Sett.}$, the fractions of BOD and TKN that redeposited; and,
- The values (1. - $BOD_{Sett.}$) and (1. - $TKN_{Sett.}$) represent the estimates of the fractions of BOD and TKN still remaining in suspension at the downstream distance, L.

Plots of BOD and TKN remaining as a function of downstream distance, L, computed for several values of L, are shown in Figure 70. The results indicate that the percentages of BOD and TKN suspended from the first segment remaining in suspension at the successive downstream manholes (DMH) are approximately as follows:

	BOD	TKN
First DMH	90%	90%
Second DMH	62%	69%
Third DMH	50%	60%

The computations also indicate that roughly 33% of the BOD and 50% of TKN suspended from the first manhole would not resettle again.

A brief assessment of the settling column results for VSS, COD, NH_3 and TP indicates that, if a similar approach used for BOD and TKN was applied, their percent remaining curves would plot on Figure 70 relative to the TSS, BOD and TKN curves, as follows: a) VSS - slightly above the TSS curve; b) COD - between the TSS and BOD curves; c) NH_3 - well above the TKN curve; and, d) TP - slightly above the TKN curve.

Notwithstanding the limitations of this procedure, a more comprehensive predictive tool could be developed using regression or normographs to give approximate estimates of the fractions remaining in suspension as a function of flush volume and rate, and pipe size, slope and length. The generalized procedure could be developed if it were repeated for a series of different pipe sizes and slopes and if the flushes were separated by volume and rate.

12.3.1 Verification

The average BOD mass removals from the second phase (serial) flushing program are shown in Table 61, with the average percent removals by flush and by manhole.

TABLE 61. AVERAGE BOD MASS AND PERCENT REMOVAL BY FLUSH AND BY MANHOLE SECOND PHASE SERIAL FLUSHING PROGRAM

Flush No.	Manhole Number 1 Mass(kg)	1 %	2 Mass(kg)	2 %	3 Mass(kg)	3 %
1	0.743	79.2	1.001	74.0	2.045	69.3
2	0.058	6.2	0.189	13.9	0.456	15.4
3	0.137	14.60	0.164	12.1	0.452	15.3
Totals	0.938	100.0	1.354	100.0	2.953	100.0

Using the curve in Figure 70 for BOD remaining in suspension and employing the same logic as given in Section 12.2.6, Table 62 was prepared. The first item presents the estimated BOD mass deposits per segment. The second item gives the percentages of BOD remaining in suspension per downstream manhole from each segment. The last two items present estimates of mass flushed out and average measured mass removals, respectively.

Comparison of the estimated and predicted BOD masses flushed out, that is, rows 3 and 4 of Table 62 show that, despite the limitations on both the estimated and measured values, the data does not seem to contradict the simplified approach used in defining the flush wave efficiency in transporting the flushed BOD loadings downstream.

TABLE 62. COMPARISON OF MEASURED VERSUS ESTIMATED AVERAGE BOD MASS REMOVALS - PORT NORFOLK ST. (SECOND PHASE) SERIAL FLUSHING

Item	Manhole Number 1	2	3
1. Estimated mass deposit (kg)	0.818	1.180	1.434
2. % remaining in suspension from:			
Segment 1	90.9	62.8	50.4
Segment 2	-	78.4	57.4
Segment 3	-	-	78.0
3. Estimated mass flushed out (kg)	0.743	1.439	2.208
4. Measured mass flushed out (kg)	0.743	1.001	2.045

12.4 Simplified Sewer System Deposition Model

Details of a procedure for obtaining estimates of the amount of daily dry weather deposition loadings within each manhole to manhole segment of a sewer collection system are provided in this section. A much simpler procedure is described in Chapter 13. A number of crude approximations and simplifications are used in this procedure and therefore, the results are not purported to be a substitute for those provided by more rigorous approaches (28,29).It is intended to provide estimates for only dry weather conditions and has no provisions for considering transient wet weather phenomena. No distinction is made between bedload, suspended load and washload deposition and resuspension characteristics. The major simplifying assumption of the model is that the amount of deposition remaining in any segment over the course of a day is computed as the residual loadings not washed or moved downstream during peak dry weather flow conditions. No detailed accounting is made of the temporal pattern of diurnal deposition, resuspension and transport phenomena. The general outline of the approach is given here but further details are available (4).

12.4.1 General Concepts

A well designed sewerage system should not only convey flows but should also minimize the deposition of sewage solids during dry weather conditions. There are in use an ample number of suitable empirical and theoretical equations for flow design but no uniform criteria have been established to prevent solids deposition.

The approach commonly used to prevent deposition is the method of minimum permissible velocity. However, the use of average velocity consideration is not necessarily the most robust criterion to use for a wide range of typical operating conditions.

A more fundamental approach is the method of fluid shear stress, τ, given by equation 34 .

$$\tau = \rho r s \tag{34}$$

where ρ = specific weight of water
r = hydraulic radius, and
s = energy slope

Yao (30) reviewed experimental results dealing with fluid shear stress measurements and concluded that the average boundary shear stress computed by equation 34 will approximate the actual local boundary shear stress within the possible region of deposition, provided that the flow depth is equal to or greater than one-third of the sewer diameter.*

Yao also concluded that a shear stress of .02 to .04 psf is adequate for self-cleaning for removal of particles in the range of 0.2mm to 1.mm in sanitary sewers, while a shear stress of .06 to .08 pfs is necessary to dislodge and transport particles of relatively larger sizes for self-cleaning of combined sewer systems. In addition, this work showed that the present practice of using a constant minimum velocity for all sewer sizes tends to under-design larger sewers and over-design smaller sewers.

Deposition Mechanisms. Shield's classic results are commonly used to predict solids deposition in sewerage systems. Shield's results, however, relate to bedload movement and specifically to uniform particles moving on the surface of the bed. In simple terms, there are two primary mechanisms involved in the transport of sewage particles: bedload transport and suspension.

The first to use bedload transport considerations to predict deposition in sewers was Camp (31). Assuming a particle specific gravity of 2.65, Shield's relationship (32) for bedload transport for large shear Reynolds numbers** is given by:

$$\tau_C/(\rho_S-\rho)p = .06 \Rightarrow \tau_C = .02p \tag{35}$$

where p = particle diameter; (mm); ρ_S = specific particle weight, and
τ_C = critical wall shear stress (psf)

The second transport mechanism is suspension. Hughmark (33) correlated 14 sets of data on slurry transport and Raths (34) conducted experiments on sand sediment in sewers. In order to prevent deposition of sand particles (specific gravity = 2.65), a critical wall shear stress must be maintained or exceeded. The results of their experiments can be summarized by the following relationship:

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*The actual or local boundary shear stress varies considerably, with the maximum occurring around the center line of the channel and the minimum near the water surface.

**Shield's constant equals 0.06 for this flow condition.

$$\tau_c = .021\ p^{1/3} \qquad (36)$$

The smaller particles (less than 0.05 mm) of Hughmark's data closely agree with the above functional form. A reasonable first order approximation is to assume that both mechanisms transport heterogenous materials through sewer systems. The geometric average of equations 35 and 36 can be used to predict transport requirements. The equation relating the critical wall shear stress, τ_c, necessary to move a particle of given diameter, p, is the following:

$$\tau_c = .02\ p^{2/3} \qquad (37)$$

12.4.2 Single Segment Deposition Model

Equation 37 and sewage particle size distributions were used to predict the quantity of suspended solids deposited from dry-weather flow over a single length of pipe. The results computed from equation 38 with two particle distributions (29,35) and the experimental results from the FMC study (12) were fitted by a simple single term power function given by equations 36 and 37 :

$$Z = 40 \left(\frac{\tau}{.004}\right)^{-1.2} \qquad \text{for } \tau > .004 \text{ psf} \qquad (38)$$

$$Z = 40 \qquad \text{for } \tau \leq .004 \text{ psf} \qquad (39)$$

where Z is the percentage of the suspended solids in the dry weather sanitary flow that is deposited if the wall shear is less than τ.

The shear stress, τ, would be computed for maximum daily dry weather flow conditions. Maximum daily peak flow, Q_{MAX}, can be computed from average dry weather flow, Q_{AV}, using:

$$\frac{Q_{MAX}}{Q_{AV}} = a\ PP^{-b} \qquad (40)$$

where PP is the contributing population in 1000's and a and b are determined from analysis of flow measurements.

12.4.3 Multi-Segment Models

In considering a series of sewer pipes having low values of fluid tractive shear, that is, characterized by low slopes or low flows (or both), the condition can arise where solids from an upstream reach can successively deposit in downstream pipes. The relative amounts deposited in any section would depend on the shear stress during peak flow in that link and also on the amounts deposited upstream. A general procedure is desired to predict the total cumulative load in any section from all upstream sources.

The procedure used is the following:

1. Segment the collection system into a network of "m" links where each link may be a section of pipe between manholes or several sections combined into a single section (similar hydraulic characteristics);

2. Establish, for all links, a list of all downstream sections that convey waste from the given link;

3. Compute cumulative upstream population at end of each link;

4. Compute average daily dry weather flow for each link using the cumulative population from step 3 and an average per capita waste rate;

5. Compute maximum daily dry-weather flow for each link using equation 40 ;

6. Compute shear stress for each link associated with the maximum daily flow, using equation 37 for the appropriate pipe shape;

7. Compute the dry-weather suspended solids deposition rates, $Z_i (i = 1,\ldots,m)$ from the shear stresses calculated in step 6, using equations 38 and 39 ;

8. Compute the suspended solids load ZL_i $(i = 1,\ldots m)$ developed along each link using population per link length and daily solids generated per capita;

9. Starting at the uppermost link, i, compute the amount of input material that will deposit, that is $Z_i \times ZL_i$;

10. Search the list of downstream links for the deposition rate, Z_j, greater than the rate at the link where the load is initially generated, and compute the amount deposited as the jth link from the ith component input load using $(Z_j - Z_i) \times ZL_i$;

11. Continue searching the list of downstream links for a deposition rate Z_k greater than Z_j and compute the deposition at the kth link from the ith component using $(Zk - Zj) \times ZL_i$;

12. Set $Z_k = Z_j$ and repeat steps 10 and 11 until the complete list of downstream links is completed;

13. Start with the next uppermost link in the system and repeat steps 9 through 12 while maintaining a running sum of all the deposited loads in each link from previous iterations; and

14. Sequentially proceed downstream until all components are completed.

In other words, a fraction of the load generated in an upstream section may deposit in that section (if the shear stress is sufficiently low) and more of that load may deposit in downstream sections only if the shear stress falls below levels experienced upstream.

The present model is coded to assume any collection system geometry with the one rule that only three segments can be considered at a given manhole. The model is coded to compute shear stress for circular, ovoid, rectangular and horseshoe shaped cross-sections with or without preset sediment beds.

An idealized example using the schematic in Figure 71 illustrates this procedure. Assume that the shear stress developed during peak dry-weather flow in links 1, 4, 5, 8 and 11 results in deposition rates of 10, 5, 5, 15, and 20 percent, respectively. Assume that the shear in all other links, i.e., 2, 3, 6, 7, 9 and 10, is sufficiently high to preclude any localized deposition. The dry weather load developed along each of the 11 links is, say, 100 units of dwf solids.

Table 63 shows contributions from all upstream links on each downstream link and the total deposition in section 4 consists of loads from

TABLE 63. DEPOSITION ANALYSIS OF IDEALIZED SYSTEM*

Link Numbers	Link Numbers: 1	2	3	4	5	6	7	8	9	10	11	Total Amount Deposited In Each Link
1	10.											10
2	-	0.										0.
3	-	0.	0.									0.
4	0.	5.	5.	5.								15.
5	0.	0.	0.	0.	5.							5.
6	-	-	-	-	-	0.						0.
7	-	-	-	-	-	0.	0.					0.
8	5.	10.	10.	10.	10.	15.	15.	15.				90.
9	-	-	-	-	-	-	-	-	0.			0.
10	-	-	-	-	-	-	-	-	0.	0.		0.
11	5.	5.	5.	5.	5.	5.	5.	5.	20.	20.	20.	100.

* The ijth in the table represents the amount deposited in link i that originated in link j. Thus element (8,1) = 5 represents the amount deposited in link 8 that originated in link 1.

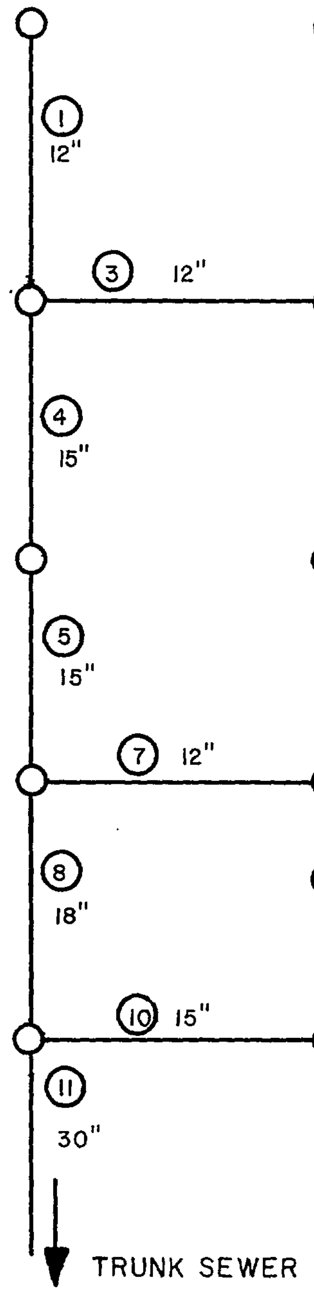

FIGURE 71 SCHEMATIC OF COLLECTION SYSTEM

sections 2, 3, 4 but not from section 1 because the deposition rate, Z_1 is greater than Z_4. At link 5, the only amount deposited is from the load developed along that link (Z_1 is greater than Z_5: no deposits, Z_2 and Z_3 less that Z_5 but Z_4 equals Z_5: no deposits; Z_4 less that Z_5: deposits). The overall deposition rate for the entire system is 20 percent (220 units deposited/1100 units total load with nearly equal loadings in links 8 and 11.

Modification to the basic existing model (4) during this project included the introduction of the following additional options: a) use of fitted slopes for the pipe segments in addition to the pipe slopes computed from manhole invert elevations; b) use of actual populations by pipe segment, in addition to average uniform figures computed on the basis of pipe length and population/100 ft of pipe, derived from global figures for the basin; and c) use of Manning's variable roughness coefficient n in the flow computations.

12.5 Verification of the Deposition Model

12.5.1 Introduction

The deposition model described in the previous section was developed to: a) identify areas of extensive collection systems subject to high degrees of deposition; b) indicate the relative degrees of deposition among different parts of the system; and, c) provide an indication of the order of magnitude of daily deposition throughout the system.

Comprehensive verification of such a model would involve applying it to one or more relatively large collection systems and verifying, by thorough inspection or more likely by a limited sampling, how well the model performed in providing the three levels of information outlined in the objectives mentioned above. It should be expected that the larger the collection system the better should be the overall performance of the model.

The present study did not include in the work program field tasks to allow a complete and formal verification of the model. The verification effort described here focuses only on the analyses of deposition loadings in the four test segments considered in the first phase field program. Both qualitative (visual) and quantitative comparative analyses will be presented for the four segments. Familiarity with the sewer segments at all four streets and occasional observations of pipe segments other than those being flushed permitted pragmatic appraisal of the results given by the model, especially in regard to the relative degrees of deposition throughout the system. With regard to the numerical prediction of localized daily solids mass deposition, the first phase field flushing results were used in conjunction with estimates of the fractions removed by repeated flushing from the second phase program to provide verification of the numerical values given by the model. It should be recognized that such verification is very limited in scope by the small size of the collection systems being analyzed.

12.5.2 Model Input Data

Physical System. All necessary information pertaining to the physical characteristics of the four collection systems upstream from the sampling manholes were prepared from as-built maps and field verified.

Population. The model option using population estimates by individual pipe segments was used in this analysis. Census tract information described in Chapter 4 of this report were used to estimate the contributing population to each pipe segment in the four streets.

Per Capita Liquid Waste Rates. Background flow measurements were used to estimate per capita waste rates of 56 and 53 gpcd for Shepton and Port Norfolk Streets, respectively. It is assumed that the waste rates at the other two streets is of the same order, that is, around 60 gpcd, since the four streets are fairly homogeneous in terms of population activity and income level. In running the model, per capita values ranging from 60 to 200 gpcd were used.

Per Capita Solid Waste Rates. The deposition loads predicted by the model are a linear function of the per capita solid waste rate. In defining an estimated average solid waste rate, to be used in the deposition model run, the averages of all measured TSS background concentrations at the four sites were used in conjunction with a per capita contribution of 60 gpcd.* Table 64 presents the estimates of solids waste rates in lb/capita/day for the four streets.

TABLE 64. ESTIMATES OF SOLIDS WASTE RATES IN LB/CAPITA/DAY

Street	Population	Mean TSS Background Concentration	Mean* Background Flow (cfs)	Mean Solid Waste Rate (lb/cap/day)
Templeton	221	1112.	0.0205	0.56
Shepton	230	393.	0.0214	0.20
Port Norfolk	94	1116.	0.0087	0.56
Walnut	71	1792.	0.0066	0.90
		Average for all Streets		0.56

*At 60 gpcd.

An average solids waste rate of 0.56 is estimated from field data collected over the course of the project at the four test segments. A value of 0.5 lb/cap/day was considered a reasonable figure to be used in the deposition model runs at the four streets.

Peak to Average Flow Coefficients. The model estimates daily deposition loads as a function of the tractive shear stress associated with maximum daily dry weather flow. The peak coefficients used in the verification runs derived from analysis of flow records covering a period of one week at all four sites. The liquid level continuous records were

*Summaries of all background flow and sewage strength data are presented in Chapter 11.

noted at time intervals of 20 minutes. A 20 minute peak to average daily flow ratio was computed for each day of the week for all four sites and are presented in Table 65. A 20 minute interval was chosen since it represented the smallest time interval that could be read from the dipper charts. The average peak flow coefficients appearing at the bottom of Table 65 were used in establishing the value of the coefficient in equation 40 .

TABLE 65. RATIOS OF 20 MINUTE PEAK TO AVERAGE DAILY FLOWS

Day of the Week	Site			
	Templeton	Shepton	Port Norfolk	Walnut
Sunday	1.72	1.13	1.09	1.23
Monday	1.54	1.29	1.07	1.57
Tuesday	2.02	1.25	1.16	1.59
Wednesday	1.73	1.32	1.15	1.21
Thursday	1.62	1.26	1.18	1.41
Friday	1.72	1.27	1.10	1.42
Saturday	1.70	1.29	1.10	1.72
Average	1.72	1.26	1.12	1.45

12.5.3 Verification Results

The deposition model used an arbitrary criteria to qualitatively rank the degree of solids deposition in pipe segments which is as follows:

Degree of Deposition	TSS Deposited Daily
None	0 - 2%
Low	2 - 6%
Moderate	6 - 15%
High	> 15%

The degree of deposition in the segments modelled for each street, using a per capita waste flow rate of 60 gpcd, are shown below. The last segment for each street shown below was the first phase test segment.

QUALITATIVE DEGREES OF DEPOSITION IN THE SEGMENTS MODELLED

Segment No.*	(Per Capita Waste Flow Rate = 60 gpcd)			
	Templeton	Shepton	Port Norfolk	Walnut
1	none	low	high	high
2	none	moderate	moderate	high
3	moderate	low	high	high
4	high	moderate	-	high
5	-	moderate	-	high

*Upstream-downstream order; in addition, in each street, the last segment no. refers to the flushing segment.

Observations of upstream segments as well as the test segments in each street at various points in time indicated that the qualitative predictions of the model could be judged with a fair degree of subjective confidence since the predicted results were found to be in reasonably good agreement with the visual observations of solids deposition in the four streets.*

Verification of the quantitative results given by the deposition model was performed using the first phase field results presented in Chapter 8. Verification was done for all four flushing sites and consisted of the following steps:

a. Estimation of the average daily mass of solids accumulated in the single pipe segment of each street. The computations considered in each street the average mass removed by the first phase flushes, and fractional estimates of the average flushed mass relative to the total average deposited mass in each segment;

b. Use of first phase sediment scrapings taken prior to flushing;

c. Use of the deposition model described in Section 12.4 to predict the daily accumulations of solids in those pipe segments; and

d. Comparison of the results derived from the average flushed masses and the sediment scrapings with those predicted by the deposition model.

The mean TSS mass removals normalized by antecedent days between flushes from the first phase for good flushing events are as follows:

Street	TSS Mass Removal (kg/day)
Templeton	2.56
Shepton	1.29
Port Norfolk	1.30
Walnut	1.72

The above estimates represent mean values of mass transported out of the respective pipe segments by a single flush and were reported in Chapter 8. Experience from the second phase serial flushes in Port Norfolk Street indicated that the mass flushed out of the first segment by the first flush (of three) represented about 76 % of the total solids mass accumulated in the pipe segment in the period between flushes. Similar estimates of the fraction flushed out for the other three sites are also necessary in order to compute the average daily accumulation rates for all four pipe segments. In the absence of any additional primary information measured, a similar flushing effectiveness rate is assumed for the other three streets.

*Previous verification of the model in the Boston area indicated that the model predicted none or low deposition in 52 out 55 segments where visual observation indicated no sedimentation; none or low deposition in 21 out 25 segments where visual observation indicated low deposition; low or moderate deposition in 9 out 12 segments where visual observation indicated moderate deposition; and moderate or high deposition in 18 out 33 segments where visual observation indicated heavy deposition (36).

The numerical predictions of the deposition model can only be verified at the pipe segments flushed in the first phase, for which numerical estimates of average deposition loads were computed. For the pipe segments where the deposition predictions and field flushing results are being compared, the optimized slopes developed for the looping stage/discharge curves in Chapter 7 for each flush segment were used in the deposition analysis. For the upstream segments the plan and profile map pipe slopes were used. The first four columns in Table 66 present the results predicted by the deposition model for per capita waste flow rates of 60, 100, 150 and 200 gpcd. The average first phase solids removals normalized by antecedent days between flushes, kg/day for each test segment are presented under the next column, labelled A. Two independent estimates of measured solids deposition rates, kg/day, are presented under columns B and C. The estimate of daily deposition given under column B is computed using the phase one flushing removal rates shown under column A and a flushing effectiveness level of 76%.* An indirect estimate of daily deposition along the segment is presented under column C using the measured sediment scrapings solids measurements taken over a one-foot section of pipe to flushing in the first phase. It is assumed that the unit deposition rates determined from the scraping operation are applicable over the entire segment. Sediment scraping information was not collected for the Walnut Street test segment.

Comparison of the deposition model predictions for the per capita waste rate of 60 gpcd with the estimates of daily accumulation from the flushing results shows reasonable agreement with the exception of Port Norfolk Street. The deposition estimates derived from the sediment scraping operation show closer agreement with Port Norfolk Street. In sum, the calibration results indicate that the deposition model should be viewed as a crude tool useful in providing rough cut estimates of dry weather solids deposition.

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* Another measure of flushing efficiency can be derived by dividing the flushed solids removals given under column A by the measured scraping given under column C. The average effectiveness for the three streets where scrapings were performed is 55%.

TABLE 66. COMPARISON OF DEPOSITION MODEL PREDICTIONS OF DAILY TSS ACCUMULATION WITH ESTIMATES OF THE FLUSHING EXPERIMENTS

Street	Deposition Model Predictions (kg/day)						
	60 gpcd	100 gpcd	150 gpcd	200 gpcd	A	B	C
Templeton	3.49	2.60	2.09	1.79	2.54	3.34	3.01
Shepton	1.69	1.28	1.02	0.87	1.31	1.73	2.71
Port Norfolk	3.33	2.54	2.01	1.71	1.30	1.71	3.80
Walnut	2.88	2.99	2.41	2.02	1.72	2.26	-

LEGEND

A - Average First Phase Flushing Results (kg/day)

B - Daily Accumulation from Flushing Results (kg/day)

C - Daily Accumulation from Measured Scraping Results (kg/day)

SECTION 13

DEVELOPMENT OF GENERALIZED PREDICTIVE DEPOSITION MODELS

13.1 Introduction

Deposition of sewage solids during dry weather in combined sewer systems has long been recognized as a major contributor to "first-flush" phenomena occurring during wet weather runoff periods. Estimation of these loadings for a given sewer system is an extremely difficult task. Measurement for extended periods is possible but extremely expensive. Some literature information is available from experiments on build-up of sanitary sewage solids in a pilot sewer study conducted by the FMC Corporation(11). Techniques presently available to estimate dry weather deposition in sewerage systems involve the use of computerized mathematical models, that are both complex and expensive and requiring more effort than appropriate for preliminary "first-cut" assessments.(4 , 28).

The objective of the analysis presented in this Chapter is to provide planners, engineers and municipal managers with readily obtainable technical information so that they can make intelligent informed decisions on potential sewer flushing programs. In this Chapter a set of generalized procedures for estimating pollutant loadings associated with dry weather sewage solids deposition in combined sewer systems is presented. A complete exposition of this analysis has been described in a planning document (37) prepared earlier in this study. A summary of that analysis is presented in this chapter.

The predictive equations relate the total daily mass of pollutant deposition accumulations within a collection system to physical characteristics of collection systems such as per capita waste rate, service area, total pipe length, average pipe slope, average diameter and other more complicated parameters that derive from analysis of pipe slope characteristics. Several alternative predictive models are presented reflecting anticipated differences in the availability of data and user resources. Pollutant parameters include TSS, VSS, BOD, COD, TKN and TP. Sewer system age and degree of maintenance was also considered. Factors are presented for estimating the increase in collection system deposition resulting from improper maintenance. A users' guide has been presented to establish the necessary data input to utilize the predictive procedures.

13.1.1 Foreword

An executive overview of the methodology used to develop the predictive simplified deposition predictive models is presented in section 13.2. A more detailed elaboration of the approach is given in section 13.3. An overview of the design of the numerical experiment is presented in section 13.4. Data input preparation for the numerical regression analyses are given in section 13.5. Summary results of the regression analysis and alternative model selections are presented in section 13.6. A user's application guide is given in section 13.7. Finally, an analysis demonstrating the numerical predictive sensitivity of the various models and approaches is presented in section 13.8.

13.1.2 Data and Information Sources

The data and information for this analysis were derived principally from three data sources: (a) sewer atlas physical data for portions of West Roxbury, Dedham, Newton and Brookline, Massachusetts for an infiltration/inflow study conducted by Environmental Design & Planning, Inc. for the Metropolitan District Commission(38);(b) sewer atlas physical data for portions of the City of Fitchburg, Massachusetts for a section of 208 combined sewer management study conducted by Environmental Design & Planning, Inc. for the Montachusetts Regional Planning Commission (5); and (c) sewer atlas physical data for portions of Dorchester and South Boston for a combined sewer management study sponsored by the Metropolitan District Commission (4).

13.2 Executive Overview of Methodology

An empirical model relating pollutant deposition loadings to collection system characteristics is the goal of this study. The approach is to use least squares to fit parameters of a postulated model. The data base used in the fitting process consists, in part, of a number of collection system parameters developed from an extensive data analysis of the physical details or several major sewerage collection systems in eastern Massachusetts. These characteristics are some of the independent variables used in the analysis. The data for the dependent variables are the total daily sewage solids deposited in these collection systems for a wide variety of different operating conditions. These quantities are estimated using an existing exogenous model that uses extremely detailed information to compute deposition loadings throughout an entire collection system network. An analysis of the detailed outputs of this model together with some of the physical data of the collection systems provided the remaining independent variables in the data base. Simply stated, the dependent variable data was generated from an exogenous predictive analysis while the independent variable data was obtained from primary collection system data and from a secondary analysis of the exogenous simulation outputs with selected collection system data.

Results of the field flushing programs have been earlier described in Chapters 8 and 9. Methodological details of an existing exogenous deposition model that predicts solids deposition in all segments of an entire sewerage collection system have been discussed in Chapter 12. In addition, calibration efforts using the field flushing results and the aforementioned model were described in Chapter 12. Those results were given to justify the application of that model to produce simulated data for the purposes of analysis described in this Chapter.

13.3 General Methodology-Detailed Overview

The general methodology used in the study is outlined in Figure 72. The first step is to define the general characteristics and parameters of the conceptual model. This discussion is presented in section 13.3.1. Next, a series of experiments is designed to generate deposition loadings using the deposition model described in Chapter 12 for a wide range of conditions likely to be encountered in practice. The regression equations would be valid for use over these ranges of conditions. The design of the experiments, described in Section 13.4 consists of defining the study areas to be used in the experiments and the hydraulic conditions under which the numerical experiments would be performed. The suspended solids per capita waste rate is discussed in the design of the experiments.

The next step involved the collection of all pertinent physical data associated with the selected collection systems, such as system configuration, pipe lengths, shapes and sizes, invert elevations, so that the deposition model referred to in Chapter 12 could be used. This physical data, together with the deposition model and the total loads deposited simulated for each of the collection systems is described in section 13.5.1.

Analyses of the collection system service areas and total pipe lengths, distributions of pipe slopes and average collection system pipe diameters are presented in sections 13.5.2 to 13.5.4, respectively. Deposition model results included the total lb/day deposited in each pipe segment of each basin and the total loads accumulated throughout each system. Information on loads by pipe segment is then used to generate curves for each basin, showing the accumulated percentages of the loads deposited against the accumulated percentage of pipe lengths where deposition took place. This part of the work is described in section 13.5.5.

Physical data of the system together with the distribution of loads by pipe length are then used to define the derived variables L_{PD} and $S_{PD/4}$ which are described in sections 13.5.6 to 13.5.8, respectively. Total loads by basin generated by the deposition model together with primary variables (pipe length, area, average slope, average diameter) and the derived variables (L_{PD}, S_{PD}, $S_{PD/4}$) are then used as input for the regression analysis described in section 13.6. Results of the regression analysis were examined and considered satisfactory and the process was complete.

13.3.1 Discussion of Model Variables

A discussion of the independent variables considered in the model and a few descriptive details of the preliminary analyses preceding the selection of the complete list of variables is given in this section.

The obvious and simplest of variables that can be used to characterize a collection system are the total service area, total pipe

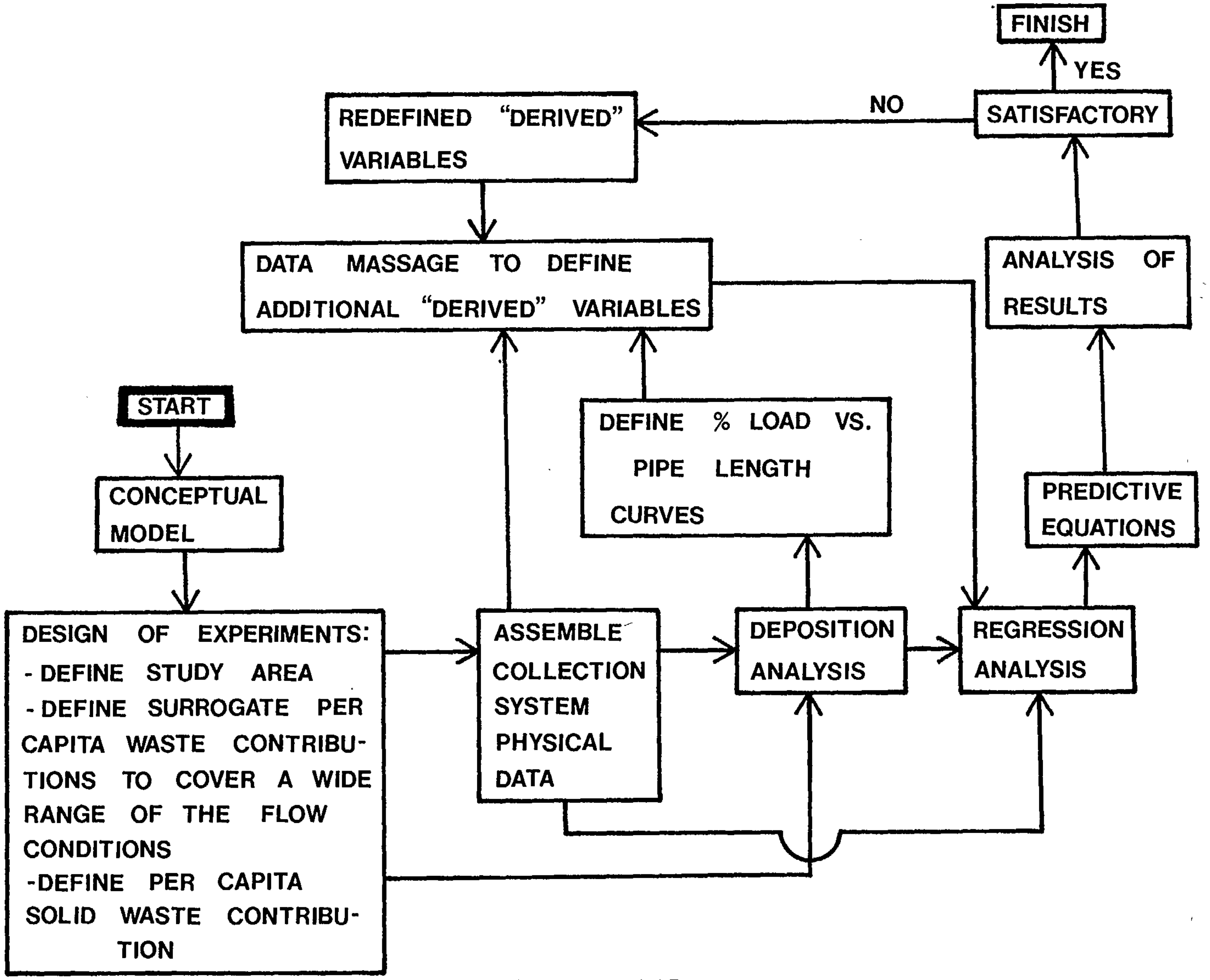

FIGURE 72
GENERAL METHODOLOGY OF THE STUDY

length, average slope and the average pipe diameter. It was believed from the onset of this study that these variables alone would not be adequate to explain the variability of the estimated loads from the deposition model. Clearly, a better characterization of the collection systems was necessary.

An exploratory analysis applying the deposition model on a number of sample collection systems revealed an interesting insight. Plots of the cumulative percentages of total loads deposited in each basin versus cumulative pipe lengths were prepared. A number of these curves can be inspected from Figures 80 and 81 presented in Section 13.5.5. The curves spread around the range of 70% to 90% of the total mass deposited suggested the use of the pipe length corresponding to 80% of the total mass deposited as a potential variable to include in the regression analysis.

Another set of plots of the cumulative distribution of pipe slopes for a few basins also suggested that the mean pipe slope alone would not be adequate to explain the effects of the pipe slopes on the variations of the deposition loads. A better characterization of the collection system pipe slopes could be obtained by defining various parameters at the flatter pipe slope range. Three other pipe slope parameters besides the mean pipe slope were initially selected for inclusion into the regression model. These parameters are as follows:

a) the pipe slope corresponding* to the percentage of the pipe length where 80% of the total load of the collection system deposits (S_{PD});

b) the average of the slopes in the basin below S_{PD} ($\bar{S}_{PD}$);

c) the slope corresponding to some fraction of S_{PD}, arbitrarily taken as the slope corresponding to 1/4 the percentage of pipe lengths below which 80% of the total mass deposits ($S_{PD/4}$).

These slope parameters can be seen in Figure 73 .

Further analysis revealed that $\bar{S}_{PD}$ and S_{PD} were very strongly correlated, so that retaining both in the regression analysis was not necessary. This finding was fortunate since the variable $\bar{S}_{PD}$ is much more difficult to determine than S_{PD}. The variable $\bar{S}_{PD}$ was excluded from the analysis.

Finally, it is clear that the deposition process is also strongly affected by the sewage flows in the system. Variations in population density and the degree of infiltration affects the dry weather flow rates. These effects are incorporated into the per capita waste rates used in the deposition model simulations and in the regression analysis. The summary list of variables considered in the regression analysis is the following:

*Note that the correspondence indicated in Figure 73 does not necessarily imply that the pipe length over which 80% of the load deposits has slope smaller than or equal to S_{PD} at all segments.

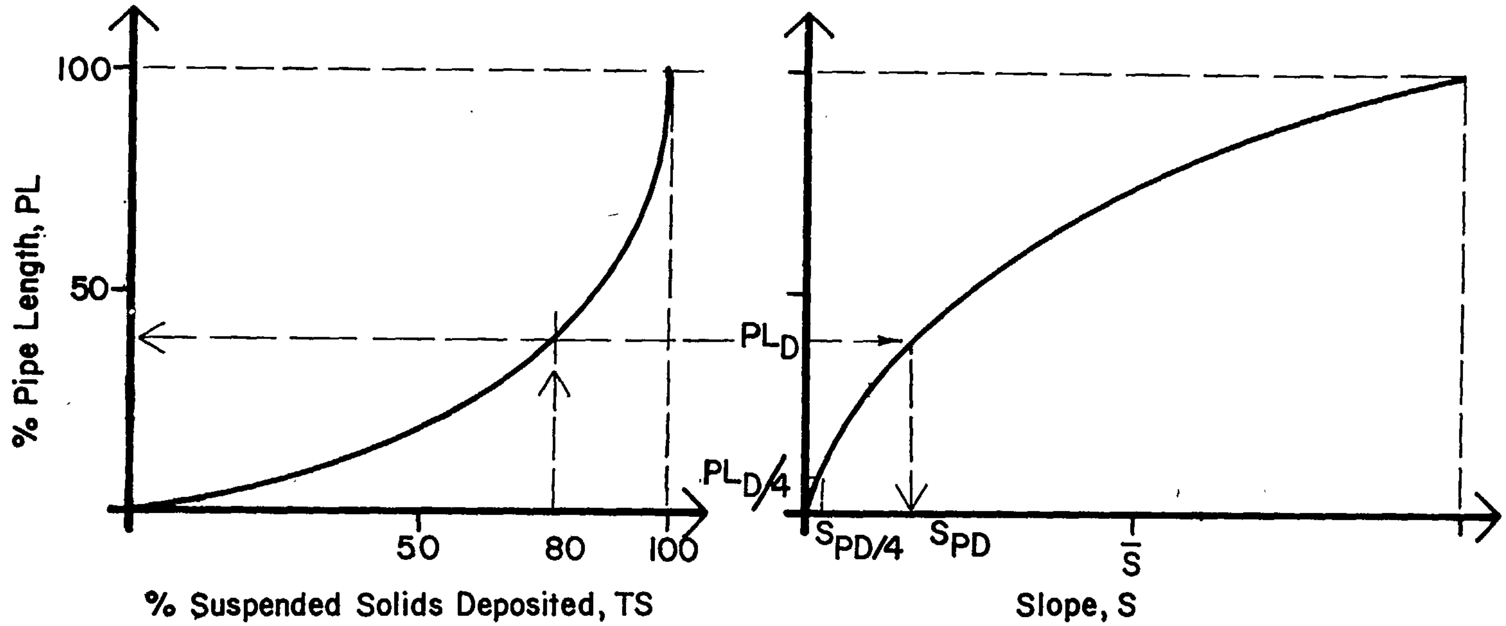

FIGURE 73 COLLECTION SYSTEM PIPE SLOPE VARIABLES

1. Total collection system pipe length (L') - ft;

2. Service area of collection system (A) - acres;

3. Average collection system pipe slope ($\bar{S}$) - ft/ft;

4. Average collection system pipe diameter ($\bar{D}$) - inches;

5. Length of pipe corresponding to 80% of the solids deposited in the system (L_{PD}) - ft;

6. Slope corresponding to L_{PD}(S_{PD}) - ft/ft;

7. Slope corresponding to 1/4 of the percentage of pipe length (PL_D) below which 80% of the solids deposit ($S_{PD}/4$) - ft/ft; and

8. Flow rate per capita, including allowance for infiltration (q) - gpcd.

With respect to the mathematical forms of the regression model both linear and alternative non-linear models were initially postulated. Non-linear fitting techniques were not needed in the analysis since the linear models, that is, the strictly additive form and the logarithmic multiplicative form converted in the log domain, resulted in excellent fitting results with the R^2 approaching 95%.

13.4 Design of Experiment

In this section an overview will be presented of how collection system data from three major sewerage systems was used to design the data base for the multivariate regression experiment. A description of the three sewer systems whose data were assumed to represent an adequate sample from the universe of all collection system is presented in section 13.4.1. A discussion of the per capita flow waste rates used in the experiment is presented in section 13.4.2. These surrogate waste rates reflect wide variations in population density and infiltration conditions encountered in practice. This parameter can be considered as a decision variable from a planning standpoint. Various sewer system age and maintenance considerations are discussed in section 13.4.3.

13.4.1 Description of Three Sewer Systems

The physical characteristics of the three major collection systems used in this analysis derived from three prior studies. The first area, covering portions of West Roxbury in Boston, Dedham, Newton and Brookline is strictly separated. The second area covering major portions of Dorchester and South Boston, two neighborhoods of the Boston metropolitan area is a mixed combined and separate area while the third basin covering a portion of the City of Fitchburg is served by a combined sewer system. The total pipe length, service area and pipe density for each basin are given in Table 67. The total pipe footage for all three areas entails 196 miles

(315.5 km) of separate and combined sewer systems encompassing a total area of 8.9 square miles (204.5 square km).

TABLE 67. SEWER DENSITY (mi/acre)

Overall System	Pipe Length (mi)	Area (acres)	Pipe Density (mi/acre)*
WRNDB** 35 basins	64.87	2464.	0.026
Dorchester (37 basins)	119.85	2753.	0.044
Fitchburg (3 basins)	11.17	485.	0.023

*Weighted averages: by pipe length: 0.036, by area: 0.034.

**Sewerage system with area covering portions of West Roxbury, Dedham, Newton and Brookline in Boston metropolitan area.

The land use in the first area in West Roxbury and neighboring communities is mostly moderate to high density single and two family dwellings with a population density ranging from 10 to 15 people/acre. The topography is mild with several hilly portions in the area. This area was investigated in a recent infiltration/inflow study and was subdivided into 35 distinct sewer collection subsystems.

The land use in Dorchester and South Boston is mostly high density multi-family dwellings with population density ranging from 30 to 60 people/acre. The topography in Dorchester is moderate with a number of hilly sections while portions of South Boston are fairly flat. There are a total of 37 distinct sewer collection systems in this study area.

The land use in the third area in Fitchburg is mixed commercial and high density multi-family dwellings with a small portion of single family homes. The population density is similar to Dorchester. The study area is subdivided into three collection systems.

A total of 75 different sewer collection systems form the data basis for the analysis. It is assumed that these basins collectively represent a wide variety of different pipe slope conditions, pipe sizes and shapes and network system configurations. Some basins serve narrow strips of land while others are broad fanned-shape with a high hierarchial network order. A central assumption is that the collection system characteristics represented by the sample of 75 sewer sheds is an adequate representation of the total universe of collection systems. This assumption is not completely valid since, for example, extremely flat collection systems were not part of the sample set. Future work should broaden this data base. This sample however is deemed reasonably complete for the purposes of this analysis.

A complete sewer atlas of manhole to manhole descriptive physical data including pipe length, slope, shape, size and network ordering designations was available for each of these systems. Much of this data had been previously processed for computer application although a considerable portion of the data had to be placed in EDP format for purpose of this study. Roughly 6000 manhole to manhole segments incorporating all of the aforementioned parameters were necessary to represent the hydraulic characterization of the 75 sewer collection systems.

13.4.2 Range of Flows

The degree of deposition in a sewer pipe is strongly dependent on the discharge. As flow increases through a pipe the depth, velocity, hydraulic radius all change resulting in higher shear stress with less deposition. Discharge therefore is an extremely important parameter in the analysis. The dry weather discharge in a sewer system is dependent upon the local population density, the domestic per capita contribution, the degree of infiltration and any industrial waste contributions.

It was envisioned that a single per capita surrogate waste rate would be generated incorporating a wide range of population density and infiltration conditions encountered in practice.* This variable would embed all these variations and be used in both the deposition model to predict daily dry weather solids deposition and in the regression model as an independent variable.

Population densities ranging from 15 people/acre up to 90 people/acre were considered. Using a factor of 0.035 miles of sewer per acre the corresponding number of people under 100 feet of sewer pipe was computed. These factors are shown in Table 68 and are used in the deposition model which requires as input the number of people per 100 feet of sewer.

The dry weather per capita contribution of 85 gpcd was considered fixed in this analysis. Four different infiltration estimates of 500, 1000, 2000 and 4000 gallons per acre per day were used to cover the range of normally encountered infiltration conditions. The adjusted per capita waste rates incorporating the various rates of infiltration for the range of population densities considered in the analysis are shown in Table 69 These per capita values are again adjusted to the mid-range of population density of 45 people/acre and are given in Table 70. This last conversion permits considering one single range of surrogate per capita flow rates using 45 people/acre as the norm. Four different flow rates are considered in the analysis and cover the full range of per capita waste rates for different population densities and infiltration conditions. The per capita waste rates used in the analysis are: 40, 110, 190 and 260 gpcd.

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*Industrial waste contributions were not explicitly considered. The user can readjust the per capita waste rates used in this analysis to reflect industrial contributions.

TABLE 68. POPULATION DENSITY (PERSONS/100 FT OF PIPE)

Density (person/acre)	Persons/100 ft of pipe*
15	8.15
30	16.30
45	24.46
60	32.61
90	48.91

*Assumes 0.035 mi/acre or 184.8 ft of pipe/acre

TABLE 69. PER CAPITA WASTE RATES FOR VARIOUS POPULATION DENSITIES AND INFILTRATION RATES*

Density	Infiltration Rate (gpad)**			
(person/acre)	500	1000	2000	4000
15	118.3	151.7	218.3	351.6
30	101.7	118.3	151.7	218.3
45	96.1	107.2	129.4	173.9
60	93.3	101.7	118.3	151.7
90	90.6	96.1	107.1	129.4

*Assumes a dry weather contribution of 85 gpcd.

**Gallons per acre per day.

TABLE 70. PER CAPITA VALUES RELATIVE TO THE DENSITY OF 45 PERSONS/ACRE

Density (person/acre)	500	1000	2000	4000
15	39.1*	50.1	72.1	116.2
30	67.3	78.2	100.3	144.3
45	96.1	107.2	129.4	173.9
60	124.4	135.6	157.7	202.3
90	179.7	190.6	212.6	256.3**

*Minimum value.

**Maximum value.

The per capita solids waste rate of 0.5 lb/capita/day was used in all computations. This parameter derives from analysis of background sample and flow monitoring results and was established in Chapter 12. All regression results presented in section 13.6 can be linearly scaled for any other desired per capita solids waste rate.

13.4.3 Age and Maintenance Conditions

The presence of long-term accumulations of organic matter, sand, gravel, grit and debris in the form of sediment beds, shoals, or bars can significantly alter the hydraulic characteristics and accordingly the degree of deposition, particularly for lateral pipes with little dry weather discharge. These accumulations can easily result in new well-constructed sewer systems with sound joints and few hydraulic obstructions such as protruding house connections. Similar deposits can occur in systems that are rodded and frequently cleaned but either are old and/or have poor joints and many hydraulic obstructions. Perforated manhole lids provide the perfect opportunity for children to jam sticks into manholes that can result in massive blockages of accumulated rags and toilet paper. The above conditions are but a few of the possible age and maintenance problems encountered in practice.

Three different categories of sewer system age and maintenance were considered in this analysis. The first category of clean pipe conditions represents good maintenance practices and well-constructed sewer systems. No sediment beds were considered in this case.

Two cases simulating different degrees of maintenance other than perfect clean pipe conditions were also considered. In the first case or the intermediate maintenance category, sediment beds ranging from 1 to 3 inches in depth were assumed for all pipes with slopes less than 0.0075. Figure 74 shows the assumed ranges of beds between pipe slopes of 0.0005 and 0.0075. In the third category, the zero maintenance care, the sediment beds range from 3 to 6 inches for the same range of pipe slopes. This range was established using judgment and also based on visual inspection of numerous combined sewer pipes in eastern Massachusetts combined sewer systems.

These three conditions were used in the deposition model analysis to compute daily collection system deposition loadings.

13.5 Data Preparation for the Regression Analysis

Descriptions of the procursory analyses necessary for generating the regression analyses input data are presented in this section. Details of the analysis for generating daily estimates of total sewerage system deposition loadings for each of the 75 collection systems are presented in section 13.5.1. Descriptions of collection system service areas and corresponding sewerage system pipe lengths are given in section 13.5.2. An analysis of statistical distributions of collection system pipe slopes is discussed in section 13.5.3. Methods for computing average pipe diameters per collection system are given in section 13.5.4. Results of predicted deposition loadings for the 75 collection systems considered in section 13.5.1

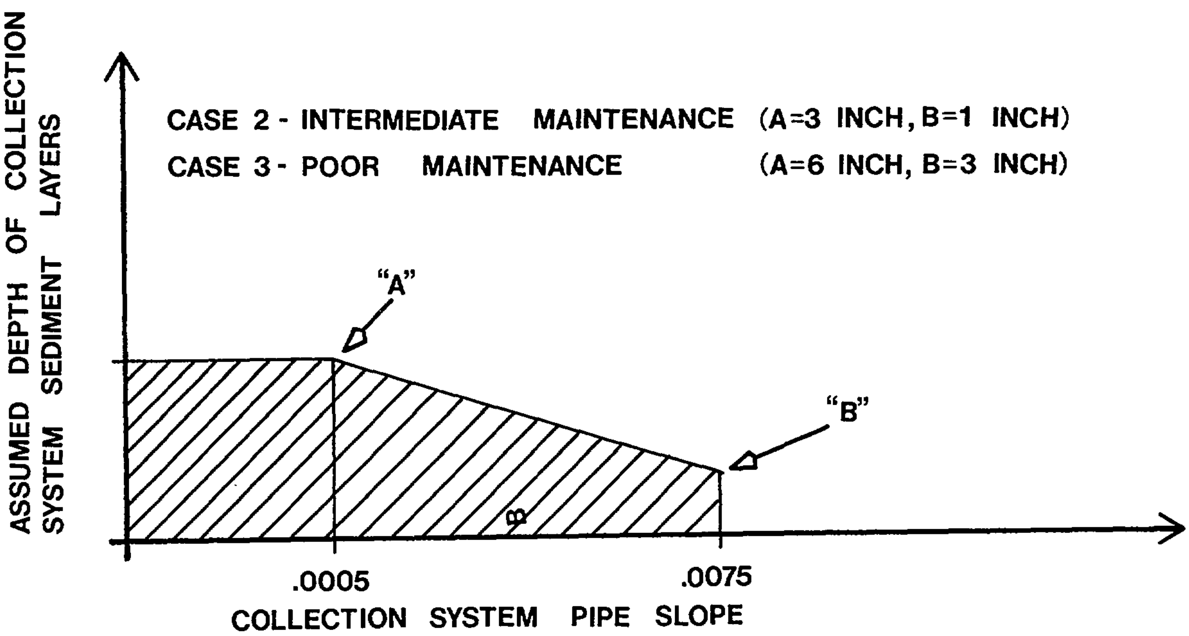

FIGURE 74 REPRESENTATION OF SEDIMENT BEDS AND PIPE SLOPE FOR TWO AGE AND MAINTENANCE CONDITIONS

are used to determine distributions of solids deposition as a function of cumulative pipe length. This analysis is described in section 13.5.5. Discussions of three other parameters, L_{PD}, S_{PD}, and $S_{PD/4}$ which derive from the analysis described in section 13.5.5, are presented in sections 13.5.6 through 13.5.8, respectively.

13.5.1 Deposition Model Results

Description of the Deposition Model. The deposition model used in this analysis to generate estimates of daily solids deposition for each of the 75 collection systems is described in detail in section 12.4 of Chapter 12. The model considers peak daily dry weather flow and uses a shear stress critera to determine the limiting diameter of the solid particles that deposit at each segment. Then, with this limiting particle size and assuming a given distribution for the particle sizes present in the dry weather sanitary flow, the model determines the percentage of the suspended solids that deposit at each pipe segment. The model also has mechanisms to account for the fact that particles of diameters up to a given size that deposit in a given pipe segment are not available for deposition in downstream segments.

Some of the results given by the model are: (1) the flow conditions at each pipe segment, including discharge, average velocity, water depth and shear stress; (2) the loads (lb/day) deposited at each pipe segment and (3) the accumulated value of the loads deposited in all upstream segments.

Input Data Required by the Deposition Model. The input data required by the deposition model consists of:

- Segment identification (by a segment number);
- Segment upstream and downstream pipe inverts;
- Segment length;
- Pipe shape (10 shapes possible);
- Pipe sizes (diameter or height and width);
- Segment type (zero, one or two tributary segments);
- Network location designation (defined by the segment type in conjunction with the next downstream segment number);
- Sediment depth in the segment;
- Population per 100 feet of pipe;
- Average daily waste flow contribution in gpcd;
- Peak daily to average flow peaking coefficients;
- Manning's resistance coefficient, n, and its variability with flow depth; and
- Total solids contribution in lb/capita/day.

Deposition Input Data Preparation. In section 13.4.1, the description of the three different sewerage systems considered in this study was presented. A total of 75 subsystems were used in this analysis including 35 separate collection systems from the WRNDB sewer system, 37 collection systems from the Dorchester and South Boston combined sewer systems; and 3 collection systems from the Fitchburg combined sewer system.

All the necessary physical data in the form of computer cards were available for the Dorchester and Fitchburg system from prior studies. For the WRNDB system all the pipe elevations, lengths, shapes and sizes were also available from a previous study, but all the segment numbering and the additional information required to establish the system configuration had to be generated in this study.

Other information on waste flow rates and solid matter contribution to the systems, necessary to run the model, were given in section 13.4.2.

Deposition Model Runs and Results. Three sets of runs were performed for all 75 basins. The first set of runs were performed assuming no previous sediment deposits present in the pipes, that is clean pipe conditions. The second set of runs were performed in which sediment depths ranging from 1 to 3 inches were assumed to represent moderate maintenance conditions. The third set of runs were performed assuming sediment depths from 3 to 6 inches intended to simulate poor maintenance.

Selected information from these runs were punched out on cards for use in future phases of the study. The values of the loads (lb/day) deposited by pipe segment were used to define for each basin the accumulated percentages of the total load versus the accumulated percentages of total pipe lengths where deposition occurs. An overall curve for all 75 basins was also prepared. These curves were useful in deriving several variables used in the regression analysis. Representative curves for individual collection systems and the overall curve are presented in section 13.5.5. The total loads per basin were used as the observed values of the dependent variable in the regression analysis.

13.5.2 Areas and Total Pipe Lengths

The total service area and total pipe lengths were known from prior studies for all 75 basins and are presented in Table 71. The first 35 basins cover portions of the WRNDB sewerage system. Basins 36 through 72 cover the Dorchester and South Boston sewerage system while the last three basins cover portions of the City of Fitchburg sewerage system. The data on Table 71 were also used for a simple regression of total pipe length on total area and is described in section 13.7. This regression may be useful in extreme cases where the total pipe length is not known or cannot be immediately determined.

13.5.3 Distribution of Pipe Slopes

The regression model proposed in section 13.4.1 included several collection system pipe slope parameters, S_{PD} and $S_{PD/4}$, that required computation of the cumulative pipe slope distributions. A computer program was prepared to compute these distributions from data on the pipe segments upstream and downstream invert elevations and segment lengths. The program computed the slope distribution weighing the segment slopes by their lengths. The mean, standard deviation, coefficient of variation, coefficient of skewness and coefficient of kurtosis of pipe slopes per collection system were also computed. The program computed the distribution and the afore-

TABLE 71. TOTAL PIPE LENGTHS AND AREAS OF THE BASINS

Basin No	Pipe Length (ft)	Area (Acre)	Basin No	Pipe Length (ft)	Area (Acre)
1	47180.	230.	39	4265.	19.
2	5945.	38.	40	3485.	17.
3	610.	5.	41	3400.	26.
4	669.	7.	42	3170.	16.
5	3309.	19.	43	4000.	25.
6	360.	5.	44	5060.	25.
7	1900.	9.	45	11325.	52.
8	2251.	13.	46	13133.	44.
9	650.	3.	47	7757.	24.
10	1160.	8.	48	7764.	42.
11	2158.	6.	49	112638.	245.
12	1410.	13.	50	875.	5.
13	15610.	84.	51	5200.	36.
14	1551.	70.	52	4501.	27.
15	990.	70.	53	11890.	51.
16	1305.	6.	54	5830.	28.
17	71621.	641.	55	2235.	15.
18	3279.	58.	56	7145.	34.
19	14415.	100.	57	5276.	91.
20	263.	6.	58	3115.	9.
21	489.	8.	59	10585.	120.
22	1146.	4.	60	8741.	65.
23	4180.	6.	61	35501.	228.
24	27385.	173.	62	13051	78.
25	12653.	90.	63	5635.	26.
26	3331.	19.	64	11325.	54.
27	14016.	82.	65	33005.	177.
28	738.	6.	66	9899.	47.
29	14997.	120.	67	5644.	30.
30	14540.	140.	68	14220.	57.
31	1374.	6.	69	4492.	24.
32	16988.	96.	70	35033.	233.
33	17131.	100.	71	134528.	315.
34	7728.	45.	72	69274.	360.
35	26981.	178.	73	31748.	264.
36	6245.	42.	74	12092.	78.
37	7735.	33.	75	14754.	143.
38	1750.	29.			

mentioned statistics for each system (WRNDB, Dorchester and Fitchburg) and finally an overall distribution and the first four moments of all data lumped into one data set.

Plots of the slope distributions for a few basins are shown in Figures 75 and 76. The concave shapes of those cumulative distributions (CDF) without a point of inflexion, suggest an exponential distribution for the pipe slopes. Several of the cumulative distributions were plotted on normal, log normal and Gumbel's probability paper. All plotted curves resulted in very non-linear shapes, indicating that the pipe slopes do not follow any of those distributions. Plots of the complementary CDF of the pipe slopes on semi-logarithmic paper, nonetheless, resulted in remarkably linear shapes shown in the illustrative cases in Figures 77 and 78. Although no formal numerical test of goodness-of-fit was performed, this fact indicates that, at least for the sample data used in this study, the distribution of the pipe slopes is exponential.

The solid lines drawn on Figures 77 and 78 were plotted using the expression of the exponential cumulative distribution function given by:

$$F_S = 1 - e^{-s/\bar{S}} \qquad (41)$$

where $F_S = P(s \leq \bar{S})$ (cumulative pipe slope distribution);

s = any given slope;

$\bar{S}$ = the mean slope computed for the basin, as indicated above and

e = the base of the natural logarithms.

Figure 79 presents the histograms for the WRNDB, Dorchester and Fitchburg sewerage systems and the overall histograms considering all data. The slope values corresponding to the intervals in Figure 79 are given in Table 72.

Two observations can be noted from these histograms:

a) the shapes of the histograms for all 3 systems are similar with minor differences between them (the same is true for the global histogram compared to any of the other three); and

b) they all indicate an exponentially decaying shape, characteristics of the exponential distribution with the CDF given by equation 1 .

A final justification for the assumption that the pipe slopes are exponentially distributed is the following. The mean and standard deviation of the coefficients of variation of the slope for all 75 basins are 0.87 and 0.25, respectively. The mean value of 0.87 for the coefficients of variation very closely approximates the theoretical value of 1.0,

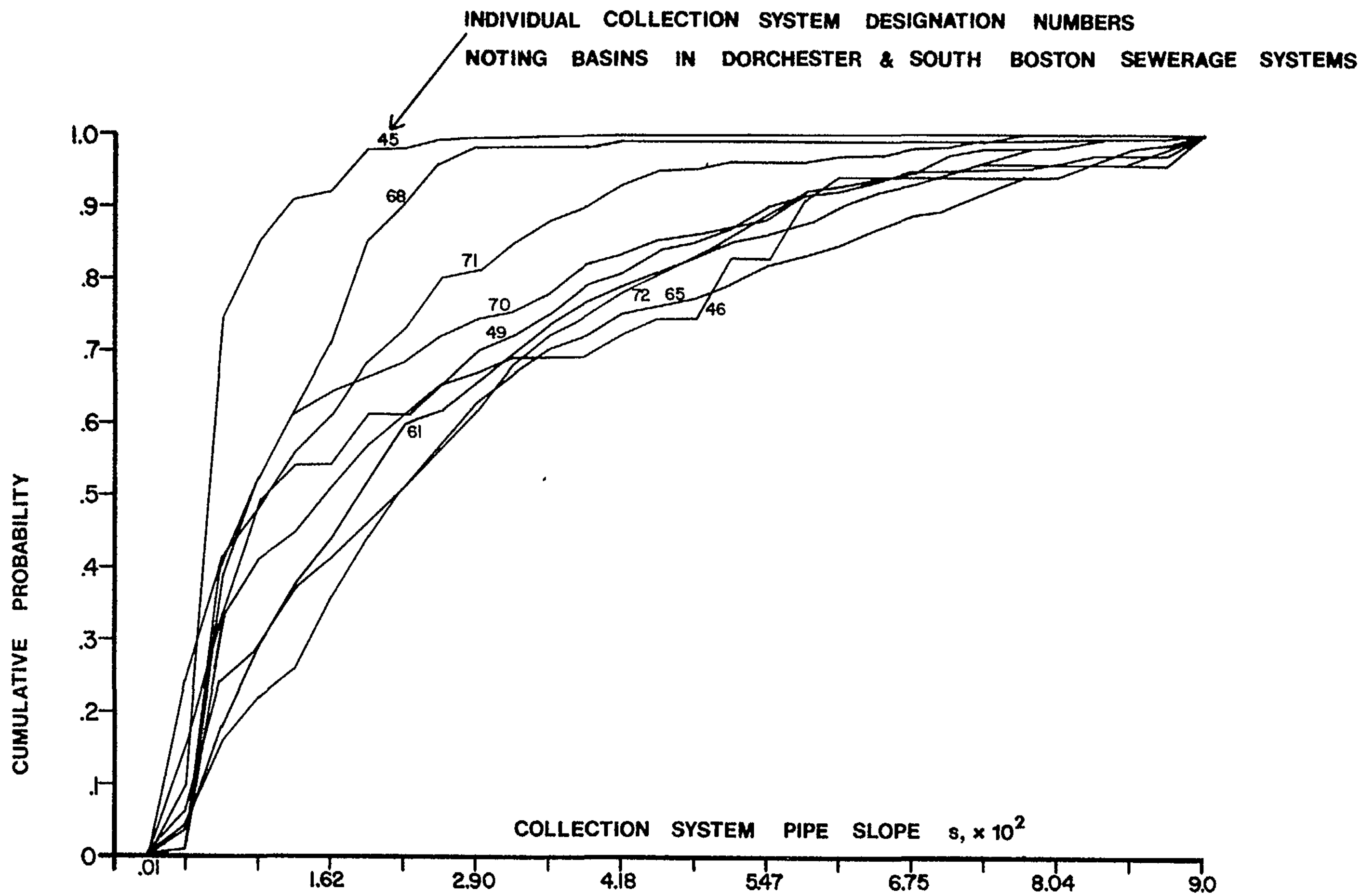

FIGURE 75 CUMULATIVE DISTRIBUTION OF PIPE SLOPES PER COLLECTION SYSTEM

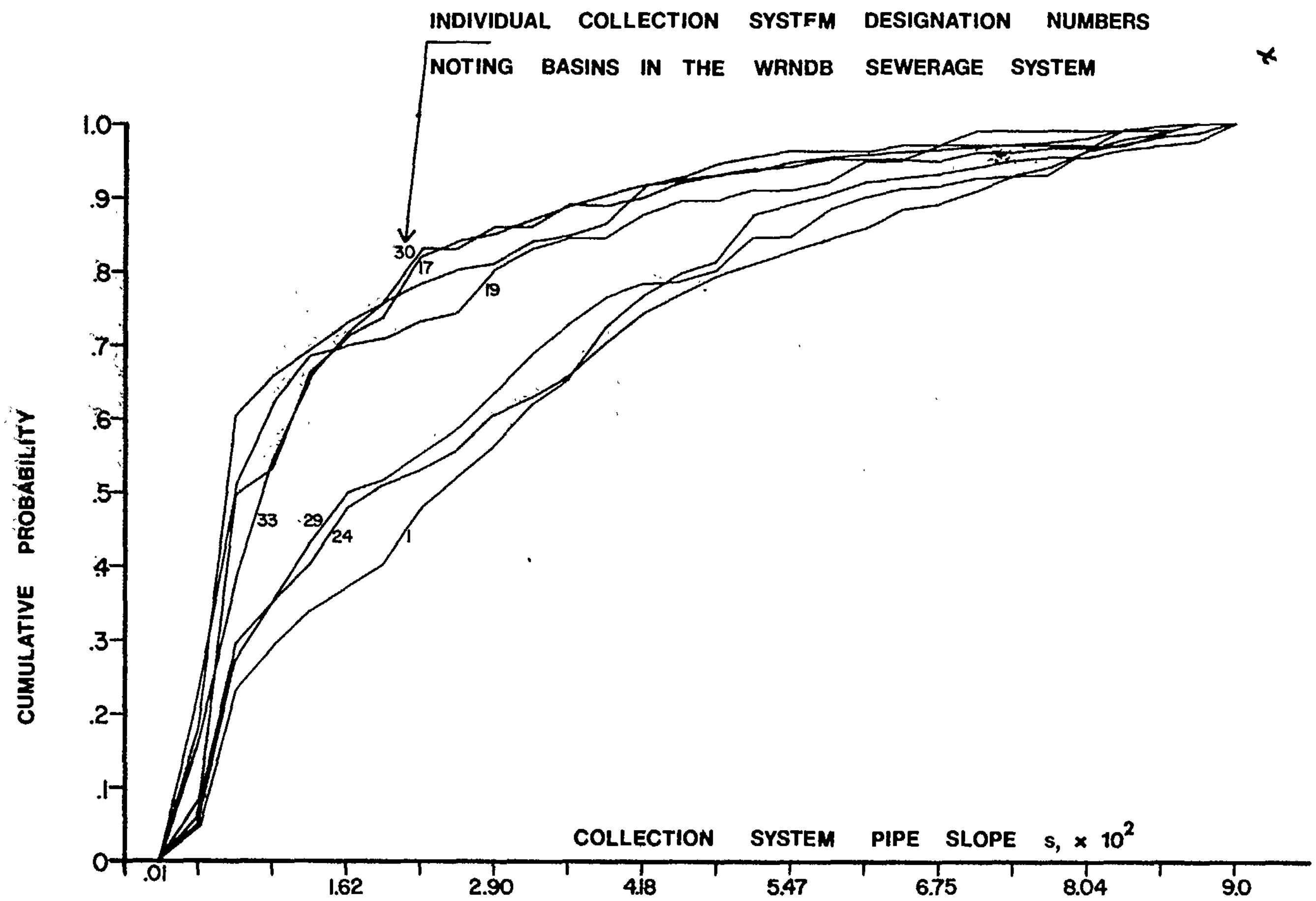

FIGURE 76 CUMULATIVE DISTRIBUTION OF PIPE SLOPES PER COLLECTION SYSTEM

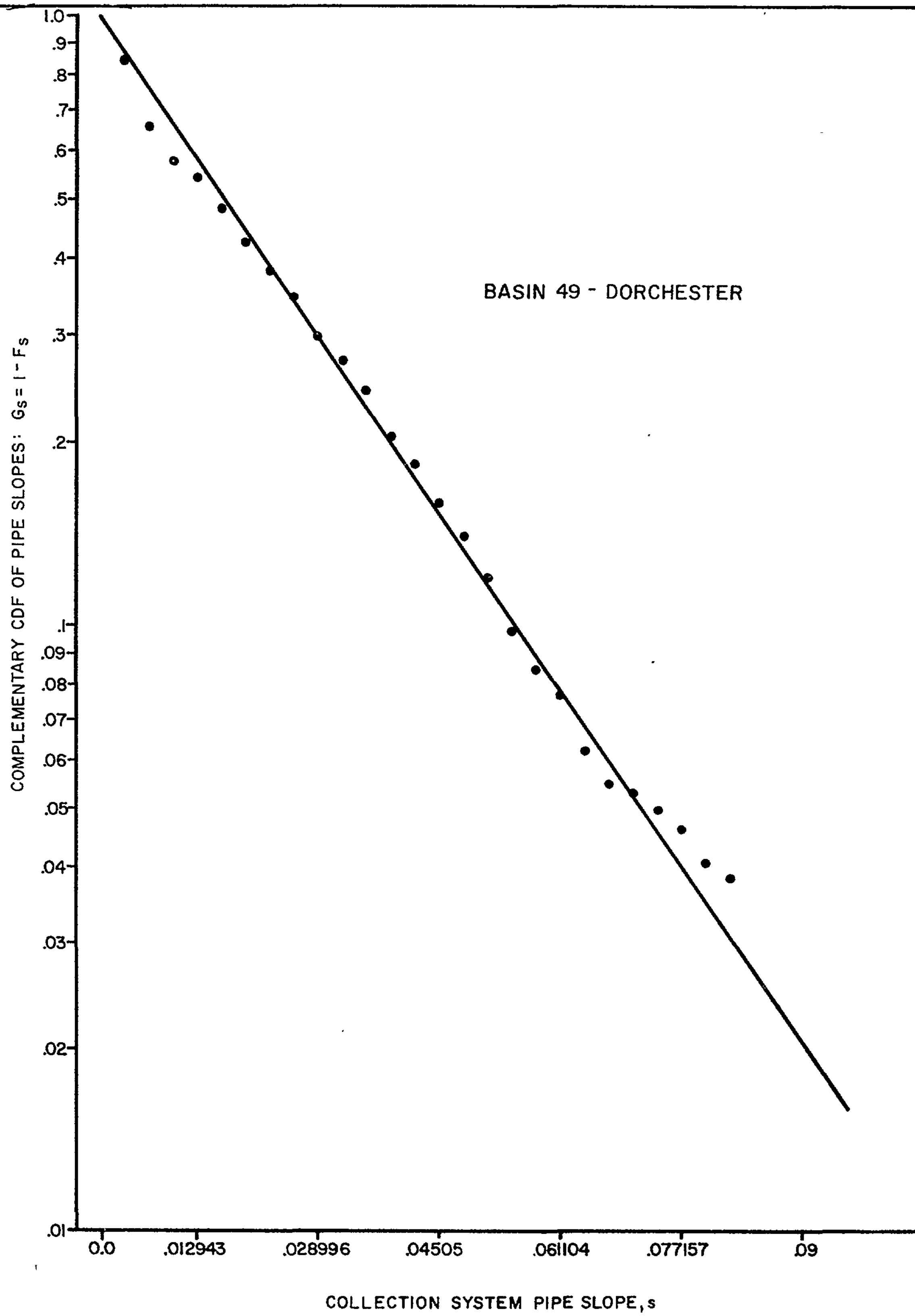

FIGURE 77 COMPLENTARY DISTRIBUTION OF PIPE SLOPES: $G_S = 1 - F_S$

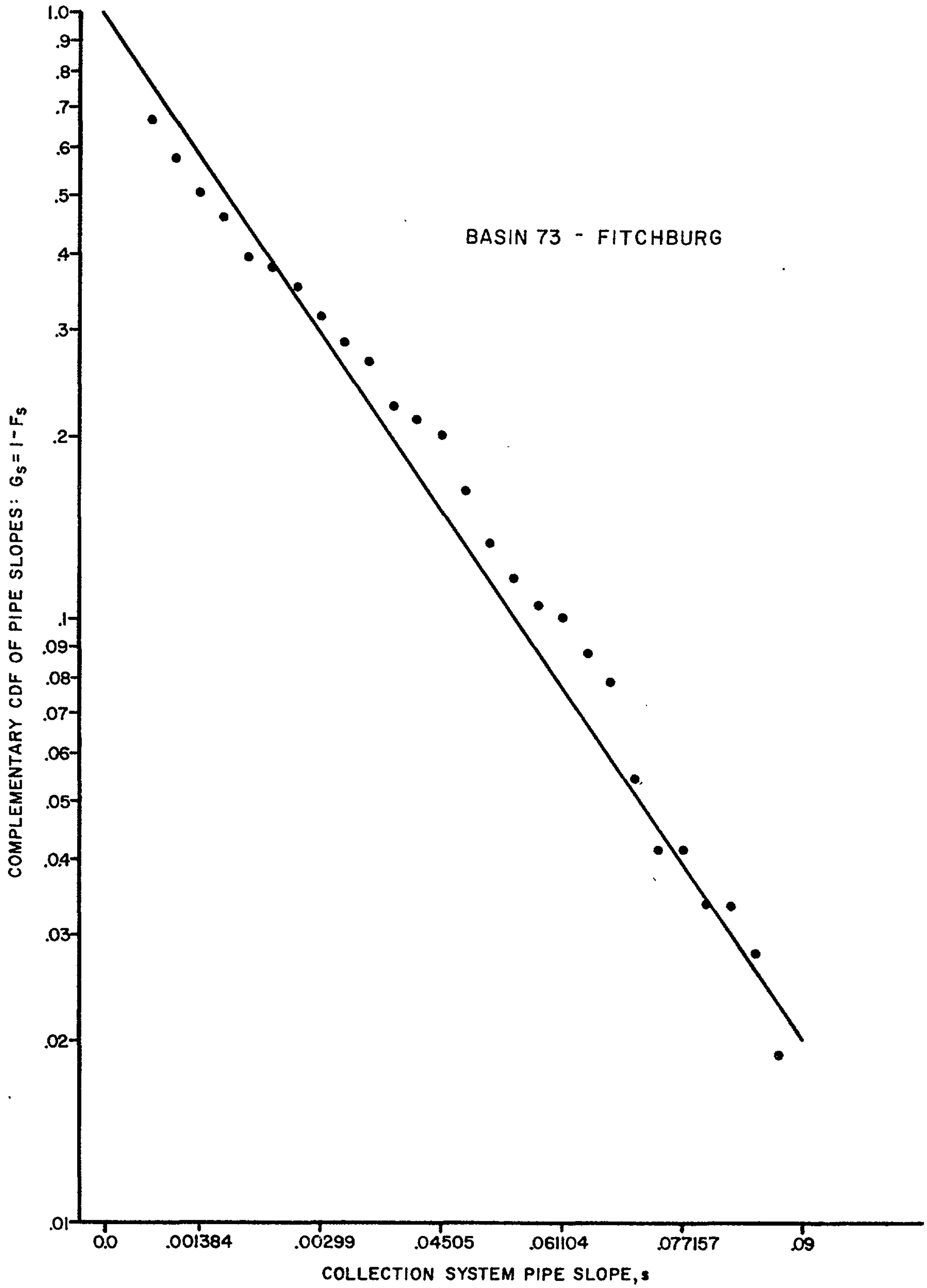

FIGURE 78 COMPLEMENTARY DISTRIBUTION OF PIPE SLOPES: $G_S = 1 - F_S$

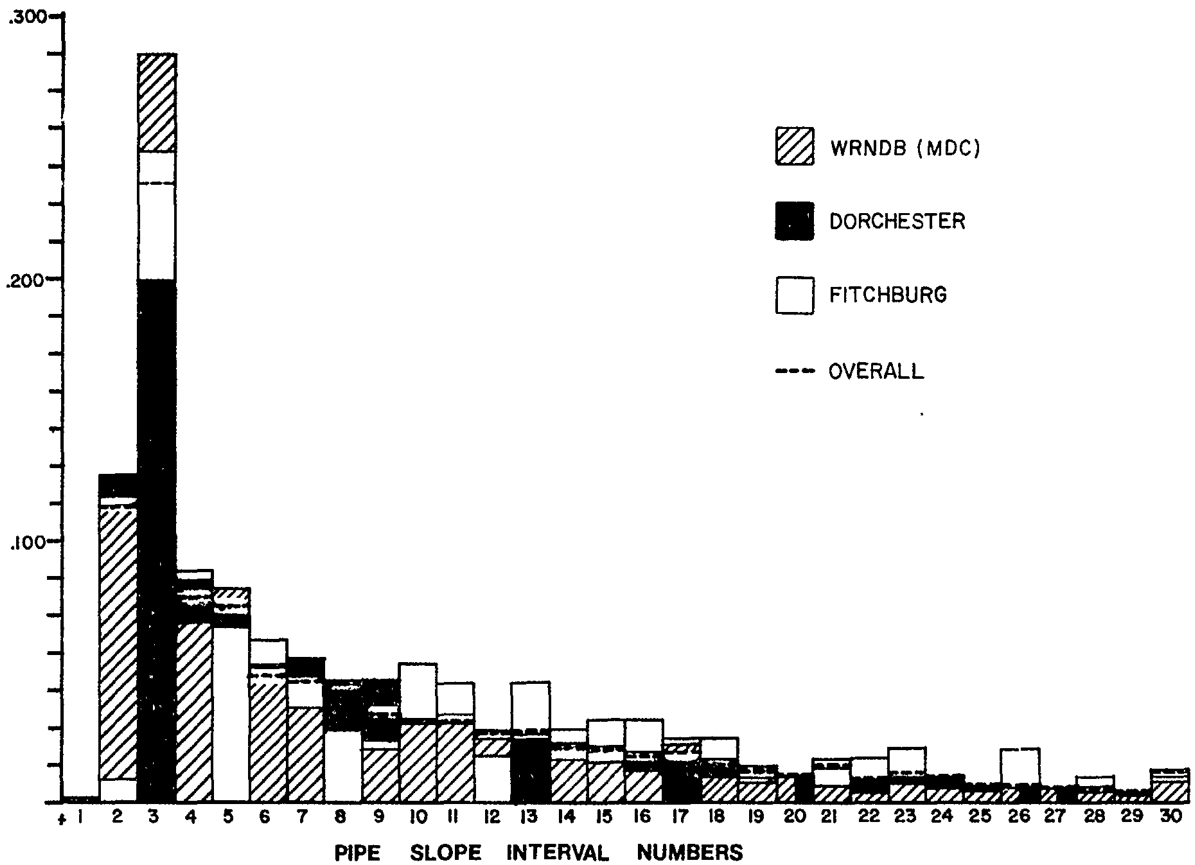

FIGURE 79 HISTOGRAM OF COLLECTION SYSTEM PIPE SLOPES

TABLE 72. SLOPES CORRESPONDING TO THE INTERVALS SHOWN ON FIGURE 79

Interval	Slope Limit of Range
1	0.000100
2	0.003311
3	0.006521
4	0.009732
5	0.012943
6	0.016154
7	0.019364
8	0.022575
9	0.025786
10	0.028996
11	0.032207
12	0.035418
13	0.038629
14	0.041839
15	0.045050
16	0.048261
17	0.051471
18	0.054682
19	0.057893
20	0.061104
21	0.064314
22	0.067525
23	0.070736
24	0.073946
25	0.077157
26	0.080368
27	0.083578
28	0.086789
29	0.090000
30	> 0.090000

characteristic of the exponential distribution.

This finding has significant importance in the practical application of the regression model developed in this study. This is especially true for cases where detailed statistical analysis of pipe slopes information is not available and only the mean value of the pipe slope is required to completely define with reasonable accuracy the pipe slope distribution.

The special slope parameters S_{PD} and $S_{PD/4}$ can then be estimated through this pipe approximation. Definition of the pipe slopes distribution using the exponential model is also important in delineating the extent and geographic location of the deposition loads in the system. This topic will be further discussed in section 13.7.2*

13.5.4 Average Pipe Diameter

The average pipe diameter of all segments within each collection system was computed by weighing circular diameters of each segment by its corresponding lengths. An equivalent circular pipe diameter was first determined for non-circular sections before the weighted average was computed. The fraction of non-circular pipes was a small percentage of the total pipe length. All pipes are circular in the WRNDB system. The number of non-circular pipes represent about 5% of the total for the Dorchester system, whereas in Fitchburg they represent less than 1% of the total pipe lengths.

The formulas used for the equivalent circular diameter for the non-linear sections are presented in Table 73 and have been derived to yield roughly the same hydraulic radius at low depths of flow. For rectangular and U-shaped pipes a simple equivalence of total areas are indicated in Table 73 These forms are non-existent in the data for this study. The average pipe diameters determined for all 75 basins are presented in Table 74 .

TABLE 73. FORMULAS FOR EQUIVALENT CIRCULAR DIAMETERS USED IN COMPUTING THE BASIN AVERAGE DIAMETER

Pipe Shape	Dimensions	Equivalent Circular D
Circular	D	D
Rectangular	X,Y	$(1.273\ XY)^{1/2}$
Ellipse	X,Y	$(X\ Y)^{1/2}$
Egg	X,Y	$5.51X^{5.38}/(X+Y)^{4.39}$
Horseshoe	X,Y	$(X+Y)/2.0$
Oval	X,Y	$0.67\ X$
Ovoid	X,Y	$5.51X^{5.38}/(X+Y)^{4.39}$
Modified Circle	X,Y	$(X\ Y)^{1/2}$
Arch	X,Y	Y
U	X,Y	$(1.273\ X\ Y)^{1/2}$

*It should be stressed here that in the generation of the slope data for the regression analysis described in section 13.6, the assumption that the pipe slopes are exponentially distributed was not used. The probability distributions of slopes for all basins were determined from their respective slope data.

TABLE 74. SUMMARY DATA ON DERIVED LENGTHS, SLOPES AND PIPE DIAMETERS

BASIN NO.	80% DEPOS. LENGTH(FT) PL_D	AVERAGE DIAM(IN) $\bar{D}$	AVERAGE SLOPE $\bar{S}$	SLOPE SPD	SLOPE SPD/4
1	15050.	10.5	0.027901	0.009965	0.003870
2	2080.	10.0	0.036746	0.022575	0.007320
3	434.	10.0	0.046065	0.058940	0.024900
4	450.	10.0	0.009118	0.008010	0.004290
5	1502.	10.7	0.021963	0.017470	0.006230
6	180.	10.0	0.008611	0.009732	0.009030
7	741.	10.8	0.018157	0.003849	0.001900
8	1019.	10.0	0.015280	0.014440	0.005050
9	36.	10.0	0.006400	0.006118	0.002200
10	560.	10.0	0.017163	0.019364	0.004560
11	304.	10.0	0.012293	0.004222	0.003480
12	685.	10.0	0.013985	0.009732	0.004290
13	6056.	10.9	0.013979	0.005995	0.002040
14	634.	10.8	0.019666	0.007681	0.003660
15	405.	10.0	0.030103	0.021642	0.010500
16	715.	10.0	0.004528	0.005510	0.000089
17	34879.	12.1	0.014539	0.006521	0.001830
18	1983.	9.5	0.009127	0.005119	0.001230
19	5938.	8.6	0.017344	0.005765	0.003430
20	83.	8.0	0.017852	0.017050	0.006270
21	171.	8.0	0.026175	0.032207	0.010400
22	648.	8.0	0.019257	0.014923	0.004780
23	1839.	8.0	0.021631	0.012943	0.004460
24	9228.	8.6	0.027730	0.009732	0.003660
25	4517.	8.7	0.019560	0.004989	0.001850
26	1370.	11.1	0.016939	0.009732	0.003810
27	5914.	11.0	0.016298	0.009732	0.001840
28	480.	12.0	0.004485	0.005012	0.002130
29	929.	10.2	0.025510	0.003311	0.001060
30	6005.	12.7	0.013617	0.005064	0.001910
31	526.	10.8	0.016449	0.006521	0.003280
32	7967.	8.7	0.016991	0.006344	0.004070
33	6903.	10.0	0.015655	0.006870	0.002180
34	3956.	11.6	0.017855	0.008449	0.002240
35	10954.	12.6	0.018203	0.006143	0.003469
36	2591.	13.3	0.014155	0.002892	0.000798
37	1461.	10.3	0.034669	0.004463	0.001100
38	936.	11.7	0.005660	0.005396	0.000380
39	1100.	12.0	0.030361	0.011499	0.004090
40	1913.	12.3	0.005365	0.004528	0.001230
41	1445.	9.3	0.027941	0.025786	0.007380
42	786.	12.0	0.037994	0.013749	0.004240
43	2540.	12.7	0.002575	0.002542	0.007100
44	1695.	12.0	0.029394	0.019364	0.004800
45	4869.	12.0	0.006341	0.004945	0.003330
46	5279.	14.1	0.024465	0.007763	0.004180
47	2893.	19.9	0.011737	0.003065	0.000841
48	2647.	12.4	0.025934	0.016154	0.006410
49	5293.	12.2	0.022566	0.001077	0.000344
50	571.	11.7	0.038857	0.047799	0.002560
51	1040.	11.4	0.035336	0.007467	0.002350
52	2295.	11.8	0.012463	0.011354	0.007200
53	1545.	14.6	0.015249	0.002514	0.000704
54	2530.	13.9	0.011539	0.007625	0.004020
55	929.	10.6	0.005995	0.005086	0.002170
56	1972.	11.2	0.041951	0.012943	0.000788
57	1461.	14.1	0.015366	0.002852	0.001370
58	1373.	13.1	0.010269	0.004675	0.001780
59	4847.	14.3	0.017349	0.009732	0.003930
60	2613.	12.7	0.024400	0.006521	0.003930
61	4934.	12.2	0.025037	0.005449	0.002020
62	6016.	12.6	0.055506	0.002802	0.000776
63	642.	12.1	0.022864	0.002143	0.000611
64	3408.	12.5	0.027835	0.009732	0.003800
65	1815.	12.9	0.029568	0.003311	0.001180
66	3434.	13.1	0.034241	0.012943	0.004610
67	2726.	14.3	0.079921	0.008435	0.001790
68	6498.	12.7	0.011681	0.008106	0.003280
69	2106.	12.0	0.018102	0.011068	0.004420
70	13312.	15.6	0.019405	0.006521	0.003020
71	26905.	13.3	0.015647	0.002787	0.000772
72	18773.	12.4	0.026287	0.008287	0.000310
73	13580.	12.4	0.002375	0.009732	0.004270
74	1306.	10.7	0.033799	0.006521	0.004020
75	4751.	13.6	0.029285	0.016154	0.005560

13.5.5 Distribution of Solids Deposited by Pipe Length

The cumulative distribution of solids deposited in the collection system versus the cumulative length of sewer pipe where the deposition occurs is the only exogenous information that the user will have to accept in applying this methodology. In other words, this distribution is the only information that cannot be derived from local data and can only be modified in a minor way by input from local conditions. These modifications will be covered later.

Some simplifications had to be made in deriving these distributions especially considering the great number of pipe segments (around 6000) that were numerically considered. The procedure established for each basin, as a function of the maximum and minimum estimated loads, a series of 200 intervals where loads of approximately equal values were accumulated, together with their corresponding pipe lengths. Whenever the basin had less than 200 pipe segments the number of intervals would be made equal to the number of pipe segments of the basin. The cumulative values of loads versus lengths were then computed for each of these intervals.

Plots of several cumulative probability functions for a few basins are presented in Figures 80 and 81 . An overall curve computed from all 75 basins is also presented in Figure 81. A total of 5000 intervals were used, in computing the overall curve, yielding a smooth curve as can be seen from Figure 81 . Plots in semi-log paper of the complementary values of the load probabilities, (1.-F load) (in the log axis) versus the cumulative probability of pipe lengths, resulted in nearly straight lines, indicating that this distribution is also exponential.

It was observed by associating the curves in Figures 80 and 81 with their respective mean slopes that the average basin slope increases moving from the lower to the upper curves. This explains comparatively higher percentages of loads depositing in lower percentages of pipe lengths. Based on this observation, Figure 82 was prepared. In its preparation a smooth curve was drawn at approximately the middle of the range of curves from Figures 80 and 81 with mean slopes corresponding approximately to the range limits in the legend of Figure 82. With the mean pipe slope known for a given basin, Figure 82 can be used to estimate the percentages of pipe lengths corresponding to given percentages of total solids deposited. Further discussion on the applications of Figure 82 will be presented in Chapter 14 which deals with flushing strategies.

The choice of the pipe length corresponding to 80% of the loads deposited as an independent variable in the regression analysis discussed in section 13.3.1, resulted solely from the observation of higher separation of the curves around the value of 80%. Other percentages of mass deposited could have given the same results. It should be noted that the loads estimated by the regression equations presented in section 13.6 correspond to 100% of the loads deposited. The length L_{PD} corresponding to 80% of the loads is only one regressor devised to explain the variation in the total loads.

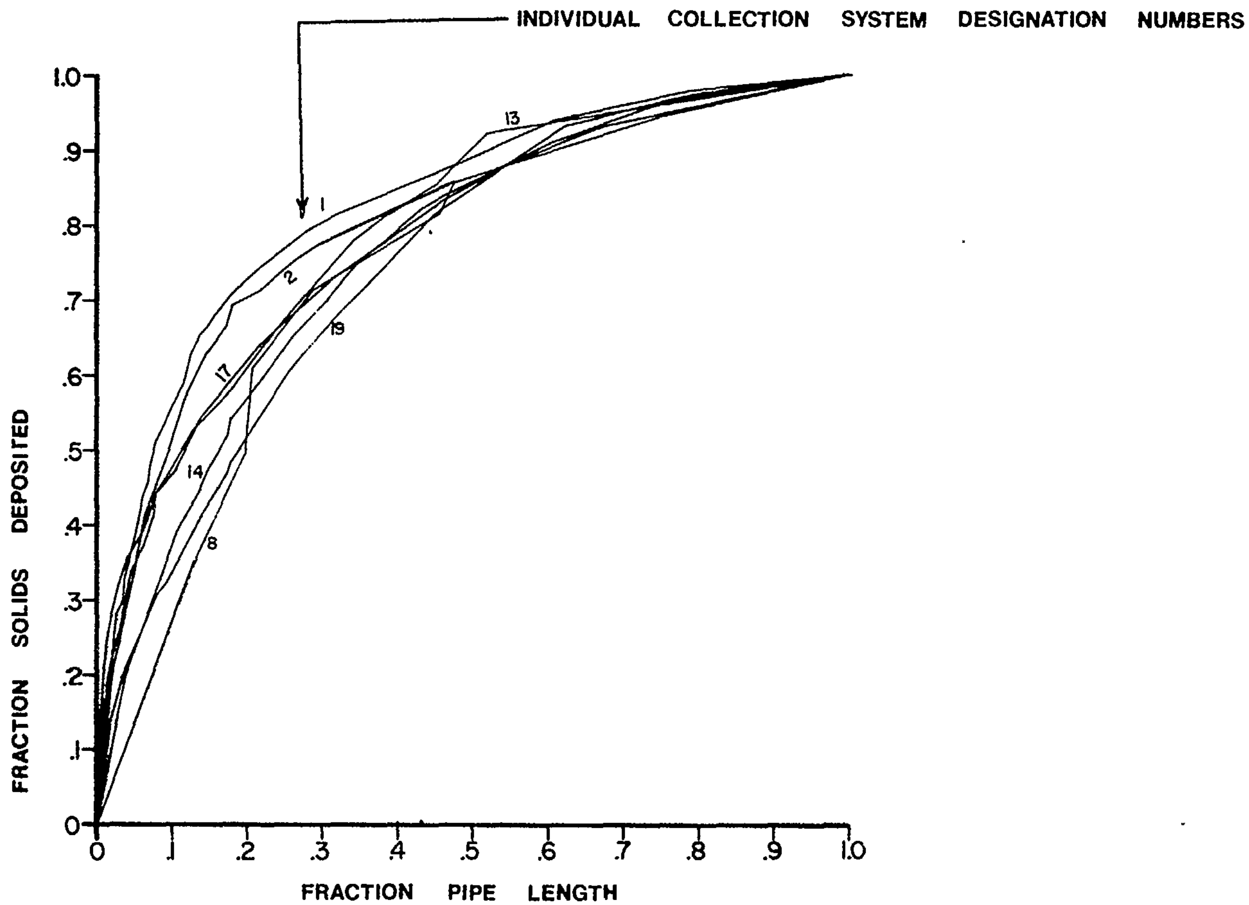

FIGURE 80 DISTRIBUTION OF SOLIDS DEPOSITED BY PIPE LENGTH – WRNDB SYSTEM

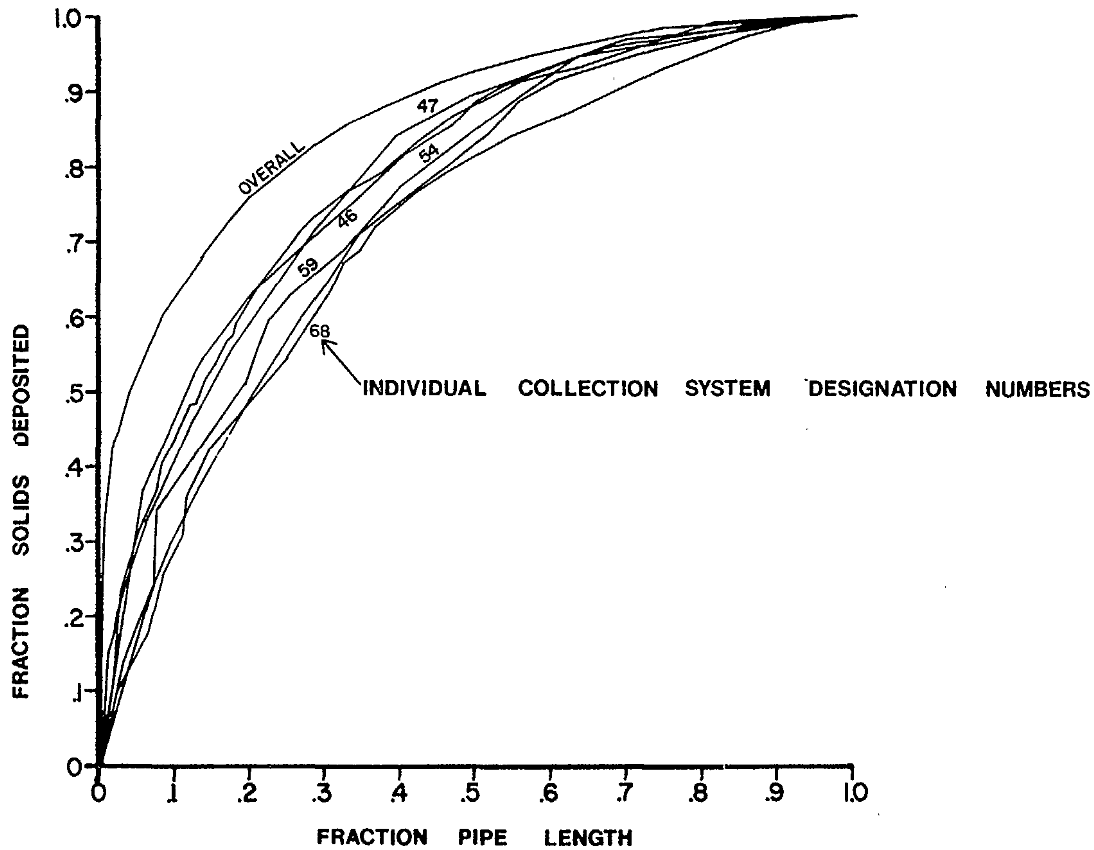

FIGURE 81 DISTRIBUTION OF SOLIDS DEPOSITED BY PIPE LENGTHS – DORCHESTER SYSTEM

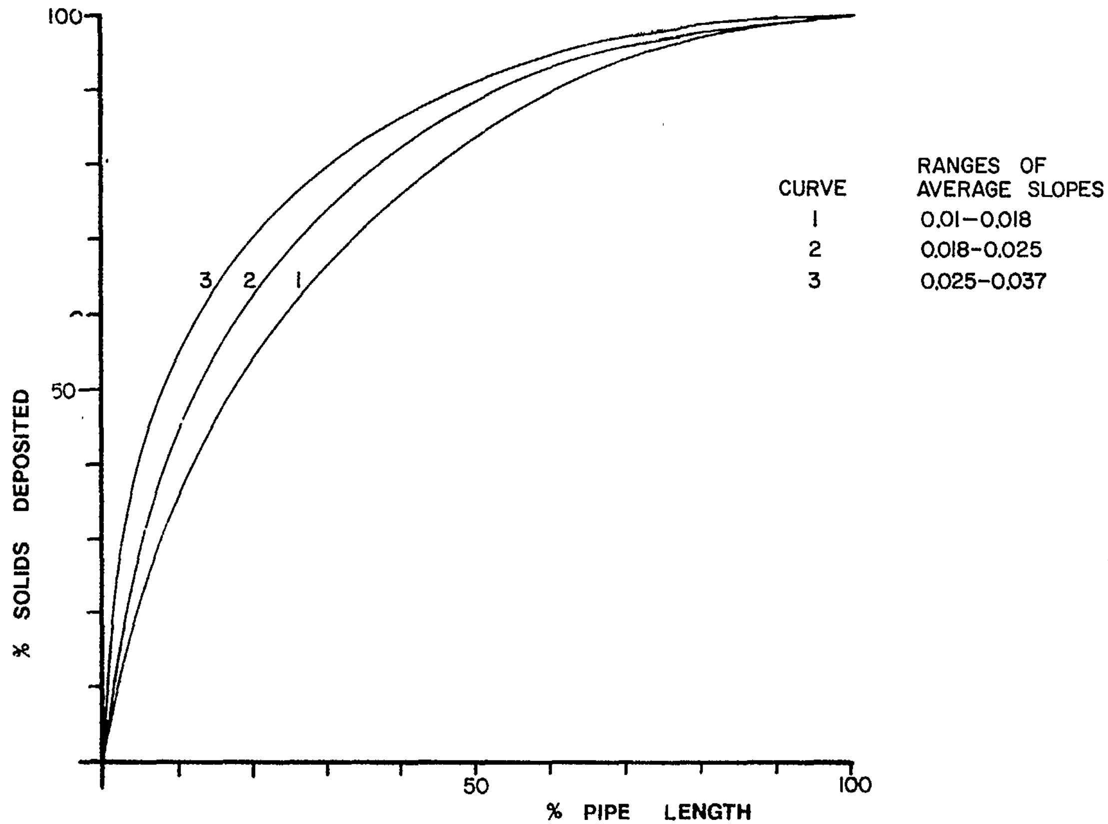

FIGURE 82 CUMULATIVE DISTRIBUTION OF SOLIDS DEPOSITED VS. PIPE LENGTH

13.5.6 Pipe Lengths Corresponding to 80% of the Loads Deposited - L_{PD}

The cumulative distribution of loads deposited versus the cumulative lengths of pipe for all basins were prepared. The values of L_{PD} to be used in the regression analysis were determined in the following manner. The computer printouts of the distributions were scanned for the percentages of pipe length (PL_D) corresponding to 80% of the total loads deposited and either read directly or more often they were interpolated. Those percentages were then applied to the total pipe lengths in the basins resulting in the L_{PD} values for all basins presented in Table 74.

13.5.7 Slope Corresponding to $PL_D(S_{PD})$

The determination of S_{PD}, the slope corresponding to the percentage of pipe (PL_D) where 80% of the total loads deposits is illustrated in Figure 73. The value of PL_D in the cumulative distribution of pipe slopes for a given basin is established and the corresponding slope value S_{PD} is determined. This was performed for all basins by reading directly or interpolating values in the tables of the cumulative distributions of the pipe slopes of these basins. The values of S_{PD} for all the 75 basins considered are given in Table 74.

There is no theoretical justification for the choice of S_{PD} as a regressor. As a matter of fact the average of the slopes ($\bar{S}_{PD}$) below S_{PD} had originally been thought of as a better regressor than S_{PD} itself, and was the first to be included in the regression analysis. A high correlation between S_{PD} and $\bar{S}_{PD}$ was observed and S_{PD} was included in the regression since its reduction of the sum of squares was slightly better than $\bar{S}_{PD}$. The variable $\bar{S}_{PD}$ was dropped from further consideration since its determination required far more effort than that for S_{PD}.

13.5.8 Slope Corresponding to $PL_{D/4}$ - ($S_{PD/4}$)

The determination of $S_{PD/4}$ is illustrated in Figure 73. First of all, multiply by 1/4 the percentage, PL_D, corresponding to 80% of the loads deposited. Next enter that value in the cumulative distribution of pipe slopes for a given basin and determine the corresponding value, $S_{PD/4}$. This step was done by reading directly or interpolating values in the tables of the cumulative distributions of the pipe slopes for the basins. The values of $S_{PD/4}$ for all 75 basins are presented in Table 74.

The choice of $S_{PD/4}$ as an independent variable in the analysis regression did not involve any theoretical consideration. Its choice resulted from an "a priori" belief that representing the lower ranges of the pipe slope distribution would be significant in the regression analysis. It is shown in section 13.6 that this assertion is true.

13.5.9 Summary of Input Requirements for Regression Analysis

Descriptions of the data preparation for the regression analysis has been presented in sections 13.5.1 through 13.5.8. The only remaining independent variable not described in those sections is the waste flow rate contribution in gpcd. This variable, as described in section 13.3.2 was fixed at the values of 40., 110., 190., and 260. gpcd, covering, therefore, a wide range of flows.

To summarize, the values obtained for the independent variables are given in Tables 71 and 74. The tabular values of L' and A are given in Table 71 and the values of L_{PD}, $\bar{D}$, $\bar{S}$, and S_{PD} and $S_{PD/4}$ determined for all 75 basins are given in Table 74. Summary statistics of the independent variables used in regression analysis are given in Table 75 including the range, the mean and the standard deviation for each variable. The results from the deposition model are lengthy and are not presented in this report.

TABLE 75. RANGES, MEANS AND STANDARD DEVIATIONS OF THE INDEPENDENT VARIABLES USED IN THE REGRESSION

Variable	Range	Mean	Standard Deviation
L' (ft)	263.-134528.	13702.	22867.
A (acre)	3.-641	76.	102.
$\bar{S}$ (ft/ft)	0.00238-0.0799	0.0210	0.0126
$\bar{D}$ (in)	8.0-20.0	11.5	2.0
L_{PD} (ft)	36.-34879	4026.	5811.
S_{PD} (ft/ft)	0.0011-0.0589	0.0101	0.0093
$S_{PD/4}$(ft/ft	0.000089-0.0249	0.0037	0.0033
q (gpcd)	40.-260.	*	*

*q is not a random variable.

3.6 Regression Analysis

In this section analyses are presented that relate daily pollutant deposition accumulations in sewerage collection systems with overall collection system physical characteristics. First of all, summary results of various regression models for estimating total suspended solids deposition in collection systems under clean pipe conditions, that is, under the assumption of a well-maintained system, are described in section 13.6.1 through 13.6.3. Secondly, regression relationships are presented in section 13.6.4 that are useful for approximating deposition loadings under conditions of existing sediment deposits, that is, mimicing the situation where poor maintenance prevails. These relationships modify the clean-pipe results given in section 13.6.3. Finally, empirical factors for computing daily accumulations for other pollutants are presented in section 13.6.5 using the total suspended solids estimates given by the procedures in section 13.6.3 and 13.6.4. These factors derive from the data reduction analyses of the field flushing data presented in Chapter 8.

Various combinations of independent variables were initially considered including only primary variates such as total pipe length, L, service acres, A, average pipe diameter, $\bar{D}$, and average collection system pipe slope, $\bar{S}$. More complicated parameters such as L_{PD}, S_{PD} and $S_{PD/4}$ were than incorporated into alternative sets of independent variables.

Multiple correlation coefficients for the untransformed data, that is, the linear additive models, ranged from 0.814 to 0.906. Similar analyses using the logarithmic transformed data yielded multiple correlation coefficients ranging from 0.923 to a maximum of 0.974. This implies that the coefficient of determination, R^2, ranged from 0.852 to 0.949. The multiplicative models were all superior to the linear forms explaining roughly 95 percent of the total variability of the dependent variable. For this reason only the multiplicative model results will be presented. Statistical details of all the alternative regression models considered are not presented but are cited in the earlier report describing this work (37).

13.6.1 Regression Method

The linear regression program used to empirically establish the relationships of the total daily suspended solids (TS) deposition within a sewerage collection system with the independent variables described in previous sections, is one that operates in a step forward manner. At each step in the analysis the particular variable entered into the regression equation accounts for the greatest amount of variance between it and the dependent variable i.e., the variable with the highest partial correlation with the dependent variable. The program is flexible to allow any independent variable to be: (1) left free to enter the regression equation by a criteria of the sum of squares reduction; (2) forced into the regression equation; or (3) be kept definitely out of the regression equation in one given selection. The procedure permits examination of several alternative considerations of the independent variables by optional selections of variables to be forced in and out of the regression equation or to be simply left free to enter the equation using variance reduction criteria.

Observation of the relative change in the standard error of estimate was used as the stopping rule in the regression analysis. An increase of the standard error at a given step indicates that the additional information realized by introducing the variable is off-set by the loss in degrees of freedom, implying that the regression equation is better off without that particular variable.

13.6.2 Regression Analyses

The values of the independent variables used in the regression analysis for all 75 basins were presented in Tables 71 and 74. The range, mean and standard deviations of the independent variables were computed from the data given in Tables 71 and 74 and are presented in Table 75. A total of 300 observations were used in the regression analysis since the deposition loadings for each of the 75 basins were computed for four per capita waste generation rates.

In this analysis various predictive models are analyzed relating total suspended solids deposition within a collection system with the aforementioned independent variables under the assumption of clean pipe conditions. These relationships are therefore applicable for situations in which the sewer piping system is properly maintained. The effects of age and improper maintenance on collection deposition loadings were examined and the results are presented in section 13.6.4. The degree of increased daily deposition resulting from improperly maintained systems was crudely simulated using several assumed depths of bottom sediments.

Both linear additive and multiplicative models were investigated. Untransformed observed values of the dependent and independent variables were initially used, leading to a strictly linear regression equation. In the second case the observed values of both the dependent and independent variables were transformed by taking their natural logarithms, leading to a linear equation in the logarithmic domain which can then be put into a non-linear multiplicative form.

13.6.3 Alternative Model Selections

In this section several regression models are recommended for user application. Alternative forms reflecting the availability of data and/or user resources will be presented. The simplest forms require little data and have the least predictive reliability whereas the more complicated models, requiring greater user resources and data availability, provide estimates with extremely high reliability.

The Elaborate Model. The highest multiple correlation coefficient, 0.974 ($R^2 = 0.949$) was obtained using the model given by equation 42.

$$TS = 0.0038\, L^{0.8142}\, S_{PD}^{-0.8187}\, S_{PD/4}^{-0.1078}\, q^{-0.5098} \quad (R^2=0.949) \qquad (42)$$

where TS is deposited solids loading in lbs/day, L is total length of sewer system in feet, S_{PD} and $S_{PD/4}$ are slope parameters defined in section 13.5.7 and 13.5.8, and q is the per capita waste rate in gpcd.

Utilization of equation 42 requires knowledge of total pipe length, the per capita contribution and the two pipe slope parameters, S_{PD} and $S_{PD/4}$. These slope parameters in turn are a function of the percentage of pipe where 80% of the total mass (TS) deposits (PL_D) and the probability distribution of the pipe slopes. The value PL_D is assumed given for the regression analysis, derived from the extensive computer analyses performed during this study and reported in section 13.5.5. The probability distribution of the pipe slopes can either be derived from histograms computed from local pipe slope data or it can be defined with reasonable approximation from the mean pipe slope ($\bar{S}$) only, as described in section 13.7.2. If the pipe slopes are not available a regression relationship of mean ground slope and mean pipe slope presented in section 13.7.2, could be used to estimate mean pipe slope.

An Intermediate Model. A simpler model than that given by equation 42, including only the primary variables, is given by:

$$TS = 0.001303\, L'^{1.18}\, A^{-0.178}\, (\bar{S})^{-0.418}\, (\bar{D})^{0.604}\, q^{-0.51} \quad (R^2 = .852) \tag{43}$$

where $\bar{S}$ is average pipe slope, and $\bar{D}$ is the average equivalent diameter in inches; and A is service area (acres).

All five independent variables present in equation 43 can be defined with good precision in any practical application. Neither the distribution of deposited loads versus pipe length nor the probability distribution of the pipe slopes are required. The mean pipe slope $\bar{S}$, again can be correlated to the mean ground slope as described in section 13.7.2, if no information on pipe slopes is available. A fitted equation for L' as a function of A was derived from the data used in this study and is presented in section 13.7.1.

If the mean pipe diameter, $\bar{D}$, is eliminated from this regression the loss in precision of the estimates is not significant, resulting in the expression:

$$TS = 0.00389\, L'^{1.2195}\, A^{-0.1866}\, (\bar{S})^{-0.4343}\, q^{-0.51} \quad (R^2 = 0.848) \tag{44}$$

The Simplest Model. The highest R^2 value that can be obtained with the least number of independent variables is given by the regression equation:

$$TS = 0.0076\, L'^{1.063}\, (\bar{S})^{-0.4375}\, q^{-0.51} \quad (R^2 = 0.845) \tag{45}$$

Comments on Model Selections. The user should note that the parameters of the regression equation estimated by the least squares procedure are a function of the data used in their estimation. These parameters are not known with certainty and represent only estimates of the true parameters of the model. The estimation of the parameter is improved as the number of data points increase and as the spread or variance of the independent variables also increases.

It is known from regression theory that the procedure provides the best estimates (least variance) around the means of the independent variables used in computing the parameters of the regression equation. As the values of the independent variables depart from the mean the uncertainty of the estimation given by the equation increases and can become large outside the range of values used as data in the regression. In other words, the uncertainty of the estimate given by the regression equation may be large when the model is used for conditions requiring extrapolation beyond the range of data used to develop the model.

13.6.4 Effects of Age and Maintenance

The regression equations presented in section 13.5.3 were derived from deposition data computed under the assumption of clean pipes, with no bottom sediments. In this section the impact of poorly maintained systems was crudely simulated by arbitrarily assuming various levels of prior sediment accumulation in the pipes. These sediment levels would change the bottom cross-sectional shape of the pipe channel and hence the depth of flow, the hydraulic radius and the shear stress characteristics.

Two cases simulating different degrees of maintenance other than perfect clean pipe conditions were considered. In the first case, or the intermediate maintenance category, sediment beds ranging from 1 to 3 inches in depth were assumed for all pipes with slopes less than 0.0075. A sediment bed of 3 inches was assumed for all pipes with slopes less than 0.0005. The bed depths then ranged linearly starting at 3 inches for a pipe slope of 0.0005 up to one inch for a pipe slope of 0.0075. This range was established using judgment and also based on visual inspection of numerous combined sewer laterals in eastern Massachusetts sewerage systems. In the second category of maintenance, the zero maintenance case, sediment beds ranging from 3 to 6 inches for the same range of slopes was considered.

Considering the two age and maintenance criteria mentioned here, the deposition model was used to estimate total deposition loadings for each of the 75 sewer systems for each of the four per capita waste generation rates of 40, 110, 190 and 260 gpcd. Before similar regression computations were performed on the deposition results obtained for pipes with bottom deposits, a comparison was made of the total deposited loads computed under the assumptions of clean and sedimented pipes.

For each basin the ratios of TS computed for sedimented pipes with sediment beds of 1-3 inches and 3-6 inches and the TS values for clean pipes were calculated for all four per capita waste rates considered, i.e., 40, 110, 190 and 260 gpcd. The resulting ratios were very stable for a given per capita waste rate for both cases of sediment deposits. The mean and coefficient of variation of these ratios are presented in Table 76 for both conditions of bottom deposits.

The results shown on Table 76 suggest that the prediction of TS in sedimented pipes could be accomplished by a simple functional multiplicative correction of the results given by any of the regression equations for clean pipes. An equation was fitted using the data of Table 76 for each of the bed deposit conditions.

These equations are:

- For a system with deposits ranging from 1 to 3 inches;

$$TS_{1-3\ inches} = 1.68\ q^{-0.076}\ TS(clean)\ (R^2=0.988) \qquad (46)$$

TABLE 76. AVERAGE VALUES OF THE RATIOS OF COMPUTED LOADS IN DEPOSITED PIPES OVER CLEAN PIPES

Ratios	Average Values of Ratios for Per Capita Waste Rates			
	40	110	190	260
$TS_{1-3inches}/TS(clean)$	1.263	1.186	1.128	1.094
	(0.18)	(0.14)	(0.07)	(0.12)
$TS_{3-6inches}/TS(clean)$	1.312	1.211	1.151	1.121
	(0.14)	(0.11)	(0.09)	(0.09)

The numbers in parenthesis indicate the coefficient of variation of the ratios.

where q = flow per capita, and TS(clean) = load of total solids computed from any of the regression equations presented in section 13.6.3.

- For a system with deposits ranging from 3 to 6 inches:

$$TS_{3-6\ inches} = 1.79\ q^{-0.084}\ TS(clean)\ (R^2=0.999) \qquad (47)$$

The R^2 values indicated above refer to the regression of the ratios of TS on the values of flow per capita. The small difference found between the two conditions of bottom deposits may well be the result of an inappropriate accounting of these factors by the deposition model. On the other hand it may simply have resulted from the particular combination of pipe diameters and sediment depths used as data, which may have led to actually small differences in flow depths above the sediment levels, and therefore small differences in shear stress between the cases.

13.6.5 Estimation of Other Pollutants Using TS Results

Results of the first phase field flushing experiments described in chapter 8 were used to develop factors for estimating other pullutant deposition loadings. The normalized flushed pollutant loadings from Tables 23 thru 27 in chapter 8 were used to compute factors for estimating relative ratios of COD, BOD, TKN, NH_3, TP and VSS to TSS. Pairs of normalized flushed pollutant loadings to solids flushed were grouped for all good non-rainfall impacted flushes from all segments and the average ratios computed. These average results are summarized in Table 77. The mean, standard deviation and number of flushing experiments used in the computation are shown in Table 76 for each pollutant. The coefficients of variation for these factors with the exception of TKN are all remarkably low.

TABLE 77. AVERAGE POUNDS OF POLLUTANTS PER POUND OF TSS* PER ANTECEDENT DAYS

	MEAN lb/lb TSS	SD	# POINTS
COD	1.247	0.269	14
BOD	0.434	0.203	26
TKN	0.041	0.114	14
NH_3	0.014	0.009	13
TP	0.009	0.005	14
VSS	0.745	0.108	45

* All good, non-rainfall impacted flushes of all 4 streets

13.7 Model Utilization

This section summarizes the steps required by the user to define on a preliminary basis the total amounts of solids and other pollutants that deposit in the sewerage system. The nature of the available data and/or the degree of resources the user can commit will define different work programs which in turn will lead to different levels of confidence in the results.

Several additional relationships derived from analysis of collection system characteristics and computational methods for deriving estimates of the various parameters in the deposition prediction models will be given. Descriptions of the necessary user steps required to use the predictive models will then follow.

13.7.1 Estimation of Total Pipe Length

The total pipe length of the system, L', and its corresponding collection area, A, are generally assumed to be known. In cases where this information is not known and where crude estimates will suffice, the total pipe length can be estimated from the total basin area using the expressions:

$$L' = 168.95\ A^{0.928}\ (R^2=0.821)\text{-low population density (10-20 people/acre)}$$
$$L' = 239.41\ A^{0.928}\ (R^2=0.821)\text{-moderate-high population density (30-60 people/acre)} \quad (48)$$

These expressions were derived from the data for the 75 basins used in this study, yielding a correlation coefficient of 0.906. Since detailed population density estimates for each collection system in the data set were not available, the following approach was used to account for the effect of population density on length of sewer. The data for the 75 basins were separated into two distinct data sets representing low population densities (the 35 basins from the West Roxbury-Newton-Dedham-Brookline sewerage system) and moderate to high population densities (the latter 40 basins from the Dorchester and Fitchburg sewerage systems). A dummy variable approach was used in the regression analysis to obtain the two expressions given in equation 48 . A more explicit treatment of population density would be possible if detailed population density estimates per collection system were available.

13.7.2 Estimation of Mean Pipe Slope $\bar{S}$

Pipe Slope Data is Available. In this case:

$$S = \frac{\sum_{i=1}^{n} S_i l_i}{\Sigma l_i} \qquad (49)$$

where: S = slope of segment i

l = length of segment i

n = total number of pipe segments

Pipe Slope Data Not Available. If data on pipe slope is not available the user will determine a mean value for the ground slope using any procedure, such as the techniques of uniform grid sampling of random sampling. With the mean ground slope value, determine the mean pipe slope by:

$$\bar{S} = 0.348\ (\bar{S}_G)^{0.818}\ (R^2=0.96) \qquad (50)$$

where: $\bar{S}_G$ = mean ground slope (ft/ft)

The expression above resulted from a regression of mean ground versus mean pipe slope for all 75 basins of this study. The resulting correlation coefficient was 0.98.

This relation was derived in the following manner. Topographic mylar overlays with ground contour at ten feet intervals were developed for the WRNDB and the Fitchburg sewer system service areas. Similar maps existed for the Dorchester and South Boston sewer systems. Using the topographic maps the collection system areas were subdivided into smaller fractions considered to have uniform slopes. The ground slopes were computed for each of the subareas. Those ground slopes were then associated with the percentages of the collection system area they covered. The basin average ground slope was then computed by weighing the individual slopes by their associate subarea percentages. Ground slopes in the smaller basins were determined by using one or two subareas whereas the ground slopes in the large basins were determined using 10 to 21 subareas.

13.7.3 Distribution of Total Solids Deposited by Pipe Segment - Determination of PL_D

The distribution of total solids deposited by pipe length is presented in Figure 82. The user can determine PL_D corresponding to 80% of the solids deposited by entering the graph with "percent of mean deposited" equal to 80% and, from the curve that best approximates the average pipe slope $\bar{S}$, read on the horizontal axis, "% pipe length", the value of PL_D. This value will be required to determine S_{PD} and $S_{PD/4}$ as described below.

13.7.4 Determination of Slopes S_{PD} and $S_{PD/4}$

Pipe Slope Data is Available. The collection system pipe slopes can be directly used to define the cumulative pipe slope distribution. In this study the pipe lengths (as integer values) associated with the observed slopes were taken as their frequencies. The user should adopt the same approach for compatability with the equations for TS derived in this study. Simplified methods may also be used, with less accurate results. If, for example, the slopes are not weighted by their lengths, this is equivalent to assuming that all pipe segments have the same length, which in some cases may be a reasonable assumption.

Pipe Slope Data Not Available. The average slope $\bar{S}$ can be computed using either equation 49 or equation 50 to define the pipe slope cumulative density function (CDF) as follows:

$$FS = 1-e^{-s/\bar{S}} \qquad (51)$$

If the probability F_S is known, with a fixed $\bar{S}$ the corresponding slopes can be computed, and vice-versa. For the basins used in this study, equation 51 was verified with excellent results as described in Section 13.5.3.

To determine the slope S_{PD}, use the value of PL_D determined in section 13.7.3, and make:

$$F_S = 1-e^{-s/\bar{S}} = PL_D$$

and determine s. Then:

$$S_{PD} = s$$

To determine the slope $S_{PD/4}$, make:

$$F_S = 1 = e^{-s/S} = 1/4PL_D$$

and determine s. Then:

$$S_{PD/4} = s.$$

13.7.5 Formula for the Average Pipe Diameter

If the system contains non-circular shapes, first compute their equivalent circular diameters using the equations for equivalent pipe diameters presented in Table 73. Then, compute the average diameter by:

$$D = \frac{\sum_{i=1}^{n} D_i l_i}{\Sigma l_i} \tag{52}$$

where: D_i = diameter of segment i

l_i = length of segment i

n = total number of pipe segments.

13.7.6 General Description of User's Steps

The generalized procedure for obtaining total solids deposited estimates is presented in Figure 83 and should be referred to in the ensuing discussion.

The first question to be answered at the onset is whether pipe slope data are available. If the answer is positive the user should use equation 49 to determine the average pipe slope $\bar{S}$; otherwise, a mean ground slope should be determined as described in section 13.7.2 and then equation 50 should be used to determine $\bar{S}$.

The next question that arises is whether the total pipe length of the collection system is known. In the negative case the total area of the collection system is assumed known and the total pipe length is estimated by equation 48 . The per capita waste rate including infiltration is then established.

The selection of the equation for the estimation of the solids deposited, TS, follows. If the simplest model is desired the user has at this point all the elements required by equation 45 to compute TS. If the intermediate model is chosen and the pipe diameter information is available the mean pipe diameter is computed by equation 52 . All information is computed to estimate TS from equation 43 . If pipe diameters are not available then use equation 44 . If the elaborate model is chosen, the value of PL_D must be determined using Figure 82 as described in section 13.7.3. The next variables to be determined are S_{PD} and $S_{PD/4}$.

If no pipe slope information is available the user has no alternative but to use the exponential approximation for the pipe slope cumulative distribution. If the pipe data are available but the user considers the exponential approximation satisfactory he may use equation 51 , associated with PL_D and S to determine S_{PD} and $S_{PD/4}$ as described in section 13.7.4. Alternatively, the user can define the pipe slope CDF from the pipe slope data and derive from it, the values of S_{PD} and $S_{PD/4}$ as described in section 13.7.4. Finally, the user can define the pipe slope CDF from the pipe slope data and derive from it, the values of S_{PD} and $S_{PD/4}$ corresponding to the percentages PL_D and $1/4PL_D$, respectively. At this point all the elements are prepared to estimate TS from equation 42 which provides the most reliable estimate of TS.

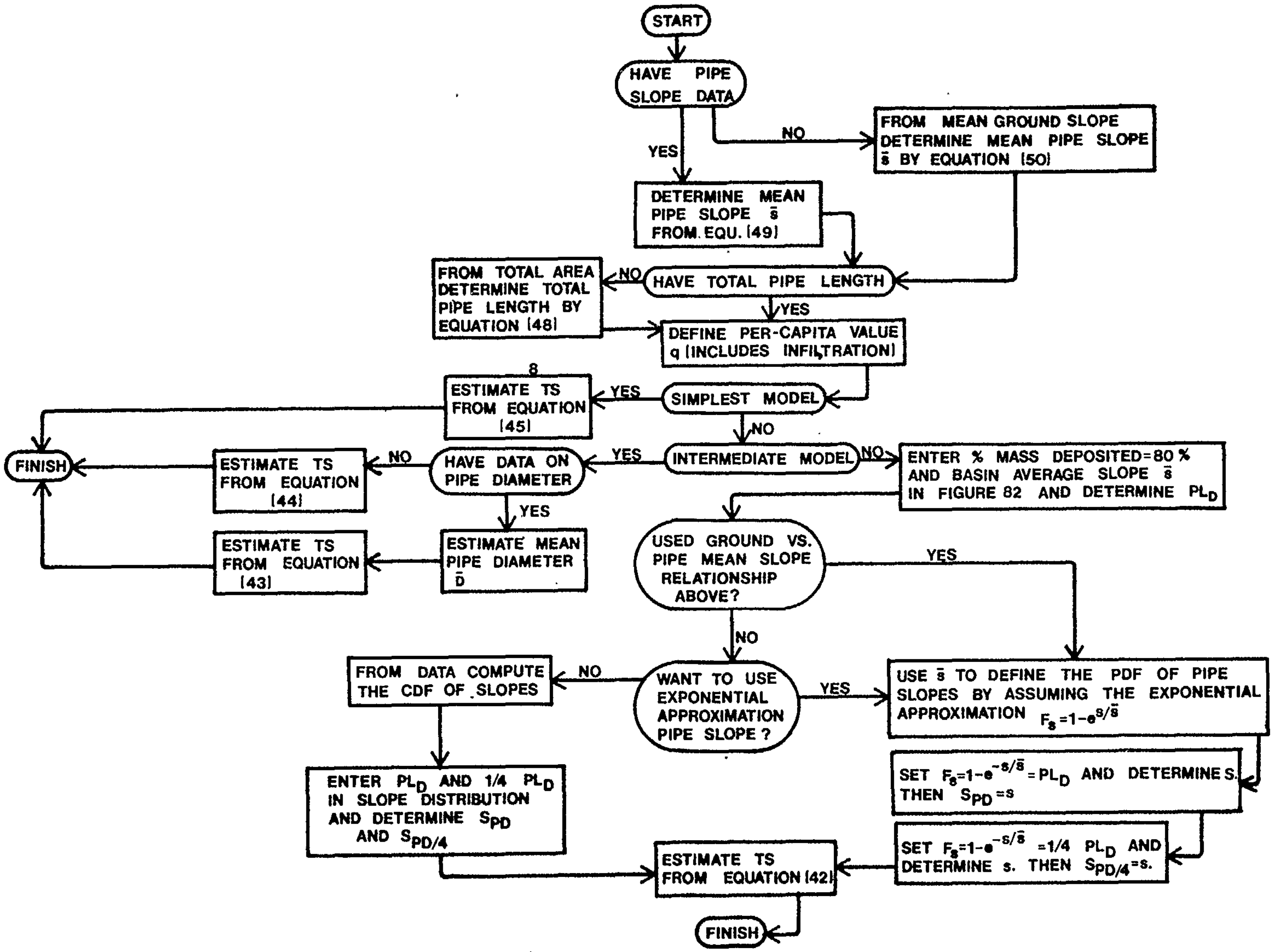

FIGURE 83 USER STEPS TO DETERMINE TOTAL SOLIDS DEPOSITED

The resulting estimate of TS is the total daily solids deposition in the collection system of interest. If the user wishes to modify this estimate for pipes with existing sediment beds the multiplicative equations in section 13.6.4 should be used. If estimates of other pollutants are desired, the deposited solids results, TS, should then be used as predictors to compute other pollutant estimates using the equations given in section 13.6.5.

13.8 Test Case Application of Prediction Procedures

In this section an example problem illustrating the methodologies developed in this report is presented. The test case is one of the collection systems in the Dorchester sewerage system. Figure 84 shows a diagram of the collection system. Estimates of total daily solids deposition in this collection system will be given for different assumptions of data availability using the simplified procedures. These estimates will also be compared with the results computed for this system using the deposition model described in Chapter 12.

The test case used in basin number 70 of the Dorchester combined sewer system. There are 190 manhole segments in this collection system. The topography of this combined sewer collection system is fairly hilly. The land use in the area is exclusively high density multi-family dwellings with a population density of 30 people/acre. The areal extent and total collection system pipe footage is about one standard deviation above the mean of all the systems used in the regression analysis. The values of all the independent variables for basin 70 are given in Tables 71 and 74.

Data Requirements. The total pipe length, L', for the basin is 35,225 feet and the total service area, A, is 233 acres. The total pipe length can also be computed using equation 48 for high population density yielding 37,740 feet, representing an overestimation error of 7.7%.

There are three possible ways to compute the values of the pipe slope variables required as input for the various regression equations. The first method involves computing these parameters from the distribution derived from actual pipe slope data. The second procedure requires knowledge of only the mean collection system pipe slope, $\bar{S}$, and assuming that the pipe slopes can be represented by an exponential distribution. The third alternative approach assumes that only ground slope information is available and that the exponential distribution is applicable to represent collection system pipe slopes.

The collection system pipe slope histogram representing the distribution of 190 pipe segments in basin 70 of the Dorchester sewerage system is given in Table 78. This table was prepared using the invert elevations for each of the 190 segments. The pipe slope histogram is divided into 30 intervals with the upper pipe slope limit of each interval given. The elements under the column labeled "frequency" are the actual pipe footages associated with each pipe slope interval. The last two columns give the interval probability computed using the pipe footage per interval relative to total pipe footage and the cumulative probability. The mean and standard deviation are also given in Table 78. The average

AREA: 223 ACRES
PIPE FOOTAGE: 35,225
NUMBER OF SEGMENTS: 190
POPULATION DENSITY: 30/ACRE
AVERAGE GROUND SLOPE 0.0359

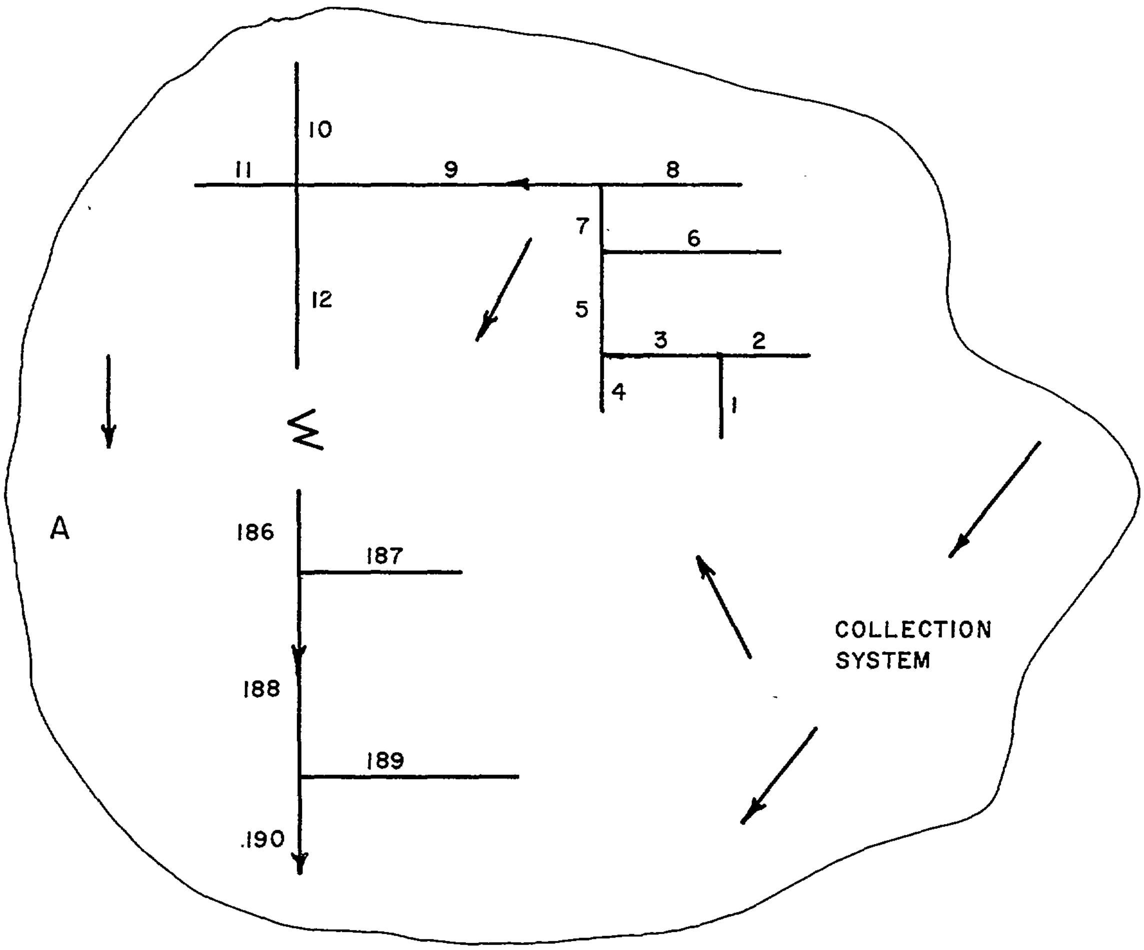

FIGURE 84 DIAGRAM AND PERTINENT INFORMATION FOR BASIN 70 - DORCHESTER

TABLE 78. DISTRIBUTION OF PIPE SLOPES FOR BASIN 70

AVG. DIAMETER = 15.6

HISTOGRAM FOR FIXED RANGES (MAX. AND MIN. ABCISSA VALUES SPECIFIED)

INTERVAL	UPPER LIMIT OF RANGE	FREQUENCY	PROBABILITY	CUMMULATIVE PROBABILITY
1	0.000100	0	0.0	0.0
2	0.003311	3680	0.1045	0.1045
3	0.006521	10477	0.2974	0.4019
4	0.009732	3239	0.0920	0.4938
5	0.012943	4099	0.1164	0.6102
6	0.016154	1189	0.0338	0.6440
7	0.019364	680	0.0193	0.6633
8	0.022575	832	0.0236	0.6869
9	0.025786	1229	0.0349	0.7218
10	0.028996	752	0.0213	0.7431
11	0.032207	472	0.0134	0.7565
12	0.035418	909	0.0258	0.7823
13	0.038629	1485	0.0422	0.8245
14	0.041839	371	0.0105	0.8350
15	0.045050	752	0.0213	0.8564
16	0.048261	247	0.0070	0.8634
17	0.051471	315	0.0089	0.8723
18	0.054682	460	0.0131	0.8854
19	0.057893	1528	0.0434	0.9288
20	0.061104	0	0.0	0.9288
21	0.064314	560	0.0159	0.9447
22	0.067525	68	0.0019	0.9466
23	0.070736	1033	0.0292	0.9759
24	0.073946	334	0.0095	0.9854
25	0.077157	16	0.0005	0.9858
26	0.080368	0	0.0	0.9858
27	0.083578	340	0.0097	0.9955
28	0.086789	126	0.0036	0.9991
29	0.090000	0	0.0	0.9991
30	> 0.090000	32	0.0009	1.0000

Mean = .0194058 Standard Deviation = .020859.

equivalent circular diameter, $\bar{D}$, is 15.6 inches and was computed using the formulas given in Table 73, and equation 52.

The average pipe slope, $\bar{S}$, is 0.0194. The slope parameter S_{PD} can be estimated in the following manner. Three curves are available in Figure 82, Section 13.5.5, for relating the cumulative distribution of solids deposited to cumulative distribution of collection system pipe length. Curve 2 is applicable in this case since the average pipe slope is 0.0194. The percentage of pipe associated with 80% of the total daily load deposited is 38%.* The value of the variable, S_{PD}, can then be determined by entering 0.38 in the last column of Table 78 and interpolating the corresponding value of slope in column two. The interpolated value of S_{PD} is 0.00629. The value of $S_{PD/4}$ is obtained by entering the same table with $0.38/4 = 0.095$, yielding $S_{PD/4} = 0.00302$.

In the second procedure the mean pipe slope can be used in the exponential cumulative distribution function given by equation 78, to compute the values of S_{PD} and $S_{PD/4}$. The value of S_{PD} is obtained by solving equation 51 for s, having set $FS = 0.38$ and $\bar{S} = 0.0194$. The result is $S_{PD}=s=0.00928$. The value of $S_{PD/4}$ is found by solving equation 51 again for s, having set $FS = \frac{0.38}{4} = 0.095$ and $\bar{S} = 0.0194$. The result is $S_{PD/4} = s = 0.00194$.

The only information required for the third procedure is the average ground slope, $\bar{S}_G$. The average ground slope for basin 70 was determined by graphical procedures to be 0.0359. An estimate of the mean pipe slope, $\bar{S}$, can be obtained using the mean ground slope in equation 50. The estimated value of mean pipe slope, $\bar{S}$, using the mean ground slope, $\bar{S}_G$, is 0.02287 differing from the actual mean pipe slope by 17%. Values of S_{PD} and $S_{PD/4}$ can now be estimated using the exponential relationship in equation 51, yielding $S_{PD} = 0.00985$ and $S_{PD/4} = 0.00229$.

Comparison of Deposition Prediction Procedures. The detailed manhole to manhole dry weather deposition model described in Section 12.4 of Chapter 12 was used to estimate total solids deposited for the entire collection system network in basin number 70. The estimated load is 169.11 lbs/day using an average per capita waste rate of 190 gpcd. This model requires detailed specification of the hydraulic parameters for each segment in the system and the use of a computer. Similar estimates will be computed from using the simplified power functions generated in this study for the three different estimates of the pipe slope variables.

The elaborate model given in equation 43 requires specification of L, S_{PD}, $S_{PD/4}$ and the per capita flow rate, q. This equation was solved for the three different estimates of S_{PD} and $S_{PD/4}$ with q = 190 gpcd. The deposited load, TS, is 155.73 lbs/day using the values of S_{PD} and $S_{PD/4}$ derived from the analysis of detailed collection system pipe slope information. The estimated load is 118.72 lbs/day using values of S_{PD} and $S_{PD/4}$ computed from the mean pipe slope and the exponential cumulative function. The estimated load is 111.11 lbs/day using values of S_{PD} and $S_{PD/4}$ for the third

*The distribution of loads by pipe length derived for basin 70 yields exactly 38% at 80% of the total load. The error involved in using Figure 82 is zero in this example.

situation where the mean ground slope is used to compute mean pipe slope for input into the exponential function.

The intermediate model given by equation 44 requires that L, $\bar{A}$, $\bar{\bar{S}}$, and q be specified. Three variations are computed using this formulation. The first estimate is determined using the measured pipe length, L and pipe slope, $\bar{\bar{S}}$, derived from data; the second case uses the measured pipe length and an estimate of mean pipe slope from ground slope estimate; and the third result is computed using the estimated pipe slope $\bar{\bar{S}}$, and an estimate of pipe length derived from equation 48 . These three estimates are 186.89, 174.02 and 190.55 lbs/day, respectively.

The simplest model given by equation 45 requires specification of L', $\bar{\bar{S}}$ and q. The estimated loads are 198.82 lbs/day using exact estimates of L' and $\bar{\bar{S}}$, 185.03 lbs/day using exact estimates of pipe length and an estimate of pipe slope ground from ground slope, and, 200.26 lbs/day using estimated values of both pipe length and mean pipe slope.

A comparison of all computed loadings for the different predictive models under different assumptions of data availability is given in Table 79. The percentage error relative to the estimate provided by the deposition model described in Chapter 12 is also provided. These results show that the estimated values given by all three regression equations are reasonably close to the value derived from the detailed collection system model. The elaborate model consistently over estimated deposition loadings.

It is an invalid conclusion to infer from this comparison that the simpler approach might be superior to the more elaborate one. It should be noted that this comparative result is only for one basin and that on the average, the elaborate model will provide consistently superior results because the coefficient of determination R^2, is higher for the elaborate model. The under estimates given by the elaborate model using approximations for the pipe slopes are explained by the fact that, for this particular basin, the exponential approximation over estimated by about 50% the slope, S_{PD}. Nevertheless, one of the simpler models would be more appropriate in cases where little data is available requiring many assumptions and approximations. Moreover, the utilization of the simpler models for planning "first-cut" purposes may be more cost effective from the standpoint of collecting, analyzing and preparing the required data inputs.

In sum the simplified procedures given by equations 42 , 43 and 45 provide estimates of daily solids deposition using exact data for basin 70 with a relative error of 8 to 18 % in comparison to the estimate given by the complicated procedure described in Chapter 12. It was shown in Chapter 12 that the complicated procedure was roughly calibrated using actual field flush information lending credulence to the adoption of the simplified procedures generated in this report.

TABLE 79. COMPARISON OF ESTIMATED DAILY SOLIDS DEPOSITED FOR BASIN 70 USING DIFFERENT PROCEDURES

Procedure	Solids Deposited (lbs/day)	Percentage Error Relative to Chapter 12 Results
Deposition Model (Chapter 12)	169.11	
Elaborate Model (Eq. 42)		
Exact Data	155.73	-7.9%
Exponentially Dist. Slopes	118.72	-30.0%
S_G and Exponentially Dist. Slopes	111.11	-34.0%
Intermediate Model (Eq. 44)		
Exact Data	186.89	+10.5%
Estimated Slope $\bar{S}$	174.02	+ 3.0%
Estimated L' and $\bar{S}$	190.55	+12.7%
Simplest Model (Eq. 45)		
Exact Data	198.82	+17.6%
Estimated $\bar{S}$	185.03	+ 9.4%
Estimated L' and $\bar{S}$	200.26	+18.4%

SECTION 14

FLUSHING GUIDELINES

14.1 Foreword

Overview comments on the use of sewer flushing as an integral component of new abatement technologies for combined sewer management is presented in section 14.2. A conceptual overview dealing with the development of a sewer flushing program is presented in section 14.3. Approximate costs of sewer flushing using both manual and automated operation is presented in section 14.4. Procedural details and guidelines dealing with the implementation of a sewer flushing program are given in section 14.5.

14.2 Management Overview

The concept of controlling depositing solids in sewer lines, although widely used around the turn of the century as a maintenance practice, is still in its infancy in regard to being viewed as a viable pollution control alternative for combined sewer systems. Even if shown to be effective, the concept of sewer flushing is still a controversial issue since it involves generally low capital first-cost investments but high operational and maintenance costs. Federal funding mechanisms presently favor high first-cost programs with low operational costs. Federal funding does not cover operational costs. Moreover, the notion of increasing the municipal commitment for greater manpower investments runs counter to the historical trend toward decreasing public works' budgets in the area of sewerage system management.

It is the authors' opinion that sewer flushing in itself should not be viewed as a substitutable alternative for structural control, but instead, when integrated together with other upstream management practices and downstream structural options where necessary, overall costs can be significantly reduced. The federal government's will and commitment to spending many billion dollars over the next decade in combined sewer control must be met with equal commitment to execute this mandate in a cost-effective manner.

Suggestive management concepts employing several state-of-the-art combined sewer controls are illustrated in Figure 85. It is assumed that thorough sewer system physical surveys aimed at identifying cost-effective pollution control malfunctions have been performed and the requirement for additional control has been established. In short, the sewer system is "tuned-up" and further pollutant reductions are necessary. The four additional controls shown in Figure 85 are sewer flushing, upstream off-line storage, swirl regulators and disinfection using chlorine and/or possibly chlorine

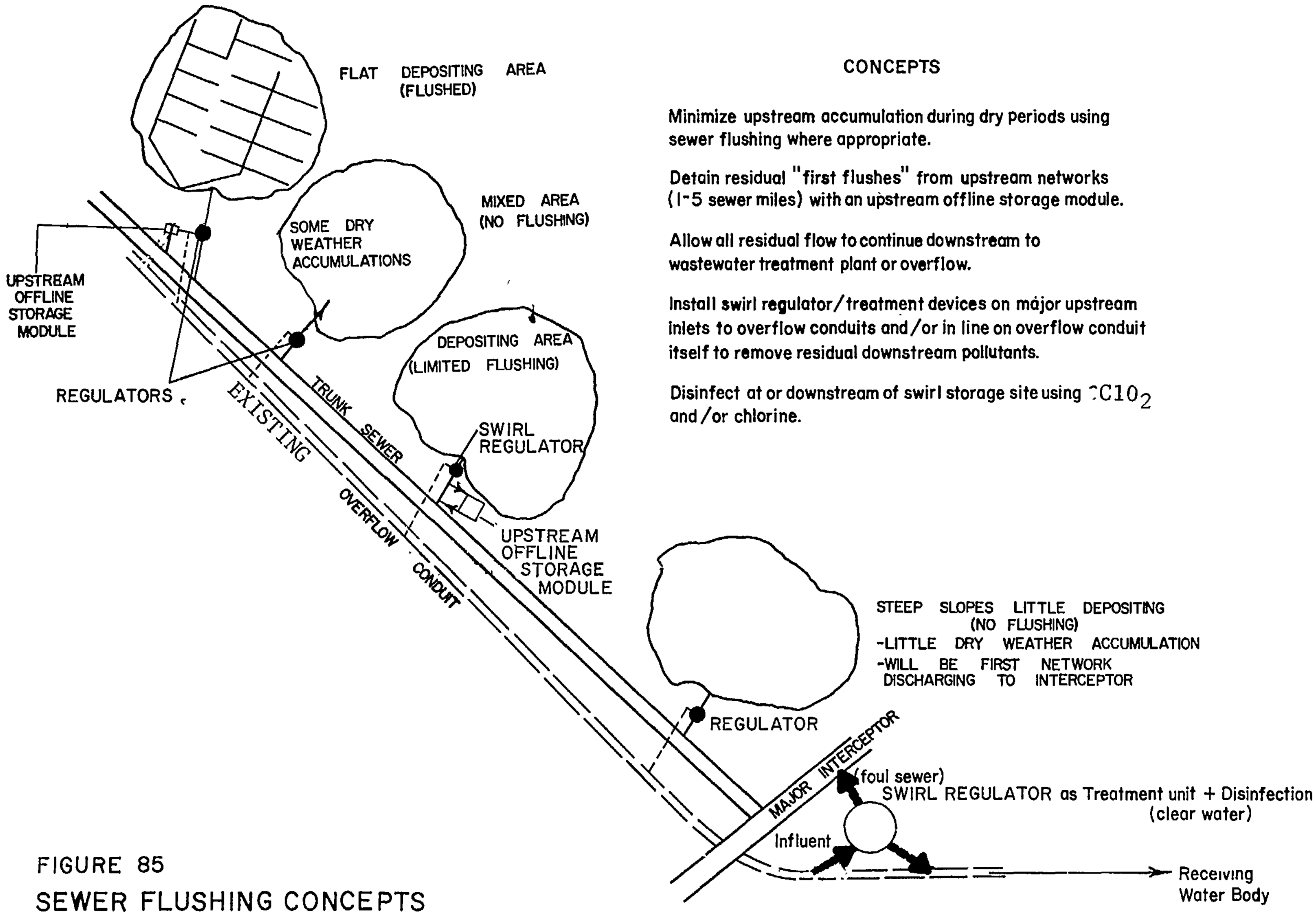

FIGURE 85
SEWER FLUSHING CONCEPTS

dioxide. Sewer flushing in flat collection systems would minimize upstream accumulations during dry weather. Upstream storage modules, optionally, could be employed to retain dry weather flushes and residual "first-flushes." Swirl regulators and/or other treatment devices at major inlets to overflow conduits can be used to remove residual pollutants. Newer forms of disinfection, including sequential dosing of chlorine/chlorine dioxide mixed by the swirl flow, having high bacterial kill in extremely short detention periods, could be then incorporated upstream of the swirl.

The upstream control concept combines flushing and off-line storage. Pollutants removed by flushing would either go directly to the treatment plant and/or alternatively to locations where storm flows and/or released captured upstream stormwater would flush the loads to off-line storage. The flushing program would be combined with an active maintenance program where malfunctions in the collection system are quickly detected and remedied.

The advantages of such an integrated approach would be the following: a) reduction in size and cost of required major downstream storage/ treatment facilities; b) improved overall solids and pollutant removal; c) more even temporal distribution of solids loadings to the wastewater treatment plant; d) improved flow stabilization during high intensity - short duration storms, and e) improved sewer maintenance practices with the resulting benefit of fewer system malfunctions which can in many instances, reduce overflow pollutant loadings.

14.3 Conceptual Overview for Developing a Sewer Flushing Program

An outline of the following steps necessary to implement a sewer flushing program are presented in Figure 86. Brief descriptions of each step are as follows:

Step A - Deposition Load Estimation

First, the sewerage system should be divided into manageable collection systems, excluding major trunk and interceptor sewers. Estimates of the daily dry weather sewage pollutant deposition must be prepared for each collection system. A series of simplified procedures for estimating gross levels of dry weather sewage pollutant deposits within collection systems are available and are described in Chapter 13. Date requirements for these procedures are also given in Chapter 13. Minimal information required to use these procedures are estimates of the service area, the average ground slope and the overall per capita waste rate, including infiltration. Procedures are also available in Chapter 13 for computing pipe length from service area estimates. These estimates of collection system deposition loadings do not include any allowances for industrial wastes deposits. Estimates of significant industrial waste contributions would be necessary.

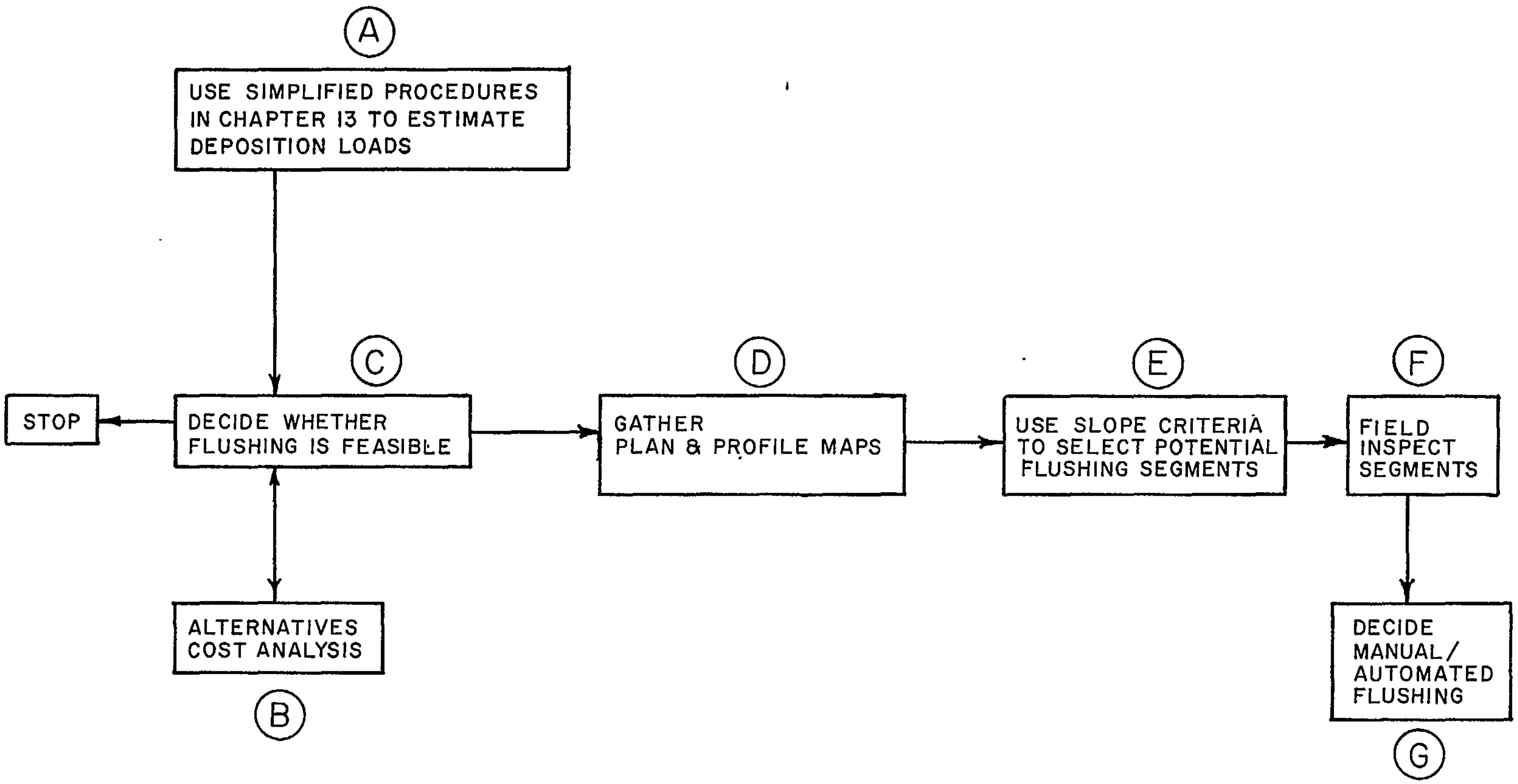

FIGURE 86 CONCEPTUAL OVERVIEW FOR DEVELOPING A SEWER FLUSHING PROGRAM

Step B - Alternatives Cost Analysis

"Desk-top" assessments can then be performed to determine whether further consideration is worthwhile. Two elements are necessary to roughly appraise the viability of sewer flushing program. First, the costs of flushing must be known and, second, the removal effectiveness must be estimated. Flushing costs are given in section 14.4. A rough rule-of-thumb is $25,000 (present worth: 20 years @ 8%) per flushing site, using either a manual or automated means.

The extent of a flushing program can be roughly estimated in the following manner. Equation 50 in section 13.7.2 can be used to estimate average collection system pipe slope using an estimate of average ground slope in the collection system. The estimate of the average collection system pipe slope can then be used in Figure 82, section 13.5.5 to select a curve depicting the cumulative percentage of mass deposited versus the cumulative fraction of pipe length. The user can select from among a family of curves prepared for different average pipe slopes. The left-hand plot in Figure 87 is an example of such a curve. The next step is to select the percentage of mass deposited, PM, to be controlled by sewer flushing. The left-hand plot can be used to estimate the fraction of pipe length, PL, in the system corresponding to the selected value of PM.

The next step is to determine the effective length controlled by a single flushing operation. The results of the sewer flushing experiments reported in Chapters 8 through 11 indicated that flushing over 700 feet was effective in removing more than half of the deposited solids and pollutant loadings. A predictive procedure was developed in Chapter 12 for estimating percent mass of solids, organics and nutrients remaining in suspension at downstream points as a result of a single upstream flush. The model projects that about 20% of the flushed solids mass would be in suspension at a downstream distance of 1000 feet, and also 45% of flushed BOD and 50% of TKN masses would remain in suspension at this distance. A simple rule is to assume that a single flushing operation could control 800 feet of pipe segment with a 30% solids removal, and 60% organics and nutrient removals. The number of flushing sites can then be estimated and the overall cost effectiveness computed. Other combinations of flushing effectiveness and controlled segment length could be investigated. Other alternatives, such as street sweeping and storage/treatment, could be prepared and their cost effectiveness computed.

Step C - Decision

If the "desk-top" assessments show that sewer flushing is attractive then the next important step is to determine those segments in the collection system where sewer flushing can be considered.

Step D - Plan & Profile Maps

Plan & profile maps of the collection system should be assembled and reviewed. Tabulations should be prepared of segment location and pipe slope.

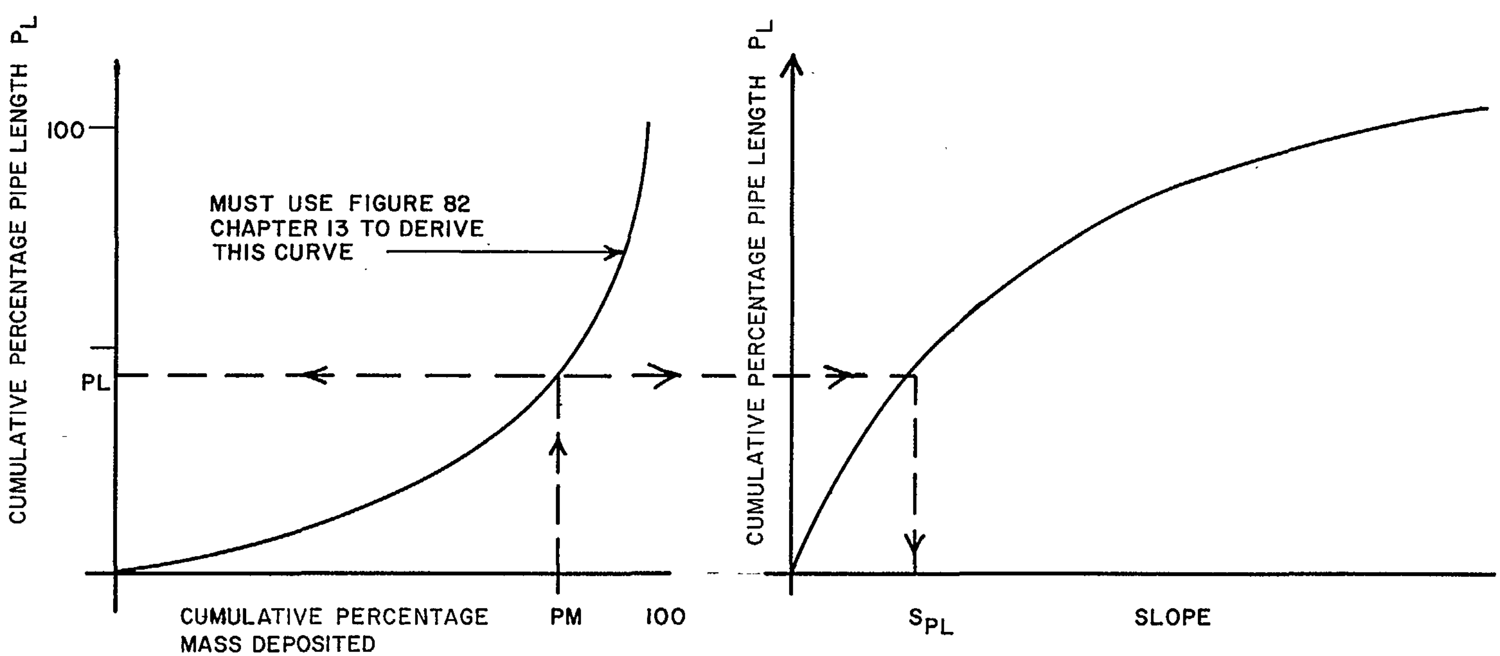

FIGURE 87 DETERMINATION OF THE CUT-OFF SLOPE FOR A PERCENTAGE OF MASS DEPOSITED

Step E - Selection of Pipe Segments

A rough-cut approach was developed in this study to identify in a preliminary fashion, potential segments for flushing considerations. Extensive statistical analyses of sewerage system pipe slopes revealed that collection system pipe slopes can be represented by an exponential probability model. The exponential distribution can be completely defined using the mean value. The average collection system pipe slope was computed in Step A from an estimate of the average ground slope. Equation 51 in section 13.7.4 can be used to define the cumulative function of collection system pipe slope, FS. A plot of a cumulative pipe slope distribution function is shown on the right in Figure 87.

The cut-off slope for locating potential segments for sewer flushing can be determined in the following manner. The percent mass, PM, to be flushed is entered in the plot on the left in Figure 87. The corresponding percentage of pipe length, PL, associated with PM, is then read off the curve. The value of PL is then entered into the ordinate scale of the cumulative pipe slope distribution function on the right side of Figure 87, and the cut-off slope, Sp_L, is determined. The percentage of total pipe length in the basin with pipe slope smaller than or equal to Sp_L is therefore known. Combining the two parts in Figure 87 does not necessarily mean that the pipes over which PM deposits all have slopes flatter than or equal to Sp_L. This may be a reasonable working assumption and, if it is made, locations of depositing segments can be determined. This step can be accomplished by noting on a sewerage system map all pipe segments with slopes equal to or smaller than Sp_L. This procedure will quantify the sewer segments associated with the percentage PM of the total load.

Step F - Inspection

Once the segments have been identified through analysis of sewer slope maps, they should be physically inspected to ascertain existing deposition levels and whether mechanical or water jet cleaning is necessary. Manholes should be inspected for soundness to determine whether remedial repairs such as rebuilding the manhole table and/or replastering manhole walls are necessary.

Step G - Flushing Method

The final decision is the choice of the flushing method. Procedural guidelines useful in weighing the pro's and con's of different approaches are discussed in section 14.5.

14.4 Sewer Flushing Costs

Realistic sewer flushing program costing information from full-scale experience is non-existent. Preliminary "desk-top" cost estimates were prepared by FMC (12). Results from a recently completed combined sewer management study in Fitchburg, MA are presented here (5,6). These estimates are again only conjecture at this point in time. The sewer flushing alternative in that study entailed flushing daily 46 combined sewer small diameter

(10-18 inch) pipe segments. The flushing program was costed using manual tanker operation and automated modules similar to the device successfully operated on Shepton Street described in Chapter 11. Further testing of such devices is necessary before recommendation for general application can be made, because no full-scale network of automated sewer flushing modules have ever been tried in practice. Costs for this type of operation are nevertheless presented for comparison with a manual operation using water tankers.

Initial costs included $1500 per flushing segment for site preparation work, which entailed repairing and, in some cases, installing manhole tables, grouting the flush manhole walls, mechanical cleaning and jetting of the flush segment and other miscellaneous contingency items. Fabrication and installation of the devices was estimated at $7,500 each. A three-man maintenance crew was assumed to ensure proper operation of the configuration of the 46 modules. Equipment component replacement costs of $300 per module per year were also assumed. A contingency factor of $10,000 per year was included to cover possible blockages and malfunctions. The estimated annual budget for this operation is $80,000. The O&M cost per module per year is $1630. The present value of O&M cost per module, assuming a 20-year discount period at 8 percent interest, is $18,745. Total present worth cost per module is $27,745. These costs (ENR = 2,000) are summarized in Table 80.

Alternative program costs consisting of manually flushing 46 segments, using three water-tankers and a six-man labor force, are shown in the second half of Table 80. Total present worth costs for this alternative are $24,880 per segment.

One of the major recommendations of this report is to perform an additional study aimed at evaluating long-term operational performance of a combination of different automated flushing techniques. An ancillary aim of the envisioned study is to develop realistic operational costs associated with sewer flushing.

14.5 Sewer Flushing Techniques - A Discussion

The following question arises: if a sewer flushing program were to be instituted within a given community, what would be the best approach for the given situation? Data and experiences generated during this study provide a general format for reaching answers to such questions. The following subsection is aimed at presenting generalized concepts, as well as known pro's and con's, of various flushing approaches.

Basically, sewer flushing as tested in this study consists of inducement of turbulent flow shock waves along a given segment which will tend to resuspend previously deposited solids with a number of mechanisms. The flush wave itself may be pictured in a simplistic sense as a rolling, gyrating ball that tends to push materials on its front side and pull materials as it passes. The severity of the roll and bounce tends to dictate the fractions of materials from heavy sand to light organic particles that will be resuspended by the wave and carried along for some distance. As the turbulence and speed decrease, the materials resuspended or carried within the flush

TABLE 80. ESTIMATED COSTS OF SEWER FLUSHING METHODS

NUMBER OF SEGMENTS: 46	
DAILY FLUSHING PROGRAM	
ALTERNATIVE 1 - AUTOMATIC FLUSHING MODULE OPERATION	
CAPITAL COST:	
- Site Preparation (Grout Manhole, Fix Base, Clean Segment)	$1,500/Segment
- Fabricate & Install Air-Operated Module	7,500
	$9,000/Segment
ANNUAL OPERATIONAL COST (TOTAL PROGRAM):	
- 3 Men @ $15,000/Yr.	45,000
- Truck Rental, Gas, Insurance	8,000
- Equipment Component Replacement $300/Yr/Module	14,700
- Sewer Cleaning Contingency	10,000
- Water	2,300
	$80,000
COST/MODULE/YR = $1,630	
PRESENT WORTH/SEGMENT = $18,745	
TOTAL PRESENT WORTH/COST/MODULE:	
- First Cost	9,000
- O&M Cost	18,745
	$27,745
ALTERNATIVE 2 - MANUAL FLUSHING MODE	
CAPITAL COST:	
- 3 Outfitted Water Tankers @ $18,000 or $1100/Segment	$54,000
OPERATIONAL COST (TOTAL PROGRAM):	
- 6 Men @ $15,000/Yr.	90,000
- Insurance, Gas, Maintenance	3,000
- Water	2,000
	$95,000
COST/SEGMENT/YR. = $1938	
PRESENT WORTH/SEGMENT = $22,287	
TOTAL PRESENT WORTH COST/SEGMENT:	
- Site Preparation	1,500
- First Cost	1,100
- O&M	22,290
	$24,880
USE $25,000/SEGMENT (ENR = 2000)	

wave tend to become lighter with the heavier sand and gravel particles dropping out.

Two factors control the removals and carrying potential of induced flush waves. These are: the flush input rate and the flush volume. Rate may be roughly equated with wave turbulence, while volume roughly equates with long-term carrying capacity and momentum. Contrary to classical hydraulics theory, the energy dissipation of the flush wave is not rapid, maintaining high degrees of turbulence for at least 300-400 feet (92-122 m). The results of this study presented in Chapter 8 tended to indicate that removals were slightly more volume-dependent than energy or input rate dependent, but a combination of both proved best. This statement basically pertains to manual methods flushing, although if jetted modules were developed the statement would pertain to them as well. Large volume flushing via backup and release methods as tested show comparable removals to jetted flushes.

Selection of the optimum flushing approach or mixed approaches is dependent on many factors. The following is a compilation of the authors' thoughts and ideas as to the pro's and con's of various flushing approaches. Sewer flushing programs have to be tailored to suit given situations. The purpose of this section is to present thoughts and ideas based on considerable field experience.

1) Sewer flushing is a mechanism to minimize, to the extent practicably possible, deposited solids within combined sewer systems that are subject to resuspension and potential overflow during storm events.

2) Sewer flushing is not meant to be a complete replacement for ongoing cleaning programs, in that no flushing method is truly effective with respect to grit removal. It has been the authors' experience that once cleaned, sewer flushing will maintain and sometimes further clean combined sewer laterals over extended periods of time. Flushing frequencies on the average of once every 3-4 days seem sufficient for previously highly depositing problem segments. From a pollution standpoint, it would seem that flushing more frequently would be more desirable, since the return period of light-moderate storms may be less than 3 to 4 days. Some preliminary evidence showing that sanitary organic deposits are not readily suspended and transported during moderate rain events is presented in Chapter 11. This evidence suggests that the intervals between flushing could be longer and not necessarily determined solely by rainfall considerations. More evidence is needed to substantiate this observation.

3) Sewer flushing can and should be incorporated as part of ongoing sewer maintenance programs to minimize additional cost, while maximizing the maintenance program effectiveness.

4) Sewer flushing can be effectively accomplished by manual or automated means. Manual methods utilizing a pressurized water tank truck system have been demonstrated to be viable and readily accomplished. On the other hand, flushing by automated devices is in its infancy. This study represents, to the authors' knowledge, the first real field application of an automated mechanical flushing device other than automated siphon manholes (39).

5) The choice of flushing by manual or automated means (assuming automated systems were perfected) poses the issue of capital vs labor intensive programs. Present funding mechanisms heavily favor capital investments, although the situation might change.

6) Widespread application of automated sewer flushing techniques is at this point unrealistic. Much development needs to be done, sensing units and mechanisms perfected and tested in an extensive program before solely automated flushing can be considered viable.

7) Automated flushing has several potential advantages: increased flushing frequency, possibility for centralized control and considerable control flexibility.

8) Flushing by automated means could have several significant disadvantages, including: potential for sticks and rags blocking devices infrequently maintained, lesser degree of "hands on maintenance" and observation of the sewer system, and complex repair problems.

9) Manual or automated flushing could readily be accomplished utilizing sewage, natural waters or even industrial wastewaters, if suitable, in water-scarce areas. Tankers could potentially be filled with strained sewage.

10) In areas where travel times are short, night flushing may be a good way of load balancing the wastewater treatment plant. Flushing sewers at night would be beneficial only if the time of travel in the collection system were short enough to move the late evening deposits all the way to the treatment facility before the next peak. This approach method would be appropriate if the branching order of the collection system were low, as occurs in most small river front towns, but not appropriate if the collection system were broad, fan-shaped with high branching order (and large areal depth). In general, since the tractive shear stresses are greater during higher flow conditions, it seems more prudent to flush upstream laterals mid-day or peak flow diurnal hours. The idea is that the suspended deposits would pass through uninterrupted zones of high tractive shear stress maximizing the flush travel.

11) Flushing of long, flat segments 1000 feet (305 m) or greater could require additional downstream or booster flushes to keep substantial deposits from accumulating if the downstream segments do not carry enough background sewage to provide solids carrying capacity. Intermittent upstream storage modules may also be considered prior to locations where solids redeposition occurs. At this time, not enough is known to define the exact location of such booster flushes.

12) Choice of proper flushing rate/volume for a given segment is somewhat dependent on the segment characteristics, that is, size, shape, slope and nature of sediments. The results of this study indicated that flush volumes between 35 and 50 cf (.99 - 1.98 cubic meters) at discharge rates in excees of 0.5 cfs (14.16 liters/sec) would be sufficient for most combined sewer laterials.

SECTION 15

ASSESSMENT OF TREATMENT COSTS WITH SEWER FLUSHING

15.1 Foreword

In this chapter two separate desk-top analysis dealing with the economic implications of sewer flushing programs on wastewater management are presented. The first analysis deals with estimation of the additional dry weather sewage treatment operational and maintenance costs resulting from increased pollutant loadings reaching the treatment plant during dry weather as a result of sewer flushing. Annual cost increases are computed for three different types of commonly encountered treatment flow sheets under seven different levels of design flow. Types of sewage treatment plants considered in this analysis include: primary, trickling filter and conventional activated sludge. Design discharge plant sizes considered in this analysis are 1, 5, 10, 25, 75 and 100 mgd. The second analysis deals with the implication of sewer flushing in regards to wet weather storage/treatment costs.

15.2 Dry Weather Analysis

Operational costs of wastewater treatment depend in part, on the amount of pollutants requiring removal. Sewer flushing programs on any significant scale would increase on the average, sewage strength beyond levels most often used in design. For example, additional suspended solids loadings would increase both the cost of wastewater treatment and to a greater degree, the cost of wastewater solids treatment. The objective of this analysis is to roughly compute the rise in annual operational costs resulting from differing yet reasonable sewer flushing program levels for hypothetical communities of different sizes.

To establish these additional costs, a range of treatment plant types will be first sized at different design flows, and the operation and maintenance cost of their unit operations found. Cost models will then be developed in the form of cost per unit loading versus design flow for each unit operation. Loading parameters include the pollutant that has the greatest impact on the cost of the operation.

If the amount of pollutant generated by the flushing is known, then the extra treatment cost could be estimated with the cost models in this form. The amount of pollutants flushed from the sewerage system tributary to a particular size plant will be crudely estimated using results of the sewer flushing experiments given in Chapter 8 and various statistical relationships between demographic and sewershed characteristics.

Total pipe length is estimated in the following way. First, population is estimated from an empirical relationship between plant wastewater flow and per capita waste flow rates (40). Next, population density is computed from an empirical relationship between population and population density (41). Finally, total pipe footage is computed from an empirical relationship between population density and pipe footage (42). Thus a particular treatment plant size is related to an estimate of total pipe footage of the hypothetical sewerage system tributary to the treatment plant.

Experience in assessing sewer flushing potential for several combined sewered communities in the Boston area indicates that most dry weather deposits of any significance lie roughly within 10 to 20 % of the total collection system pipe footage (4, 6, 13). In this analysis two hypothetical sewer flushing programs are considered that would flush 100 percent of the estimated loadings assumed to deposit in the range of collection system pipe footage cited above. It is further assumed that the amount of flushed solids and BOD loadings reaching the treatment plant would be 25 to 50 % of the estimated flushed loadings. This assumption derives from the conclusions of the settleability analysis of flush waves presented in Chapter 10. It is also assumed that flushing would be conducted by either automated or manual means using sewage as the flushing source. No additional hydraulic loading to the treatment plant is considered.

Additional pollutant loadings resulting from the two hypothetical flushing operations are crudely estimated using 10 and 20 % of the total estimated pipe footage and the average pollutant removals in pounds per day per foot of pipe, scaled by the aforementioned transport efficiency factors. It is assumed that the deposition loadings and flushing removals characteristics of the four test segments in Dorchester would be typical of all pipe segments where flushing is assumed in this analysis.

The estimated flushed pollutants are then used in the cost models to determine additional operation and maintenance costs. These costs are modified by utilization factors to account for the increased pollutant loadings since marginal treatment costs will change.

In this analysis two sets of cost models are used. One set of models will be used for estimating nominal operation and maintenance costs of unit operations that are used in primary, secondary, and advanced treatment. The other set of models will be for estimating the additional operation and maintenance costs incurred by flushing programs for primary and secondary plants. Flow sheets for the primary, trickling filter and conventional activated sludge plants considered in this analysis are shown in Figure 88.

15.2.1 Discussion of Pertinent Wastewater Treatment Plant Cost Models

Several empirical models were reviewed that present overall operation and maintenance costs for a given type of treatment plant (43, 44, 45, 46, 47). These models were developed from regression analyses of actual treatment plant audits. Models that estimate cost for specific unit process

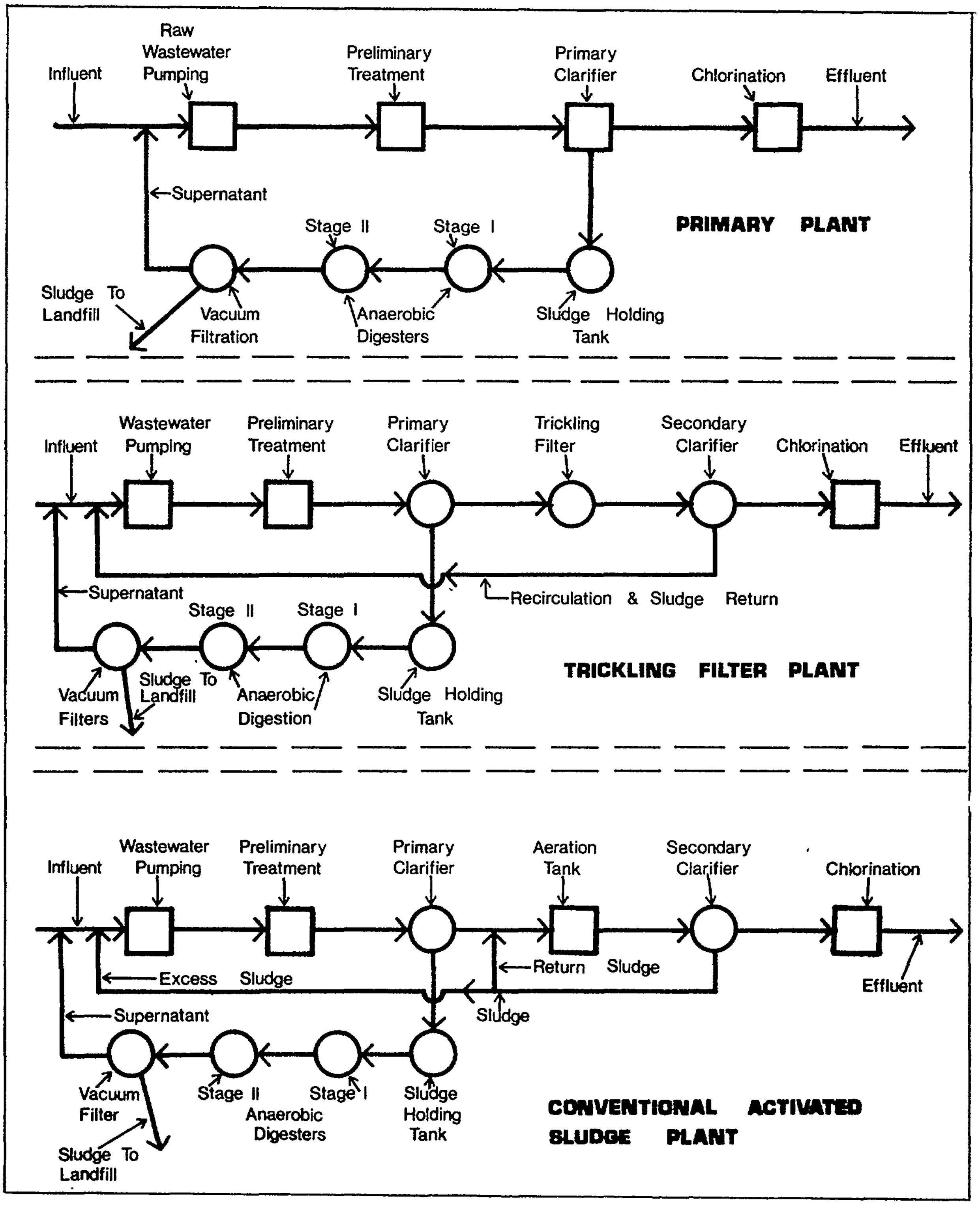

FIGURE 88: FLOWSHEETS OF PRIMARY TRICKLING FILTER AND CONVENTIONAL ACTIVATED SLUDGE TREATMENT PLANTS

are more useful, since the flushed loadings have different cost implications for the various unit operations in treatment plants.

The methodology developed by Van Note (48) costed individual processes for primary, secondary and tertiary treatment plants. The costs of treatment are divided into unit operation costs. Each unit operation is depicted on a unit operation sheet which consists of the process flow diagram the equipment and sub-unit operations making up the unit operation, the influent and effluent wastewater quality, and the design parameters used in the unit operation. Associated with each unit operation sheet are cost formulae for variable and fixed operation and maintenance costs.

W. L. Patterson and R. F. Banker (46) provide a more refined procedure in that the treatment plants are subdivided into their basic unit operations and costs are provided for each unit operation. In their work, operation and maintenance labor and material costs are presented in graphical form. Furthermore costs for unit processes were plotted against design parameters. For example, sedimentation cost was plotted against surface area, and vacuum filtration cost was plotted against dry solids filtered. The costing methodology used in this study is the following. Suitable design criteria were used to size the individual unit operations for a given treatment flowsheet, so that the required independent variables were obtained for the cost models. The cost models in graphical form were then used to find the annual labor manhours and the material and supply costs. Appropriate estimates of manpower cost factors were then used to determine annual labor costs.

15.2.2 Development of Unit Operation - Operational Cost Models

Primary, Trickling Filter and Conventional Activated Sludge Plants. Utilization of the operational cost methodology developed by Patterson and Banker requires that pertinent design variables be specified for each unit process. Several commonly used design criteria (50, 51, 24, 52) were used to estimate these unit process design variables for each plant type and for each design flow. These variables are as follows:

Operation	Variable
Raw waste pumping	average wastewater flowrate pumped (mgd)
Preliminary treatment	average plant flow (mgd)
Sedimentation	surface area of clarifier (ft^2)
Trickling filter	surface area of filter (ft^2)
Aeration-diffused system	blower capacity (cfm)
Recirculation or intermediate pumping	pumping capacity (mgd)
Chlorination	chlorine used (ton/year)
Primary sludge pumping	pumping capacity (gpm)
Sludge holding tank	sludge volume (cf)

Sludge digestion	sludge volume (cf)
Vacuum filtration-sludge to landfill	dry solids filtered (ton/year)

Various assumptions used in the preliminary treatment plant designs are cited in Table 81.

Estimates of sewage strength as a function of design flow were derived from several empirical relationships given by Michels et al. (40). These results are shown in graphical form in Figure 89 and are used in this analysis. The BOD and TSS curves were sufficiently close so that one curve was used for both parameters. These results were used in the preliminary design analysis for establishing sewage strength as a function of design flow.

Design variables for each unit process were then used in Patterson's and Banker's procedure to obtain annual payroll manhours and material and supply costs. The earnings for nonsupervisory workers (53) in December 1976 was $190.00 per week or $4.75 per hour based on a forty-hour week. This figure was increased to $5.00 per hour as some of the workers are supervisory. Indirect labor cost was taken as fifteen percent of the direct labor cost. Material and supply costs cited in Patterson's and Banker's work for January, 1971, were adjusted using a trend factor to December 1976 level. The trend factor used is the Wholesale Price Index (54) for industrial commodities for December 1976 of 187.4 referenced to the January, 1971 index of 112.2.

Operational cost estimates for each unit process for all plant types and scale, together with estimates of intra-process applied pollutant loadings derived from the preliminary plant designs, were used to develop a new set of operational cost curves directly relating cost to applied loadings for different design flows. The various unit operation cost functions and associated cost parameters developed in this analysis are summarized in Table 82 for each of the three treatment plant types. In sum, twenty-five operation and maintenance cost models were prepared. The sludge pumping cost functional was later modified to reflect unit cost per pounds of sludge settled.

Factors used to compute the applied loadings onto each unit process from the plant influent solids loadings and BOD loadings are given in fractional form in Table 83.

15.2.3 Operational Unit Operation - Cost Model Modifications

The operational cost models described in section 15.2.3 are in the form of cost per applied loading versus average design flow. The additional dry weather operation and maintenance cost resulting from sewer flushing can be computed using these procedures. The cost models were however, modified to reflect the fact that additional pollutant loadings would change plant utilization and hence marginal unit operation treatment costs.

TABLE 81. ASSUMPTIONS USED IN PRELIMINARY DESIGN OF PRIMARY, TRICKLING FILTER AND ACTIVATED SLUDGE PLANTS

A. Primary Plants	
. Sedimentation overflow rate =	700 gpd/ft^2
. Peak flow factor =	2.6
. Suspended solids removal by sedimentation =	55%
. Primary sludge solids concentration =	5%
. Sludge pumping =	4 hours/day
. Sludge holding tank detention =	5 days
. Chlorination dose =	10 mg/l
. Two stage digestion detention =	14 days/25 days
. Mean solids concentration in secondary digestor =	8%
. % Volatile to digestors =	70%
. Reduction volatile in digestors	55%
B. Trickling Filter Plant	
. Recirculation ratio =	.75
. BOD removal in primary sedimentation =	35%
. Applied organic load to filter =	45 lb BOD/1000 cf/day
. Depth of filter =	6 feet
. Sludge generation (% plant influent solids) =	85%
. Raw solids sludge concentration =	4.5%
. Chorination dose =	8 mg/l
. All other parameters same as primary plants	
C. Conventional Activated Sludge Plant	
. Applied BOD loading aeration tank =	35 lb/1000 cf/day
. Mixed liquid suspended solids =	2500 mg/l
. Aeration tank detention =	5 hours
. Sludge age =	8 days
. Return sludge concentration =	10,000 mg/l
. Air requirement =	1500 cf/ lb BOD
. Sludge generation (% plant influent solids) =	90%
. Chlorination dose =	5 mg/l
. All other parameters same as primary plants	

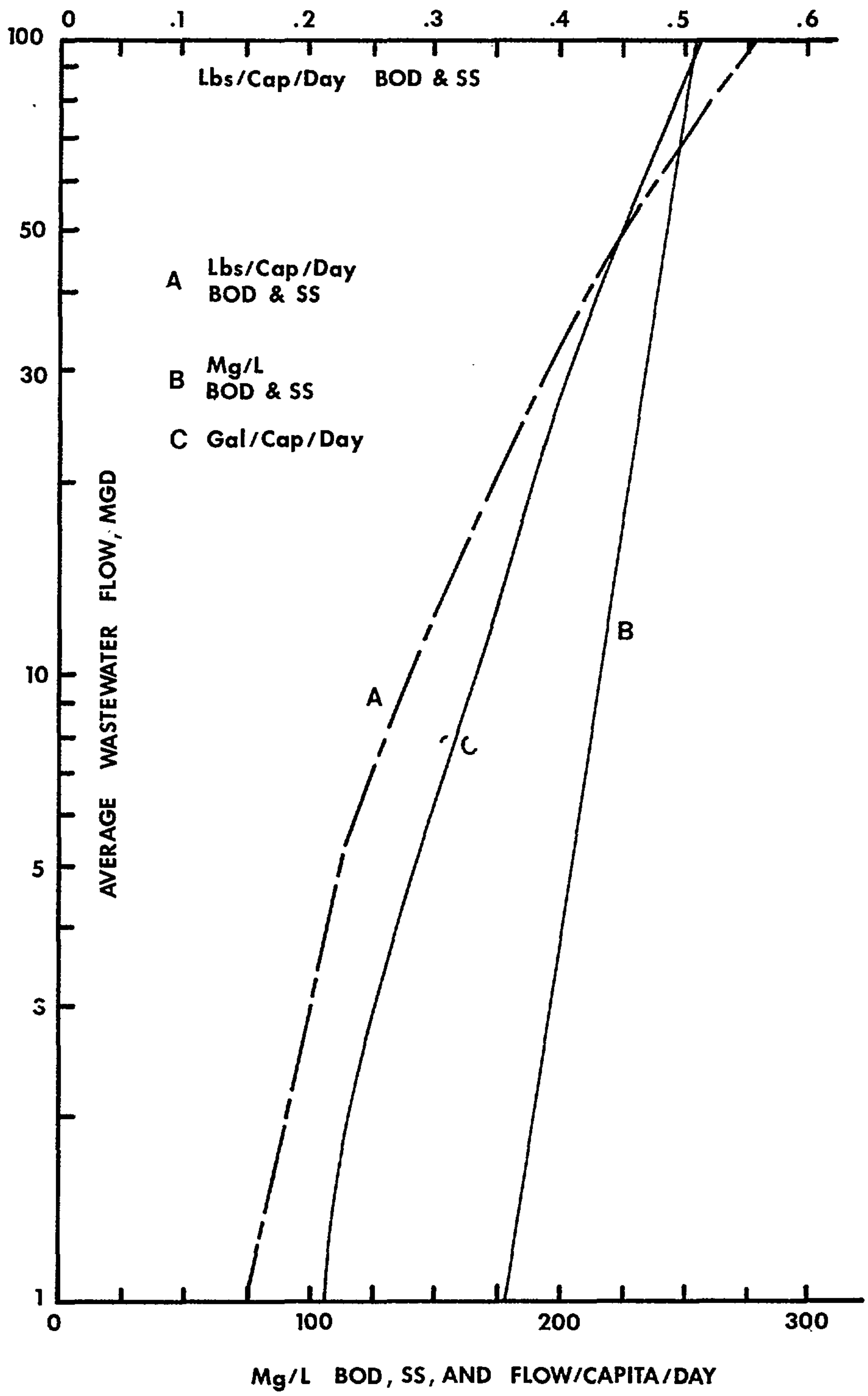

FIGURE 89. SEWAGE STRENGTH VERSUS DESIGN FLOW

TABLE 82. PARAMETERS FOR OPERATION AND MAINTENANCE COST MODELS PRIMARY, TRICKLING FILTER AND CONVENTIONAL ACTIVATED SLUDGE PLANTS

OPERATION	COST PARAMETER*	ALL PLANTS	PRIMARY	TRICKLING FILTER	ACTIVATED SLUDGE
Wastewater Pumping	Cents/1000 gallons	X			
Primary Sludge Pumping	Cents/1000 gallons		X	X	X
Recirculation Pumping	Cents/1000 gallons			X	X
Sedimentation	Cents/lb sludge settled		X	X	X
Sludge Digestion	Cents/lb sludge applied to digestion		X	X	X
Vacuum Filtration	Cents/lb sludge filtered		X	X	X
Sludge Holding Tank	Cents/lb sludge applied to tank		X	X	X
Trickling Filter	Cents/lb BOD applied to filter			X	
Activated Sludge Aeration Aeration	Cents/lb BOD applied to aeration				X
Chlorination	Cents/1000 gallons	X			
Yardwork	Cents/1000 gallons	X			
Laboratory	Cents/1000 gallons		X	**	X
Administration and General	Cents/1000 gallons	X			

*Cost model in form of cost parameter versus average design flow (mgd).

**Laboratory cost function for trickling filter same as for Primary Plant.

TABLE 82. PARAMETERS FOR OPERATION AND MAINTENANCE COST MODELS
PRIMARY, TRICKLING FILTER AND CONVENTIONAL ACTIVATED SLUDGE PLANTS

OPERATION	COST PARAMETER*	ALL PLANTS	PRIMARY	TRICKLING FILTER	ACTIVATED SLUDGE
Wastewater Pumping	Cents/1000 gallons	X			
Primary Sludge Pumping	Cents/1000 gallons		X	X	X
Recirculation Pumping	Cents/1000 gallons			X	X
Sedimentation	Cents/lb sludge settled		X	X	X
Sludge Digestion	Cents/lb sludge applied to digestion		X	X	X
Vacuum Filtration	Cents/lb sludge filtered		X	X	X
Sludge Holding Tank	Cents/lb sludge applied to tank		X	X	X
Trickling Filter	Cents/lb BOD applied to filter			X	
Activated Sludge Aeration Aeration	Cents/lb BOD applied to aeration				X
Chlorination	Cents/1000 gallons	X			
Yardwork	Cents/1000 gallons	X			
Laboratory	Cents/1000 gallons		X	**	X
Administration and General	Cents/1000 gallons	X			

*Cost model in form of cost parameter versus average design flow (mgd).

**Laboratory cost function for trickling filter same as for Primary Plant.

TABLE 83. FACTORS USED TO COMPUTE APPLIED LOADINGS PER UNIT PROCESS

	Influent Suspended Solids Loadings			Influent BOD Loadings		
	Plant Type			Plant Type		
	A	B	C	A	B	C
Preliminary Treatment	1.	1.	1.	-	-	-
Sedimentation	.55	.85	.9	-	-	-
Trickling Filter Bed	-	-	-	-	.65	-
Activated Sludge Aeration	-	-	-	-	.65	-
Sludge Pumping	.55	.85	.90	-	-	-
Sludge Holding Tank	.55	.85	.90	-	-	-
Anaerobic Digesters	.55	.85	.90	-	-	-
Vacuum Filters	.34	.49	.55	-	-	-

A - Primary Treatment Plant
B - Trickling Filter Plant
C - Conventional Activated Sludge Plant

The results of an analysis (55) depicting the variation operation and maintenance cost per unit treated for discharge levels other than design flows for 2.5 and 10.0 mgd primary, trickling filter and activated sludge plants were used to prepare Figure 90 . The curve shows how the utilization factor changes with plant utilization. The utilization factor is the cost per unit treated at a particular utilization divided by the cost per unit treated at 100% utilization. It was assumed that this formulation would be applicable for approximating marginal cost changes under conditions of only increased pollutant loadings since the sewer flushing programs would not increase hydraulic loadings. Utilization factors derived from this curve were later used in estimating the additional flushing costs to modify the operation and maintenance cost model estimates which were prepared for 100% utilization.

15.2.4 Estimation of Additional Treatment Costs

Total collection system footage tributary to each of the seven different sized treatment plants were estimated in the following way. First, Figure 89 was used to relate design flow to per capita waste rate for an estimated population. Population density for communities served by combined sewers was computed using the following equation 41 :

$$\text{Population Density (Capita/Acre)} = 0.3\ (\text{Population})^{.304}$$

Footage of installed sewers per capita (42) was computed from population density using the following equation:

$$\text{Feet of Installed Sewer/Capita} = 54\ (\text{Capita/Acre})^{-0.65}$$

Additional treatment plant pollutant loadings resulting from the sewer flushing programs were computed in the following manner. First, an estimate of the daily flushed load per foot of sewer was derived by averaging the pollutant removal results of the four test segments in the

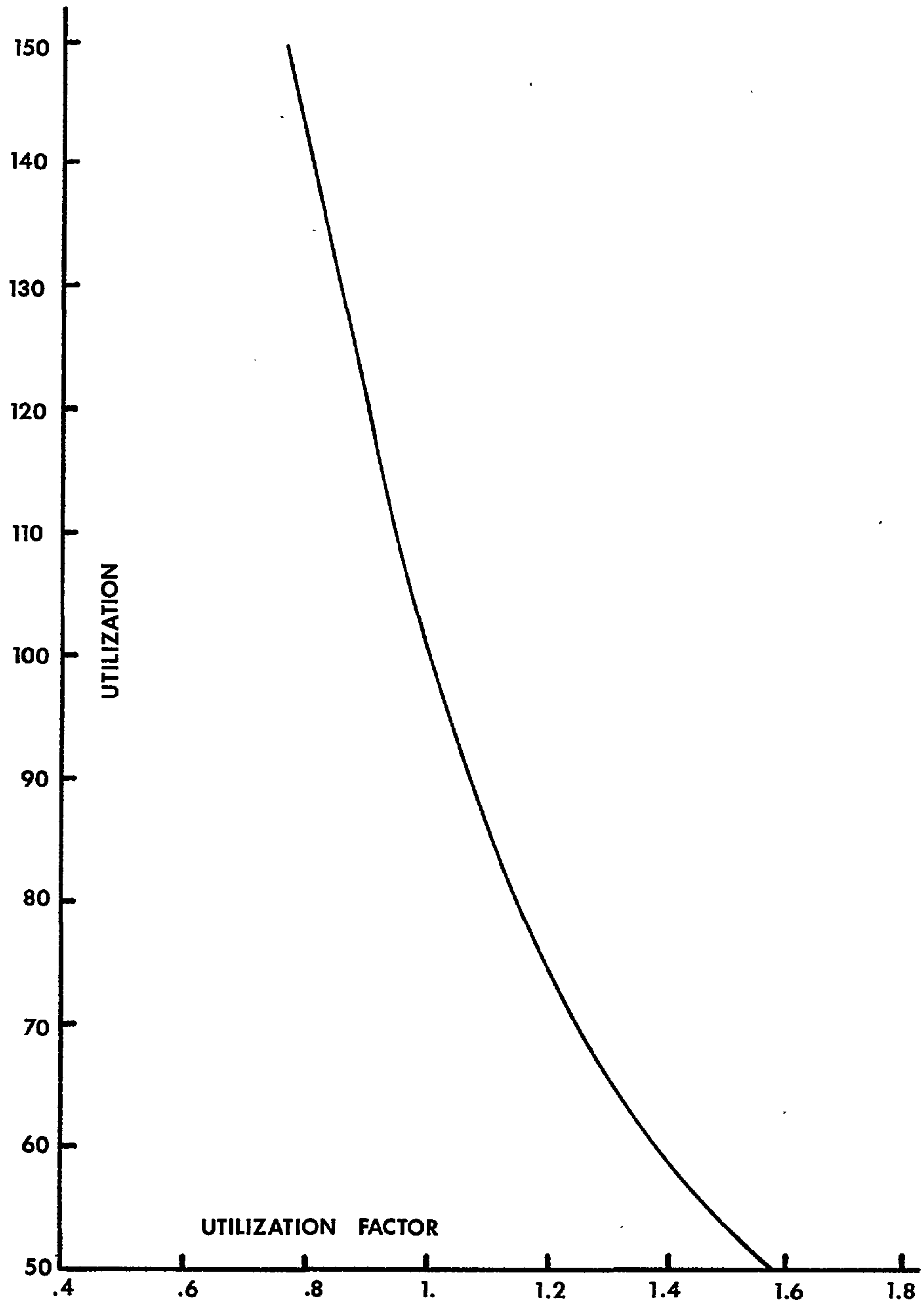

FIGURE 90. TREATMENT PLANT UTILIZATION VERSUS UTILIZATION FACTOR

first phase. The average normalized pollutant mass removals in pounds per day for each street were divided by test segment footages. These mean results per street were then averaged to yield the factors used in this analysis. The overall average flushed TSS, VSS and BOD loadings used in this analysis are 0.0205, 0.0143 and .0084 lbs/foot of sewer flushed/day. These factors were then adjusted by a factor of 22% for suspended solids and 45% for BOD to reflect the findings of the settleability experiments.* These factors were then multiplied by 10 and 20% of the total estimated collection system footage tributary to each of the seven different size treatment plants.

Additional operation and maintenance costs were computed for each unit operation within each of the three treatment plants for the seven design flow conditions using the estimated dry weather loadings resulting from the hypothetical sewer flushing programs. The unit process operation and maintenance cost model estimates described in section 15.2.3 were modified to reflect increased plant loading utilization using the utilization versus utilization factor curve given in Figure 90. Nominal plant utilization for pre-flushing conditions was assumed at 100%.

Summary cost results for the three treatment plants types are given in Table 84. The estimated additional annual operation and maintenance cost resulting from the increased pollutant loadings from sewer flushing programs for the seven design flow conditions are presented along with the relative increase in the total estimated annual yearly plant operation and maintenance cost. The total annual yearly cost included wastewater pumping, chlorination yardwork, administration and laboratory operational costs besides the operational costs for those particular unit processes affected by the increased solids and organic sewer flushing loadings. The results show that the two hypothetical sewer flushing programs would have neglible impact on the operational cost of dry weather sewage treatment. The cost increases range from 3 to 6% for the 1 mgd plant and decrease to around 1% for larger sized plants.

The distribution of the additional operational costs for the various unit operations relative to the total additional increase revealed that the vacuum filtration units for the primary and trickling filter plants exhibited the highest overall increase, averaging about 42%. This is not surprising since this process has a high chemical and power cost. The largest relative cost increase in the conventional activated sludge plant occurred for the aeration tank which ranged from 25 to 35 % depending on the plant size. The vacuum filtration operational cost increase for the activated sludge plant about equalled the aeration tank cost increase.

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* These estimates were crudely taken as the percent TSS and BOD remaining in suspension 800 feet downstream from a flushing manhole. See Figure 70, Chapter 12.

TABLE 84. SUMMARY OF ADDITIONAL TREATMENT OPERATION AND MAINTENANCE COSTS FOR PRIMARY, TRICKLING FILTER AND ACTIVATED SLUDGE PLANTS

DESIGN FLOW (mgd)	PRIMARY TREATMENT PLANT A	PRIMARY TREATMENT PLANT B	TRICKLING FILTER PLANT A	TRICKLING FILTER PLANT B	CONVENTIONAL ACTIVATED SLUDGE PLANT A	CONVENTIONAL ACTIVATED SLUDGE PLANT B
A. Flushing Program (10% Segments)						
1	1820	3.4	2410	3.3	3020	3.0
5	2060	1.4	2970	1.5	4070	1.7
10	2375	1.0	3420	1.1	4925	1.3
25	3090	0.7	4480	0.8	6260	0.9
50	3950	0.5	5810	0.6	8300	0.7
75	4700	0.5	6400	0.5	9100	0.6
100	5090	0.4	-	-	10600	0.5
B. Flushing Program (20% Segments)						
1	3350	6.2	4460	6.1	5620	5.7
5	3990	2.8	5760	3.0	7910	3.3
10	4380	1.9	6640	2.2	9550	2.5
25	6060	1.4	8800	1.5	12260	1.7
50	7750	1.0	11390	1.1	16315	1.3
75	9300	1.0	12660	1.0	18470	1.1
100	10070	0.8	-	-	21020	1.0

A - Additional Annual Operation & Maintenance Treatment Costs Resulting From Flushing Programs (\$/year)

B - Percentage of Increase Relative to Normal Total Operation and Maintenance Cost

15.3 Wet Weather Considerations

The results of the sewer flushing R&D program conducted over the last two years clearly showed that flushing combined sewer laterals definitely removed and transported on a remarkedly consistent basis pollution related contaminants from both single pipe segments and from serial segments. The program did not, however, document the long-term mass pollutant removal reduction from a reasonably sized collection system. It can be inferred, however, that if pollutant removals over, say, a 1000 feet stretch of lateral were documented (which they were), and if these laterals discharged into trunk sewers with adequate shear stress transport capacity, then in a hypothetical sense, removal of dry weather pollution deposition loadings on a larger scale could in fact be projected with reasonable certainty. Intense sedimentation can still occur in flat trunk sewers.

If, however, sewer flushing would only partially transport pollutants down through a system, that is, stopping short of the sewage treatment facilities, there is still a strong benefit occurring from even this limited effectiveness. Except for extreme storm events characterized by intense frontal waves, "first flushes" would have a higher likelihood of being transported either to the treatment headworks before overflows occur and/or to storage/treatment capture points. Storage designed to capture "first flushes" would invariably be more effectively utilized and, accordingly, the costs would be reduced.

SECTION 16

USER DETAILS

16.1 Foreword

This chapter contains examples of two procedures briefly described earlier in this report, represented with more explanatory detail.* The first procedure is the multi-segment dry weather sewage deposition model described in section 12.4. An example of the computations is presented using the Walnut Street collection subsystem to illustrate the computations. Details of this example are provided in section 16.2. The second procedure described in this chapter is the application of the empirical model described in section 12.2 for predicting the amount of pollutants remaining in suspension at downstream points resulting from an upstream flush. This procedure is described in section 16.3. This procedure is extremely useful in the estimation of flushed pollutant loads along a small subsystem. Its use on a broad spatial scale is not recommended.

16.2 Application of Multi-Segment Deposition Model

A description of the deposition model fundamentals and a short description of a hypothetical example illustrating some of the aspects of the methodology used in the model were presented in section 12.4. For this section, a more detailed example is presented with equations and computation of numerical values. In presenting this example the output of one of the runs made for the Walnut Street test segment is described and some of the numerical values shown in the output table are verified by manual computations using the appropriate equations, as described in section 12.4. It should be noted that the manually computed numbers will not always match exactly those shown in the table, due to differences in the number of significant digits used in the manual computations and those used by the IBM 370/168 which computed the table.

Figure 91 presents a schematic of the Walnut collection system. Table 85 is a sample output of the deposition model, computed for a per capita waste flow rate of 100 gpcd. The columns labeled A through R in Table 85 are sufficiently clear so that a description of each column is not necessary. The schematic and some of the labeled columns shown in the table will be used as needed in illustrating the series of computational steps described in section 12.4.3. The steps described below follow closely the steps presented in section 12.4.3.

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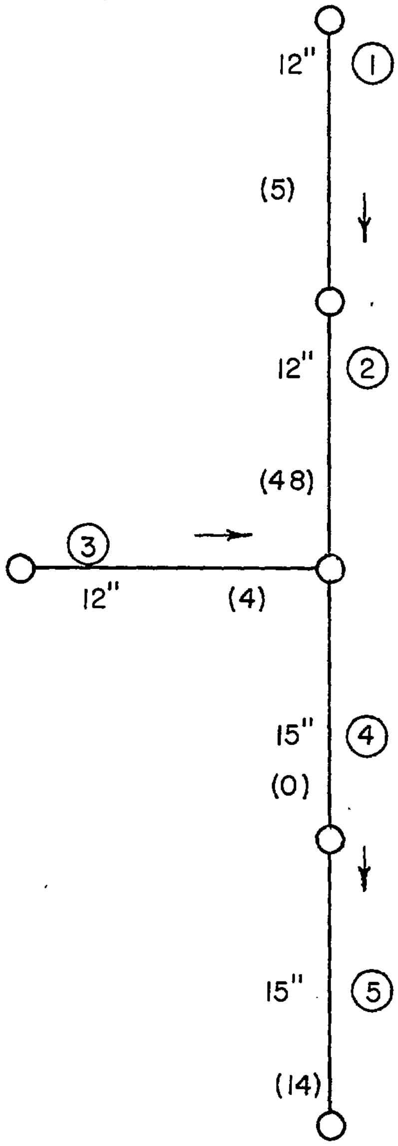
*Pertinent equations are repeated here for convenience.



LEGEND
1 LINK NUMBER
(5) POPULATION CONTRIBUTING TO LINK
12" PIPE SIZE (CIRCULAR)

FIGURE 91 SCHEMATIC OF COLLECTION SYSTEM - WALNUT STREET

TABLE 85. EXAMPLE OF APPLICATION OF DEPOSITION MODEL

PER CAPITA=100.

LOCATION : SEWER FLUSHING RESEARCH - DEPOSITION ANALYSIS : WALNUT STREET (COMBINED SEWER)

* DENOTES TRUNK LINE

A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
----	HYDRAULIC CHARACTERISTICS					----	----	FLOW ANALYSIS			----	----	DEPOSITION ANALYSIS				----
COMP. NO.	SITE	SLOPE	WIDTH (IN)	HEIGHT (IN)	LENGTH (FT)	TYPE	POP. SERVED	QFULL (CFS)	QAVE (CFS)	QMAX (CFS)	DEPTH (INCH)	SHEAR (PSF)	PERCENT DEPOSIT	LOAD/DAY *	SUM LOAD PER DAY	OVERALL FRAC.DEP	DEG DEP
* 1	WALNUT	0.0028	12.00	0.0	82.	CIRC	5.	1.64	0.001	0.001	0.3	0.003	40.00	1.00	1.00	0.400	HIGH
* 2	WALNUT	0.0027	12.00	0.0	259.	CIRC	53.	1.61	0.008	0.011	0.8	0.007	20.39	4.89	5.89	0.222	HIGH
3	SQUARE	0.0192	12.00	0.0	184.	CIRC	4.	4.29	0.001	0.001	0.2	0.010	12.99	0.26	0.26	0.130	MOD
* 4	WALNUT	0.0048	15.00	0.0	141.	CIRC	57.	3.90	0.009	0.012	0.7	0.011	12.32	0.0	6.15	0.216	MOD
* 5	WALNUT	0.0013	15.00	0.0	136.	CIRC	71.	2.05	0.011	0.015	1.0	0.004	35.63	6.60	12.76	0.359	HIGH

* lbs./day

1. The collection system was segmented into 5 links, as shown in Figure 91, each link corresponding to one actual pipe segment between two consecutive manholes;

2. A list of downstream links that convey wastes from each indicated link is as follows:

Link	Downstream Conveying Links
1	2, 4, 5
2	4, 5
3	4, 5
4	5
5	-

3. The cumulative population at the downstream end of each link is presented in column A of Table 85;

4. Using the cumulative populations of column H and the per capita waste flow rate of 100 gpcd, the average daily dry weather flows at the downstream end of each link were computed, expressed in cfs, and are presented in column J of Table 85. The numbers shown in column J were rounded off for printing purposes only. Considering, for example, link no. 5, the daily average flow in cfs can be estimated as:

$$71 \text{ persons} \times 100 \text{ gpcd} \times 0.1337 \text{ ft}^3/\text{gallon} \div 86400 \frac{\text{sec}}{\text{day}} = 0.011 \text{ cfs}$$

5. The maximum daily dry weather flows, Q_{MAX} at each link were computed using equation:

$$\frac{Q_{MAX}}{Q_{AV}} = aPP^{-b} \qquad (53)$$

The computed values are presented in column 11 of Table 85. The values of the coefficients a and b above were fixed at:

$a = 1.157$
$b = -0.08$

and PP is the cumulative population at the downstream end of the link, in thousands. The value of b was established in a previous study conducted in the Dorchester Bay area (4). The value of a was adjusted so that the peak coefficient at the downstream end of link 5 equals the average value of 1.43, determined from data at that point, and presented in Table 65, Chapter 12.

Again, for link no. 5, the peak daily flow computed by equation 53 is:

$Q_{MAX} = a * PP^{-b} * Q_{AV}$ or

$Q_{MAX} = 1.157 * (0.071)^{-0.08} * 0.011 = 0.0157$ cfs

6. The shear stress is computed by the equation:

$$\tau = \rho r s \qquad (54)$$

where: ρ = specific weight of water
r = hydraulic radius (r=flow area/wetted parameter)
s = energy slope

Equation 54 requires knowledge of the hydraulic radius corresponding to the maximum daily dry weather flow. An iterative procedure is used to compute the uniform flow depth corresponding to the maximum daily flow rate, given the pipe shape, size and slope. The flow depth is then used in computing the hydraulic radius, r.

The iterative procedure used in the model determines flow depths corresponding to the estimated maximum daily discharges for each link, which are presented in column L of Table 85. Using link no.5 once more as a numerical example, the depth of 1 inch of flow in a 15-inch diameter pipe implies a hydraulic radius of

$$r = 0.0538 \text{ ft.}$$

Therefore, the shear stress can be estimated by equation 54 as:

$$\tau = 62.4 \text{ lb/ft}^3 * 0.0538 \text{ ft} * 0.001335 \frac{\text{ft}}{\text{ft}} = 0.00448 \text{ lb/ft}^2$$

7. The deposition rates are computed as a function of the shear stress by equations 55 and 56, which are:

$$Z = 40 \left(\frac{\tau}{0.004}\right)^{-1.2} \quad \text{for} \quad \tau > 0.004 \text{ psf} \qquad (55)$$

$$Z = 40 \quad \text{for} \quad \tau \leq 0.004 \text{ psf} \qquad (56)$$

The suspended solids deposition rates estimated by the equations above are presented in column N of Table 85. In verifying the deposition rate for link no. 5, equation 55 is used, yielding:

$$Z = 40 * \left(\frac{0.00448}{0.004}\right)^{-1.2} = 34.9\%$$

8. The daily loads of suspended solids generated in each link are computed using the contributary population of each link and an estimated solids waste rate which is defined in section 12.4.2 as 0.5 lb/cap/day. These values are not printed in Table 85, and therefore are presented below.

Link No.	Population (Capita)	Solids Generated Daily (lb/day)*
1	5	2.5
2	48	24.0
3	4	2.0
4	0	0.0
5	14	7.0

*at 0.5 lb/cap/day

9. Starting at link no. 1 and using the estimated deposition rate of 40.00% shown in column N of Table 85, the deposition load in that link is:

$$0.4 * 2.5 \text{ lb/day} = 1.0 \text{ lb/day}$$

10. Searching the list of segments downstream from link no. 1, that is, links 2, 4 and 5 (refer to step no. 2 above), it can be observed from column N in Table 85 that none has a higher deposition rate than link 1. This implies that a fraction (40%) of the load generated in link no. 1 will deposit in that link, but none of the particles that reach the end of link 1 will settle in downstream segments since they all have shear stresses higher than that of link 1.

11. The search now moves to link 2. Searching the list of links downstream from link no. 2, namely links 4 and 5, it can be observed that link 5 has a higher deposition rate than that of link 2, respectively 35.63% against 20.39%. Therefore, 20.39% of the load generated in link 2 will deposit in that link, whereas 35.65%-20.39% = 14.24% of that same load will deposit in link 5. This logic can be explained by noting that each particle size is associated with a shear stress value, by equation:

$$\tau = 0.02p^{2/3} \qquad (57)$$

where τ = fluid shear stress, psf
p = particle diameter, mm

Use of equation 57 implies that all particles having sizes larger than or equal to p will settle at the corresponding shear stress τ, and smaller particles will be carried downstream in suspension. In this manner, the downstream segment with the lowest shear stress (conversely with the highest deposition rate) defines the smallest particle size generated in an upstream link which will deposit in the entire system. In this case, link 5, with a shear stress of 0.004 psf, defines the smallest particle size, p, of the load generated in link 2, which will deposit in the entire system. Given an assumed distribution for the particle sizes in the sewage, represented indirectly by equations 55 and 56, it was estimated that the particles that deposit under the shear stress of 0.00448 psf at link 5 account for 35.63% of the total mass generated in link 2. A portion of the coarser particles generated in link 2 will settle in that link, which has a shear stress of 0.007 psf. These coarser particle sizes that deposit in link 2, which account for 20.39% of the total load generated in that link, will not be available for deposition in link no. 5. Therefore link 5 can only deposit the difference of 35.63%-20.39% = 15.24%, of the total mass generated in link 2.

12. Following a similar approach for all downstream links, the results can be summarized as follows:

FRACTION OF LOADS GENERATED IN THE LINKS NUMBERED HORIZONTALLY THAT WILL DEPOSIT IN THE LINKS NUMBERED VERTICALLY

DEPOSITION LINKS	Links Originating Loads					Total load per link* (lb/day)
	1	2	3	4	5	
	Loads generated by Link (lb/day)					
	2.5	24.0	2.0	0.0	7.0	
1	0.4					1.0
2	0.0	0.2039				4.89
3	0.0	0.0	0.1299			0.26
4	0.0	0.0	0.0	0.1232		0.0
5	0.0	0.1524	0.2264	0.2331	0.3563	6.6

* Computed as the horizontal summation of the products of the fractions shown in the table times the daily deposition loads appearing at the top of the table. For link 5, for example, the total load is:

$$0.1524*24.0 + 0.2264*2.0 + 0.2331*0.0 + 0.3563*7.0 = 6.6 \text{ lb/day}$$

16.3 Downstream Transport of Solids in Suspension

This section is intended to provide an example utilizing the methodology described in section 12.2. For simplicity, the verification case presented in section 12.2.6 for the second phase flushes at Port Norfolk Street is repeated in greater detail.

Assume that the estimated mean deposits from the second phase flushing program along the three successive pipe segments in Port Norfolk Street are known to be 4.22 kg, 6.09 kg and 7.40 kg (a schematic of the three pipe segments is presented in Figure 92). This procedure provides a rough estimate of the loads flushed out of the last pipe segment if an average flush (in volume and in injection rate) is injected into the most upstream manhole.

Using the values of coefficients a and b of equation 24, which were described in section 12.2.2, equation 24 can be written as

$$V = 0.5\, e^{0.789}(1+e^{-0.000634L}) \qquad (58)$$

where: V = average velocity of the flush wave over the distance L, fps;

L = downstream distance from the flush injection point, ft; and

e = 2.71828.

The estimation of the masses from the first pipe segment that remain in suspension at the end of the first, second and third segments, that is, at downstream distances of 162, 409 and 659 ft, respectively, is accomplished by the following steps:

1. Solve equation 58 for values of L equal to 162 ft, 409 ft and 659 ft, thus determining three values of average wave velocity, V;

2. Use some iterative procedure associated with Manning's equation, or any other dynamic equation, to determine the three water depths, h, associated with the wave velocities above;

3. With these three sets of values of V, h and L, solve equation 59 :

$$V_{sett} = \frac{h.V}{L} \qquad (59)$$

   determining three values of settling velocities which will define the particles that remain in suspension after the first, second and third downstream manholes;

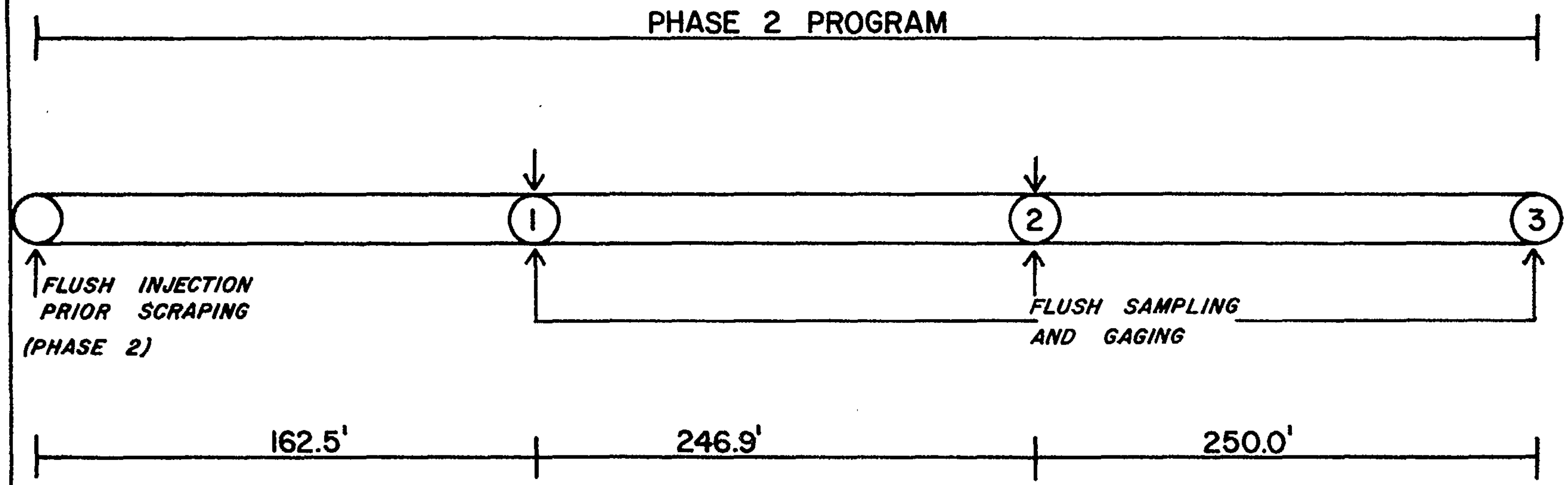


FIGURE 92 SCHEMATIC OF SECOND PHASE FIELD OPERATIONS

4. Enter the values of the settling velocities determined above in the "Conservative Turbulent Settling Curve" shown in Figure 69 of Chapter 12, or use equation 60 :

$$P = 424.\times 10^5\, V^3_{sett} - 960.\times 10^3\, V^2_{sett} + 123.\times 10^2 V_{sett} + 10.8 \qquad (60)$$

where P is percentage of mass remaining in suspension.

Then determine the percentages of material suspended from the first pipe segment that will remain in suspension at the end of the first, second and third segments. These values were computed as being respectively 67.5%, 34.2% and 24.2% and are presented on the first line of item number 2 in Table 86.

TABLE 86. COMPARISON OF MEASURED VERSUS ESTIMATED AVERAGE TSS MASS REMOVALS - PORT NORFOLK ST. (SECOND PHASE)

1. Estimated mass deposited (kg)	4.22	6.09	7.40
2. % remaining in suspension from:			
Segment 1	67.5	34.2	24.2
Segment 2	-	46.8	27.9
Segment 3	-	-	41.2
3. Estimated mass flushed out (kg)	2.85	4.29	5.77*
4. Measured mass flushed out (kg)	2.85	4.03	7.39

*0.242 x 4.22 + 0.279 x 6.09 + 0.412 x 7.40 = 5.77.

The percent removals from the successive pipe segments are computed in a similar manner except for the fact that the settling velocity values given by equation 59 corresponding to the distance L from the flush injection, should be corrected for the length down to the new pipe segment being considered. In this case equation 31 , Chapter 12 applies and is given by:

$$V_{Sett.} = \frac{h \cdot V}{L-L_o} \qquad (61)$$

where

L_o = 0 ft. for materials removed from the first segment;

L_o = 162 ft. for materials removed from the second segment; and

L_o = 409 ft. for materials removed from the third segment.

It should be noted that discretization by pipe segment as described above is crude. A finer discretization, that is using smaller pipe sections would be more desirable.

The percentages of the loads from the second pipe segment still in suspension at the end of the second and third pipe segments were estimated as 46.8% and 27.9%. The percentage of the loads from the third pipe segment still in suspension at the end of the third segment was estimated to be 41.2%. All those percentages are shown under item 2 of Table 86. The masses of solids transported out of the first, second and third pipe segments can now be estimated as:

First segment (162 ft.): 0.675 x 4.22 = 2.85 kg
Second segment (409 ft.): 0.342 x 4.22 + 0.468 x 6.09 = 4.29 kg
Third segment (659 ft.): 0.242 x 4.22 + 0.279 x 6.09 + 0.412 x 7.40 = 5.77 kg

These values compare reasonably well with the average measured values shown as item 4 in Table 86. The first value, 2.85 kg was modified by adjusting the mass composited in the first pipe segment, by 12.5% as was described in Section 12.2.6.

The procedure is summarized as follows: (1) computation of the average velocity V; (2) solution of Manning's equation for h; (3) solution of equation 59 for V_{Sett}; and (4) solution of equation 60 for P. A more expedited solution can be obtained by using the curves of Figure 69, or, in the case of TSS, directly solving equation 30 , section 12.2.2, which is:

$$P(\%) = -1.33\ (\ln L)^3 + 33.86\ (\ln L)^2 - 288.57\ (\ln L) + 834.32 \qquad (62)$$

To obtain the percentages of solids from the first pipe segment that are carried beyond the end of the first, second and third segments, equation 62 is solved for L equal to 162 ft., 409 ft. and 659 ft., respectively, obtaining values of 67.5%, 34.2% and 24.2%, as before. To obtain the percentage of solids from the second pipe segment leaving in suspension the end of the second and third pipe segments, equation 62 is solved for L equal to 247 ft. and 497 ft, yielding 49.84% and 29.64%, respectively. Finally, equation 62 is solved for L = 250 ft to obtain the percentage of mass from the third pipe segment remaining in suspension at the end of the third pipe segment. This estimate is 49.40%. With these new values, the masses of solids leaving the end of the first, second and third pipe segments are evaluated as:

First segment (162 ft): 0.675 x 4.22 = 2.85 kg
Second segment (409 ft): 0.342 x 4.22 + 0.498 x 6.09 = 4.48 kg
Third segment (659 ft): 0.242 x 4.22 + 0.296 x 6.09 + 0.494 x 7.40 = 6.50 kg

It was pointed out in Section 12.2.6 that the use of the curves in Figure 69, or equation 62 for TSS provides the same results for the first segment as the 4 step procedure described before, nonetheless it overestimates the percentages of materials removed from the second and third pipe segments. Such application is equivalent to assuming that the upstream end of these downstream pipe segments is at the point of the flush injection.

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16. ABSTRACT This report summarizes the results of a two year study aimed at addressing the feasibility, cost-effectiveness and ease of application of upstream solids control as an integral part of overall combined sewer management. The project was functionally divided into four major phases. In the first phase four test segments on different streets in the Boston sewerage system were field flushed over an extended period to quantify the effectiveness of flushing deposition accumulation from a single pipe segment, and to estimate deposition characteristics within collection system laterals. The second phase was concerned with the problem of flushing a long flat stretch of combined sewer laterals. Flushes were injected into the uppermost manhole and pollutant levels in the flush wave passing three downstream manholes were monitored, providing insights into flushing effectiveness over three segments of pipe. Settleability tests were also performed for the purpose of estimating how far beyond the flushing monitoring manholes would the materials be carried. In the third phase, an automatic sewer flushing module was designed, fabricated, installed and operated on a single segment for an extended period. The purpose of this work was to begin to develop operational experience, using automated flushing equipment. In the fourth phase, various predictive deposition loading and flushing criteria were generated from the large data base developed during the field programs. These formalisms allow for scanning of large-scale sewer systems to identify problem pipes with respect to deposition.

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