United States
Environmental Protection
Agency

Municipal Environmental Research
Laboratory
Cincinnati OH 45268

EPA-600/2-79-134
August 1979

Research and Development

Disinfection/ Treatment of Combined Sewer Overflows

Syracuse, New York

RESEARCH REPORTING SERIES

Research reports of the Office of Research and Development, U.S. Environmental Protection Agency, have been grouped into nine series. These nine broad categories were established to facilitate further development and application of environmental technology. Elimination of traditional grouping was consciously planned to foster technology transfer and a maximum interface in related fields. The nine series are:

1. Environmental Health Effects Research
2. Environmental Protection Technology
3. Ecological Research
4. Environmental Monitoring
5. Socioeconomic Environmental Studies
6. Scientific and Technical Assessment Reports (STAR)
7. Interagency Energy-Environment Research and Development
8. "Special" Reports
9. Miscellaneous Reports

This report has been assigned to the ENVIRONMENTAL PROTECTION TECHNOLOGY series. This series describes research performed to develop and demonstrate instrumentation, equipment, and methodology to repair or prevent environmental degradation from point and non-point sources of pollution. This work provides the new or improved technology required for the control and treatment of pollution sources to meet environmental quality standards.

This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

EPA-600/2-79-134
August 1979

DISINFECTION/TREATMENT OF COMBINED SEWER OVERFLOWS

Syracuse, New York

by

Frank J. Drehwing
Arthur J. Oliver
Dwight A. MacArthur
Peter E. Moffa
O'Brien & Gere Engineers, Inc.
Syracuse, New York 13201

Grant No. S802400 (11020HFR)

Project Officer

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

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

DISCLAIMER

This report has been reviewed by the Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

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

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic 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 upon characterization of these flows in both a quantity and quality sense. This report describes the results of a full-scale facilities demonstration study of a number of treatment devices for controlling the quality of combined sewer overflow discharges.

Francis T. Mayo
Director
Municipal Environmental Research Laboratory

ABSTRACT

The Syracuse demonstration program was designed to evaluate treatment of combined sewer overflows (CSO) for the Onondaga County Department of Drainage and Sanitation.

The demonstration study covered field evaluations of high-rate screening and disinfection by the following unit processes: fine-mesh screening by three separate microscreening devices, swirl regulator/concentrator, and disinfection utilizing chlorine (Cl_2) and chlorine dioxide (ClO_2). Applied flowrates to the microscreening devices ranged from 210 to 1660 gpm (13.2 to 104.6 l/sec), and applied flowrates to the swirl regulator/concentrator ranged from 140 to 5280 gpm (8.8 to 332.6 l/sec).

The screening facilities operation covered a total of 16 overflow events during the period of March 1975 through October 1976. A total of 11 overflow events were studied using the swirl unit during the period of May 1974 to September 1975. All studies evaluated the effects of hydraulic and pollutant loadings and influent quality on system performance.

The three microscreening units were a 5-ft (1.5m) diameter Sweco Wastewater Concentrator utilizing 105μ screen apertures, a 6ft x 6ft (1.8m x 1.8m) Zurn Micromatic utilizing 71μ screen apertures, and a 7.5ft x 5ft (2.3m x 1.5m) Crane Microstrainer utilizing 23μ screen apertures. Storm average hydraulic loading rates employed in the evaluations ranged from 11.2 to 66.4 gpm/ft^2 (27.3 to 162.0 m/hr) for the Sweco unit, 3.0 to 12.3 gpm/ft^2 (7.3 to 30.0 m/hr) for the Zurn unit, and 1.7 to 7.7 gpm/ft^2 (4.1 to 18.8 m/hr) for the Crane unit. Using multiple regression analysis techniques, mathematical performance models were developed for each of the microscreening units relating SS removal rates to influent hydraulic loading and to influent SS concentrations. Average SS removal rates in terms of concentration were approximately 32 percent for the Sweco unit, 45 percent for the Zurn unit, and 58 percent for the Crane unit. In terms of mass removal, the Sweco unit averaged 48 percent - a significantly higher removal rate than exhibited in terms of SS concentration removal efficiency. The increased level of solids mass removal is due to the large fraction of influent flow returned to the intercepting sewer. Comparisons of the performance results with results published in the literature indicated similar trends of SS removal.

A 12.33 ft diameter swirl regulator/concentrator was evaluated for SS removal efficiency at flowrates ranging from 0.2 to 7.6 MGD (0.5 to 20 cum/min) or hydraulic loading rates ranging from 1.2 to 44.4 gpm/ft^2 (2.9 to 108.3 m/hr). Mathematical models were developed relating SS removal efficiencies to influent hydraulic loading rates and influent SS concentrations. Results indicated SS concentration removal efficiencies ranging from

18 to 55 percent and SS mass removal efficiencies ranging from 33 to 82 percent. A settling velocity distribution analysis of solids particles was conducted to determine the predicted SS removal under a given set of operating conditions for comparison with actual measured SS removal efficiencies. The results tended to confirm the predicted performance curve determined in previous model studies.

Multiple regression modeling of the Cl_2 and ClO_2 disinfection data yielded statistically significant performance equations for the high-rate disinfection systems. The results of the study at Maltbie Street indicate that application of high-rate disinfection processes can result in significant reduction of bacterial populations in CSO. ClO_2 dosages in the order of 6 to 12 mg/l applied in the initial stages of overflows reduced FC levels to 200 counts/100 ml. Applied dosages of 4 mg/l after first-flush loadings had passed through the treatment system, maintained the 200 counts FC/100 ml level in the majority of the samples collected. Application of Cl_2 at dosages of from 12 to 24 mg/l during the initial stages of overflow also were able to achieve 3 to 4 log reductions of FC, while lower dosages (12 mg/l) produced similar reductions after the first 30 to 45 min. Sequential addition of disinfectants (2 mg/l ClO_2 followed by 8 mg/l Cl_2 after 15 sec) at a total contact time of 1 min produced 3 to 4 log reductions of FC. The limited data obtained in the sequential addition tests precludes a comparison of this method of disinfection with the application of Cl_2 or ClO_2 separately.

Regression analyses of the disinfection data collected indicated that removal of SS would improve the reduction of bacterial populations by the disinfection processes. The effects of solids removal are slightly more pronounced for ClO_2 than for Cl_2 disinfection with both exhibiting improved FC kills of about one-half to one order of magnitude.

Capital and operating cost estimates indicated that disinfection facilities would be less expensive utilizing Cl_2 as the disinfectant rather than ClO_2.

Preliminary evaluations were conducted on the impact of transmitting CSO treatment residuals to the Metropolitan Syracuse Sewage Treatment Plant (Metro). The analysis included the effects of transmission of CSO sludges as well as transmission of dilute residuals from on-site sludge dewatering processes on both the primary and secondary treatment facilities at Metro. The effects of transmitting CSO sludges directly to the Metro sludge handling facilities were also examined. Results of the analysis indicated that return of CSO treatment residuals would result in a solids overload of the primary facilities at Metro and that the secondary treatment facilities would suffer an organic overload if the bleedback of CSO residuals occurred at flow conditions higher than average dry-weather conditions.

Transmission of CSO treatment residuals directly to the dry-weather plant sludge handling facilities would result in hydraulically overloading the gravity thickeners, thereby reducing the solids retention time in the thickeners from a design retention time of 7 hours to 4 hours or less for a 1 year - 2 hour storm. Hydraulic overload would also be evident in the sludge digesters and would result in a reduced solids retention time. A return of incomplete digestion products to the head of the treatment plant would be expected. The impact of CSO residual bleedback was also evaluated from a solids loading viewpoint. Results indicated that the bleedback would cause the solids loading to be approximately equal to the peak allowable solids loading, and the sludge handling facilities would be limited in their capacity to handle solids loadings from back-to-back storm events.

Capital and operating costs are presented for point-source treatment of all overflows in Syracuse based on a 15 MGD (56,775 cu m/day) capacity satellite CSO treatment facility serving a 115 acre (46.5 ha) drainage area. Costs were then extrapolated for the entire Syracuse CSO drainage area. These costs are compared to previous estimates which considered centralized treatment adjacent to Metro and point-source treatment at individual overflow sites utilizing conventional disinfection design parameters. The study projected annual costs for CSO treatment facilities in Syracuse as follows:

Treatment Device	Cl_2 Disinfection	ClO_2 Disinfection
Sweco	$ 12,294,500	$ 12,665,700
Zurn	7,669,900	8,076,700
Crane	5,888,900	6,307,500
Swirl	3,264,300	3,641,900

Special studies were conducted during the demonstration program and are also presented in this report. The special studies included virus inactivation, bench-scale ultraviolet light disinfection, on-line monitoring of SS, continuous monitoring of TOC, and a brief discussion of chlorinated hydrocarbon analyses.

This report was submitted in fulfillment of Grant No. S802400 (formerly 11020HFR) by O'Brien & Gere Engineers, Inc. under the partial sponsorship of the U.S. Environmental Protection Agency. This report covers the period from 1972 to 1976, and work was completed as of January 1978.

CONTENTS

FIGURES

TABLES

ABBREVIATIONS AND SYMBOLS

ABBREVIATIONS

APWA	--American Public Works Association
ATP	--adenosine triphosphate
BOD5	--5 day biochemical oxygen demand at 20^oC
C	--centigrade
cm	--centimeters
COD	--chemical oxygen demand
CSO	--combined sewer overflow
CSS	--Onondaga County Comprehensive Sewerage Study
cu m	--cubic meters
DC	--direct current
dia	--diameter
DWF	--dry-weather flow
ENR	--Engineering News Record
ft	--feet
ft^2	--square feet
ft^3	--cubic feet
EDTA	--ethylenediaminetetraacetate
EPA	--U.S. Environmental Protection Agency
FC	--fecal coliform
fps	--feet per second
FS	--fecal streptococcus (or - cocci)
G	--velocity gradient (sec^{-1})
gal	--gallons
gpm	--gallons per minute
ha	--hectares
hr	--hour
in	--inches
kg	--kilograms
KWH	--kilowatt hours
l	--liters
lb	--pounds
m	--meters
m^2	--square meters
m^3	--cubic meters
ma	--milliamperes
Metro	--Syracuse Metropolitan Sewage Treatment Plant
MG	--million gallons
mg/l	--milligrams per liter
MGD	--million gallons per day
MIS	--Main Intercepting Sewer
ml	--milliliter

ABBREVIATIONS (CONT'D)

MLSS	--mixed liquor suspended solids
mm	--millimeter
min	--minute
$\mu w/cm^2$	--microwatts per square centimeter
nm	--nanometers
NYSDEC	--New York State Department of Environmental Conservation
O&G	--oil and grease
O&M	--operation and maintenance
OrgN	--organic nitrogen
PFU	--plaque forming units
rpm	--revolutions per minute
SCSO	--simulated combined sewer overflow
sec	--second
Sett-S	--settleable solids
sq km	--square kilometers
sq mi	--square miles
SS	--suspended solids
SWMM	--Stormwater Management Model
TC	--total coliform
TDS	--total dissolved solids
TIP	--total inorganic phosphorus
TKN	--total kjeldahl nitrogen
TOC	--total organic carbon
TS	--total solids
TSS	--total suspended solids
T-Alk	--total alkalinity
UV	--ultraviolet
VAC	--volts of alternating current
VS	--volatile solids
VSS	--volatile suspended solids
yr	--year

SYMBOLS

Å	--angstroms
Al	--aluminum
$AlCl_3$	--aluminum chloride
Ca	--calcium
Cd	--cadmium
Cl^-	--chloride ion
Cl_2	--chlorine
ClO_2	--chlorine dioxide
ClO_2^-	--chlorite ion
CO_2	--carbon dioxide
HCl	--hydrochloric acid
HOCl	--hypochlorous Acid
K	--potassium

SYMBOLS (Cont'd)

Mg	--magnesium
NaOH	--sodium hydroxide
NH_3N	--ammonia nitrogen
Ni	--nickel
NO_2	--nitrites
NO_3	--nitrates
OCl^-	--hypochlorite ion
P	--phosphorus
Pb	--lead
Zn	--zinc

ACKNOWLEDGMENTS

O'Brien & Gere Engineers, Inc. gratefully acknowledges the cooperation of the Onondaga County Department of Drainage and Sanitation. Appreciation is expressed to Mr. John J. Hennigan, Jr., Commissioner, Mr. John M. Karanik, Deputy Commissioner, and Mr. Randy Ott, Project Engineer for their cooperation and assistance.

The support of this effort by the Storm and Combined Sewer Section, Edison, New Jersey of the USEPA Municipal Environmental Research Laboratory, Cincinnati, Ohio, and especially of Richard Field, Chief, Storm and Combined Sewer Section, USEPA, for their guidance, suggestions, and contributions is acknowledged with gratitude.

Valuable assistance to the program was furnished by Dr. James E. Smith of Syracuse University and his staff through the design, conduct, and analysis of the virus studies at the Maltbie Street facility.

SECTION 1

INTRODUCTION

GENERAL

Discharge of untreated domestic and industrial waste and of primary sewage treatment plant effluent has long been recognized as a major source of pollution in the nation's waterways. More recently, within the past two or three decades, the importance of storm and combined sewer overflow (CSO) as a major source of contamination has also been recognized. The significance of CSO has increased as a result of the addition of wastewater from new residential, commercial, and industrial developments to the finite capacity of existing combined sewer systems.

Although many advances have been made in waste treatment technology in recent years, receiving water quality cannot be consistently maintained without controlling and/or treating the high volume discharges from combined and storm sewers. It was the purpose of the study described in this report to demonstrate the feasibility of treating a CSO at the overflow point rather than at a centralized treatment facility through high-rate treatment techniques. These techniques included microscreening followed by high-rate disinfection, and high-rate quality and quantity control through application of the swirl regulator/ concentrator.

BACKGROUND

Description of Study Area

Onondaga County, located in the central region of New York State has a total area of 792 sq mi (2,050 sq km) and a population (estimated 1976) of approximately 473,000. Most of the county is located in the Oswego River Basin. Onondaga Lake, which drains to the Oswego River, is included in the lowland region which extends across the northern part of the county. The lake is shown in Figure 1.

Soils in the county are predominently glacial till, which is composed of silt, sand, and gravel. The soil contains numerous cobbles and boulders, deposited by receding glaciers. Precipitation is generally distributed evenly across the county. The average annual precipitation is about 36.9 in./yr (94 cm/yr).

The City of Syracuse is located approximately at the center of the county. The city has a land area of about 16,000 acres (6,480 ha) and a

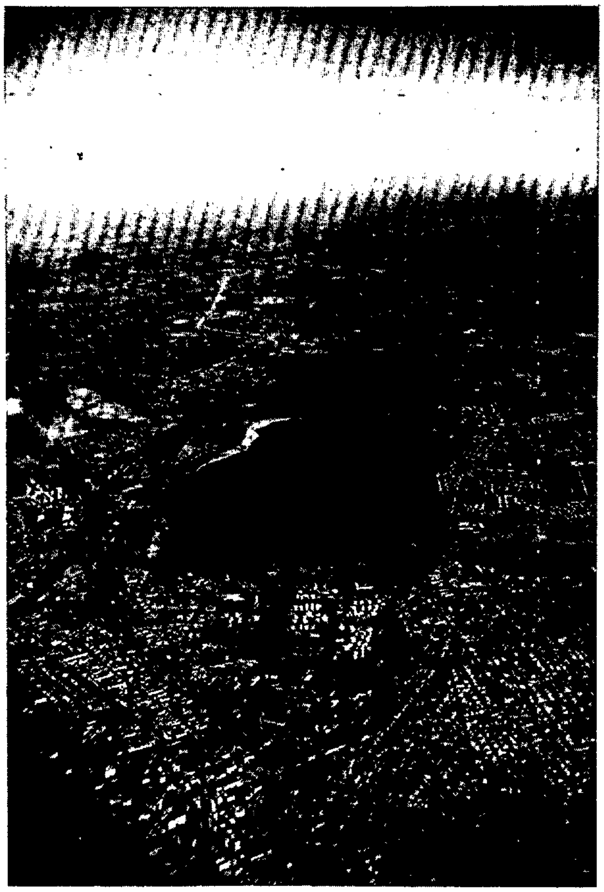


FIGURE 1. Onondaga Lake

population (estimated 1976) of approximately 173,000. Sewage from the central urbanized area of the county, consisting of the city and adjacent sections of the suburban towns of Dewitt, Salina, and Geddes, is conveyed either to the Metropolitan Sewage Treatment Plant (Metro) or to the Ley Creek Sewage Treatment Plant. Primary treatment is presently provided at the Ley Creek plant. All flow from the Ley Creek plant, including treated and untreated excess flows, is pumped to Metro.

All sewage is conveyed in one of three intercepting systems. The Main Intercepting Sewer (MIS), which follows Onondaga Creek, and the Harbor Brook Intercepting Sewer, which follows Harbor Brook, are both tributary to Metro. The Ley Creek intercepting system is tributary to the Ley Creek Sewage Treatment Plant.

The MIS runs from Metro at Hiawatha Boulevard south along Onondaga Creek to Pacific Avenue, a distance of about 27,000 ft (8,200 m). It serves a total area of about 11,750 acres (4,760 ha) in the City of Syracuse and the Town of Dewitt.

The Harbor Brook Intercepting Sewer runs from Metro southwesterly to Velasko Road, a distance of approximately 15,700 ft (4,800 m). It serves a total area of about 2,150 acres (870 ha).

The Ley Creek trunk sewer serves an area of about 8,750 acres (3,500 ha) in and adjacent to the northeastern section of the city. The trunk sewer is approximately 30,000 ft (9,200 m) long.

An area of approximately 9,000 acres (3,650 ha) in the central urbanized area is served by combined sewers. This combined system is within the Main and Harbor Brook Interceptor Systems. The collection system tributary to the Ley Creek trunk sewer is a separated system, collecting and conveying only sanitary and industrial wastes.

The MIS and the Harbor Brook Intercepting Sewer together have a maximum capacity of 150 MGD (6.6 m^3/sec), which is about twice the anticipated dry weather flow from the served areas. Diversion devices are located in manholes at points where combined sewers intersect the intercepting sewers. Wastewater in excess of the capacity of the intercepting sewers is discharged directly into Onondaga Creek or Harbor Brook at these diversion manholes. The diversion devices include leaping weirs, side overflow weirs, dam and orifice devices, and drop manholes, and are in varying states of structural condition.

Previous Studies

The Federal Water Pollution Control Act Amendments of 1972 were enacted on October 18, 1972. The objective of this legislation was: "to restore and maintain the chemical, physical, and biological integrity of the nation's water". Included in the goals and policies were the following declarations: "It is the national goal that the discharge of pollutants into the navigable waters be eliminated by 1985.", and "It is the national policy that a major research and demonstration effort be made to develop the technology necessary to eliminate the discharge of pollutants into the navigable waters.....".

In 1974 the Environmental Protection Agency (EPA) published a report entitled, "Urban Storm Water Management and Technology: An Assessment" (1). This report stated that "During the next decade, it is expected that billions of dollars will be spent in the United States to combat the degradation of streams and other water bodies by pollutants released through storm discharges and combined sewer overflows." The statements above emphasize the Federal government's concern about the quality of the nation's waters and demonstrate its determination to find a solution to CSO discharges through control and/or treatment.

The County of Onondaga has shared this concern with the Federal government. It is the overall goal of the county to reduce or totally eliminate pollutional discharges to the waters of Onondaga Lake and by so doing, to upgrade the water quality of the lake and protect downstream waters, which include the Seneca River, Oswego River and Lake Ontario. To achieve this goal the Onondaga County Department of Drainage and Sanitation in conjunction with EPA has undertaken a number of studies to define and ultimately abate pollution to the lake.

The studies completed to date are as follows:

1. The Onondaga County Comprehensive Sewerage Study (2)
2. An Industrial Waste Study (3)
3. An Onondaga Lake Study (4)
4. The Onondaga Lake Monitoring Program (5)

Two of the above studies, the Comprehensive Sewerage Study and the Onondaga Lake Study, have direct bearing on the demonstration studies described in this report.

The Comprehensive Sewerage Study (CSS) was published in November, 1968. Its objective was to set forth a broad master plan of sewage requirements for Onondaga County which could be implemented to assure acceptable quality for the receiving waters in the county. The master plan formed the basis upon which more specific facilities planning could be conducted.

The CSS included the assembly and evaluation of all pertinent existing information, inspection and investigation of existing private and municipal sewage facilities (including overflow points and combined sewers) and evaluation of major municipal and industrial wastewater discharge outlets. From the resulting data, alternate solutions for collection systems were prepared. Proposals for treatment of wastewater within the study area were developed and recommendations for the most feasible solutions to the problem of wastewater discharge were presented. Within the central urban area of the county, a total of 87 CSO locations to Onondaga Lake or its three tributaries, Ley Creek, Onondaga Creek, and Harbor Brook were identified. Numerous discharges were found to occur at these overflows, primarily due to inadequate or reduced wet-weather capacity of conveyance systems.

The CSS indicated that capacities for design peak dry-weather flow rates are commonly from two to four times the average dry-weather rates, estimated to be realized in the design year. The average rate of sewage flow is estimated to be equivalent to the runoff rate of about 0.01 in/hr (0.025 cm/hr) from the area served. Intercepting sewers, thus, are usually designed to have capacities not greater than for equivalent storm-water runoff rates of about 0.02 to 0.04 in/hr (0.05 to 0.10 cm/hr). The capacities of Syracuse intercepting sewers fall within this range. The combined sewer system, not including the intercepting sewers, is reported to have a capacity to carry storm flows equivalent to a runoff rate of about 0.50 in/hr (1.3 cm/hr), in addition to dry-weather flow.

Even at times of generation of sanitary sewage at average or less than average rates, there is little capacity in intercepting sewers for mixed sanitary sewage and stormwater. Thus, it is true, that even for storms of low rainfall intensity, the greater proportion of mixed sewage and stormwater cannot be accepted by the intercepting sewer system from the combined sewers and must discharge through outlets to the streams or to Onondaga Lake.

After evaluating various methods of treating CSO, the Comprehensive Study recommended that the CSO be conveyed to a centralized treatment facility adjacent to Metro. The estimated construction cost for centralized treatment totaled $170,000,000 at an ENR Index of 1475 in 1971. After application of the September 1978 ENR Index of 2861, the estimated cost in terms of 1978 dollars was $330,000,000. The CSS estimated the cost of point-source treatment at $250,000,000 in 1971. This estimated cost is $485,000,000 in terms of 1978 dollars.

The high projected costs for point-source treatment were based on construction and operation of chlorine contact tanks sized for a minimum of 10 min detention at design flow. Due to the excessive costs and land requirements associated with conventional chlorination, it was decided that high-rate disinfection would be investigated. If successful, high-rate disinfection (detention times of approximately 1 to 2 min) would greatly reduce the size of the contact tanks and significantly reduce the cost of treatment facilities.

The Onondaga Lake Study was conducted to determine the trophic status of the lake in terms of physical, chemical, and biological water quality parameters. Engineering evaluations were conducted to determine the effects future pollution abatement facilities would have on the lake. In addition, a monitoring program was established to provide continuous updating of the lake water quality data.

One conclusion of the lake study was that although lake bacterial concentrations during dry-weather periods usually fall below contact recreation limits established by the New York State Department of Environmental Conservation (NYSDEC), the bacterial input of CSO discharges precludes any guarantee of public safety.

PURPOSE AND SCOPE

As a result of the previous studies, three major areas of concern became evident to the responsible government agencies:

1. That low dissolved oxygen in the lake would continue because of the high organic concentrations discharged to the lake, primarily in the effluent from Metro, but also from the CSO.

2. That bacterial and viral contamination of the lake would continue during and for some period after storm events as long as the CSO remain untreated.

3. That significant addition of nutrients to the lake could result from CSO.

The total organic loading to the lake will be significantly reduced by the upgrading of Metro to tertiary treatment. The upgrading was mandated by the New York State Health Department in mid-1969. Construction of the required additional treatment facilities was started in February, 1975, under an EPA construction grant. The upgrading and expansion will provide for 85 percent removal of biochemical oxygen demand (BOD_5) and suspended solids (SS), and for tertiary treatment specifically for reduction of phosphorous concentrations to less than 1 mg/l in the plant effluent (6). It is expected that upgrading will reduce the average rate of BOD_5 discharged to the lake at Metro from 8,000 lb/day (3,600 kg/day) to 3,600 lb/day (1,600 kg/day). Discharge of SS is expected to be reduced from 38,000 to 21,400 lb/day (17,300 to 9,700 kg/day). These reductions will tend to emphasize the significance of untreated CSO.

In 1971, EPA awarded Demonstration Grant No. 11020 HFR (now S802400) to Onondaga County, in order to provide partial funding for a demonstration study of various aspects of point-source treatment of CSO. The broad objectives of the study were as follows:

1. To investigate the feasibility of high-rate disinfection techniques;

2. To investigate removal of nutrients, particularly nitrogen and phosphorus;

3. To investigate techniques for removal of suspended solids, and the impact of solids removal on high-rate disinfection.

Treatability of CSO, in terms of bacteria, viruses, nutrients, and SS, was to be demonstrated using commercially available treatment equipment. Emphasis was placed on efficiency of removal, automatic response and control during storm events, and definition of operation and maintenance requirements.

The activities and results of the studies conducted under 11020 HFR (S802400) are summarized in this report. In addition, two separate reports have been prepared under the same grant, as follows:

1. Bench-Scale High-Rate Disinfection of Combined Sewer Overflows with Chlorine and Chlorine Dioxide, (EPA-670/2-75-021) (7).

2. High-Rate Nutrient Removal for Combined Sewer Overflows, (EPA-600/2-78-056) (8).

SECTION 2

CONCLUSIONS

GENERAL

1. High-rate treatment processes for solids reduction and disinfection are viable and economical alternatives in CSO abatement and management. These treatment processes are adaptable and well suited for point-source applications.

2. Initial field studies provide important physical and quality data on CSO flows and loadings for the selection of treatment facilities.

3. Analysis of long term rainfall data provides a good basis of correlation between projected and monitored overflows.

4. The Simplified Stormwater Management Model is an excellent tool for analysis of projected loadings and preliminary abatement planning.

MICROSCREENING

1. Operational problems were encountered during start-up and initial performance of prototype screening devices, particularly as related to the Crane Microstrainer.

2. The Sweco unit achieved an average SS mass removal efficiency of 48 percent and a concentration removal efficiency of 32 percent, operating in a range of 4.9 to 61.7 gpm/ft^2 (12 to 150 m/hr).

3. The Zurn unit provided an average SS mass removal efficiency of 45 percent and a concentration removal efficiency also of 45 percent, operating in a range of 3.3 to 13.7 gpm/ft^2 (8 to 33 m/hr).

4. The Crane unit achieved an average SS mass removal efficiency of 58 percent and a concentration removal efficiency of 58 percent also, operating at loading rates of 1.7 and 7.7 gpm/ft^2 (4.1 and 18.8 m/hr).

5. Relationships of SS removal efficiency to hydraulic loading rate were developed for each of the units and generally indicated that efficiency decreased as hydraulic loading rate increased.

6. The Sweco and Zurn units removed proportionately greater amounts of organic solids than total SS while the Crane unit removed proportionately lesser amounts.

7. Significant and/or consistent removal of heavy metals was not achieved by any of the screening units.

8. The performance data of the screening units in this study corresponded closely with the reported results of other microscreening studies and noted that better SS removals were achieved at smaller screen apertures. However, sustained hydraulic loading rates could not be achieved for the Zurn Micromatic and only for one short time interval (5 min) was the design hydraulic loading rate of 30 gpm/ft^2 (75 m/hr) achieved. The Sweco unit appeared capable of achieving its design loading rate of 60 gpm/ft^2 (150 m/hr) although most evaluations were at lower hydraulic loading rates.

SWIRL REGULATOR/CONCENTRATOR

1. The swirl regulator/concentrator was found to function satisfactorily as a concentrator of CSO solids when operating within the design flow range.

2. Operational problems with peripheral equipment installed at the site (pumps, instrumentation, etc.) prevented a more detailed evaluation of the swirl regulator/concentrator.

3. Analysis of the settling velocities for a range of particles found at the selected overflow tended to confirm the predicted performance curve determined from previous model studies.

4. The SS concentration removal efficiency of the swirl regulator/concentrator ranged from 18 to 55 percent and from 33 to 82 percent in terms of mass removal.

5. Regression analysis of SS removal data indicated a slight increase in SS concentration removal efficiency with increased foul fraction of the swirl unit.

6. The BOD_5 concentration removal efficiency of the swirl regulator/concentrator ranged from 29 to 79 percent and from 51 to 82 percent in terms of mass removal.

7. The overall average TOC and VSS concentration removal efficiencies were 33 and 34 percent, respectively.

8. Significant and/or consistent removal of heavy metals was not achieved by the swirl regulator/concentrator.

SOLIDS HANDLING CONSIDERATIONS

1. Transmission of CSO treatment facilities concentrated underflows (residuals) from the entire Syracuse CSO drainage area to the primary treatment facilities of Metro would not result in a hydraulic overload for any of the four solids removal devices investigated.

2. A solids overload at Metro would result from transmission of CSO treatment residuals for a 1 year-2 hour storm for all of the treatment devices considered.

3. The organic loading to the Metro secondary clarifiers would not exceed the acceptable limit of 1.5 times the average dry weather flow (DWF) loading for the two storms considered upon transmission of CSO treatment residuals to Metro.

4. No hydraulic overload would result from transmission of dilute residuals to Metro.

5. No solids overload would result for the Zurn, Crane and swirl units used as CSO treatment devices when CSO dilute residuals are transmitted to Metro. However, if the Sweco unit were utilized, a solids overload would result from a 1 year-2 hour storm.

6. Transmission of CSO treatment residuals directly to the Metro sludge handling facilities would result in drastically overloading the gravity thickeners hydraulically.

7. The total solids loadings from CSO treatment residuals transmitted directly to the existing Metro sludge handling facilities would be close to the peak recommended solids loading rates under average DWF conditions.

DISINFECTION

1. Application of high rate disinfection processes can result in significant reduction of bacterial populations in CSO.

2. Chlorine (Cl_2) dosages of 12 to 24 mg/l during initial stages of overflow were able to achieve 3 to 4 log reductions of fecal coliform (FC).

3. Cl_2 dosages of 12 mg/l achieved similar log reductions of FC after the first 30 to 45 min of the overflow event.

4. Chlorine dioxide (ClO_2) dosages of 6 to 12 mg/l in the initial stages of overflow reduced FC levels to 200 counts/100 ml.

5. ClO_2 dosages of 4 mg/l following first flush loadings of overflow events maintained 200 counts FC/100 ml level.

6. There were limited data to evaluate disinfection by sequential addition of Cl_2 and ClO_2.

7. Regression analyses of disinfection data indicated that removal of SS improved reduction of bacterial populations.

8. High disinfectant residuals of ClO_2 and Cl_2 could be expected in the effluent after a contact time of 1 min.

9. Chlorination equipment functioned safely and reliably under intermittent operations.

10. Prototype ClO_2 generation equipment exhibited continuous mechanical problems and required close attention during operations, but produced results to warrant consideration for full development.

CAPITAL AND OPERATING COSTS

1. The capital and operating costs of actual facilities provided the basis of cost comparisons between treatment facilities. The capital and operating costs developed were:

Treatment Device	Capital Costs Using Cl_2 $/Acre	Capital Costs Using Cl_2 $/MGD	Capital Costs Using ClO_2 $/Acre	Capital Costs Using ClO_2 $/MGD	O&M Costs Using Cl_2 $/Acre	O&M Costs Using Cl_2 $/MGD	O&M Costs Using ClO_2 $/Acre	O&M Costs Using ClO_2 $/MGD
Sweco	11,120	82,250	11,420	87,500	290	2,220	310	2,350
Zurn	5,860	44,960	6,150	47,210	240	1,830	260	1,950
Crane	3,700	28,350	4,000	30,610	230	1,780	250	1,910
Swirl	1,950	14,950	2,240	17,200	140	1,090	160	1,210

Note: Acres x 0.4047 = hectares
MGD x 3875 = cu m/day

2. Actual capital and operating costs were the basis of projected annual costs for treatment of all Syracuse overflows. The projected annual costs based on 20-year amortization at 7% were:

Treatment Device	Cl_2 Disinfection	ClO_2 Disinfection
Sweco	$13,179,000	$13,577,000
Zurn	8,222,000	8,658,000
Crane	6,313,000	6,761,000
Swirl	3,500,000	3,900,000

Note: Acres x 0.4047 = hectares
MGD x 3875 = cu m/day

3. The swirl regulator/concentrator was found to be a more cost-effective alternative then microscreening.

4. Disinfection by Cl_2 was projected to be less costly than ClO_2 as indicated in the data presented above.

5. Treatment of CSO by application of point-source high-rate application processes would be significantly less expensive than would be centralized CSO treatment or point-source treatment utilizing conventional application processes.

VIRUS STUDIES

1. The Aquella virus concentrator and its system of selective adsorption of viruses by filter media are reasonably well suited for study of CSO. However, difficulties were experienced with continuous operation, and it was found that the batch concentration capability of the equipment is very limited. Prospects for use of the Aquella concentrator for frequent or continuous monitoring of viruses in CSO seem poor.

2. The population of wild viruses in CSO is generally at a low level, with high sample variation. It appears questionable whether a meaningful measure of disinfection effectiveness can be made on the basis of observed reductions in wild viruses found in CSO.

3. Seeding of CSO with three indicator organisms (coliphages f2 and ØX174, and poliovirus Sabin K-1) was found to be a satisfactory method for increasing viral populations in CSO to reliably measurable levels. However, phage f2 proved not to be a good simulant of enteroviruses in CSO.

4. Use of massive doses of Cl_2 (24 mg/l with a Cl_2 residual of 1.0-1.4 mg/l) in a high-rate application resulted in virtually complete kill of seeded organisms.

5. Screening had no influence on virus inactivation in CSO.

6. Simultaneous reductions in bacterial and viral titers were common but there was no direct proportional relationship between them.

7. Sequential addition of disinfectants did not result in increased viral kills.

SPECIAL ANALYSIS

1. Analyses for total chlorinated hydrocarbon species in disinfected and undisinfected CSO indicated values of 1 to 50 μg/l with respect to aldrin. A variety of compounds (10 to 20 in number) contributed to the total value.

2. Containment time in the delivery system of ClO_2 generated on-site was determined to significantly affect the strength of the ClO_2 at the point of injection to the microscreened CSO.

3. Bench-scale analyses of the interaction of Cl_2 and ClO_2 with organic species in CSO indicated a lack of correlation of chlorinated organics formation versus dosage upon addition of either Cl_2 or ClO_2 or upon sequential addition of ClO_2 and Cl_2. However, 5 out of 8 samples indicated significantly increased levels of chlorinated organics after addition of Cl_2 and/or ClO_2. The change in chlorinated organics levels ranged from -2.2 to +200 percent.

4. Bench-scale testing of the formation of volatile chlorinated organics upon addition of 12 mg/l Cl_2 or 8 mg/l ClO_2 to simulated CSO indicated that only low levels of such organics are produced when ClO_2 is applied as a disinfectant. Formation of volatile chlorinated organics upon addition of Cl_2 and Cl_2/ClO_2 combinations is more significant. Results indicated that tetrachloroethylene is the most significant volatile chlorinated organic produced, which increased by as much as 150 percent at a detention time of 10 min. Only a slight increase was observed at a detention time of 1 min. The background levels of both chloroform and tetrachloroethylene in the simulated CSO were significant (5.1 µg/l and 8.1 µg/l, respectively).

SPECIAL INVESTIGATIONS OF CSO

1. Disinfection by ultraviolet (UV) radiation is feasible and is reported to have no residual toxic effects.

2. Chlorination at a dosage of 10 mg/l for one minute had approximately the same disinfecting power as a UV lamp radiating at an intensity of 5800 $\mu w/cm^2$ for one minute.

3. The initial rate of bacteria inactivation was greatest when UV radiation and chlorination were used simultaneously.

4. Chlorination followed by UV radiation produced higher bacteria kills than UV radiation followed by chlorine.

5. The highest bacteria kills were produced when in contact with Cl_2 for a minimum of 45 sec prior to UV radiation.

6. No definite advantage was demonstrated for sequential application of UV radiation alone.

7. Bench-scale results indicated that a target level of 2400 counts/ 100 ml of TC bacteria could be achieved by chlorinating with 8 mg/l for 40-45 sec and irradicating with UV for 15-20 sec.

8. Cost analysis data indicates that a UV disinfection system would be substantially higher than Cl_2 and ClO_2 systems.

9. Consideration must be given to the turbidity of treated CSO to evaluate the effectiveness of UV disinfection.

10. Current determinations of adenosine triphosphate (ATP), utilizing the luciferin-luciferase bioluminescent assay system, and indicator bacteria from CSO have indicated the feasibility of using ATP as a reliable and rapid indicator to control the disinfection process. In the dosages required for high-rate disinfection, Cl_2, ClO_2 and sodium thiosulfate did not significantly interfere with the ATP assay.

11. Although operational problems were experienced, the concept of an on-line TOC monitor for continuous monitoring of TOC in CSO is feasible.

12. Laboratory testing indicated that an on-line SS Monitor for continuous measurement of SS concentrations in CSO is feasible. The prototype was shown to be insensitive to the color of dissolved solids or to the shape and size of SS particles within the particle concentration range tested of 10 to 100,000 mg/l.

13. The size of the SS Monitor transducer probe and associated instrumentation allows operation in confined spaces and permits portable operation.

14. Field testing of the SS monitor at the Tulsa, Oklahoma, Mohawk Park Sewage Treatment Plant and at the Syracuse demonstration facilities indicated good qualitative correspondence to measured SS concentrations.

SECTION 3

RECOMMENDATIONS

1. Design and installation of demonstration facilities should be preceded by intensive field studies.

2. Future demonstration programs should include the use of the Simplified Stormwater Management Model for projection of flows and loadings to CSO facilities.

3. Future development work should be continued by manufacturers of microscreening equipment to eliminate or minimize operational difficulties such as blinding, drive malfunction, and screen breakage.

4. Consideration should be given to further research on the feasibility of microscreening in the range of solids removal above that attainable with the swirl regulator/concentrator.

5. Full-scale swirl units, covering a wide range of CSO hydraulic and solids loading conditions, should be evaluated.

6. Future swirl units should include provision for hydraulic regulation of the foul fraction.

7. Although the actual SS removal efficiency of the West Newell Street swirl unit appeared to confirm the SS removal efficiency as predicted from model studies, pilot and prototype scale swirl units should be tested side-by-side under actual field conditions to verify the scaleup procedures from model to full-scale swirl units.

8. In-depth research studies are required on residual CSO solids and sludges to promote solutions for handling and disposal.

9. Bleedback of CSO sludges to sewerage systems should be re-evaluated for specific abatement alternatives and programs.

10. Possible deleterious by-products of disinfection by Cl_2 and ClO_2 should be more comprehensively identified in subsequent demonstration programs.

11. Further research should be performed to optimize high-rate dosages of Cl_2 and ClO_2 for CSO applications.

12. More detailed evaluation of sequential addition of disinfectants in a subsequent demonstration program should be considered.

13. Consideration should be given to the development of fully operational equipment for field generation and feeding of ClO_2.

14. Site specific factors should be considered in the design of CSO facilities, particularly in regard to pumping requirements.

15. Viruses found in CSO discharges require further studies on identification and quantification.

16. Further independent research into the mechanisms governing the existence of viral organisms in CSO, and their response to disinfectants, should be conducted.

17. The formation of chlorinated organics, especially volatile chlorinated organics, and other refractory residuals in high-rate disinfection systems using Cl_2 and ClO_2 should be more thoroughly evaluated.

18. Further research is necessary to develop positive control of injecting the generated ClO_2 into the wastewater.

19. Full scale studies should be performed on ultraviolet (UV) disinfection.

20. The development of an instrument to monitor ATP in unattended operation is a realistic venture and should be actively pursued, with particular attention directed to evaluation of instrument stability and the cost of the enzyme reagents. ATP monitoring could be applicable for automated disinfection.

21. Further evaluations of on-line TOC monitoring are required to eliminate problems resulting from intermittent operation of CSO treatment facilities.

22. Encouragement should be given to further technical development of a SS monitor for CSO and other monitoring and control applications.

23. Future design of swirl regulators should consider the criteria of using a design treatment flowrate (Q_d) for frequent storm occurrences, e.g. 6 to 25 per year. A peripheral relief weir should then be included for flows in the range of 1.5 to 2.0 times Q_d.

SECTION 4

EXPERIMENTAL PLAN

PLAN OBJECTIVES

The specified objectives of the Syracuse CSO demonstration study are summarized as follows: (1) to demonstrate the feasibility of mechanical screening for solids removal, (2) to demonstrate the feasibility of swirl flow regulation/concentration for overflow regulation and solids removal, (3) to demonstrate the feasibility of high-rate disinfection for reduction of microbial and viral populations with and without preliminary screening, and (4) to develop design parameters and cost estimating information for high-rate CSO treatment facilities.

In addition, certain special considerations were added to the program objectives during the course of the study. These peripheral investigations included either an analysis of related research and demonstration work which would enhance this and possible future work on CSO or field testing of specific applications. The areas covered under special considerations included analyses, investigations and instrumentation and are presented in Section 14.

An experimental plan was devised to provide for a specific, rational program for the accomplishment of the overall objectives of the demonstration study. Considerations involved in this planning, and the major elements of the plan itself, are outlined in the following subsections.

SOLIDS REMOVAL CONSIDERATIONS

CSO's frequently contain high concentrations of SS, which when discharged to receiving waters may settle to the bottom or accumulate in shoals, creating noxious conditions, excess oxygen demands on overlying waters, and other physiochemical and aesthetically displeasing effects. The receiving water's capacity to assimilate downstream wastes is reduced, magnifying other pollution problems downstream from CSO. It has also been hypothesized that solids removal may enhance disinfection (7, 9, 10).

The available techniques for solids removal were considered to be the following:

1. Sedimentation
2. Mechanical fine mesh screening (microscreening)
3. Swirl concentration

The feasibility of using each of these techniques was considered.

Feasibility of Sedimentation

Normal sedimentation techniques for solids removal were not considered practical for the Syracuse CSO demonstration study. The space available in the vicinity of the Syracuse CSO outfalls was found to be inadequate for sedimentation tanks sized to handle anticipated peak discharge rates.

Feasibility of Mechanical Fine Mesh Screening (Microscreening)

In general, a microscreen consists of a rotating drum, covered with finely woven fabrics of stainless steel with a range of aperture sizes of approximately 20 to 120 microns. It is constructed to rotate around either a horizontal or a vertical axis. Water enters the inside of the drum, flows radially through the drum screen into the outlet chamber, and deposits suspended solids on the inside of the drum screen. This mat of deposited solids on the screen increases the resistance to flow through the unit. As a result the head differential across the unit increases. The screening unit must have some provision for the removal of the solids mat. Generally, backwashing (either continuous or intermittent) has been used.

Glover and Herbert (9) concluded that high-rate fine mesh, mechanical screening, using screens with apertures in the range of 23 to 35 microns, followed by high-rate disinfection, is a practical method for CSO treatment. The process demonstrated in Philadelphia was termed Microstraining, which is a copyrighted name under Crane Company. For purposes of this study, mechanical screening of this general type has been designated microscreening.

Maher (10), expanding on previous work by Glover and Herbert, observed that when a microstrainer having openings of 23 microns was operated at an influx rate of approximately 26 gpm/ft^2 (63.4 m/hr), suspended solids in CSO were reduced from 50-300 mg/l to 40-60 mg/l. Maher concluded, based on the findings of his study, that the microstrainer could be operated at differentials of about 24 in. (60 cm) of water, and at an influx range of 15 to 25 gpm/ft^2 (36.0 to 60.0 m/hr), resulting in effluent suspended solids of 40 mg/l. Organic matter, in the form of volatile suspended solids was reduced by about 70 percent.

An economic study by Keilbaugh, et al. (11), on the cost of various methods for treating CSO concluded that microstraining followed by high-rate disinfection was the least expensive and most compact of several methods investigated. Combinations of 12 basic treatment processes were compared. Only a surface impounding basin located at the CSO outfall with pumpage to a sewage treatment plant was determined to be less expensive for CSO treatment. However sufficient land area for this method is often not available in urban areas. Consideration of the swirl regulator/concentrator was not included in their economic study.

Bench-scale demonstrations were conducted under the Syracuse study to test the feasibility of CSO treatment by microscreening followed by disinfection. Based on the results of the bench-scale work it was decided that microscreening would be tested in a full-scale demonstration study. Consequently, microscreening was used for solids removal in one of the two Syracuse full-scale demonstration facilities in order to determine solids removal efficiencies and subsequent effects on disinfection processes.

Feasibility of Swirl Regulator/Concentrator

The hydraulic concept leading to the development of the swirl concentrator as an overflow regulating device was first presented in the United States in a report published by EPA in 1970 (12). This report presented the results of a study conducted by the American Public Works Association (APWA) Research Foundation in 1968-1970 on combined sewer overflow regulator and control facilities found in the United States, Canada and selected foreign countries.

The discussion pertinent to the development of the swirl concentrator was on a device known as the circular "vortex", which had been developed in the City of Bristol, England. The "vortex" device was designed as a regulator, but was also found to effectively separate and concentrate solids found in CSO. The report subsequently recommended additional investigation.

An opportunity to confirm and supplement the work carried out at Bristol and to to develop a basis for a similar regulator device suitable for the different CSO conditions found in the United States was provided by the plan of the City of Lancaster, Pennsylvania to construct a CSO regulator facility upstream of a proposed CSO control/treatment facility.

The APWA was retained by the City of Lancaster to conduct an intensive study aimed at achieving these goals. This study was accomplished through use of laboratory hydraulic modeling and development of a mathematical model calibrated to provide the best possible match with experimental model results. The study was sponsored, in part, by the Office of Research and Development, EPA, under Demonstration Grant No. 802219 (formerly 11023GSC). The final study reports were published in 1972, 1973 and 1974 in the EPA Technology Series (13, 14, 15).

A study finding indicated that an unimpeded free vortex must be avoided with the large flows and minimum sized chambers associated with CSO in the United States. A different hydraulic phenomenon, the swirl, was determined to effectively separate solids from CSO. The report concluded that:

1. A practical, simple facility has been developed which offers a high degree of performance in reducing the amount of settleable solids contained in CSO, as well as enabling the quantity of flow to the interceptor to be controlled, all with a minimum of moving equipment.

2. The design of the swirl concentrator has been developed for rapid calculation of its different elements enabling ready transferability to the regulation of various quantities of flow.

3. The swirl concentrator is very efficient in separating both grit and settleable solids in their middle and larger grain size range (>0.2mm). By weight, these fractions represent about two-thirds of the respective materials in the defined combined sewage. Separation of the smaller grain sizes was less efficient, although still appreciable.

It recommended that:

1. A demonstration facility should be constructed of sufficient size to be totally effective for flows of 103 cfs (2.92 m^3/sec). The facility should be monitored to verify the hydraulic and mathematical modeling which was accomplished in the study.

2. Additional hydraulic and mathematical modeling should be accomplished to determine the effectiveness of the swirl concentrator concept in the various phases of primary sewage treatment. Such research should also have application in many industrial waste situations.

3. Further investigation should be made to determine if better efficiency could be obtained with two or more concentrators operated in parallel or in series.

The Lancaster hydraulic study demonstrated the potential of the swirl concentrator as a CSO quality and quantity regulating device. Under the Syracuse study, it was decided to construct and operate a prototype demonstration facility based on the data obtained during the Lancaster study, in order to prove the effectiveness of the swirl for solids removal during full-scale operation.

DISINFECTION CONSIDERATIONS

Investigation of disinfection in this demonstration study was based on the application of chlorine and chlorine dioxide and limited bench-scale analysis of ultraviolet radiation.

Feasibility of High-Rate Disinfection

Chlorine (Cl_2) is used extensively for disinfection of sewage. Chlorination of sewage can be thought of as a two-stage process:

1. Addition of sufficient Cl_2 for satisfaction of Cl_2 demand. In this stage, free Cl_2 combines with other material in the waste. The major combined forms of Cl_2 are chloramines NH_2Cl, mono-; $NHCl_2$, di-; and NCl_3, tri-), which are formed by the reactions of ammonia with free chlorine. Minor reactions

occur with organic material, particularly with reducing agents.

2. After satisfaction of the Cl_2 demand, further addition of Cl_2 results in maintenance of free chlorine, in the form of Cl_2, HOCl (hypochlorous acid) or OCl^- (hypochlorite ion).

Free Cl_2 is a fast-acting disinfectant. Cl_2 in combined forms disinfects much more slowly. In standard disinfection of sewage effluent, addition of sufficient Cl_2 to satisfy the Cl_2 demand is not attempted, and it is assumed that only combined Cl_2 is present. To provide reasonable assurance that disinfection is achieved detention times of at least 15 minutes have been required by state and federal regulatory agencies.

In general, provision of 15 minutes or more of detention time following addition of Cl_2 to CSO was considered infeasible because of the high flows encountered. Therefore, chlorination planning for this study was based on a concept of adding enough Cl_2 to overcome the side reactions quickly and provide free Cl_2 for rapid disinfection. It was hypothesized that Cl_2 added in sufficient amounts to overflows could reduce bacteria and viruses to acceptable levels with contact times of two minutes or less. Bench-scale tests indicated that target levels of bacteria and viruses could be accomplished within two minutes in simulated CSO with a Cl_2 dosage of 25 mg/l (7). The following target levels were achieved at this dosage rate:

1. Total coliform bacteria (TC) - 1,000 colonies per 100 ml.
2. Fecal coliform bacteria (FC) - 200 colonies per 100 ml.
3. Fecal streptococci bacteria (FS) - 200 colonies per 100 ml.
4. Poliovirus, Sabin K-1 (Poliovirus 1) - Five log reduction in population.
5. Coliphage (ØX174 and f2) - Five log reduction in population.

No federal or state regulation specifies target levels of viruses, but it was assumed that a five log reduction in population would reduce the highest anticipated viral counts in CSO to essentially zero. Work by Crane Company (10) and by Glover and Herbert (9) concluded that it is practical to use Cl_2 for high-rate disinfection of CSO. The Syracuse demonstration study, therefore, included facilities for Cl_2 addition and high-rate disinfection. The effects of screening, contact times, mixing, pH, and temperature on disinfection by Cl_2 were to be investigated.

Use of chlorine dioxide (ClO_2) for disinfection is a concept of relatively recent origin. It appears that ClO_2 may have certain advanvantages over Cl_2. ClO_2 is a good disinfectant at high pH values, does not combine with ammonia to form chloramines, and destroys phenolic compounds that combine with other types of Cl_2 compounds to produce chlorinated phenols which yield undesirable taste and odors in drinking water.

The lack of a specific test to determine ClO_2 concentration in sewage has prevented detailed studies of the behavior of ClO_2

during the disinfection process. A technique known as electron spin resonance was recently developed that may be useful for this purpose in the future. Bench-scale studies indicated that ClO_2 on a weight basis, is about twice as effective as Cl_2 in reducing bacterial and viral counts to target levels (7). It is not understood exactly how Cl_2 and ClO_2 act on bacteria and viruses to accomplish disinfection. However, during the bench-scale work, it appeared that lipid (organic fat) content in the cell membrane may be a factor. Cells with relatively low lipid content in the membrane appeared to be relatively more resistant to disinfection. There is some basis to assume that lipids may be more sensitive to ClO_2 than to Cl_2, which might explain the superior disinfection ability of ClO_2. The Syracuse demonstration study was to include ClO_2 addition to compare its disinfectant capability with Cl_2 under similar conditions of screening, contact time, mixing, pH, and temperature.

VIRUS CONSIDERATIONS

The hazards to public health posed by the presence of water-borne viruses have been well documented (16, 17, 18). The principal sources of these viruses are the alimentary tracts of humans and domestic animals. A broad spectrum of viruses is contributed whose members have only one general feature in common -- they are naked, nonenveloped virions for the most part: picornaviruses, adenoviruses, reoviruses, parvoviruses and perhaps papovaviruses. The viruses have relatively long thermal decay times and are not readily inactivated by osmotic shock resulting from high dilutions in water, by pH variations normally associated with water supplies, by adsorption to natural colloids in water or by the action of other types of microorganisms. On this basis alone it is not surprising that animal viruses are transmitted virtually unchanged when domestic sewage and storm runoff are combined.

Although there is no evidence to date that any virion is completely resistant to disinfection by Cl_2 and Cl_2 compounds, there are ample data to demonstrate that the method of their application and the presence of interfering organic substances control the efficiency of this treatment (19, 20, 21). Previous research (22) had shown that ClO_2 as well as Cl_2 (as HOCl) reduced the titers of several enteroviruses in simulated and real CSO. These attempts to disinfect CSO and to evaluate the relative merits of both Cl_2 and ClO_2 had been limited largely to bench-scale studies. A follow-up was required to determine if the same principles could be applied to disinfection of CSO on a full scale.

DEVELOPMENT OF EXPERIMENTAL PLAN

The experimental plan which was developed to satisfy the overall objectives of the demonstration study was based on use of facilities at the Maltbie Street overflow to demonstrate screening techniques and use of facilities at the West Newell Street overflow to demonstrate the swirl concentrator. High-rate disinfection was planned at both sites. The procedures for selection of sites for demonstration purposes are outlined in Section 5.

Maltbie Street Facilities

Microscreening--

It was decided that the facilities at Maltbie Street would include three screening devices, operated in parallel. The screens would be commercially available units, of comparable design. This would allow evaluation of the relative merits of each under roughly identical loading and operating conditions. An economic analysis was made to determine the feasibility of designing the screening and disinfection facilities for the peak anticipated overflow of 30 MGD (78.9 cu m/min). A reduced design flow of 15 MGD (39.4 cu m/min) was selected due to physical and financial constraints. Flow was to be split evenly between the three units, with each receiving 0 to 5 MGD (0 to 13.2 cu m/min), depending on overflow quantity.

The three screening devices to be tested in this program consisted of two microscreens each rotating on a horizontal axis (a Crane Microstrainer and a Zurn Micromatic) and a third microscreen (a Sweco Wastewater Concentrator) rotating on a vertical axis. Details of the design of each device are contained in Section 6. The screening devices were equipped with different size screen apertures to determine what effect the removal of different levels of suspended solids would have on disinfection processes. The screen aperture sizes were, specifically, 23, 71 and 105 micron stainless steel mesh. It was expected that these different screen sizes would produce different suspended solids levels in the effluents to the disinfection contact chambers. All other parameters being equal, an evaluation of varying solids concentrations on the disinfection process could be made.

Evaluations of the screen loading rates were to be investigated to determine the optimum loading rates in terms of operation, maintenance, and efficiency of solids removal for each unit, in order to minimize the capital and operating costs of treatment facilities.

In addition to varying the screen loading and flowrates, two overflow events were to be evaluated for reduction of bacterial levels with no screening prior to disinfection. This evaluation would further indicate the relative importance of solids levels on disinfection.

Disinfection - Bacteria --

The basic objective of the disinfection experiments at the Maltbie Street facility was to optimize the disinfectant dosages required to reduce microbial populations to acceptable levels.

Microscreens were believed to have a significant advantage regarding disinfection in that microorganisms which would otherwise be protected from disinfection by larger grease and solids clumps would be more vulnerable to disinfection in the relatively small clumps passing through the screen. In addition, it was believed that microscreening would have the following downstream advantages:

1 As a result of reducing the particle sizes, the time for natural assimilation of oxygen-demanding substances in the receiving stream would be shortened.

2. Some removal of oxygen-demanding substances would be accomplished, in that some fraction of the organic matter in the waste stream would be included in the screenings.

For the purpose of the demonstration phase of this study, target levels for the receiving stream were established as follows: total coliforms of less than 1,000 counts/100 ml and fecal coliforms of less than 200 counts/100 ml. This indicated that acceptable levels in the discharge could be as follows:

$$TC_d = \frac{Q_s}{Q_d} \times 1000/100 \text{ ml}$$

$$FC_d = \frac{Q_s}{Q_d} \times 200/100 \text{ ml}$$

where: Q_s is stream flow, after CSO discharge
Q_d is CSO discharge
TC_d is total coliform count in Q_d
FC_d is fecal coliform count in Q_d

In practice, it was impossible to pace disinfectant dosage using the Q_s/Q_d ratio since its value was not known. In order to be certain of achieving the target levels in the stream, it was decided that the acceptable levels in the discharge would be defined as being exactly equal to the target levels in the stream (i.e. $Q_s/Q_d = 1$).

The conclusions of the bench-scale study (7) had indicated that a Cl_2 dosage of 25 mg/l, a ClO_2 dosage of 12 mg/l, or a two-stage (sequential) addition of 2 mg/l of ClO_2 followed by 8 mg/l of Cl_2 in 15 to 30 sec, would reduce bacteria counts to the defined acceptable levels in a two-minute exposure period (7). These dosages were to be verified or optimized in the full-scale facility.

It was hypothesized that two-stage disinfection by sequential addition might enhance disinfection beyond the additive effects of the respective doses. This might occur in two ways: (1) The first disinfectant would precondition the waste so that the second disinfectant could work more efficiently; and (2) interactions between two disinfectants could lead to improved efficiency of one or both. Four combinations are theoretically possible for two-stage chlorination with Cl_2 and ClO_2: Cl_2 followed by Cl_2, Cl_2 followed by ClO_2, ClO_2 followed by ClO_2, and ClO_2 followed by Cl_2. Bench scale tests showed that disinfection was enhanced by addition of ClO_2 followed by addition of Cl_2 in 15 to 30 seconds. It was speculated that ClO_2, which is a stronger disinfectant, may be regenerated through interaction of the chlorite ion (ClO_2^-) and Cl_2. Sequential addition of the same disinfectant did not enhance disinfection beyond the expected additive effects of the chemicals. Two-stage disinfection was demonstrated at the Maltbie Street facility only.

It was also felt that rapid mixing of the disinfectant with the CSO could result in the achievement of acceptable levels of bacteria at reduced disinfectant dosages. Thus, provisions were made in the experimental plan to investigate various mechanical mixing techniques, including single flash mixing at the point of injection, sequential flash mixing, no mixing, and a previously reported technique (9) utilizing corrugated baffles throughout the length of the disinfection tanks. Evaluations of the bacterial level reduction achieved by these mixing techniques versus cost were to be made.

Disinfection - Viruses --

Although it seems patently obvious that combined storm and domestic sewage overflows should yield a spectrum of enteric pathogens, both the qualitative and quantitative nature of these pathogens are largely unknown. This is particularly true regarding our knowledge of virus pathogens from humans and animals since the viruses generally are diluted greatly and the technology for quantitative recovery of viruses from large volumes of water is relatively new.

A wide variety of methods for quantitative recovery of viruses have been studied including precipitation, ion exchange, electroosmosis, two-phase separations, high speed continuous centrifugation and gel filtration (22, 23, 24). However, only high speed filtration methods combined with selective adsorption appear to have the capacity to remove viruses selectively from tens or even hundreds of gallons of water (22). Wallis, et al. (25, 26) have developed the most successful system for this purpose and a commercial version of their instrument has been marketed under the trademark Aquella (Carborundum Co., Buffalo, N.Y.). Numerous published accounts of the development and performance of this instrument have appeared recently (21, 22, 25, 26, 27, 28, 29, 30).

The Aquella virus concentrator has been applied successfully to tap water (25, 27, 28), sewage (22), estuarine waters (30, 31) and sea water (30, 31). Both wild viruses and virus seed have been recovered with efficiencies approaching 77 to 100 percent when infective units were less than one per gallon (27, 28).

Thus the Aquella concentrator appeared to be the best available choice for quantitating viruses in CSO. A recent paper (32) revealed that BGM, a continuous line of African green monkey kidney, could detect enteroviruses in storm sewage (and incidentally, more efficiently than primary African green MKC or rhesus MKC). Since a study was underway to determine the effectiveness of ClO_2 as a viricide for CSO it was decided to attempt the use of the Aquella to measure the number of wild viruses which survive ClO_2 treatment of storm overflows. An effort was also made to use phage f2 as a virus indicator which would mimic enteroviruses and could serve as an internal control for the concentration of viruses, measuring the recovery efficiency.

The primary objectives in the first phase of the virus investigations were: (1) to determine the feasibility of using the Aquella virus concentrator to detect virus pathogens in CSO; (2) to determine levels of virus pathogens in CSO at the Onondaga County demonstration facilities on Maltbie Street and West Newell Street, Syracuse, N.Y.; (3) to investigate the use of bacteriophage indicators as standards for Aquella performance and efficiency; and (4) to systematize the survey methods for animal viruses in CSO.

A second phase of the project was conducted at the Maltbie Street treatment facility during the summer and fall of 1976 with the following objectives: (1) to compare Cl_2 and ClO_2 for their relative efficiency in disinfecting CSO seeded with indicator viruses; (2) to determine if a combined application of Cl_2 and ClO_2 provided enhanced activity; (3) to determine if the treatment reduced the wild type enteroviruses naturally occurring in CSO; and (4) to study the correlation between reduced seed virus titers and reduced enterobacterial counts.

Results of these virus investigations are presented in Section 13.

West Newell Street Facilities

At the West Newell Street site, experiments on treatment of CSO were to be investigated with the use of the swirl regulator/ solids concentrator with pre-and post-disinfection by Cl_2 and ClO_2.

Swirl Regulator/Solids Concentrator --

The swirl regulator/concentrator operates with relatively little mechanical equipment, thus providing a marked contrast in operating and maintenance expenses when compared to other primary treatment devices. During an overflow condition, swirling action produced by the momentum of flow into the swirl chamber and the geometry of the tank causes solid particles suspended in the flow to move toward the outside of the swirl chamber and toward the bottom, where they are removed through a foul sewer line. Approximately 20 overflow events were to be investigated for removal of SS and BOD_5 by swirl concentration.

A series of reports from 1967 through 1972 (12, 13, 33, 34, 35, 36) outlined considerations and developed parameters for design of the swirl regulator/concentrator. In general, these reports advanced two design approaches. The first approach was developed from the results of hydraulic model studies conducted by the LaSalle Hydraulic Laboratory Ltd. (Laboratoire d'Hydraulique), LaSalle, Quebec, Canada. The second design approach was the product of mathematical modeling calibrated with experimental data from the LaSalle model study. The mathematical model was developed by the Re-Entry and Environmental Systems Division of General Electric Company.

The physical model design approach used anticipated flow to the swirl chamber as the basis for deriving design dimensions. The mathematical model used solids concentrations, specific gravity of solids, and particle size distribution to determine design dimensions.

Although some disparity existed between the design dimensions obtained from the two approaches, it was considered to be outside the scope of this study to evaluate their relative merits, particularly since neither approach had been previously proven on the prototype scale. The LaSalle model approach (13) was chosen to provide the design dimensions for the swirl chamber at the West Newell Street facility.

Disinfection --

The basic objective of the disinfection experiments at the West Newell Street facility was to develop data on required dosages after treatment by the swirl regulator/concentrator. Provisions were made for injection of disinfectant to that portion of flow spilling over the weir (post-disinfection) as well as to the total flow entering the swirl chamber (pre-disinfection), to take advantage of the mixing action inherent in the swirl. During the initial operation of the facility, an Englehard Chloropac sodium hypochlorite generating system (Englehard Industries Division, Englehard Minerals and Chemicals Corporation, East Newark, N.J.) was to be installed. The system was to utilize an existing brine supply as its influent, and through the use of Englehard's electrolytic process, sodium hypochlorite was to be generated, stored on site, and fed to the swirl concentrator effluent during overflow conditions. An evaluation of the use of sodium hypochlorite in the two-stage disinfection process was to be made.

ClO_2 was to be generated on-site by means of a Nitrosyl Chloride generation system (U.S. Patent 3754079, Chemical Generators, Inc., Rochester, N.Y.).

Various combinations of pre- and post-addition of disinfectants were to be investigated. Table 1 indicates the various combinations that were proposed for field application.

TABLE 1. WEST NEWELL STREET DISINFECTION EVALUATION SCHEDULE

	ClO_2 (mg/l)		Cl_2 (mg/l)	
Trial	Pre	Post	Pre	Post
1		12		
2	15			
3	15	2		
4				25
5			8	20
6		4	8	

SECTION 5

PRE-CONSTRUCTION STUDIES

PRELIMINARY MONITORING

General

The activities involved in evaluations of high-rate treatment of CSO in Syracuse have been divided into pre-construction and post-construction studies. The pre-construction studies included a preliminary monitoring program and bench-scale investigations.

In order to establish a rational basis for designing full-scale demonstration facilities, a one-year sampling and monitoring program was conducted at three selected overflow sites. Selection was based on size of outfall and associated drainage area, accessibility for sampling, and availability of sufficient land to accommodate the required monitoring structures. The sites selected were the overflows at Maltbie Street, Rowland Street and West Newell Street.

A monitoring program was established at these sites to collect background data on water quality characteristics, flowrates and volumes, and rainfall intensities. Figure 2 shows a typical overflow event being monitored at Maltbie Street. The water quality parameters that were determined during the preliminary monitoring program are presented in Table 2.

The geometric mean has been used in the statistical data analysis presented in this section in preference to the arithmetic mean. It is felt that for most design purposes, it is more appropriate to design on the basis of modal conditions, since by definition these conditions will be most frequently encountered in actual operation. Most natural phenomena are zero-limited and have a right-skewed distribution. For this type of distribution, the geometric mean is closer to the modal value than the arithmetic mean. In addition, 'outliers' which occur in the measurement of natural phenomena tend to occur to the right of the median value, and therefore further emphasize the right-skewness of the distribution. This effect is reduced by use of the geometric mean, since the geometric mean is always to the left of the arithmetic mean, and is closer to the mode than the arithmetic mean.

TABLE 2. PRELIMINARY MONITORING - WATER QUALITY PARAMETERS

Parameters	
Flow	Ammonia Nitrogen (NH_3N)
Rain Intensity	Nitrates (NO_3)
pH	Nitrites (NO_2)
Total Coliform (TC)	Settleable Solids (Sett-S)
Fecal Coliform (FC)	Total Solids (TS)
Fecal Streptococcus (FS)	Volatile Solids (VS)
Adenosine Triphosphate (ATP)	Suspended Solids (TSS)
Alkalinity (T-Alk)	Volatile Suspended Solids (VSS)
Total Inorganic Phosphorus (TIP)	Dissolved Solids (TDS)
Chemical Oxygen Demand (COD)	Chlorides (Cl)
Biochemical Oxygen Demand (BOD_5)	Calcium (Ca)
Total Organic Carbon (TOC)	Magnesium (Mg)
Oil and Grease (O&G)	Sodium (Na)
Organic Nitrogen (Org N)	Potassium (K)

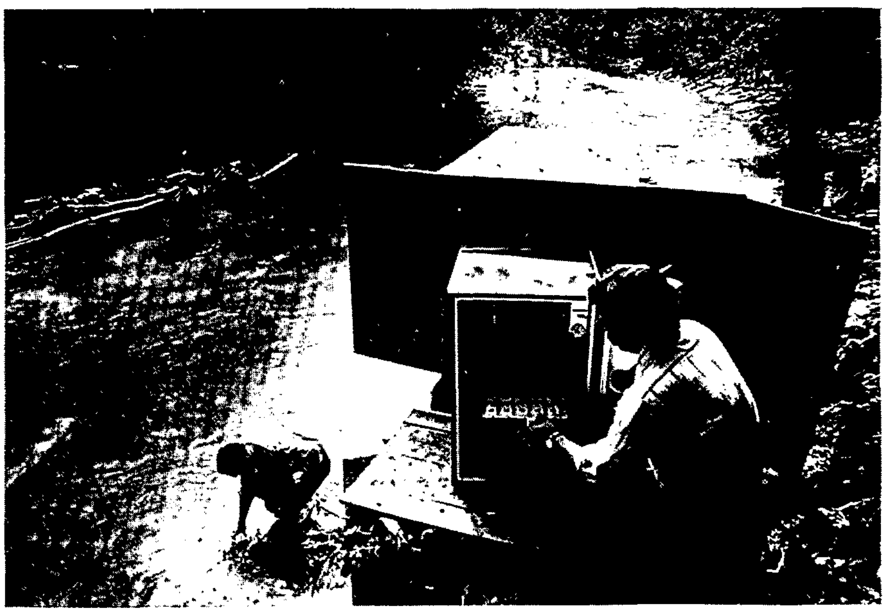


FIGURE 2. Typical Overflow Event Being Monitored at Maltbie Street

The geometric mean is defined as the nth root of the product of n values. The spread factor is defined as the antilog of the standard deviation of the logs of the n values. Expressed mathematically, these definitions are as follows:

$$\text{Geometric mean (G)} = (X_1 \ldots X_n)^{1/n}$$

$$\text{Spread factor} = \text{antilog} \left[\frac{\sum_{i=1}^{n} \left[(\log x_i - \log G)^2 \right]}{n-1} \right]^{\frac{1}{2}}$$

These parameters, for a right-skewed distribution, are comparable to the arithmetic mean and standard deviation of a normal distribution.

Subsequently, Maltbie Street was selected as the site for demonstration of rotary screening for solids removal and high-rate disinfection of bacteria and viruses, and West Newell Street was chosen for demonstration of a swirl regulator/concentrator for solids removal and high-rate disinfection of bacteria.

Summary of Results

During the preliminary phase of the project, the quality and quantity of overflows at Maltbie Street, West Newell Street and Rowland Street were monitored. Tables 3 to 5 give the results of a statistical analysis performed on the data gathered at each of the sites. The drainage area tributary to the Maltbie Street overflow consists of approximately 115 acres (47 ha). Predominant land use is by commercial and light industrial establishments. The estimated population of the area is 1,350, based on the 1970 census. In addition, dry weather flow is contributed by industrial establishments with a total estimated population equivalent of 4,500. The combined sewer in West Newell Street drains about 54 acres (22 ha) of medium-density residential area. The major sources of stormwater are runoff from roof drains and overland runoff. The estimated population of the tributary area is 1,200. In addition, dry-weather flow is contributed by commercial establishments with a total estimated population equivalent of 300. The drainage area tributary to the Rowland Street overflow consists of 125 acres (50 ha), primarily medium-density residential. The population of the area is approximately 2,800. Statistical analysis of data collected from Onondaga Creek at Dorwin Avenue (upstream of the overflows) and at Spencer Street (downstream of most overflows) is shown in Tables 6 and 7.

As can be seen in Tables 3 to 5 the numbers of total coliforms discharged during periods of wet weather were much higher from the residential land use areas of West Newell and Rowland Streets than from the commercial/industrial area of Maltbie Street, by a factor of two to five times. Fecal coliform (FC) counts from the residential areas were three to three and one-half times as high as from the commercial/industrial area. Fecal streptococcus (FS) counts were inconclusive, with values

TABLE 3. COMBINED SEWER OVERFLOW QUALITY - MALTBIE STREET OVERFLOW

Parameter	No. Points	Geometric Mean	Spread Factor	Upper Confidence Level 95.0 Percent	Upper Confidence Level 99.5 Percent
Overflow Volume, MG	9	0.26×10^6	7.28	6.81×10^6	43.58×10^6
Total Coliform count/100 ml	189	6.38×10^5	7.36	0.17×10^8	1.10×10^8
Fecal Coliform count/100 ml	166	1.25×10^5	10.98	0.64×10^7	6.05×10^7
Fecal Strep count/100 ml	181	3.76×10^4	3.60	3.09×10^5	10.26×10^5
BOD_5, mg/l	151	27	3.60	226	747
TOC, mg/l	224	26	2.45	112	259
SS, mg/l	173	159	2.40	672	1526
VSS, mg/l	113	40	3.09	255	733
TKN, mg/l	226	3.09	1.88	8.70	15.67
NH_3N, mg/l	228	0.72	2.16	2.58	5.30
OrgN, mg/l	261	2.03	2.12	6.98	14.08
NO_2N, mg/l	263	0.07	2.21	0.27	0.57
NO_3N, mg/l	259	0.40	3.22	2.77	8.27
T-IP, mg/l	262	0.56	3.02	3.46	9.72
COD, mg/l	258	34	2.45	150	347
TDS, mg/l	203	289	1.95	868	1622
Cl, mg/l	260	42	2.14	145	296
T-Alk, mg/l	258	100	2.06	328	645

Samples obtained during 15 storm periods.

TABLE 4. COMBINED SEWER OVERFLOW QUALITY - WEST NEWELL STREET OVERFLOW

Parameter	No. Points	Geometric Mean	Spread Factor	Upper Confidence Level 95.0 Percent	Upper Confidence Level 99.5 Percent
Overflow Volume	10	2.00×10^4	6.45	4.29×10^6	2.45×10^6
Total Coliform count/100 ml	117	13.39×10^5	54.89	9.74×10^8	411.92×10^8
Fecal Coliform count/100 ml	94	4.67×10^5	27.60	10.96×10^7	242.76×10^7
Fecal Strep count/100 ml	101	1.44×10^4	15.71	13.35×10^5	173.37×10^5
BOD_5, mg/l	129	59	1.87	166	297
TOC, mg/l	172	69	1.65	157	251
SS, mg/l	154	87	2.88	498	1342
VSS, mg/l	152	48	2.85	269	718
TKN, mg/l	171	10.24	1.78	26.56	45.65
NH_3N, mg/l	171	4.91	1.71	11.90	19.68
OrgN, mg/l	189	3.55	3.07	22.53	64.36
NO_2N, mg/l	188	0.06	2.68	0.31	0.78
NO_3N, mg/l	161	0.20	3.63	1.66	5.56
T-IP, mg/l	177	1.51	2.48	6.74	15.76
COD, mg/l	190	85	2.86	498	1278
TDS, mg/l	134	708	2.53	3261	7770
Cl, mg/l	190	81	4.28	883	3443
T-Alk, mg/l	190	197	2.86	1108	2959

Samples obtained during 10 storm periods.

TABLE 5. COMBINED SEWER OVERFLOW QUALITY - ROWLAND STREET SITE

Parameter	No. Points	Geometric Mean	Spread Factor	Upper Confidence Level 95.0 Percent	Upper Confidence Level 99.5 Percent
Overflow Volume, MG	16	248,000	3.22	1.70×10^6	5.07×10^6
Total Coliform count/100 ml	264	34.02×10^5	10.57	1.65×10^8	14.94×10^8
Fecal Coliform count/100 ml	249	4.46×10^5	13.74	3.32×10^7	38.49×10^7
Fecal Strep count/100 ml	243	8.23×10^4	4.06	8.27×10^5	30.69×10^5
BOD_5, mg/l	242	29	2.05	96	188
TOC, mg/l	319	28	2.12	95	191
SS, mg/l	274	80	2.79	431	1126
VSS, mg/l	265	38	2.78	202	526
TKN, mg/l	312	4.50	1.97	13.73	25.87
NH_3N, mg/l	312	2.33	2.31	9.20	20.11
OrgN, mg/l	348	1.53	2.71	7.88	20.04
NO_2N, mg/l	352	0.07	2.35	0.27	0.61
NO_3N, mg/l	338	0.35	2.52	1.58	3.73
T-IP, mg/l	350	0.79	3.09	5.04	14.44
COD, mg/l	350	32	2.17	114	235
TDS, mg/l	253	326	2.00	1018	1944
Cl, mg/l	346	59	2.90	341	921
T-Alk, mg/l	345	117	1.92	342	628

Samples obtained during 18 storm periods.

for residential areas both less than and greater than the recorded values for the commercial/industrial area.

The West Newell Street overflow contained the highest concentrations of BOD_5, TOC, TKN and NH_3N as indicated in Table 4. The geometric mean BOD_5 at West Newell Street was approximately twice that for either Rowland Street or Maltbie Street. Similar comparisons of TOC values show West Newell Street to be twice as high in TOC as either Maltbie or Rowland Streets. These results tend to indicate that the CSO at West Newell Street is much higher in organic matter than either of the other two sites monitored. This conclusion is further supported by examination of the TKN and NH_3N data where, again, the geometric mean is significantly higher at West Newell Street than at either Maltbie Street or Rowland Street. The relatively high values found for the West Newell Street overflow may be due to a greater rate of dry-weather organic deposition in the West Newell Street tributary area, which has relatively flat sewers.

The volatile suspended solids (VSS) at the three overflows were roughly equivalent. However, the level of fixed suspended solids (SS-VSS) at Maltbie Street was approximately three times that at either West Newell Street or Rowland Street, indicating the presence of relatively large amounts of grit in the Maltbie Street overflow.

In general, grit particles can be expected to settle out in transporting conduits more readily than lighter organic material. In a commerical/industrial section such as the Maltbie Street area, relatively greater amounts of grit would be expected to enter the sewer system and settle out. When a storm occurs, the first flush picks up these inert materials and transports them to the outfall, resulting in a relatively higher solids loading. In contrast, in residential sections, such as the West Newell Street and Rowland Street areas, lesser amounts of gritty material are deposited in the transporting conduits during dry weather. This effect is seen in a comparison of the ratio of fixed suspended solids to total suspended solids for Maltbie Street (0.75), vs. West Newell Street (0.45) or Rowland Street (0.53).

Tables 6 and 7 demonstrate that there is considerable deterioration of the quality of Onondaga Creek as it passes through the City of Syracuse. Substantial increases are seen in BOD_5, TOC, SS, VSS, TKN, Org-N, and chlorides. Relatively insignificant changes are seen in nitrites, nitrates, TIP, COD, and alkalinity. CSO are unquestionably major sources of contamination to the creek and lake, although the total contribution cannot be known without comprehensive monitoring of the overflows.

Table 8 lists the quantities of pollutants discharged from the CSO at Maltbie Street, West Newell Street and Rowland Street, in terms of pounds per acre-inch (kg/ha-cm) of runoff. Quantitatively, Maltbie Street discharged the greater quantity, 268,000 gal (1,020 m^3) per overflow event. Rowland Street discharged a mean of 248,000 gal (940 m^3) per event and West Newell Street discharged a mean of 20,000 gal (75 m^3) per event. These overflow quantities are consistant with the rainfall data

TABLE 6. WET-WEATHER ONONDAGA CREEK QUALITY UPSTREAM OF CSO's AT DORWIN AVENUE

Parameter	No. Points	Geometric Mean	Spread Factor	Upper Confidence Level 95.0 Percent	Upper Confidence Level 99.5 Percent
Total Coliform Count/100 ml	10	1.9×10^5	15.16	1.64×10^7	2.11×10^8
Fecal Coliform count/100 ml	10	2.5×10^3	13.02	1.70×10^5	1.88×10^6
BOD_5, mg/l	2	24	-	-	-
TOC, mg/l	111	11	1.54	22.3	33.4
SS, mg/l	88	82	1.80	217	377
VSS, mg/l	84	15	2.04	47	93
TKN, mg/l	89	0.74	1.63	1.65	2.61
NH_3N, mg/l	89	0.27	1.69	0.65	1.06
OrgN, mg/l	79	0.46	1.99	1.42	2.70
NO_2N, mg/l	112	0.03	1.49	0.05	0.07
NO_3N, mg/l	112	0.50	1.89	1.41	2.56
T-IP, mg/l	98	0.18	3.25	1.28	3.85
COD, mg/l	110	7.8	2.28	30	65
Cl, mg/l	112	47	1.54	95	142
T-Alk, mg/l	112	205	1.18	269	314

Samples obtained during 5 storm periods.

TABLE 7. WET-WEATHER ONONDAGA CREEK QUALITY DOWNSTREAM OF CSO'S AT SPENCER STREET

Parameter	No. Points	Geometric Mean	Spread Factor	Upper Confidence Level	
				95.0 Percent	99.5 Percent
Total Coliform Count/100 ml	11	4.97×10^5	2.74	2.61×10^6	6.70×10^6
Fecal Coliform count/100 ml	2	3.4×10^4	127.95	9.94×10^7	9.28×10^9
BOD_5, mg/l	2	128	-	-	-
TOC, mg/l	147	16.4	1.58	34.7	53.2
SS, mg/l	121	144	2.23	538	1138
VSS, mg/l	116	21	2.56	100	242
TKN, mg/l	125	1.08	1.48	2.07	2.99
NH_3N, mg/l	124	0.40	1.72	0.97	1.60
OrgN, mg/l	116	0.64	1.94	1.90	3.53
NO_2N, mg/l	147	0.04	1.89	0.10	0.19
NO_3N, mg/l	147	0.36	1.45	0.66	0.94
T-IP, mg/l	140	0.20	2.20	0.74	1.55
COD, mg/l	147	9.6	2.41	41	93
Cl, mg/l	147	101	1.68	237	384
T-Alk, mg/l	147	189	1.24	269	328

Samples obtained during 4 storm periods.

TABLE 8. QUANTITIES OF CSO POLLUTANTS DISCHARGED

Paramater	Maltbie Street Geometric* Mean	Maltbie Street Spread Factor	West Newell Street Geometric* Mean	West Newell Street Spread Factor	Rowland Street Geometric* Mean	Rowland Street Spread Factor
BOD_5	6.22	3.60	13.36	1.87	6.57	2.05
TOC	5.79	2.45	15.63	1.65	6.34	2.12
SS	36.00	2.40	19.71	2.88	18.12	2.79
VSS	9.06	3.09	10.87	2.85	8.61	2.78
TKN	0.70	1.88	2.32	1.78	1.02	1.97
NH_3N	0.16	2.16	1.11	1.71	0.53	2.31

*Expressed in lbs/ac-in.

Conversion: 1 lb/ac-in.= 0.441 kg/ha-cm

obtained for the storm events at each site. The average rainfall intensity was approximately 0.20 in./hr (0.51 cm/hr) with an average peak intensity of 1.00 in./hr. (2.5 cm/hr). At West Newell Street the average rainfall intensity for monitored storms was approximately 0.07 in./hr. (0.18 cm/hr) with an average peak intensity of 0.29 in./hr. (0.74 cm/hr). At Rowland Street the average rainfall intensity was 0.10 in./hr (0.25 cm/hr) with an average peak intensity for all monitored storms of 0.56 in./hr (1.42 cm/hr). The difference in rainfall intensities between sites considered together with the effective runoff areas (total rainoff area x runoff coefficient) of 63 acres (25.5 ha) at Maltbie Street, 53 acres (21.4 ha) at Rowland Street, and 18 acres (7.3 ha) at West Newell Street account for the Maltbie Street overflow discharging the largest volume of overflow event and West Newell Street discharging the least volume of overflow/event.

The West Newell Street overflow had the higher contaminant levels in terms of pounds per acre-inch as indicated in Table 8. As noted earlier, the higher contaminant levels are believed to be due to a greater rate of dry-weather organic deposition in the relatively flat trunk sewer serving the West Newell Street tributary area.

In the analysis of selecting sites for full scale demonstration of the swirl regulator/concentrator, the Rowland Street site was not considered feasible based on operational expenses, cost estimates and collected data. Therefore the West Newell Street site was chosen for demonstration of the swirl unit.

Subsequent to this demonstration study, a comprehensive CSO monitoring program was conducted in Onondaga County to determine the variability in quality and quantity of the CSO from individual drainage areas in the combined sewer system. For informational purposes only, Tables 8a and 8b present summary data for selected parameters as obtained in the monitoring program, details of which are available in the referenced report.

BENCH SCALE STUDIES

A detailed literature search revealed that very little data was available to establish a basis for design and operation of prototype high-rate treatment facilities. A series of bench-scale studies were conducted prior to preliminary design in order to obtain this information. The major conclusions and recommendations of the bench-scale studies are summarized in this section. Detailed information about the bench-scale studies is presented in a separate report (7).

The bench-scale study resulted in several tentative conclusions, contingent on verification in the full-scale demonstration of prototype treatment facilities at Syracuse. The degrees of mixing that occur in full-scale facilities could not be simulated on a bench scale. Complete mixing was therefore used in the bench-scale study, and the conclusions of that study are based on complete mixing. The following conclusions were made.

TABLE 8a. SUMMARY OF QUALITY DATA FOR SELECTED PARAMETERS ON A SITE-BY-SITE BASIS IN ONONDAGA COUNTY +

SITE NO.	Geometric Mean of Selected Parameters					
	SS	VSS	BOD_5	TKN	TIP	FC
003	573	207	104	2.74	0.32	1,548,760
004	283	67	43	1.20	0.14	3,108,610
014	262	93	53	2.28	0.27	1,583,510
015	148	40	10	2.27	0.17	1,950,460
020	125	62	97	3.21	0.44	2,226,620
021	224	82	52	1.67	0.16	1,078,920
022	564	133	45	0.35	0.07	77,549
025	13	3	110	21.70	1.60	32,011
026	179	103	44	1.08	0.18	594,155
027	392	247	116	3.45	0.56	-
028	4	1	-	-	-	10,000
029	177	59	53	1.18	0.15	626,230
030	367	152	91	5.17	0.64	1,284,940
031	664	67	219	2.16	0.15	-
033	544	110	44	5.97	0.29	-
034	103	61	23	1.62	0.23	4,157,600
035	134	84	92	-	0.31	999,987
036	563	283	130	5.95	0.44	1,201,530
037	53	47	18	1.38	0.50	56,322
039	139	76	60	2.74	0.36	1,271,180
040	47	23	50	9.11	1.07	999,976
042	189	99	74	5.86	0.46	4,795,000
043	443	153	87	5.63	0.40	1,845,580
044	333	141	51	2.27	0.23	2,632,920
046	216	92	118	3.30	0.28	1,325,280
051	472	119	38	2.20	0.26	1,228,130
052	442	156	55	3.38	0.66	1,437,630
058	316	66	12	0.11	0.07	585
059	-	-	-	-	-	-
060	197	65	56	2.06	0.29	622,433
063	633	105	48	3.46	0.21	80,738
073	198	86	35	0.70	0.16	222,199
074	188	40	23	1.22	0.11	6,830,100
076	1,272	178	18	0.44	0.07	92,507
077	704	202	69	2.62	0.31	1,009,560
080	651	58	45	0.84	0.13	116,436

+ Adapted from: Progress Report, Combined Sewer Overflow Abatement Program, Department of Drainage and Sanitation, Onondaga County, New York. O'Brien and Gere Engineers. 1978.

TABLE 8b. ILLUSTRATION OF "FIRST-FLUSH" EFFECTS FROM ONONDAGA COUNTY CSO+

	SS			BOD_5			TKN			FC		
Site	0-30*	30-60*	60-90*	0-30	30-60	60-90	0-30	30-60	60-90	0-30	30-60	60-90
003	819	523	214	195	97	50	4.83	1.58	1.47	1,322,022	2,773,920	2,699,414
004	630	210	99	102	25	19	1.27	0.76	0.67	3,346,813	-	2,778,559
014	580	333	179	142	62	43	4.00	1.88	2.03	3,474,542	963,126	1,958,384
020	160	68	-	-	-	-	2.95	1.74	-	-	-	-
021	343	146	52	68	13	17	1.84	0.60	1.51	1,424,458	80,000	189,228
022	520	1370	-	38	31	-	0.32	0.28	-	67,954	2,106,644	-
026	163	133	220	56	36	37	1.08	1.02	1.27	592,682	590,591	590,055
027	527	389	215	171	105	71	4.49	2.78	2.90	-	-	-
029	223	207	83	76	54	21	1.40	1.16	0.98	717,863	939,065	491,224
030	498	420	413	135	125	90	6.88	5.89	3.70	1,326,850	2,217,945	1,285,137
031	515	-	-	219	-	-	2.16	-	-	26,671	-	-
034	307	259	53	129	54	15	2.62	1.40	1.41	8,825,250	2,800,000	2,234,643
036	546	484	456	128	181	88	5.35	6.39	4.65	936,215	7,955,378	4,802,279
039	170	186	180	75	63	57	3.59	3.74	2.97	970,900	1,381,639	1,063,662
042	261	473	192	74	162	80	6.19	6.80	5.11	5,118,182	4,411,049	3,692,996
043	566	295	318	109	67	49	5.02	7.63	5.03	2,003,141	1,620,059	924,979
044	369	447	276	134	42	33	3.40	2.36	2.37	3,576,146	2,135,924	4,097,124
052	795	534	330	151	46	40	5.85	2.24	2.57	4,686,115	3,500,000	3,600,000
058	316	-	-	12	-	-	0.11	-	-	585	-	-
060	500	116	59	91	42	29	3.42	1.68	0.98	951,358	656,175	277,346
063	919	450	391	63	27	27	5.17	3.22	1.67	79,015	119,642	55,807
073	228	357	207	39	33	20	0.81	0.82	0.65	228,876	644,000	303,926
074	313	148	104	26	28	17	1.02	1.34	1.46	8,843,971	5,948,316	4,370,282
076	1767	-	-	23	4	-	0.79	0.10	-	302,571	10,000	-
080	599	933	814	47	39	55	0.81	0.89	0.97	90,938	238,831	50,043

* 0-30 = First 30 minutes of overflow
30-60 = Second 30 minutes of overflow
60-90 = Third 30 minutes of overflow

+ Adapted from: Progress Report, Combined Sewer Overflow Abatement Program, Department of Drainage and Sanitation, Onondaga County, New York. O'Brien & Gere Engineers. 1978.

1. Total coliform (TC) bacteria in simulated combined sewer overflows (SCSO) were reduced to the target levels of 1,000 colonies per 100 ml in two minutes by the following disinfectant dosages:

   a. 25 mg/l Cl_2 (only 50 percent of all trials)

   b. 12 mg/l ClO_2

   These same conditions reduced FC and FS bacteria to 200 colonies per 100 ml in two minutes.

2. These same conditions also achieved five log reductions in poliovirus-1 and ØX174 coliphage. Although target levels of viruses are not specified as part of federal or state water quality effluent criteria, five log reductions in virus populations would reduce the highest anticipated viral counts in actual overflows essentially to zero.

3. High-rate treatment by microscreening, followed by disinfection, is a feasible method of reducing microbial contamination of CSO to an acceptable level.

4. The enhanced disinfection by using two-stage (sequential) addition of ClO_2 followed by Cl_2 in 15 to 30 sec may be due to the regeneration of ClO_2 through the interaction of chlorite ion (ClO_2^-) and Cl_2.

5. There is no enhancement of disinfection beyond the expected additive effects when sequential addition of the same disinfectant is practiced.

6. On a weight basis, ClO_2 is approximately twice as effective as Cl_2 in reducing bacterial and viral populations to target levels.

7. In the disinfection of contaminated waters with Cl_2, the initial rapid disinfection is accomplished by free Cl_2, which is converted to the less potent combined Cl_2 species in one to two minutes. ClO_2 is converted to the less potent ClO_2^- in the same time period.

8. Microscreening had no measurable effect on high-rate disinfection with Cl_2 and only a slight positive effect with ClO_2. A possible explanation is that the increased rate of reaction between disinfectant and demands that resulted from the shredding of particulates upon screening offset the increased numbers of exposed bacteria to yield no net increase in disinfection.

9. Microscreening alone decreased SS, but in some cases increased BOD_5 and bacteria counts.

10. An advantage of microscreening is that only the smaller SS particles will pass through fine mesh screens thereby increasing the disinfectant penetration potential.

11. Cl_2 and ClO_2 demands can be attributed to different substances in wastewaters.

12. Within the dosages required for acceptable disinfection, Cl_2 and ClO_2 do not measurably change pH, TOC, BOD_5, COD, TKN or NH_3N.

13. The temperature variations associated with the northeastern climate had only a slight positive effect on disinfection of wastewaters with Cl_2 and ClO_2. This deviation from the large, positive temperature effects observed in no-demand waters is most likely due to a wide variety of competing chemical reactions that occur in wastewaters.

14. Microorganism aftergrowth was not observed to be a significant factor in this study. However, the results may be more a reflection of the difficulties in simulating the conditions for aftergrowth than aftergrowth itself.

15. Significant decreases in adenosine triphosphate (ATP) concentration that parallel bacterial reductions have been observed during the disinfection process. The results of ATP measurements point to the potential of using this indicator parameter as an effective means of measuring bacterial concentration after disinfection or controlling disinfectant dosages.

16. The effects of screening and temperature upon disinfection were difficult to observe because they were of similar magnitude as the variations in duplicate trials.

17. The order of resistance of bacteria to disinfection with Cl_2 and/or ClO_2 is FS > TC > FC.

As a result of the bench-scale studies, the following recommendations were made.

1. The results of the operation of the full-scale demonstration units be correlated with the bench-scale results to determine the validity of using these bench-scale studies for certain design parameters.

2. The role of SS in disinfection of CSO be evaluated on a full-scale to determine the effect of screening on the disinfection process.

3. The comparison of the effects of disinfection between the swirl regulator/concentrator and the various microscreens receive particular attention in the operation of the full-scale facilities in view of the conclusions about microscreening previously mentioned.

4. The full-scale facilities be operated in different seasons to evaluate the effects of temperature on disinfection.

5. The effect of chlorite (ClO_2^-) in receiving waters be investigated before the widespread use of ClO_2 is implemented.

6. ClO_2 be considered as a disinfectant pursuant to the previous recommendation.

7. The effects of mixing on high-rate disinfection be thoroughly investigated on a full-scale study.

8. Two-stage disinfection with Cl_2 and ClO_2 be investigated to determine the mechanism through and conditions under which enhanced disinfection occurs.

9. For bench-scale comparisons of the factors that affect the disinfection of CSO, a SCSO may be satisfactory, if SCSO is prepared properly.

10. The procedural difficulties in running bench-scale studies such as these must be recognized. The greatest care must be taken to preserve the intended experimental conditions and to maintain sample integrity.

11. Blending be adapted as a preliminary step in the bacteriological examination of waters that contain significant amounts of particulate matter. Because of the differences in individual blenders, a study of bacterial counts vs blending time should be performed to determine the optimum time for each model.

In general, the bench-scale studies indicated that high-rate treatment of CSO, consisting of solids removal followed by high-rate disinfection, is feasible. Although additional conclusions and recommendations were presented as a result of the bench-scale studies, those concerned only with the full-scale studies have been presented above. Bench-scale results were valuable in developing the full-scale prototype treatment facilities which are described in detail in Section 6.

SECTION 6

FACILITIES DESCRIPTION AND OPERATION

MALTBIE STREET FACILITY

Description of Drainage Area

The drainage area tributary to the Maltbie Street overflow consists of approximately 115 acres (46.5 ha) located west of Onondaga Creek. The principal land use is for commercial and light-industrial purposes. The tributary area which has an estimated population (based on the 1970 census) of 1,350, is served by approximately 15,500 ft (4,725m) of trunk and lateral sewers. The sewers convey sanitary and combined sewage to the main intercepting sewer via an 8 in.(20.3 cm) siphon under Onondaga Creek. Upstream from the siphon there is a diversion device, consisting of a side overflow weir, located in a manhole at the intersection of Leavenworth Avenue and Evans Street. The overflow from the diversion structure is a 30 in.(76.2 cm) diameter concrete pipe, which originally ran directly to the west bank of the creek. This overflow provides the combined sewage treated at the Maltbie Street facility.

An average time of concentration (including inlet time and transport time to the point of overflow) was determined for the Maltbie Street trunk sewer based on an inlet time estimated at 15 min. The actual time of concentration as observed from start of rainfall to start of overflow in this study varied between 20 to 45 min and averaged about 26.5 min. Each individually measured time of concentration varied with storm intensity. The basic physical characteristics of this site are listed in Table 9.

TABLE 9. MALTBIE STREET OVERFLOW CHARACTERISTICS

Drainage Area Characteristics		Overflow Outfall Characteristics
Size- 115 acres (46.5 ha)		Length - 3,571 ft (1088 m)
Runoff Coefficient - 0.55		Diameter - 30 in (76.2 cm)
Population-Tributary - 1,350		Slope - 0.0043
Industrial Population Equivalent	4,500	Inlet Time - 15 min
Total Population Equivalent	5,850	Transport Time - 5 to 30 min Time of Concentration - 20 to 45 min

Objectives and Implementation of Prototype Facilities

The primary objective of the facilities designed for installation at the Maltbie Street overflow was to demonstrate under field conditions the feasibility of high-rate, fine-mesh screening for solids removal and disinfection enhancement followed by high-rate disinfection. Secondary objectives were to:

1. Evaluate the relative performance of commercial screening units under similar hydraulic and solids loading conditions.

2. Investigate different means of high-rate disinfection.

3. Investigate the effect of solids removal on high-rate disinfection.

4. Investigate the effects of various mixing techniques on high-rate disinfection.

5. Investigate the impact of additional solids volumes from CSO treatment on existing treatment facilities.

These objectives, together with the physical characteristics of the site and the specific constraints discussed in Section 5, dictated the following major elements for implementation of the experimental plan:

1. A means of pumping controlled amounts of CSO to wet-weather treatment units.

2. Screening units, with parallel flow pattern.

3. Instrumentation to monitor flows and to vary flows to specific units.

4. Parallel disinfection units, with the flexibility to apply Cl_2, ClO_2 or both.

5. Piping to convey treated overflow to the creek.

6. Monitoring and sampling equipment to record data needed for evaluation of equipment performance.

7. Fresh water and electric power for system operation.

Facilities Installed

The prototype facilities installed at the Maltbie Street overflow for implementation of the experimental plan included the following major components:

- pumping station
- screens
    - Zurn Micromatic

Sweco Centrifugal Wastewater Concentrator
Crane Microstrainer

- disinfection basins
- control instrumentation
- electrical service
- fresh water piping
- chlorination equipment
- chlorine dioxide generating and dosing equipment
- influent and treated effluent piping

The overall configuration of the Maltbie Street facilities is shown in Figure 3, and Figure 4 illustrates the site location with the pumping station, screening building and effluent discharge depicted.

Pumping Station--

A pumping station was constructed at the Maltbie Street facility to convey overflow to the screening units. The pumping station structure was divided into three compartments: (1) an influent chamber; (2) a metering chamber; and (3) a wet well, and pump chamber as shown on Figure 5.

The influent chamber was equipped with a bar screen to remove coarse solids and an emergency bypass to allow bypassing of the total overflow in the event of pump failure or flows in excess of pumping capacity.

The metering chamber contained a 30 in. (76.2 cm) magnetic flowmeter (Brooks Instrument Division, Model 7100 Series, Emerson Electric Company, Hatfield, Pennsylvania) to measure the total flow entering the treatment facilities from the overflow regulator. A signal from the flowmeter activated a circuit to start the treatment units and sampling equipment.

The third chamber acted as a wet well for the pumping systems located above the chamber. The overall pumping scheme had three parallel pumping systems, each consisting of a 2.5 MGD (6.6 cu m/min) constant speed drive and a 2.5 MGD (6.6 cu m/min) variable speed drive.

The pumping systems provided either constant or variable flow to each of the three screening units. The variable speed pumps were activated by a level sensor in the wet well. Under constant flow conditions the variable speed pump was manually inactivated, and the constant speed pump was automatically started when a pre-determined wet well level was sensed.

Under variable flow operations, each pump combination operated in sequence. The variable speed pump was activated first, and as this pump approached maximum capacity, a relay was energized in the pump controller to activate the constant speed pump. Once the latter pump reached full speed, the variable speed pump equalized the pumping rate with incoming flowrate.

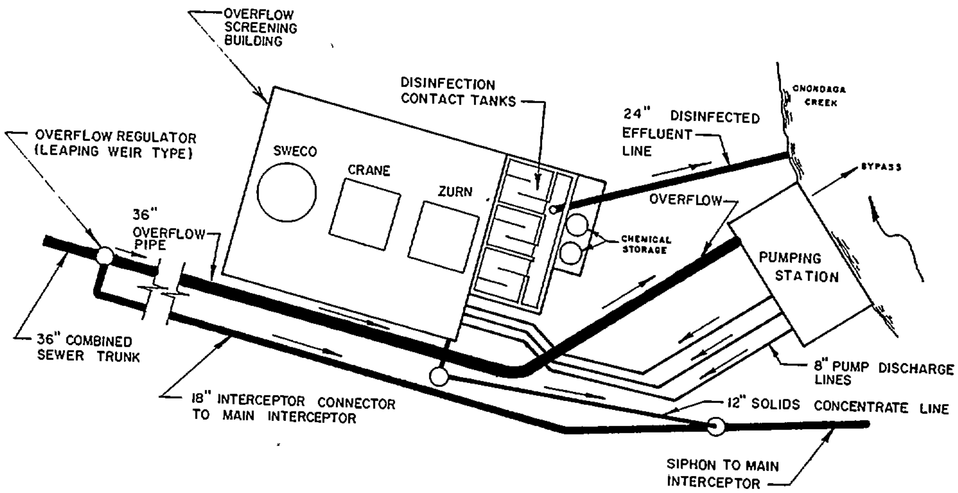


FIGURE 3. Maltbie Street Site Plan

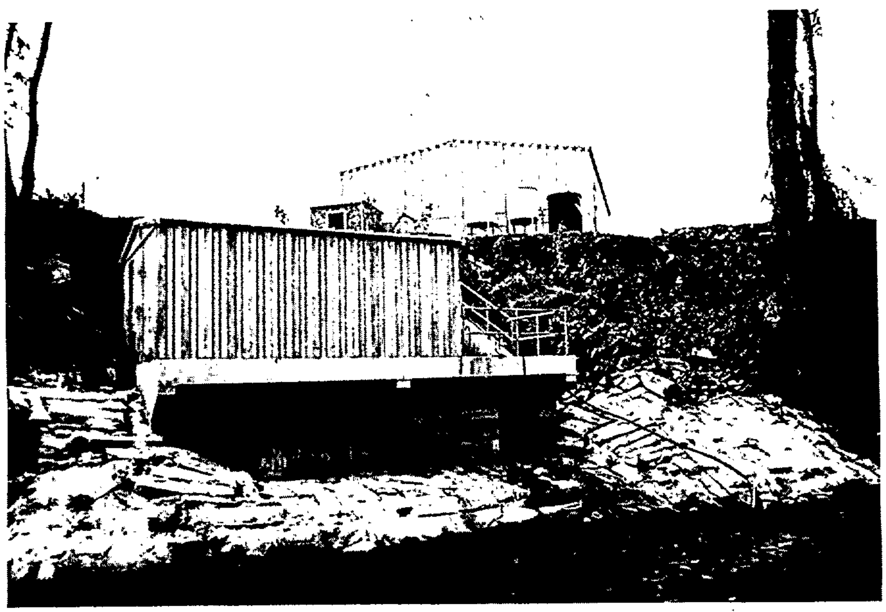


FIGURE 4. Maltbie Street Site Location. Pumping Station in Foreground, Overflow Screening Building in Background.

Control System--

The control system at the Maltbie Street Facility provided automatic activation of pumps, screens, disinfection equipment and sampling equipment.

At a predetermined level of flow, a signal from the 30 in. (76.2 cm) inlet flowmeter activated a current trip relay, which completed a 120 V AC power supply circuit. Elements of this circuit included the following:

1. Screening unit drives
2. 120 V AC electrical outlets for sampler operation
3. A time-delay relay to activate a 4-20 ma DC pump control circuit

A flowmeter on each of the pump discharges provided process control for the following equipment:

1. The chlorinator associated with each individual pump discharge
2. The chlorine dioxide feed pumps associated with each individual pump discharge.

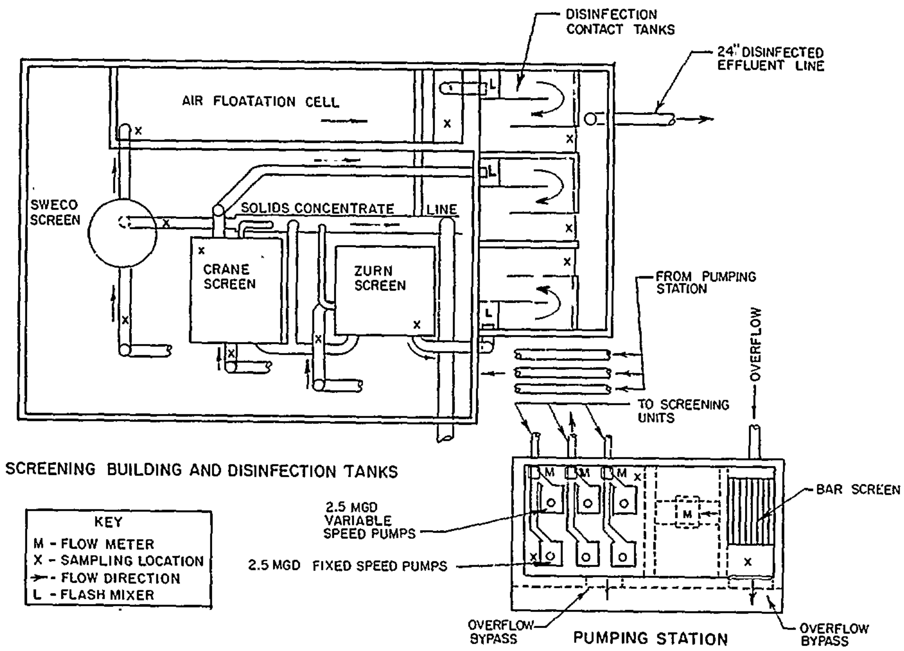


FIGURE 5. Maltbie Street Process Orientation

Signals from the inlet flowmeter and the individual flowmeters on the pump discharges were telemetered to the offices of O'Brien & Gere Engineers, Inc. Flows were also recorded on strip charts at the Maltbie Street facility.

Microscreens--

The three screening devices selected for demonstration at the Maltbie Street facility were:

1. Crane Microstrainer, manufactured by Crane Company - Cochrane Environmental Systems, King of Prussia, Pennsylvania.

2. Zurn Micromatic, manufactured by Zurn Industries, Inc., Erie, Pennsylvania.

3. Sweco Centrifugal Wastewater Concentrator, manufactured by Southwestern Engineering Company, Massilon, Ohio.

Operating characterics of the three units are listed in Table 10 and photographs of each presented in Figures 6 through 8, respectively.

The Crane Microstrainer was designed to provide a maximum of 5 MGD (13.1 cu m/min) at a hydraulic loading rate of approximately 40 gpm/ft^2 (98 m/hr) of submerged screen area. In 1973 a report (9) indicated that effective removal of suspended matter could be achieved by the microscreening process at hydraulic loading rates of 35 to 45 gpm/ft^2 (85 to 110 m/hr). Further study in 1974 (10) indicated that the latter flux were not achieved by the Crane unit but were limited to a maximum of 23 gpm/ft^2 (56 m/hr) for treatment of the CSO. However, after coagulent aids were added to the overflow throughput was increased to 39 gpm/ft^2 (95 m/hr). Since construction of the Syracuse facilities preceded the final 1974 study referenced above, the design flux of the Crane unit in Syracuse had been selected as 40 gpm/ft^2 (98 m/hr), or 1500 gpm. The Zurn unit design provided a maximum flux of 30 gpm/ft^2 (75 m/hr), or 2160 gpm (490 cu m/hr), utilizing 71 micron screens.

The Sweco unit was designed for hydraulic loadings up to 60 gpm/ft^2 (150 m/hr), or 3470 gpm (788 cu m/hr), based on Sweco's experience with similar units in the past. A 1974 study by the Ontario Ministry of the Environment (37) on a comparably-sized Sweco unit handling a treatment plant bypass during storm events resulted in 22 percent removal of total SS using 105 micron screens at a hydraulic loading rate of 66 gpm/ft^2 (160 m/hr).

The Crane and Zurn units both utilize drum screens rotating on a horizontal axis at a variable speed of 4.5 to 6.5 rpm, dependent on the head differential between influent and effluent at the screen (See Figure 9). These units are equipped with backwash jets which wash deposited solids off the inside of the screens, into washwater troughs and then to the sewer. Washwater consumption is normally 0.5 to 3 percent of the throughput flow.

The Sweco unit contains a series of screens attached to a cage which revolves around a vertical axis at 55 rpm. Flow enters the inside of the screen cage at the bottom and flows upward to a deflection plate at the top of the unit as schematically depected in Figure 10. Deflected

TABLE 10. MICROSCREEN OPERATING CHARACTERISTICS

Parameter	Crane	Zurn	Sweco
Drum Size	7.5 ft x 5 ft	6 ft x 6 ft	5 ft dia
Screen Aperture, microns	23	71	105
Screen Mesh	230	100	150
Rotating Speed, rpm	4.5 to 6.5	4.5 to 6.5	55
Design Flow, gpm**	3470	2160	1500
(cu m/hr)	790	490	340
Total Screening Area ft^2	94	108	25
(m^2)	8.7	10	2.3
Effective Screening Area, ft^2*	90	70	25
(m^2)	8.4	6.5	2.3
Design Hydraulic Loading Rate			
gpm/ft^2	40	30	60
(m/hr)	100	75	150
Backwash Volume, percent of inflow	0.5 to 3	0.5 to 3	0.5 to 3
Backwash Pressure lb/sq in.	40	30	80
(kg/sq cm)	2.8	2.1	5.6

* Effective Screening Area is the actual area of screen used to remove solids. In the case of the Zurn and Crane units, it is equivalent to the submerged area of screen.

**These design flowrates are based on the specified screen aperture and screen panel support material as obtained from the equipment manufacturers.

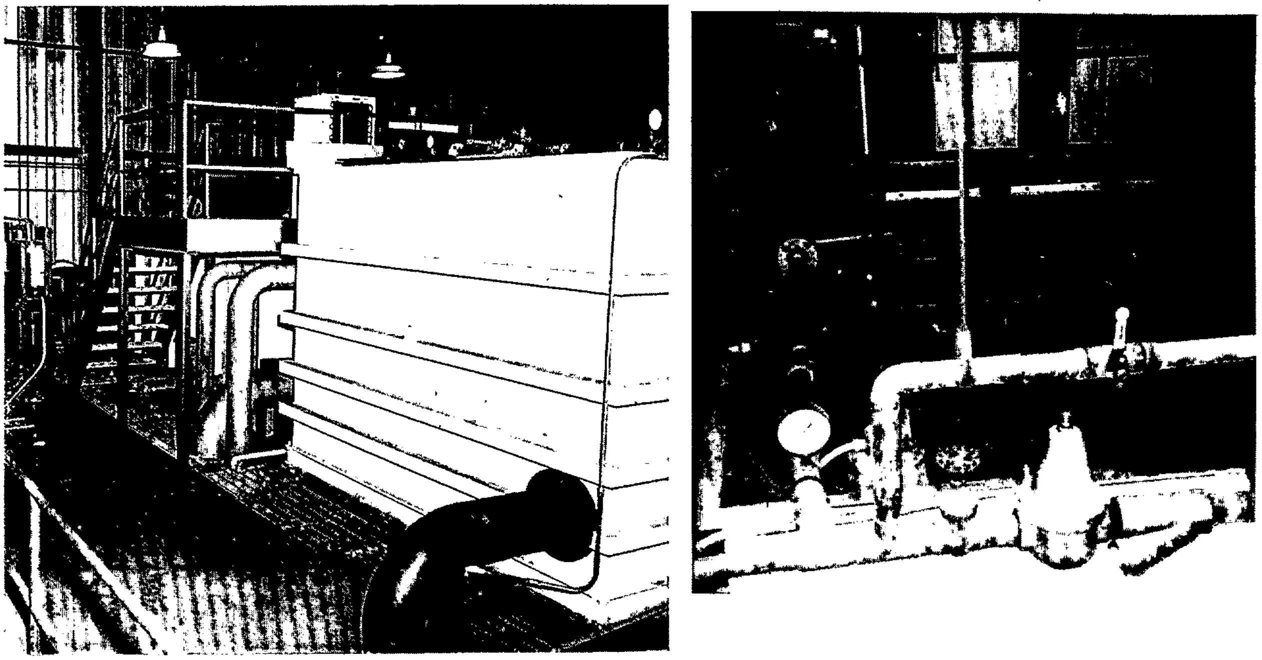


FIGURE 6. Crane Microstrainer

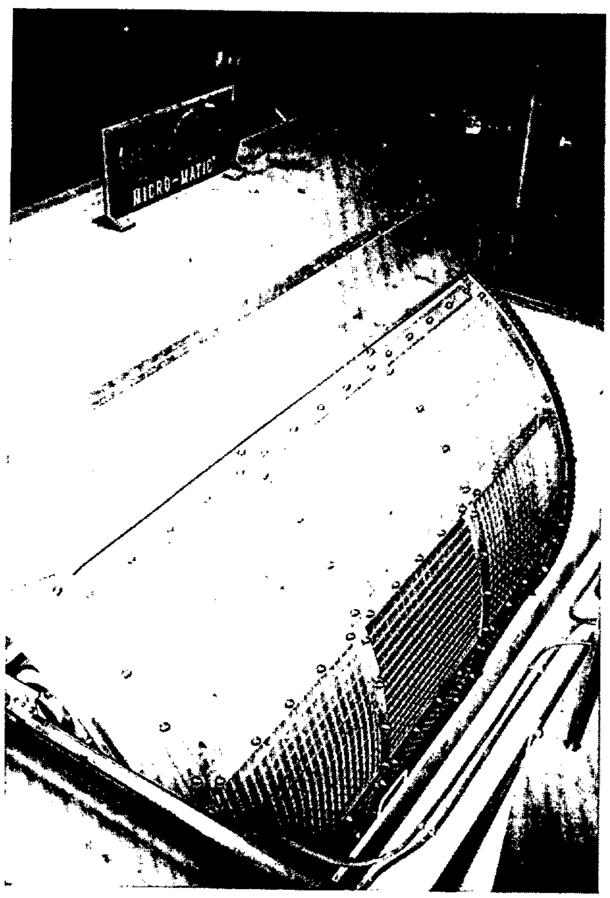


FIGURE 7. Zurn Micromatic

flow passes through the screens and is collected outside the cage. Solids which are entrapped on the screens, together with a fraction of the throughput, are withdrawn at the bottom of the cage and conveyed to the interceptor.

The Sweco Concentrator method of operation utilizes a flow "split" operation such that up to 25 percent of incoming flow is directed back to the dry-weather interceptor with the solids screened from the CSO, The Zurn and Crane units utilize only backwash water of up to 3.0 percent of the total throughput to the unit to carry the screened solids back to the interceptor.

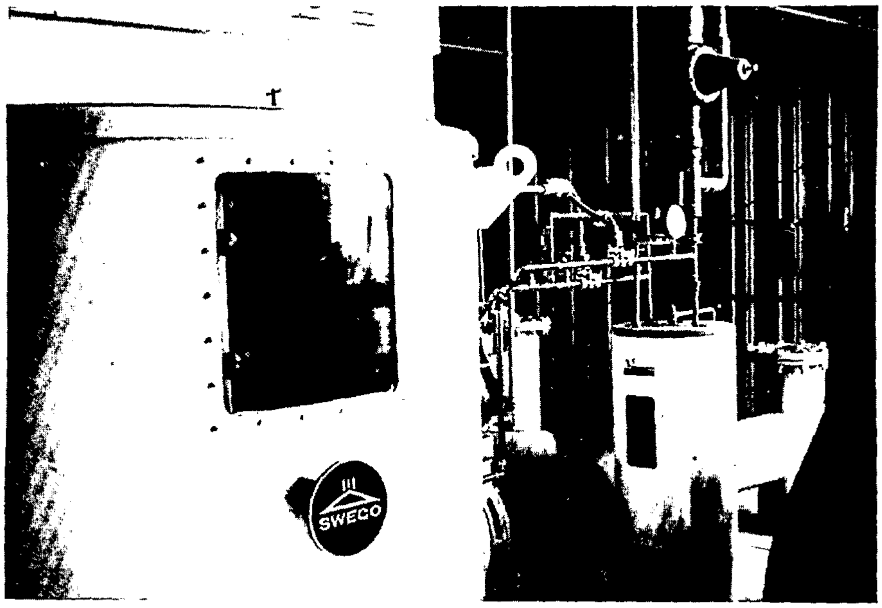


FIGURE 8. Sweco Wastewater Concentrator

By returning up to 25 percent of the incoming flow to the interceptor, the Sweco unit mass loading calculations will indicate removals of a portion of the solids even if the screens were not effective in filtering the solids, since the mass balance equation for this unit is:

$$Q_1C_1 = Q_2C_2 + Q_3C_3$$

where

Q_1 = influent flow to the unit, MGD (cu m/min)

C_1 = SS concentration of the influent flow, mg/l

Q_2 = effluent flow from the unit to the disinfection process, MGD (cu m/min)

C_2 = SS concentration of the effluent flow, mg/l

Q_3 = effluent flow from the unit returned to the interceptor, MGD (cu m/min)

C_3 = SS concentration of flow returned to interceptor, mg/l

If $C_1 = C_2 = C_3$, the overall efficiency of the unit in removing SS would be equal to $Q_3/Q_1 \times 100$, even though the concentration levels in each

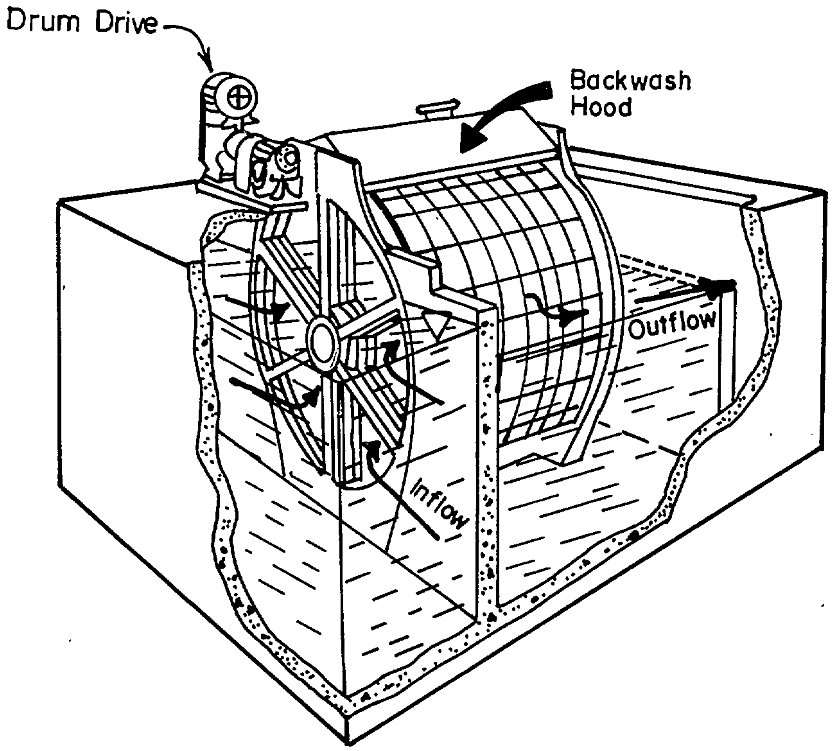


FIGURE 9. Schematic of Horizontal Shaft Drum Screen

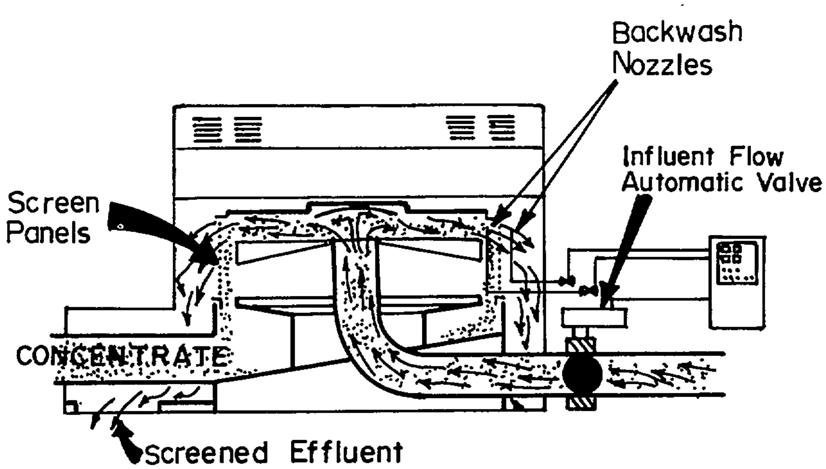


FIGURE 10. Schematic of Vertical Shaft Drum Screen

wastewater stream do not change. Thus, when comparing the SS removal efficiencies of the two units, the concentration removal efficiency should be considered.

However, when attempting to predict the effects of the concentrated SS being diverted to the dry-weather treatment facility, the mass loading removal efficiencies should be considered since the total poundage of SS returned to the interceptor could cause upsets in the operation of the dry-weather treatment facility during wet-weather periods. In addition, the evaluation must consider that many intercepting systems do not have the capacity to accept 25 percent of the total overflow from CSO treatment facilities. For example, during wet-weather events at Maltbie Street, the intercepting system has essentially zero capacity to handle the runoff from that drainage area.

The Sweco unit was followed by an air flotation cell to achieve further reduction of SS by taking advantage of dissolved air entrained in the wastewater during the screening process. The flotation cell was a 8 x 38 x 2 ft (2.4 x 11.6 x 0.6 m) concrete basin with a manually-operated scum collector for skimming off floating solids. Dimensions were as specified by the Sweco manufacturer and resulted in a surface loading rate of approximately 5 gpm/ft^2 (12.2 m/hr) at the design flow of 2.2 MGD (5.8 cu m/min).

The screens were operated during storm events in 1974, 1975 and 1976. Various operating and sampling problems effectively invalidated the 1974 screening data. These problems were resolved, and the analysis of results found in Section 8 is based on the 1975 and 1976 data.

During screen operation, the influent and effluent from each unit were sampled, in order to provide data to evaluate the relative performance of the microscreens and the effects of different solids levels on disinfection. The previous bench-scale studies did not result in any conclusive finding regarding the effect of solids on disinfection. Periodic analyses of BOD and of metals (Pb,Cr,Fe,Zn,Cu,Cd,Ni,Hg) were also performed.

The operation and maintenance requirements of each screening unit were determined. Data was accumulated on the ability of the units to restart after both long and short periods of operation, on the effectiveness of their backwash cycles, and on the durability of the screening material. Other screen operating parameters examined included power requirements, head losses and rotational speeds.

Disinfection Equipment--

Equipment was provided at the Maltbie Street facility for the addition of chlorine and chlorine dioxide to the effluent from each screening unit.

Chlorine gas was stored at the site in 150 lb (68 kg) cylinders and delivered to the wastewater flow by Fisher-Porter solution-feed, vacuum type gas chlorinators (Model 70C1751). The chlorination equipment is shown in Figure 11. It should be noted that the Cl_2 used in the bench-scale studies was obtained as a 5 percent solution of sodium hypochlorite (94.25 percent available Cl_2). It has been assumed that this difference in source of Cl_2 between the bench-scale and full-scale studies had no bearing on the results, since results in all cases were related to dosage of free Cl_2, regardless of source.

Chlorine dioxide was generated at the site by means of a Nitrosyl Chloride generation system (U.S. Patent 3754079) furnished by Chemical Generators, Inc., Rochester, New York. The process used in this system consisted of separately pumping equal amounts of sodium chlorate-sodium nitrite ($NaClO_3$-$NaNO_2$) slurry and nitric acid (HNO_3) into a lucite reaction chamber, in which they were mixed. The resulting reaction was expected to produce a 12 percent solution of ClO_2 which could be fed directly to a disinfection contact tank. The equipment used is shown in Figure 12. It should be noted that the Nitrosyl Chloride system involved a new and relatively unproven process. The state-of-the-art in ClO_2 technology prior to this project was that field generation of ClO_2 at the time of disinfection is necessary (since ClO_2 deteriorates rapidly in storage), and that large-scale, low unit cost methods for field generation are generally lacking. The ClO_2 used in the bench-scale studies was generated in small amounts according to the laboratory procedure then described in the 13th Edition of Standard Methods.

During operation, the rate of delivery of Cl_2 and/or ClO_2 to each disinfection basin was controlled by a 4-20 ma DC signal from a flowmeter on each of the screening unit pump discharges. Disinfectant strengths of Cl_2 and ClO_2 solutions were monitored in the field. Bacteriological and viral samples were taken before and after disinfection for analysis.

Particular attention was given to determining and controlling the strength of ClO_2 solutions produced, since the performance of the generation equipment was found to be erratic.

The disinfection contact facilities consisted of three parallel tanks with an approximate flowpath distance of 30 ft (9.1 m). The tanks were designed for a one-minute contact time at all flows which was provided for by use of a proportional weir on the downstream end of the disinfection tanks. This constant contact period facilitated comparisons of disinfection techniques.

In order to demonstrate the effectiveness of different mixing applications, experiments were attempted during the 1976 facility operations in which a common header from the Zurn unit to all three contact tanks was installed. This modification was to provide influent of virtually identical quality to each of the tanks, thus maximizing the validity of any comparison of mixing procedures. However, the amount of flow which could be passed through the Zurn unit screens was not sufficient to operate the three disinfection basins in parallel at design levels. These experiments were then performed by removing the screen panels of the Zurn unit and using unscreened CSO.

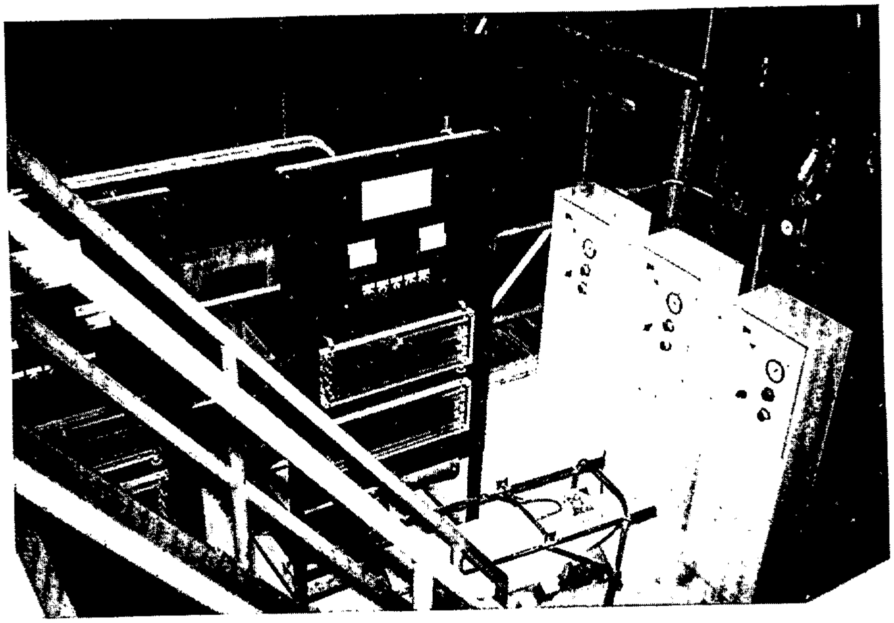


FIGURE 11. Maltbie Street - Chlorine Disinfection Equipment

FIGURE 12. Maltbie Street - Chlorine Dioxide Generators

One of the three contact tanks was equipped with two flash mixers, located at the point of disinfectant injection (tank entry) and at the downstream end (approximately 10 ft (3.05 m) of the first longitudinal baffle. Each of the other two tanks was equipped with a single flash mixer at the point of disinfectant injection. This allowed for comparison of single-flash mixing with sequential-flash mixing. Positive results had previously been observed by Kruse during a full-scale sewage treatment plant study of sequential flash mixing (38).

Previous pilot work by Glover (39) indicated that an increase in turbulence throughout the length of a contact tank increases the efficiency of the disinfection process. Glover's design objective in the high-rate pilot contact chamber was to achieve a GT value of 10,000 (GT is a unitless measure of mixing intensity where G is the velocity gradient in sec^{-1} and T is the contact time in sec). In order to maintain GT at a level of 10,000 without using long contact time, or a large tank size, G was increased by inserting corrugated, closely-spaced baffles in the tank parallel to the flow. In Glover's work a flash mixer was also used at the head of the contact tank.

WEST NEWELL STREET FACILITY

Description of Drainage Area

The drainage area tributary to the West Newell Street overflow consists of approximately 54 acres (21.9 ha) located within the City of Syracuse, east of Onondaga Creek. The area is primarily residential with only one major commerical establishment, a laundromat. The population of the area at the 1970 census was 1,200. The tributary area is served by approximately 6,400 ft (1,950 m) of combined sewers. All dry weather and combined sewage from the area is collected in a 24 in. (61 cm) dia trunk sewer and dry-weather flow is conveyed to the main intercepting sewer on the west bank of Onondaga Creek via an 8 in. (20.3 cm) siphon under the creek. Upstream from the siphon a diversion device directed excess storm flows into an overflow pipe. Table 11 lists the overflow characteristics of the West Newell Street site.

TABLE 11. WEST NEWELL STREET OVERFLOW CHARACTERISTICS

Drainage Area Characteristics		Overflow Outfall Characteristics
Size- 54 acres (21.9 ha)		Length - 2,202 ft (671 m)
Runoff Coefficient - 0.34		Diameter - 24 in (61 cm)
Population-Tributary - 1,200		Slope - 0.003
Industrial Population Equivalent	300	Inlet Time - 15 min
Total Population Equivalent	1,500	Transport Time - 5 to 30 min
		Time of Concentration-20 to 45 min

Objectives and Implementation of Prototype Facilities

The primary objectives of the facilities designed for installation at the West Newell Street overflow were:

1. To determine under field conditions the reliability and operating parameters of the swirl regulator/concentrator as a device for reduction of suspended solids in CSO.

2. To demonstrate the feasibility of a concept combining solids reduction by the swirl regulator/concentrator with disinfection to accomplish various CSO pollution abatement levels.

These objectives required the following major construction elements at the West Newell Street site:

1. Gravity diversion of all flow in the West Newell Street trunk sewer through the treatment facility.

2. Construction of a swirl regulator/concentrator.

3. Disinfection equipment.

4. A pump downstream of the swirl regulator/concentrator to provide pump-out of dry-weather flow.

5. Monitoring and sampling equipment to record data needed for evaluation of equipment performance.

6. Fresh water and electrical power.

Facilities Installed

The facilities installed at the West Newell Street overflow for implementation of the experimental prototype included the following major components:

- flow diversion
- swirl regulator/concentrator
- control instrumentation
- electrical service
- fresh water piping
- chlorine dioxide generating equipment
- effluent pumping

The site plan of the West Newell Street facilities is shown in Figure 13, and a photograph of the constructed facilities is shown in Figure 14.

Flow Diversion--

In order to divert combined sewage to the swirl regulator/concentrator, the upstream trunk sewer was intercepted and a 24 in. (61 cm) dia concrete diversion pipe was installed. An inflatable plug was placed in the

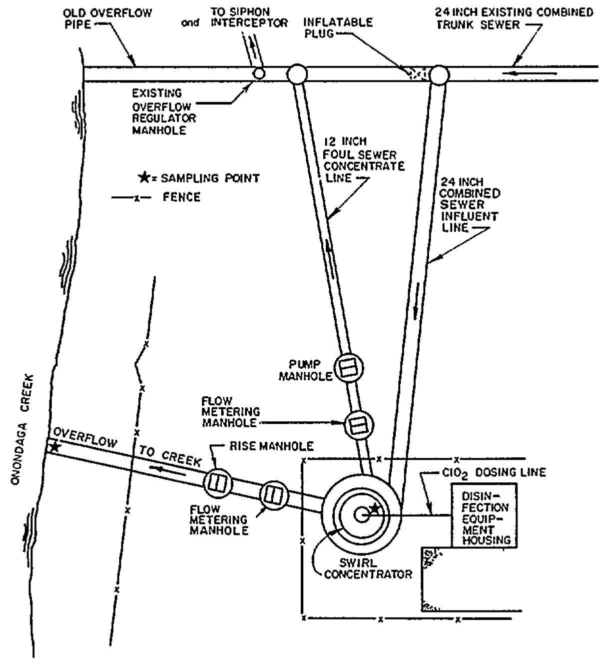
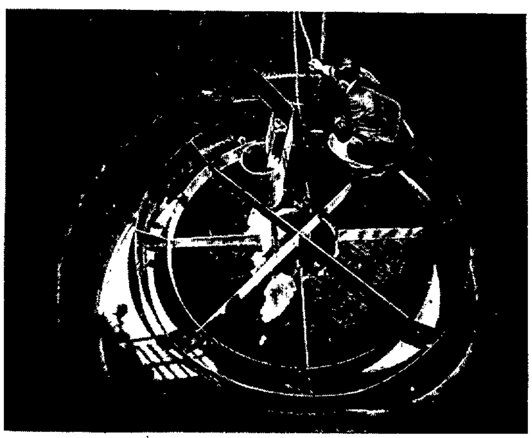


FIGURE 13. West Newell Street Site Plan

a. Site Location



b. Swirl Unit

FIGURE 14. West Newell Street Swirl Regulator/Concentrator

trunk sewer downstream from the new diversion manhole, to force flow into the diversion pipe.

The original overflow pipe was sealed off with sandbags. A new 24 in. (61 cm) dia overflow line was run from the downshaft of the swirl regulator/concentrator to Onondaga Creek. A 12 in. (30.5 cm) dia foul sewer (concentrate line) was installed between the swirl regulator/concentrator and the original trunk sewer in order to return solids removed from the CSO discharge to the sanitary intercepting sewer.

New manholes were constructed as follows:

1. A flow monitoring manhole was constructed in the new overflow line.

2. A rise manhole was constructed in the new overflow line so as to maintain full pipe conditions upstream, in order to provide for proper operation of the overflow flowmeter.

3. A flow monitoring manhole was constructed in the foul sewer.

4. A so-called "grit" manhole was constructed in the foul sewer, to a depth necessary to establish gravity flow through the swirl concentrator. A submersible pump was installed in the "grit" manhole to remove grit and dry-weather flow.

Swirl Regulator/Concentrator--

Design dimensions for the swirl regulator/concentrator were obtained by applying the design approach developed by the LaSalle Hydraulic Laboratory, LaSalle, P.Q., Canada (13). The design approach was based on anticipated flows to the swirl unit as the foundation from which all design dimensions were obtained. Supplemented design approaches have since been developed (14) which provide design parameters to give the greatest solids removal efficiencies under specific hydraulic conditions. The flows pertinent to the West Newell Street swirl regulator/concentrator were: (1) a maximum inlet flow of 8.9 MGD (23.4 cu m/min) corresponding to a flood flow, (2) a design flow of 6.8 MGD (17.9 cu m/min), (3) a dry-weather flow range of 0.50 to 0.75 MGD (1.3 to 2.0 cu m/min), and (4) an inverted siphon flow to the sanitary interceptor of 1.2 MGD (3.2 cu m/min). The major dimensions are presented in Table 12.

The foul sewer (concentrate line) was sized to convey maximum dry-weather flow and prevent line blockage by large solids. A diameter of 12 in. (30.5 cm) was determined to be the minimum size necessary to fulfill these conditions.

The swirl regulator/concentrator was constructed with a gently-sloped concrete floor, with floor gutters in place and vertical steel sides. The various appurtenances associated with the swirl chamber, e.g. inlet ramp, flow deflector, scum ring, weirs, spoilers, floatables trap, and floor gutters were installed as recommended in the physical model design approach. Figure 15 presents an isometric view of the swirl regulator/concentrator.

Control System--

The control system was based on two independent measurement systems, described as follows:

1. An ultrasonic flow monitoring system (Badger Meter, Inc., Precision Products Division, Tulsa, Oklahoma) on the foul sewer to control operation of the submersible pump in the grit manhole.

2. A magnetic flow monitoring system (Brooks Instrument Division, Model 7100 Series, Emerson Electric Co., Hatfield, Pennsylvania) on the overflow discharge pipe, to control operation of the disinfection system and the automatic samplers.

The basic control strategy at the West Newell Street facility was such that dry-weather flow diverted from the trunk sewer flowed through the swirl regulator/concentrator and the foul sewer to the grit manhole. (Refer to Figure 16.) During periods of low (dry-weather) flow, the submersible pump in the grit manhole was automatically activated at a level set to prevent surcharge in the swirl regulator/concentrator, and dry-weather flow was pumped back to the sanitary interceptor via the West Newell Street trunk sewer, at a point downstream from the diversion manhole. During periods of high (wet-weather) flow, a signal from the flowmeter in the foul sewer deactivated the submersible pump, and the concentrate (underflow) was forced through the foul sewer system by the raised liquid level in the swirl which eventually caused overflow. As a result of pump deactivation, the grit manhole continued to surcharge until the level of sewage reached a 12 in. (30.5 cm) dia gravity relief line located 7.79 ft (2.38 m) above the invert of the manhole. This relief line allowed gravity return to the trunk sewer leading to the siphon and interceptor.

During periods of high flow, a gravity flow regime was established, with diverted flow from the trunk sewer entering the swirl regulator/con-concentrator, and exiting either via the gravity relief line from the grit manhole or via the overflow to the creek directly from the swirl unit.

When high flow subsided, a signal from the foul sewer flowmeter reactivated the submersible pump, drawing down the system and preventing wastewater from standing in the swirl chamber.

The swirl regulator/concentrator was operated during eleven storm events in 1974 and 1975. As discussed in greater detail in subsequent sections of this report, a constant operational difficulty was the lack of sufficient storm flow to simulate design conditions. Figures 17 and 18 illustrate the swirl under dry- and wet-weather conditions, respectively.

TABLE 12. SWIRL REGULATOR/CONCENTRATOR DESIGN DIMENSIONS

Basic Design Dimensions	Size ft	m
Inside Chamber Diameter	12.33	3.76
Inlet Diameter	2.00	0.61
Scum Ring Diameter	8.00	2.44
Weir Diameter	6.67	2.03
Overflow Outlet Diameter	2.00	0.61
Radius of Inlet gutter		
0° - 90°	4.67	1.42
90° - 180°	1.00	0.30
Radius of Secondary gutter		
90° - 270°	1.25	0.38
9° - 90°	2.25	6.78
270° - 360°	7.33	2.23
Offset Distance for Determining Gutter Radii	0.33	0.10
Floor to top of Circular Weir	3.00	0.91
Inlet Pipe Invert to Chamber Bottom	1.67	0.51
Depth of Circular Weir Skirt	1.00	0.30
Depth of Scum Ring	0.67	0.20

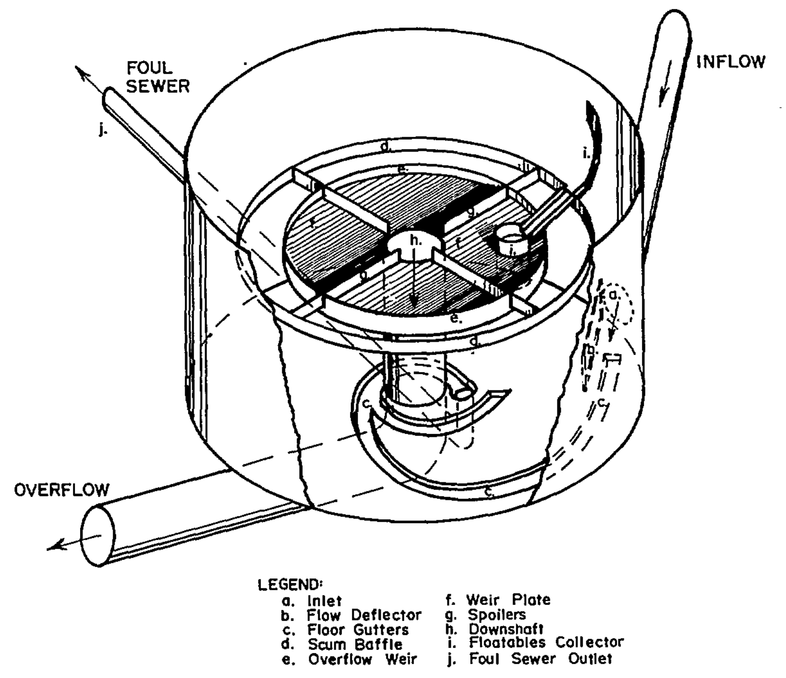


FIGURE 15. Isometric View of Swirl Regulator/Concentrator

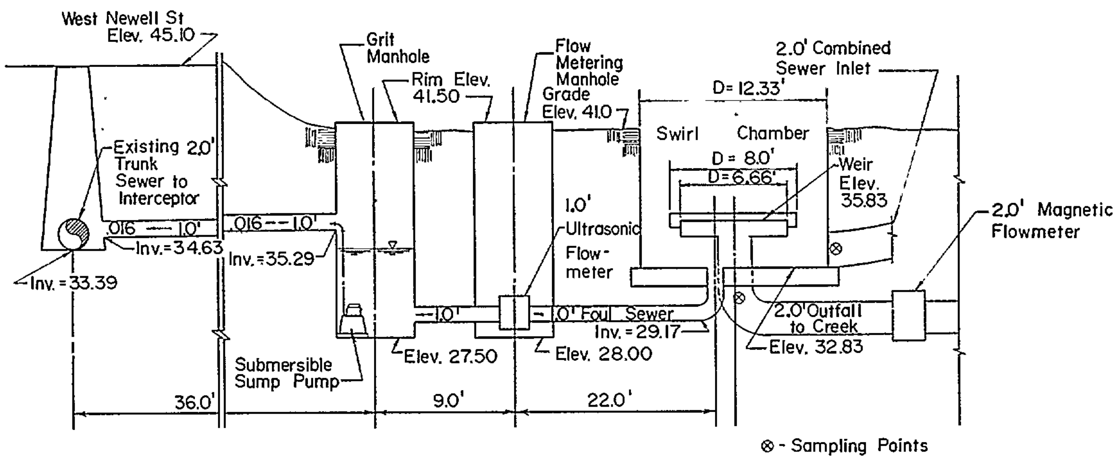


FIGURE 16. Schematic Profile - West Newell Street CSO Facilities

Samplers were installed to collect samples from the swirl influent, the swirl effluent prior to disinfection, and the swirl effluent after disinfection.

Disinfection Equipment--

The high-rate disinfection system installed at the West Newell Street facility (Figure 19) was a Nitrosyl Chloride generation system identical with that previously described in this report for the Maltbie Street facility. Provision was made to introduce the disinfectant either at the inlet to the swirl concentrator (pre-disinfection) or at the bottom of the concentrator downshaft (post-disinfection). ClO_2 was to be injected at a rate proportional to the quantity of flow measured in the discharge pipe.

No valid disinfection data was obtained during the 1974 and 1975 storms, due to rapid deterioration of the disinfection equipment and a continuous lack of cooperation on the part of the supplier to make his equipment operable.

Corrosivity--

Corrosivity problems at this site were especially severe since the chemical storage tanks were housed in the same area as the instrumentation controls and ClO_2 generating equipment. Copper water pipes were severely corroded and one terminal strip in the instrumentation panel had to be replaced. Also, leaks in the ClO_2 piping system resulted in additional corrosion problems. Storage of the chemicals should be accomplished in a separate storage area that is well-ventilated and protected from sunlight.

Flow Sensing Devices--

A magnetic flowmeter was installed in the swirl effluent line discharging to Onondaga Creek. When calibration procedures were necessary, a technician was required to descend into a damp, crowded manhole to calibrate the flowhead. Then the signal converter mounted in the equipment building had to be calibrated. Oils, grease and solids attached to the probes presented difficulties in calibrating the unit. In contrast, an ultrasonic

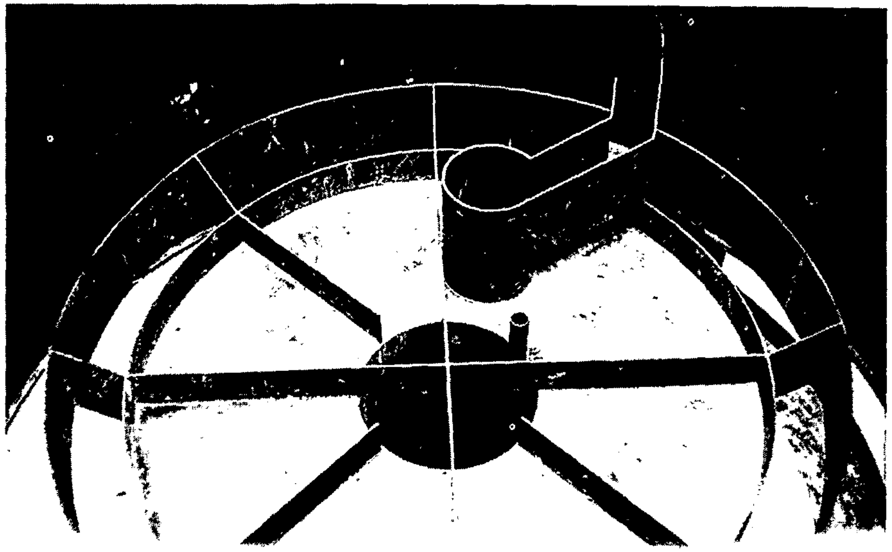


FIGURE 17. Swirl Regulator/Concentrator - Dry-Weather Operation

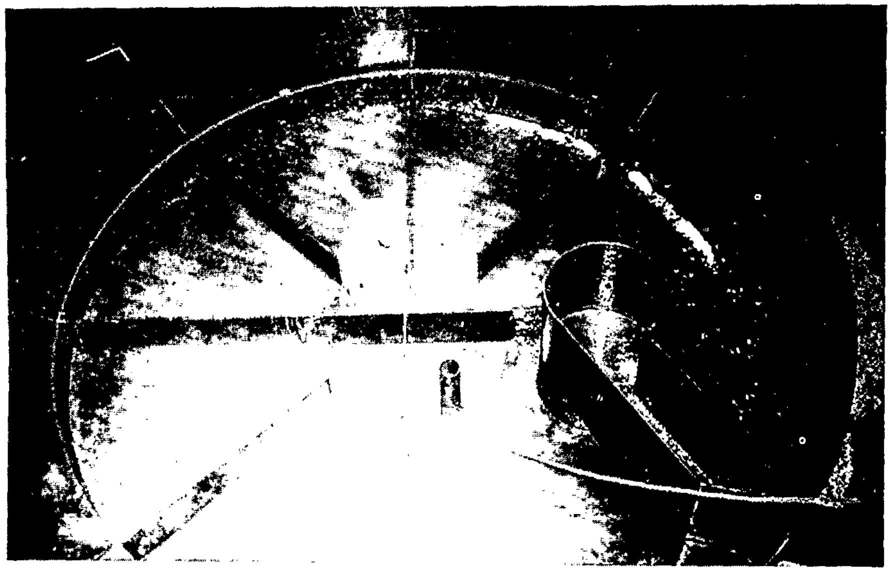


FIGURE 18. Swirl Regulator/Concentrator - Wet-Weather Operation

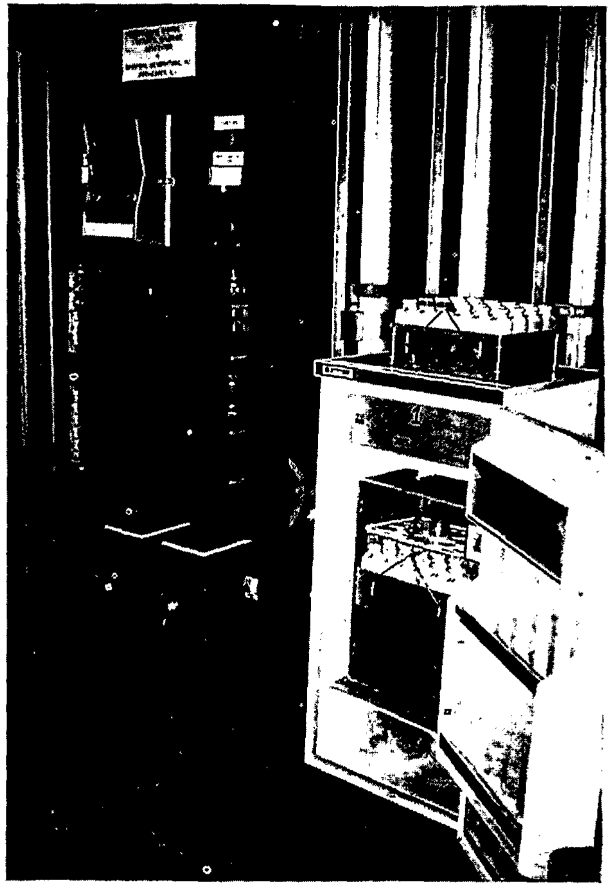


FIGURE 19. West Newell Street - Chlorine Dioxide Generators

flowmeter installed in the foul sewer (with probes attached to the outside of the pipe) was easily calibrated from the equipment building. Constant calibration of the magnetic flowmeter was required while only occasional calibration of the ultrasonic flowmeter was required. A non-contact flowmeter is preferred over a magnetic flowmeter.

Insulation--

The West Newell Street equipment building was insulated at the beginning of the project in anticipation of winter operation. A portable, electric heater was provided to maintain above-freezing temperatures and although this precaution was taken, water pipes burst on one occasion during the winter when a power failure occurred in the area. CSO facilities should be adequately insulated to minimize freezing problems.

Pumping Control--

Where a pump is required for dry-weather operation of a swirl regulator/concentrator, the pumping should be controlled by the clear effluent flowmeter rather than the foul sewer flowmeter. Such an arrangement would allow a pump to continue to operate up to a predetermined flow in the overflow pipe, at which point the flowmeter would activate a relay to remove the pump from service and allow the swirl unit to operate under gravity conditions.

At the West Newell Street facility the capacity of the pump installed exceeded the desired predetermined overflow rate. Thus, when the pump was operating during dry-weather conditions, the pump itself occasionally created a flow through the ultrasonic flowmeter which deactivated the pump entirely resulting in standing water depth of three feet in the swirl chamber. (See Section 5 for a description of the hydraulic limitations of this site.) A pump operating mode controlled by a flowmeter in the overflow line would avoid this problem.

Washdown Facilities--

After each overflow event, manual washdown of the swirl chamber was required. This requirement could be eliminated by installation of an automatic washdown system. References are available in the literature (13,15) for an effective means of incorporating such a system.

SECTION 7

CSO LOADINGS AT DEMONSTRATION SITES

RAIN DATA ANALYSIS

Precipitation records (40) from the U.S. Weather Bureau's station at Hancock Airport for the period 1948 to 1973 were statistically analyzed to develop rainfall frequency-duration-intensity curves for the Syracuse area. Technical Paper No. 40 published by the U.S. Department of Commerce was also used in the development of these curves (41). Examination of the existing records indicated that precipitation patterns in the Syracuse area are highly variable. High intensity-short duration events are usually associated with thunderstorms; less intense, longer duration rainfall events are usually caused by cyclonic activity. The computed relationships of intensity versus duration for different storm return periods is illustrated in Figure 20.

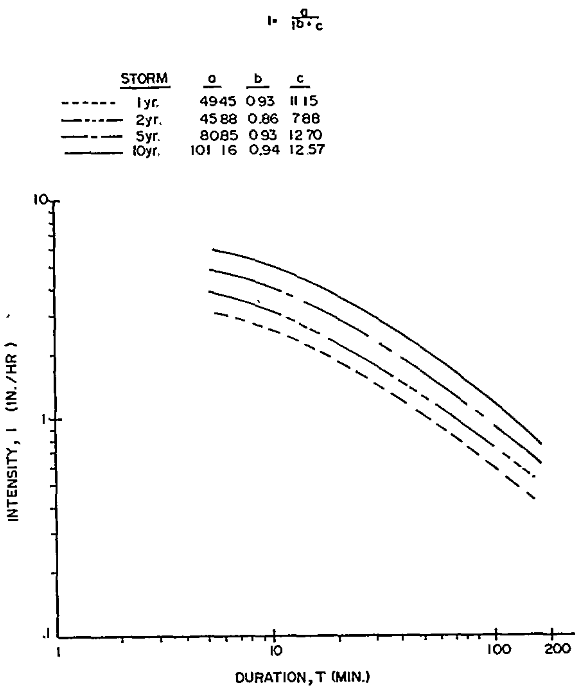


FIGURE 20. Syracuse Area-Rainfall Intensity vs. Duration

FIELD PROGRAM

During 1974, 19 overflow events were monitored at the Maltbie Street demonstration facility during a period from May 1974 through November 1974. The durations of the individual rainfall events ranged from a minimum of 1.0 hr to a maximum of 11.0 hr with a mean duration for all storms of 4.8 hr. Average intensitites of individual storms ranged from 0.05 in./hr (0.1 cm/hr) to 0.60 in./hr (1.5 cm/hr) with a mean intensity for all storms of 0.17 in./hr (0.4 cm/hr). The average intensities of individual storms were calculated by dividing the total rainfall by the duration of the rainfall for each event. The mean intensity of all monitored storms was calculated by averaging the individual storm intensitites on a duration-weighted basis. To attempt to define the severity of individual storms, a maximum hourly intensity was calculated by determining the maximum intensity during any 10-min period. The maximum hourly intensitites for individual storms were determined to range from 0.02 in./hr (0.05 cm/hr) to 1.30 in./hr (3.3 cm/hr). All of these storms had a return frequency of less than one year, as can be seen by entering Figure 20 with values of 1.30 in./hr (3.3 cm/hr) and a 20 min duration.

Between March and November 1975, 10 overflow events were monitored at the Maltbie Street site. The rainfall durations of individual events ranged from 1.0 hr to 12.0 hr with a mean duration of 6.5 hr for all storms. Average intensitites of individual storms ranged from 0.03 to 0.26 in./hr (0.08 to 0.66 cm/hr) with a mean intensity for the 10 storms of 0.10 in./hr (0.25 cm/hr). Maximum hourly intensities were calculated to estimate the severity of individual storms based on the maximum rainfall during any 30 min period. The results indicated a range of maximum hourly intensitites of 0.12 to 1.50 in./hr (0.30 to 3.8 cm/hr) for the 10 storms monitored. As in 1974, all of these rainfall events fall into a category of less than one-year storms in terms of return frequency (see Figure 20).

From March 1974 to September 1975, a total of 11 storms were monitored at the West Newell Street demonstration site. The durations of individual rainfall events ranged from a minimum of 2.25 hr to 37.0 hr. The average intensities of individual storms ranged from 0.04 to 0.40 in./hr (0.10 to 1.0 cm/hr) with a time-weighted average intensity for all storms of 0.16 in./hr. An effort to determine the severity of each rainfall was attempted by determining the maximum intensity occurring in any 20 min period during the rainfall. Maximum rainfall intensities ranged from 0.08 to 1.95 in./hr (0.20 to 5.0 cm/hr). All of these storms had return frequencies of less than 1 year.

Runoff coefficients for the two demonstration sites were determined from rainfall and runoff measurements. The ratio of total runoff measured to the total rainfall for each demonstration site resulted in an average runoff coefficient of 0.55 for the Maltbie Street drainage area and 0.34 for the West Newell Street drainage area. From the data gather, no correlation of runoff coefficient with preceding dry-weather intervals could be developed.

SIMPLIFIED SWMM ANALYSIS

Using techniques of Simplified Stormwater Management Model (Simplified SWMM) analysis (42), relationships between mass emissions of various pollutants from the Maltbie and West Newell Street overflows and the total rainfall for given storm events have been established. Figures 21 through 26 illustrate the mass emission of BOD_5, TSS, VSS, TKN, NH_3N and TIP, respectively, for both the Maltbie and West Newell Street overflows for various storm events. The Simplified SWMM approach provides an accounting of the runoff from rainfall events by mass balance calculations considering such factors as drainage area, runoff coefficient, and total rainfall (or average storm intensity times storm duration). The mass emissions are determined by superimposing the runoff quality on the volume of discharge. No adjustment has been provided for the variability of the runoff coefficient within given storm events. Actual data measurements obtained in the demonstration study are plotted on the graphs also.

As indicated in Figures 21 through 26 Maltbie Street overflows result in higher pollutant loadings for all parameters, except NH_3N, than does the West Newell Street overflow. This is a direct result of the larger drainage area and higher runoff coefficient for the Maltbie Street location. West Newell Street overflows result in higher quantities of NH_3N being discharged than at Maltbie Street, largely as a result of the larger NH_3N concentrations at West Newell Street, e.g. 4.4 mg/l NH_3N at West Newell Street as compared to 0.8 mg/l NH_3N at Maltbie Street.

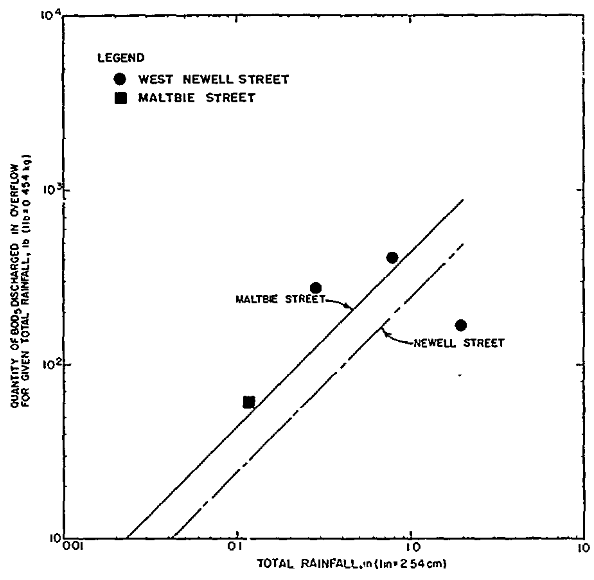


FIGURE 21. Mass Emission of BOD5 vs Total Rainfall

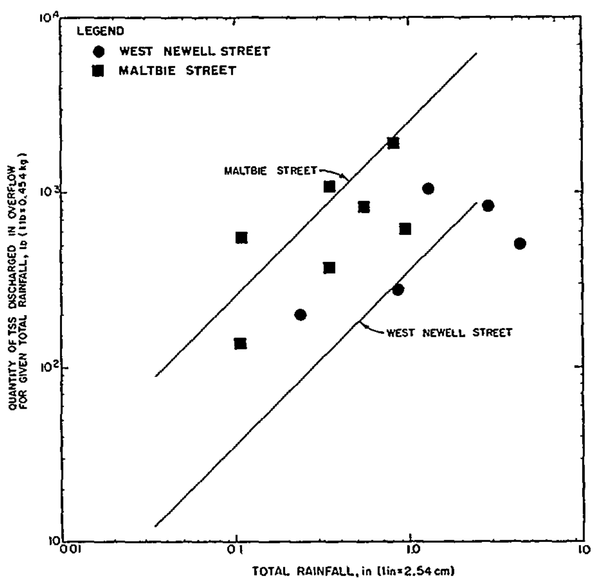


FIGURE 22. Mass Emission of SS vs Total Rainfall

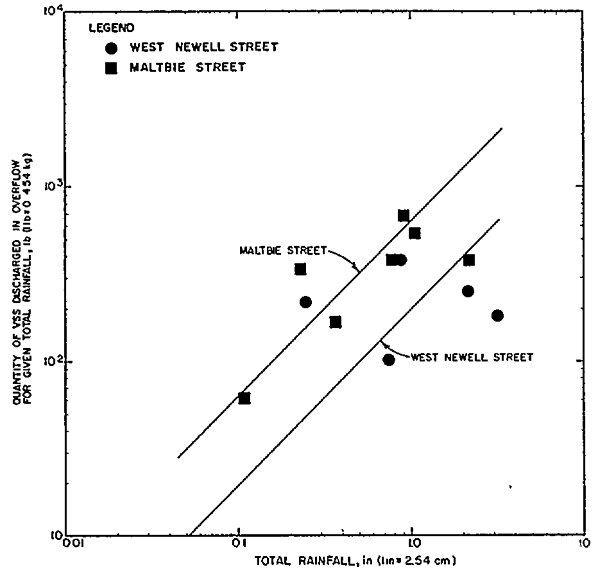


FIGURE 23. Mass Emission of VSS vs Total Rainfall

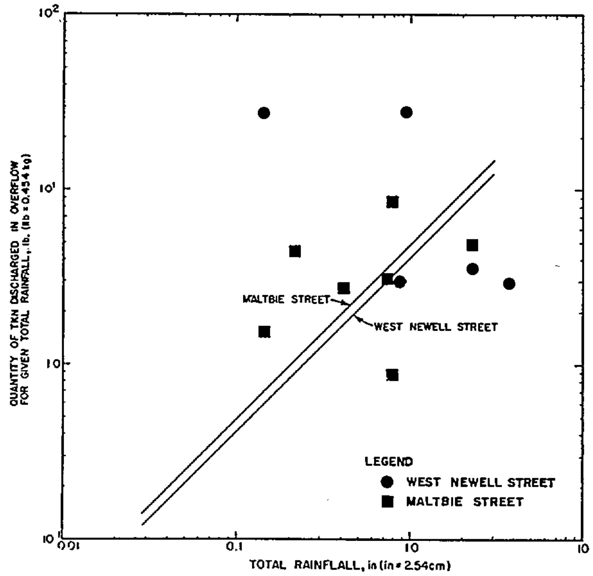


FIGURE 24. Mass Emission of TKN vs Total Rainfall

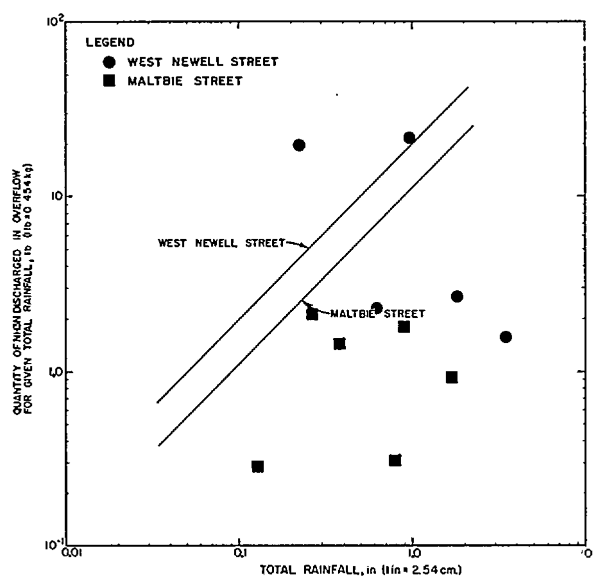


FIGURE 25. Mass Emission of NH_3N vs Total Rainfall

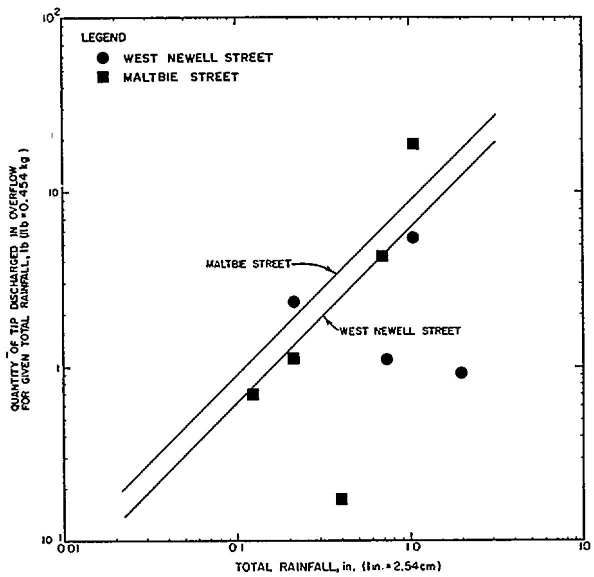


FIGURE 26. Mass Emission of TIP vs Total Rainfall

The pollutant loadings expressed in lb/acre-in (kg/ha-cm) as measured during the demonstration program were:

Parameter	Maltbie St. Site mg/l	Maltbie St. Site lbs/acre-in	Maltbie St. Site Kg/ha-cm	West Newell St. Site mg/l	West Newell St. Site lb/acre-in	West Newell St. Site Kg/ha-cm
SS	325	73.46	32.45	276	62.38	27.55
VSS	163	36.84	16.27	132	29.83	13.18
TKN	2.56	0.58	0.26	8.46	1.91	0.84
NH_3N	0.82	0.19	0.08	4.39	0.99	0.44
TIP	1.23	0.28	0.12	0.85	0.19	0.08

SECTION 8

MICROSCREENING - RESULTS AND DISCUSSION

GENERAL

During the 1974 Maltbie Street facilities operation, many operational problems were encountered relating to equipment start-up and performance. Evaluation of sampler performance indicated that intake velocities of 0.1 ft/sec (3.0 cm/sec) and less were much too low for obtaining representative samples of the process wastewaters, although the latter also flowed at velocities generally less than 0.1 ft/sec (3.0 cm/sec). In all cases, samples were lifted distances of at least 5 ft (1.5 m) from the process streams and settling of solids was observed to occur in the sample tubing. In late 1974, the samplers were modified to provide intake velocities up to 3.0 ft/sec (0.9 m/sec), thus yielding more representative samples. Other operational problems such as replacement of punctured microscreen panels and electrical malfunctions of disinfection equipment delayed the start of meaningful data collection until the spring of 1975.

SUSPENDED SOLIDS REMOVALS

During 1975, ten overflow events were monitored at the Maltbie Street demonstration site, during which time the Sweco and Zurn units were evaluated for solids removal capabilities. The Crane microscreen unit was functional during the 1975 monitoring period for only Storm 16. Table 13 presents the screen loading rates for the units during this period. It should be noted that the Sweco unit was operated at much higher hydraulic loading rates than the Zurn unit. The Zurn unit was operated at the maximum pressure differential possible between influent chamber and overflow weir level that would not result in influent bypass via the fixed elevation emergency weir. Nevertheless, flux for the Zurn unit was limited to less than 15 gpm/ft^2 (36.6 m/hr).

Solids removal capability of the Crane Microstrainer was investigated for two additional overflow events in 1976, as indicated in Table 13. However, since the 1976 evaluations focused on disinfection investigations, variable loadings to the Crane were not attempted; instead, flowrates were held constant.

The results of the solids removal evaluations for the three microscreens are discussed in the following paragraphs.

TABLE 13. MALTBIE STREEET OPERATION SCHEDULE - MICROSCREENING

Overflow	Avg. Screen Loading Rate (gpm/ft^2)			Screen Aperture Size (microns)		
	Sweco	Crane	Zurn	Sweco	Crane	Zurn
2-75	40	-	12	105	-	71
3-75	36	-	11	105	-	71
4-75	32	-	7	105	-	71
5-75	28	-	9	105	-	71
6-75	43	-	10	105	-	71
7-75	33	-	8	105	-	71
8-75	66	-	4	105	-	71
10-75	-	-	12	-	-	71
11-75	50	-	9	105	-	71
16-75	11	2	3	105	23	71
1-76	-	-	no screen	-	-	no screen
2-76	-	-	no screen	-	-	no screen
3-76	-	8	no screen	-	-	no screen
4-76	-	8	no screen	-	23	no screen
5-76	-	8	no screen	-	23	no screen
6-76	-	-	no screen	-	-	no screen

Note: $gpm/ft^2 \times 2.44 = m/hr$

Sweco Centrifugal Wastewater Concentrator Performance

Results of the performance data for the Sweco unit are presented in Table 14. Average influent SS concentrations for individual storm overflows ranged from 106 to 529 mg/l with an overall time-weighted SS concentration of 284 mg/l. Effluent SS concentrations ranged from 65 to 396 mg/l with an overall time-weighted average of 196 mg/l. SS concentration removal efficiencies ranged from 7 to 65 percent with an overall time-weighted average of 32 percent. The operation of the Sweco unit utilizes a "flow-splitting" technique whereby up to 25 percent of the influent volume is returned to the sewer system in the form of a concentrated slurry. This method of operation effectively results in physical removal of a portion of the raw wastewater from the overflow discharge. When this removed volume is accounted for in the mass removal efficiency calculations, the mass removal efficiency of SS achieved by the Sweco unit ranged from 30 to 74 percent for individual storms with an overall time-weighted average of 48 percent. The 25 percent of total flow entering the Sweco unit which is returned to the interceptor may be too large a volume to be accepted

TABLE 14. SWECO CENTRIFUGAL WASTEWATER CONCENTRATOR SUSPENDED SOLIDS REMOVAL

Storm No.	Total Time Min	Total Volume gal	Suspended Solids Raw mg/l	Suspended Solids Effluent mg/l	Concentration Removal Efficiency Percent	Average Flow Rate gpm	Loading Rates Hydraulic gpm/ft^2	Loading Rates Solids lb/day/ft^2	Solids Influent lb/min	Average Flow Rate Effluent gpm	Solids Effluent lb/min	Mass Removal Efficiency Percent
2-75	360	360,000	324	242	25.3	1,000	40.0	156	2.70	750	1.51	44.1
3-75	225	204,750	240	157	34.6	910	36.4	105	1.82	680	0.89	51.1
4-75	50	40,500	181	163	9.9	810	32.4	70.4	1.22	610	0.83	32.0
5-75	120	82,800	217	76	65.0	690	27.6	71.9	1.25	520	0.33	73.6
6-75	120	129,600	529	396	25.1	1,080	43.2	274	4.76	810	2.67	43.9
7-75	30	24,900	277	212	23.5	830	33.2	110	1.91	620	1.10	42.4
8-75	75	124,500	499	314	37.1	1,660	66.4	398	6.91	1,250	3.27	52.7
11-75	45	56,250	124	115	7.3	1,250	50.0	74.5	1.29	940	0.90	30.2
16-75	165	46.200	106	65	38.7	280	11.2	14.3	0.25	210	0.11	56.0
Time-Weighted Average			284	196	32.3	900	35.9	137.5	2.39	670	1.25	47.9

Conversions: gal x 3.785 x 10^{-3} = cu m
gpm x 3.785 x 10^{-3} = cu m/min
gpm/ft^2 x 2.44 = m/hr
lb/day/ft^2 x 4.89 = kg/day/m^2
lb/min x 0.454 = kg/min

by the dry-weather treatment facilities and/or interceptor systems in many municipalities. This feature should be evaluated when considering use of Sweco units.

Zurn Micromatic Performance

Average influent SS concentrations to the Zurn unit for individual overflow events ranged from 120 to 748 mg/l with an overall time-weighted average of 308 mg/l, as presented in Table 15. Effluent SS concentrations ranged from 46 to 340 mg/l with an overall time-weighted average of 172 mg/l. SS concentration removal efficiencies ranged from 25 to 62 percent with an overall time-weighted removal efficiency of 45 percent. In terms of mass removal efficiency, the Zurn Micromatic SS removal efficiencies were the same for individual storms as were the concentration removal efficiencies since the Zurn unit did not utilize a flow-splitting method of removing screened solids. Rather, backwash water from the city water supply system in the order of 0.5 to 3.0 percent of influent flowrate was used to return screened solids to the sewer system. For the storms monitored in this study, 3.0 percent of the average flowrate would amount to approximately 30 gpm or 43,200 gpd for one Zurn unit in operation. Although most interceptors would have the capacity to accept this volume of flow, the backwash flow volumes generated should be evaluated on an individual basis. Provision can be made to utilize screened effluent for backwash water.

Crane Microstrainer Performance

The Crane Microstrainer was operated for a total of three storms during the demonstration study, as indicated in Table 16. Influent SS concentrations ranged from 118 to 971 mg/l for individual storm events with an overall time-weighted concentration of 619 mg/l. Effluent SS concentrations ranged from 42 to 588 mg/l with an overall time-weighted concentration of 290 mg/l. SS concentration removal efficiencies ranged from 39 to 79 percent with an overall time-weighted average of 58 percent. As in the case of the Zurn Micromatic, the mass removal efficiencies were the same as the process removal efficiencies, since no volume of wastewater is removed from the process flow. Backwash water from the city water supply was utilized to return screened solids to the sewerage system. However, provisions can be made to utilize screened effluent for backwash water, which would reduce operating cost by eliminating city water use for the purpose of backwashing.

Comparison of Screening Unit Performance Data

Regression analyses were performed on the process efficiency data for each of the microscreens relating process efficiency to the hydraulic loading rate. Although the correlation was not as high as might be desired between these two parameters ($r = 0.3$ to 0.5), the equations developed are considered to represent the performance trends for the three microscreens. The correlation results are presented graphically in Figures 27 and 28.

TABLE 15. ZURN MICROMATIC SUSPENDED SOLIDS REMOVAL

Storm No.	Total Time min	Total Volume gal	Suspended Solids Raw mg/l	Suspended Solids Effluent mg/l	Concentration Removal Efficiency Percent	Average Flow Rate gpm	Loading Rates Hydraulic gpm/ft^2	Loading Rates Solids lb/day/ft^2	Solids Influent lb/min	Solids Effluent lb/min	Treatment Efficiency Percent
2-75	260	223,600	380	230	39.5	860	12.3	56.1	2.73	1.65	39.5
3-75	240	182,400	291	143	50.8	760	10.9	38.1	1.84	0.91	50.8
4-75	50	24,000	186	138	25.8	480	6.9	15.4	0.74	0.55	25.8
5-75	120	76,800	277	132	52.3	640	9.1	30.3	1.50	0.70	52.3
7-75	100	53,000	277	175	36.8	530	7.6	25.3	1.22	0.77	36.8
8-75	75	21,000	748	340	54.5	280	4.0	35.9	1.75	0.79	54.5
10-75	120	98,400	321	240	25.2	820	11.7	45.1	2.20	1.64	25.2
11-75	45	27,900	124	80	35.5	620	8.9	13.3	0.64	0.41	35.5
16-75	150	52,500	120	46	61.7	210	3.0	4.3	0.21	0.08	61.7
Time-Weighted Average			308	172	44.5	616	9.1	34.5	1.68	0.97	44.5

Conversions: gal x 3.785 x 10^{-3} = cu m
gpm x 3.785 x 10^{-3} = cu m/min
gpm/ft^2 x 2.44 = m/hr
lb/day/ft^2 x 4.89 = kg/day/m^2
lb/min x 0.454 = kg/min

TABLE 16. CRANE MICROSTRAINER SUSPENDED SOLIDS REMOVAL

Storm No.	Total Time min	Total Volume gal	Suspended Solids Raw mg/l	Suspended Solids Effluent mg/l	Concentration Removal Efficiency Percent	Average Flow Rate gpm	Loading Rates Hydraulic gpm/ft^2	Loading Rates Solids lb/day/ft^2	Solids Influent lb/min	Solids Effluent lb/min	Treatment Efficiency Percent
16-75	75	11,475	118	42	64.4	153	1.7	2.4	0.15	0.053	64.4
4-76	100	69,400	971	588	39.4	694	7.7	89.8	5.62	3.40	39.4
5-76	75	52.050	652	140	78.5	694	7.7	60.3	3.77	0.81	78.5
Time-Weighted Average			619	290	58.6	532	5.9	54.7	3.42	1.62	58.6

Conversions: gal x 3.785 x 10^{-3} = cu m
gpm x 3.785 x 10^{-3} = cu m/min
gpm/ft^2 x 2.44 = m/hr
lb/day/ft^2 x 4.89 = kg/day/m^2
lb/min x 0.454 = kg/min

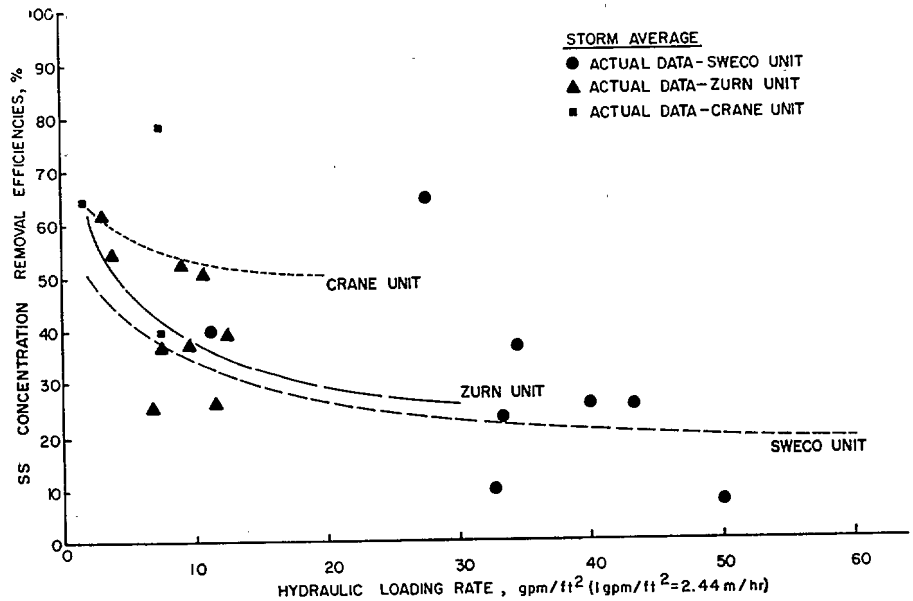


FIGURE 27. Microscreening - SS Concentration Removal vs Hydraulic Loading Rate

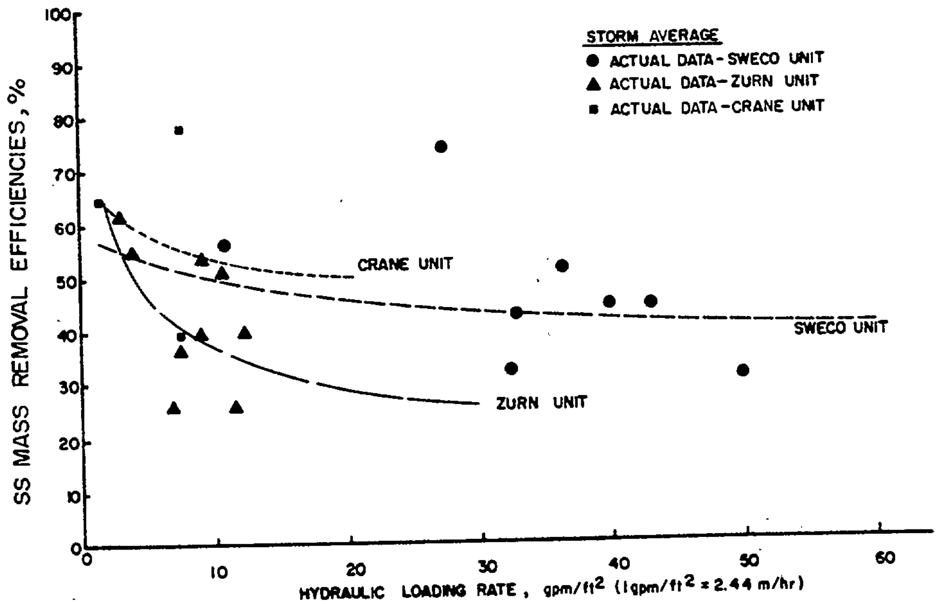


FIGURE 28. Microscreening - SS Mass Removal vs Hydraulic Loading Rate

In Figure 27, the SS concentration removal efficiencies for each of the microscreens is seen to decrease as the hydraulic loading rate is increased.

The highest storm-average hydraulic loading rate at which the Sweco unit was operated approached 66.4 gpm/ft^2 (150 m/hr). On the basis of the regression equation results, the SS concentration removal efficiency at this loading rate should be about 18 percent. At the lower hydraulic loading rate near 4.9 gpm/ft^2 (12 m/hr), SS concentration removal efficiencies should exceed 45 percent. Corresponding mass removal efficiencies for the Sweco unit for the above hydraulic loading rate would be 40 to 55 percent, respectively as indicated in Figure 28.

The range of storm-average hydraulic loading rates at which the Zurn unit operated was 3.3 to 13.7 gpm/ft^2 (8 to 33 m/hr). The concentration removal efficiencies (and mass removal efficiencies) as represented by the regression equations in Figures 27 and 28 ranged from 56 to 35 percent. The highest instantaneous hydraulic loading rate attained by the Zurn unit during the study was 30 gpm/ft^2 (73 m/hr) which resulted in a concentration removal efficiency of 19 percent. Attempts to apply loading rates in excess of 30 gpm/ft^2 (73 m/hr) resulted in the creation of excessive differential levels greater than 6 in. (15.2 cm) between the unit's influent and effluent chambers and subsequent bypass of a portion of the influent flows to the receiving water. The hydraulic loading rate appeared to be limited by the relatively fine mesh size (71 microns) and inherent support material of the microscreen, and to some degree to the blinding of the screens by oil and grease, since the backwash system did not incorporate any detergent or other oil and grease removers. The backwash spray system operating at 36 gpm (136 l/min) and 40 psig (2.8 kg/cm^2) was not sufficient to thoroughly remove the oil and grease particles. Increased drum rotational speed, although automatically increased as head loss across the screen increased, did not significantly increase the flow-through capacity of the Zurn Micromatic.

The Crane Microstrainer was operated at only two hydraulic loading rates: 1.7 and 7.7 gpm/ft^2 (4.1 and 18.8 m/hr) during the demonstration study. These hydraulic loading rates are significantly lower than hydraulic loading rates investigated on a Crane-Glenfield microstrainer in Philadelphia, Pa., which operated at an average flowrate of 16 gpm/ft^2 and achieved an average SS concentration removal efficiency of 70 percent.(10) The data obtained at these loading rates was not considered adequate enough to define the performance capabilities of the unit; however, the regression analysis did indicate that higher SS removals might be achieved by this unit than by the Zurn or Sweco units. The SS removal trend is consistent with the removals expected by a finer mesh screen of 23 microns as compared to the screen sizes of 105 microns on the Sweco unit and 71 microns on the Zurn unit. The testing of the Crane unit was not sufficient to define possible problems with screen blinding.

SS concentrations removal efficiencies as determined for the Syracuse microscreens indicated that better SS removals were achieved at smaller screen sizes. At a hydraulic loading rate of 15 gpm/ft^2 (36.6 m/hr) the Sweco unit would produce a SS removal efficiency of 29 percent, the Zurn unit

32 percent, and the Crane unit 51 percent. However, consideration must be given to the fact that the Zurn unit was limited to a maximum hydraulic loading rate of 30 gpm/ft^2 (73.2 m/hr) and the Crane unit to 7.7 gpm/ft^2 (18.8 m/hr), whereas the Sweco unit treated up to 60 gpm/ft^2 (146.4 m/hr), although at reduced concentration removal efficiency.

In terms of the SS mass removal efficiency as depicted in Figure 28. the Sweco unit performed much better than the Zurn unit and nearly as well as the Crane unit. As an example, at the 15 gpm/ft^2 (36.6 m/hr) hydraulic loading rate, the Sweco unit achieved about 47 percent removal of SS, the Zurn unit achieved 32 percent removal and the Crane unit achieved 51 percent removal. However, the increased mass removal performance of the Sweco unit (as compared to the concentration removal performance) was due to the physical removal of up to 25 percent of the total influent volume, as a result of the hydraulic split. The impact on the existing dry-weather treatment plant and collection system of returning the latter volume to the existing sewer system must be considered.

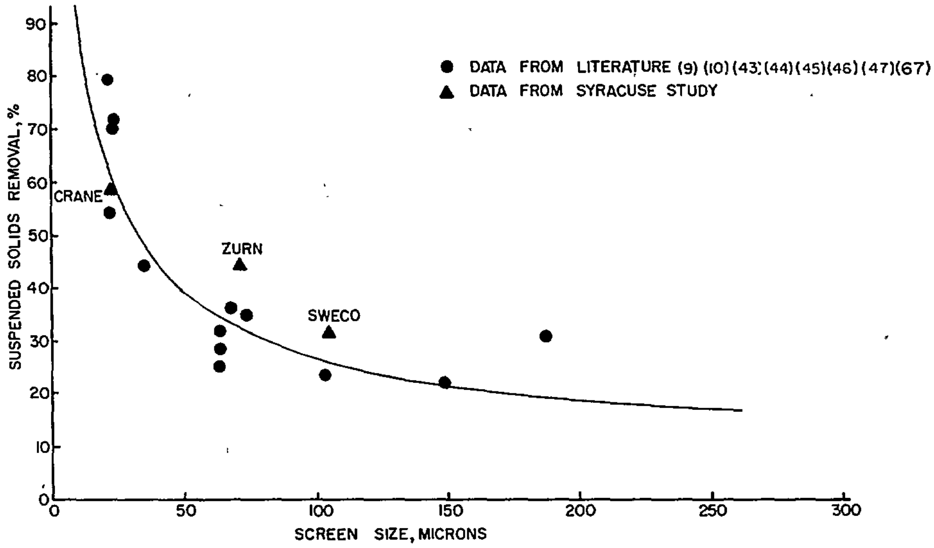


FIGURE 29. Microscreening-SS Removal vs Screen Aperture Size

When the performance data of the Maltbie Street microscreening units are compared with the performance results of other studies, the data are seen to correspond closely, as illustrated in Figure 29. Suspended solids removals were slightly greater for the Sweco and Zurn units than was reported elsewhere, and the Crane unit produced removals about the same as others have reported (9, 10, 43, 44, 45, 46, 47, 48).

The relationship between the SS removal efficiencies versus influent SS concentrations is illustrated in Figure 30. The data gathered in the Syracuse study lie within the general range found in other projects and summarized in Reference 48. The data from this project are quite scattered, however, and do not show the increase in removal with increase in influent SS indicated by the curves from that reference shown in Figure 30.

ORGANIC SOLIDS REMOVAL

The organic solids removals for individual storms for each of the screening units are presented in Table 17 through 19. A summary of average values follows:

Time-Weighted Concentration Removal

Parameter	Sweco In	Sweco Out	Sweco Removed	Zurn In	Zurn Out	Zurn Removed	Crane In	Crane Out	Crane Removed
SS,mg/1	284	196	32	308	172	45	619	290	55
VSS, mg/1	143	79	45	172	74	57	212	127	40
TOC, mg/1	128	57	55	112	54	52	158	100	37

The removals of VSS achieved by the Sweco and Zurn units were greater than the removals for total SS. The percent VSS/SS for the Sweco, Zurn and Crane units were 50, 56 and 34 percent, respectively. These values are lower than the 60 to 85 percent VSS/SS ratios typical of sewage treatment influents and are somewhat indicative of the contribution of inorganics in combined sewer overflows due to storm runoff. Removals of TOC paralled the removal rates of VSS. These two parameter removal rates indicate that the organic matter contained in the combined sewer overflow is more related to the suspended matter than to dissolved material, and that the inorganic portions of the total SS measurement consisted of considerable quantities of fine materials.

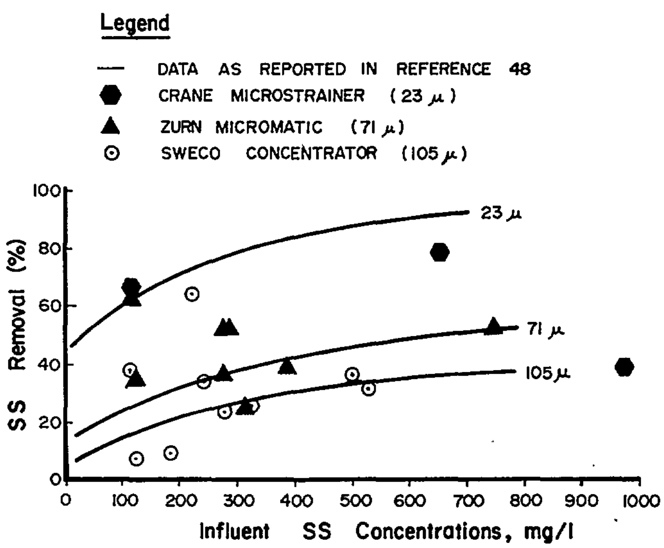


FIGURE 30. Microscreening-SS Removal vs Influent SS Concentrations

HEAVY METALS REMOVAL

Heavy metal concentrations in the combined sewer overflow at Maltbie Street were in the range of concentrations reported elsewhere (48, 49, 50) with the exception of nickel which was up to five times higher than other values reported. Since the area tributary to this overflow is a mixture

TABLE 17. SWECO CWC ORGANIC SOLIDS REMOVAL

Storm No.	Duration min	TSS, mg/l In	TSS, mg/l Out	TSS, mg/l Percent Removed	VSS, mg/l In	VSS, mg/l Out	VSS, mg/l Percent Removed	TOC, mg/l In	TOC, mg/l Out	TOC, mg/l Percent Removed
2-75	360	324	242	25	200	106	47	-	-	-
3-75	225	240	157	35	112	52	53	63	35	44
4-75	50	181	163	10	42	25	40	54	43	20
5-75	120	217	76	65	75	54	28	96	38	60
6-75	120	529	396	25	212	115	46	339	110	68
7-75	30	277	212	23	163	99	39	-	-	-
8-75	75	499	314	37	290	157	46	150	99	34
11-75	45	124	115	7	51	30	42	27	26	4
16-75	165	106	65	39	44	38	14	-	-	-
Time-Weighted Average		284	196	32	143	79	45	128	57	55

TABLE 18. ZURN MICROMATIC ORGANIC SOLIDS REMOVAL

Storm No.	Duration min	TSS, mg/l In	TSS, mg/l Out	TSS, mg/l Percent Removed	VSS, mg/l In	VSS, mg/l Out	VSS, mg/l Percent Removed	TOC, mg/l In	TOC, mg/l Out	TOC, mg/l Percent Removed
2-75	260	380	230	39	317	106	66	-	-	-
3-75	240	291	143	51	166	50	70	154	62	60
4-75	50	186	138	26	45	18	60	56	32	43
5-75	120	277	132	52	94	81	14	101	57	44
7-75	100	277	175	37	163	106	35	-	-	-
8-75	75	748	350	55	305	159	48	183	106	42
10-75	120	321	240	25	86	67	22	50	25	50
11-75	45	124	80	35	51	14	73	28	20	29
16-75	150	120	46	62	81	29	64	-	-	-
Time-Weighted Average		308	172	45	172	74	57	112	54	52

TABLE 19. CRANE MICROSTRAINER ORGANIC SOLIDS REMOVAL

Storm No.	Duration, min	TSS, mg/l In	TSS, mg/l Out	TSS, mg/l Percent Removed	VSS, mg/l In	VSS, mg/l Out	VSS, mg/l Percent Removed	TOC, mg/l In	TOC, mg/l Out	TOC, mg/l Percent Removed
16-75	75	118	42	65	86	37	57	-	-	-
4-76	100	971	588	39	378	224	41	239	141	41
5-76	75	652	140	79	118	89	25	50	45	10
Time-Weighted Average		619	290	55	212	127	40	158	100	37

of commercial, light industrial and residential, higher concentrations of metals than is normally associated with residential areas were expected.

Significant and consistent removal of heavy metals by the microscreening units was not achieved during this study. The influent and effluent concentrations for all storms are presented in Table 20 and 21, as well as the range of influent values encountered during the study.

TABLE 20. SWECO CONCENTRATOR HEAVY METALS REMOVAL

Heavy Metal	Mean Concentration, mg/l		Range of Influent Values	
	Influent	Effluent	Low	High
Fe	1.93	2.14	0.00	17.20
Cr	0.22	0.31	0.00	1.60
Cu	0.09	0.08	0.00	1.60
Pb	0.20	0.17	0.00	0.90
Zn	0.19	0.24	0.02	2.40
Cd	0.00	0.00	0.00	0.02
Ni	0.70	0.69	0.00	19.00

TABLE 21. ZURN MICROMATIC HEAVY METALS REMOVAL

Heavy Metal	Mean Concentration, mg/l		Range of Influent Values	
	Influent	Effluent	Low	High
Fe	1.65	1.13	0.00	17.20
Cr	0.18	0.25	0.00	1.60
Cu	0.08	0.07	0.00	1.60
Pb	0.19	0.09	0.00	0.90
Zn	0.24	0.26	0.02	2.40
Cd	0.00	0.00	0.00	0.02
Ni	0.52	0.60	0.00	19.00

In general, it was concluded that screens could remove metals only via particulate removal either through direct removal of heavy metal compound precipitate or as a result of a dissolved metal being sorbed on a particle. The only metal that showed removals at measurable and significant concentrations was Pb. All other metals showed highly variable removals and/or relatively low concentrations. The removal of Pb is most probably due to the fact that Pb compounds are insoluble or only slightly soluble in water and would be removed relatively easily with other suspended matter.

Data collected during this study are considered insufficient to draw conclusions about heavy metals removal capabilities of the microscreening units. Whatever removals that were achieved are considered incidental to the microscreening process. Since reduction of heavy metals in terms of concentration was so erratic, no attempt is made here to determine the mass removal rates of the individual units.

SECTION 9

SWIRL REGULATOR/CONCENTRATOR - RESULTS AND DISCUSSION

GENERAL

Studies of the hydraulic model at LaSalle Hydraulic Laboratories determined that particle removal effectiveness was a function of the particle effective diameter and specific gravity, or in other words the particle settling velocity (13). In the model studies, removals of grit with specific gravity of 2.65 and greater than 0.33 mm in diameter were greater than 90 percent. This removal percentage for grit decreased to less than 40 percent for 0.1 mm particles. For settleable solids of specific gravity 1.20, efficiency ranged from 80 to 100 percent for particles larger than 1.0 mm, 30 percent for 0.5 mm particles, and no less than 20 percent for 0.3 mm sizes.

In all of the model studies, the hydraulic loading rates for individual tests were held constant. Also, since simulated solids were injected at constant rates, the SS loading rates were also constant for individual tests. In the Syracuse prototype swirl regulator/concentrator at the West Newell Street site, these uniform loading rates were not possible to obtain since the swirl was installed directly on a combined sewer. This situation makes it difficult to compare results obtained in the field to results obtained in a highly controlled laboratory model analysis. However, a series of grab samples totaling 30 gal (113 l) of inflow to the Syracuse swirl prototype was collected during one overflow event for particle settling velocity analysis, and comparison of the observed particle removal efficiency to the predicted model results was made. The following paragraphs describe the settling velocity tests and the overall performance of the swirl unit in terms of SS, VSS, TOC and heavy metals removal.

SETTLING VELOCITY TESTS

Settling velocity relationships of sanitary sewage and stormwater runoff have been presented previously (51). In tests of the settling velocity distributions in sanitary sewage, the median settling velocity observed was 0.00018 fps (0.054 cm/sec) with 31 percent of the particles having settling velocities of less than 0.00033 fps (0.01 cm/sec). Similar tests of urban stormwater showed that 78 percent of the particles have settling velocities less than 0.00033 fps (0.01 cm/sec) with a mediam velocity of less than 0.000033 fps (0.001 cm/sec). In this particular case, it is not known under what storm intensity conditions the urban stormwater sample was collected. If the storm intensity was low, the runoff may not have had sufficient velocity to carry larger-sized particles to the storm conduits.

This possibility could account for the lower settling velocities obtained in the urban stormwater sample than was found in the sanitary sewage sample. A second test conducted on a portion of the same urban stormwater sample after storage of six days at 4°C showed a median settling velocity of 0.00023 fps (0.007 cm/sec) with only 56 percent of the solids having settling velocities less than 0.00033 pfs (0.01 cm/sec), indicating improved settling characteristics after storage, apparently due to agglomeration of small particles during storage.

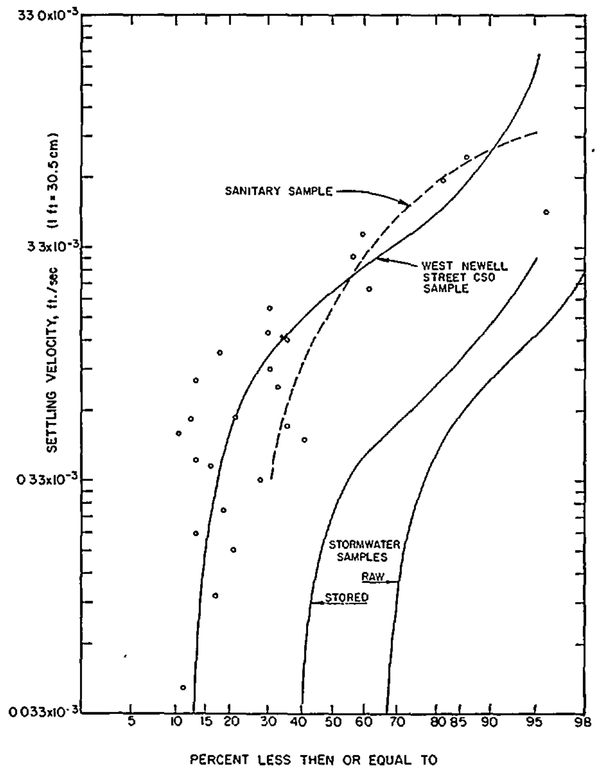


FIGURE 31. Settling Velocity Distribution Curve - West Newell Street CSO

A similar settling velocity test was conducted on the West Newell Street overflow during one overflow event. As seen in Figure 31, only 17 percent of the solids were found to have a settling velocity less than 0.00033 fps (0.01 cm/sec). A mean settling velocity of 0.0021 fps (0.064 cm/sec) was determined. These data indicate that the mean settling velocity of the solids at West Newell Street was similar to the sanitary sewage sample, although the range of settling velocities was smaller. The results are compared against those obtained for the stored stormwater sample by Dalrymple, et al (51). In the stored stormwater sample, 56 percent of the solids had settling velocities less than 0.00033 fps (0.01 cm/sec) compared to 17 percent for the West Newell Street sample. Dalrymple, et al, determined that nearly 40 percent of the particles had a settling of less than 0.000033 fps (0.001 cm/sec) while the West Newell Street sample showed that only 13 percent had a settling velocity of 0.000033 fps (0.001 cm/sec) or less. Also, 52 percent of all particles in the West Newell Street CSO had settling velocities between 0.00033 fps (0.01 cm/sec) and 0.0033 fps (0.10 cm/sec). The similar sample tested by Dalrymple, et al, indicated that only 40 percent of the particles in urban stormwater had settling velocities between 0.00033 fps (0.01 cm/sec) and 0.0033 fps (0.10 cm/sec), while 56 percent were less than 0.00033 fps (0.01 cm/sec). Unfortunately, at the time of the Dalrymple report, a suitable sample of CSO was not available for settling velocity analysis and thus no comparison of the West Newell Street CSO sample with another CSO sample can be made here.

In summary, the CSO at West Newell Street for the one storm event where settling velocity characteristics were determined, contained particles with settling velocities somewhat higher than particle settling velocities of sanitary sewage and stormwater runoff. The mean settling velocity of CSO should lie within a range determined by the mean settling velocity of sanitary sewage as the upper range limit, and the mean settling velocity of stormwater runoff as the lower range limit, since CSO is considered to consist of some mixture of sanitary sewage and stormwater runoff. The mean settling velocity of CSO at West Newell Street is only slightly greater than for sanitary sewage; however, further testing to define the settling characteristics of CSO is warranted.

No particle size distribution data was gathered at the West Newell Street site. Table 22 presents a condensed form of the size distribution relationships presented by Dalrymple, et al, in the previously cited report.

TABLE 22. PARTICLE SIZE DISTRIBUTION (PERCENT) AND SOLIDS RANGES IN SANITARY SEWAGE, STORMWATER RUNOFF, AND COMBINED SEWER OVERFLOWS.*(51)

Parameter	Sanitary Sewage	Stormwater Runoff	Combined Sewer Overflows	W. Newell CSO
Sett-S (<100μ)	45	90	27	
SupraColloidal (1 to 100μ)	35	9	50	
Colloidal (1 mμ to 1μ)	20	1	23	
Total	100	100	100	
VSS Ranges	70-85	10-40	25-86	24-87
Sett-S Ranges	37-65	70-90	37-87	19

*Expressed as a percentage of SS

From Table 22 it is seen that the VSS fraction of the West Newell Street CSO compares with the VSS fraction as determined in other studies of CSO, having a very wide range. The settleable solids value as determined on one sample used in the settling velocity tests is lower than the range yielded by other test data, 19 percent for West Newell Street as compared to a range of 37 to 87 percent in other studies.

Based on the settling velocity data gathered during the storm event of July 19, 1975, a comparison of predicted SS removals and actual SS removals was made.

Adjustments of the performance curve as predicted by the model studies were necessary to arrive at the corresponding performance curve predicted for a 12.3 ft (3.7 m) unit since the results of the model studies were scaled from a 3 ft (0.9 m) diameter unit (13). The performance curves for swirl units of various sizes are related by the Froude number since the latter is the parameter upon which the flows and settling velocities of the simulated solids used in the model are based. From Froude number relationships, the equations relating the performance curves of various size swirl units are described below:

$$\frac{V_{12.3}}{V_3} = \left[\frac{d_{12.3}}{d_3}\right]^{1/2}$$

where

$V_{12.3}$ = settling velocity of particles in a 12.3 ft (3.7 m) diameter unit

V_3 = settling velocity of particles in a 3.0 ft (0.9 m) diameter unit

$d_{12.3}$ = diameter of 12.3 ft (3.7 m) unit

d_3 = diameter of 3.0 ft (0.9 m) unit

$$V_{12.3} = V_3 \left[\frac{12.3}{3}\right]^{1/2} = 2.025\ V3$$

The settling velocity curve referenced above was therefore adjusted by multiplying all values by 2.025. Flowrate, Q was adjusted by a similar value derived from the Froude number relationship of

$$\frac{Q_{12.3}}{Q_3} = \left[\frac{d_{12.3}}{d_3}\right]^{5/2}$$

where

$Q_{12.3}$ = design flowrate of the West Newell Street unit

Q_3 = design flowrate of a 3.0 ft (0.9 m) diameter unit

$d_{12.3}$ = diameter of a 12.3 ft (3.7 m) unit

d_3 = diameter of a 3.0 ft (0.9 m) unit

$$Q_{12.3} = Q_3\,(4.1)^{5/2} = 34.04\,Q_3$$

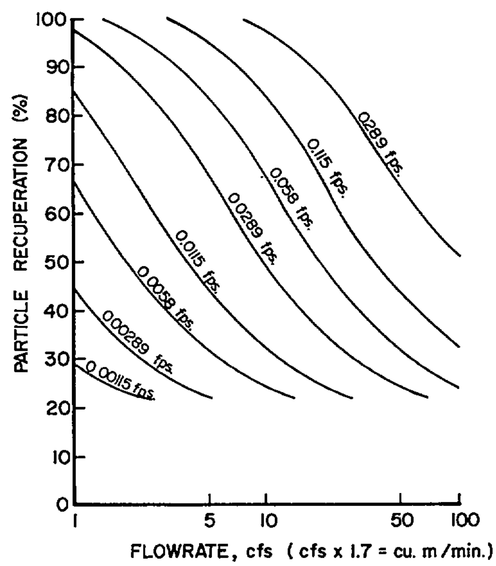


FIGURE 32. Predicted Performance of Prototype Swirl vs Flowrate

Figure 32 represents the resulting curve of predicted performance of the West Newell Street prototype swirl concentrator. The family of curves presented in Figure 32 are representative of the performance of the prototype swirl at a foul sewer flow of 22 percent of the total inflow. (the foul sewer fraction of the July 19, 1975 storm). Table 23 illustrates the predicted performance and the actual performance for the July 19, 1975 event. The predicted performance is arrived at by entering Figure 32 at the desired flowrate and determining the percent removal of particles of the various settling velocity ranges. The percent removal of particles for each settling velocity range is then multiplied by the fraction of total particles represented by the particles of each given settling velocity range. The sum of the individual particle removals is the predicted removal for the swirl regulator/concentrator. The procedure is outlined in Table 23.

As Table 23 indicates, the mass removal of total SS for the July 19, 1975 storm should have been approximately 32 percent, while the SS mass removal measured on the day of actual overflow was determined to be 33 percent. The predicted removal is slightly lower than the actual removal despite the fact that the West Newell Street settling velocity sample was stored for five days as described earlier. Dalrymple, et al, reported that the settling characteristics of solids appear to be improved with storage (51). Since the settling characteristics of West Newell Street

TABLE 23. PREDICTED VERSUS ACTUAL SS REMOVAL - SWIRL REGULATOR/CONCENTRATOR

Predicted Removal - Analysis for Stored Sample

Particle Settling Velocity, ft/sec	Percent of Total Particles With Given Settling Velocity	Percent of Particles With Given Settling Velocity That Are Removed	Percent of Particles Removed as Percent of Total Particles
0.00003	13	22	2.9
0.00028	7	22	1.5
0.00085	10	22	2.2
0.00138	10	24	2.4
0.00174	10	25	2.5
0.00236	10	26	2.6
0.00298	10	28	2.8
0.00387	10	34	3.4
0.00574	10	42	4.2
0.01082	5	56	2.8
0.03279	5	86	4.3
Total Percent SS Removed			31.6

Actual Removal of SS

From Table 24, Storm 12-75:
Total Mass in = 551 lbs (250 kg)
Total Mass out = 370 lbs (168 kg)
Percent Removal = 33%

Conversions: 1 ft/sec = 30.5 cm/sec

overflow were determined after storage and the actual removal determined on unstored sample, these results tend to indicate that the actual performance of the swirl unit is better than predicted performance.

In summary, the one settling velocity distribution performed on the West Newell Street swirl influent tends to confirm the predicted performance curve determined from the model studies (13).

SUSPENDED SOLIDS REMOVALS

Relatively good SS removal efficiencies were determined over the entire storm-flow range for this prototype (Table 24). Total mass loading and concentration removal efficiencies ranged from 33 to 82 percent and 18 to 55 percent, respectively, as flowrates ranged from relatively minor flow of 0.2 MGD (0.5 cu m/min) to a high of 7.6 MGD (20 cu m/min).

Under dry-weather flow conditions, most regulators are designed to direct the entire flow and associated solids to the intercepting sewer. When flow conditions exceed the maximum capacity of the regulator to direct flow to the the interceptor, overflows result whereby flows in excess of the regulator capacity are then discharged from the sewer system. The swirl has the added advantage of concentrating solids as well as conventionally diverting flow during overflow events. This concentrating effect is evidenced by removal efficiencies for the swirl in terms of SS concentrations varying from 18 to 55 percent (Table 24), as previously stated; whereas conventional regulators are assumed not to concentrate solids and result in zero percent removal. (See Table 24, Footnote 3)

If a hypothetical conventional regulator was developed that did not concentrate solids, the net mass loading reductions (attributable to the SS conventionally going to the intercepted underflow) would have ranged from 17 to 64 percent as compared to a more effective range of 33 to 82 percent for the actual swirl unit. This may be a better way to compare the effectiveness of the swirl to conventional CSO regulators.

For low-flow storms, approaching the maximum dry-weather capacity of the interceptor, the advantages of swirl concentration are reduced as would be expected based on the physical principle involved. In other words, as the ratios of inflow to foul outlet underflow (or weir overflow to foul outlet underflow) decrease, the SS removal advantage from swirl concentrating also decreases, since the intercepted hydraulic loading to underflow becomes more significant in the net mass loading calculation of the hypothetical or conventional regulator, according to the equation:

$$\text{Hypothetical Regulator Percent SS Mass Removal} = \frac{Q_iC_i - Q_oC_o}{Q_iC_i} \times 100 = \frac{Q_i - Q_o}{Q_i} \times 100 = \frac{Q_f}{Q_i} \times 100$$

Where Q_i = influent flow

Q_o = overflow discharged to receiving stream

Q_f = foul sewer flow

TABLE 24. SWIRL REGULATOR/CONCENTRATOR SUSPENDED SOLIDS REMOVAL[1]

		Swirl Concentrator						Conv. Regulator			
		Average SS per storm, mg/l			Mass Loading lb			Mass Loading lb			
Storm No.	Avg. Flow MGD	Inf.	Eff.	Rem.[2]	Inf.	Eff.	Rem.[2]	Inf.	Underflow	Rem.[3]	
2-74	2.2	535	345	36	824	394	52	824	222	27	25%
3-74	1.0	182	141	23	152	75	51	152	73	48	3%
7-74	2.8	110	90	18	205	134	34	205	44	22	12%
10-74	2.9	230	164	29	564	295	48	564	108	19	29%
14-74	2.2	159	123	23	218	126	42	218	57	26	16%
1-75	0.5	374	167	55	227	53	77	227	145	64	13%
2-75	1.5	342	202	41	1020	368	64	1020	374	34	30%
6-75	1.4	342	259	24	247	137	45	247	68	27	18%
12-75	2.8	291	232	20	551	370	33	551	106	19	14%
14-75	2.4	163	119	27	367	216	41	367	80	22	19%
15-75	3.3	115	55	52	258	46	82	258	159	61	21%
Mean	1.9	276	176	36	496	221	55	496	164	33	

36.66%

37%

1. SS removals were calculated from actual measured data sets of influent and effluent samples.

2. Data reflecting negative SS removals at tail end of storms not included.

3. For the conventional regulator removal calculation, it is assumed that the SS concentration of the foul underflow equals the SS concentration of the inflow.

4. Conversion: 1 lb x 0.454 = kg

$C_o = C_f = C_i$ = SS concentration in influent

This phenomenon can be illustrated by comparing the removals shown in Table 24 of Storm 10-74 in which the average flow was relatively high (2.91 MGD) (7.7 cu m/min) to Storm 1-75 in which the average flow was relatively low (0.50 MGD) (13 cu m/min). Comparison of the removal efficiency of the swirl versus a conventional regulator shows that for Storm 10-74, the SS removal by the swirl was 48 percent compared to 19 percent for the conventional regulator, a difference of 29 percent. For Storm 1-75, the difference in SS removal was only 13 percent.

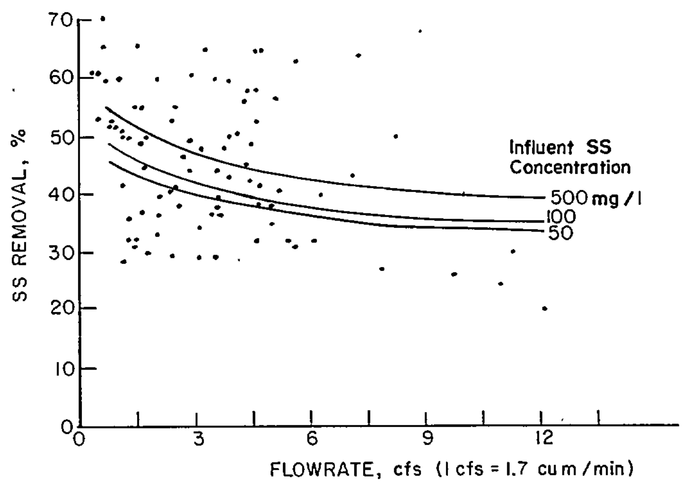


FIGURE 33. Prototype Swirl Regulator/Concentrator SS Removal

Despite the apparent decreased advantage of swirl concentration at low flows it is important to consider that many overflow outfalls are designed to pass 20, 100 or even 1,000 times average dry-weather flow, as opposed to West Newell Street which, at best, passes only 10 times average dry-weather flow. For the former cases, the swirl concentrating effect will exhibit distinct advantages over conventional regulators for SS removal.

Figure 33 illustrates that at increased flowrates, the percent concentration removal of SS is decreased. In addition, increased influent SS concentrations tended to result in increased removals of SS. It is believed that the SS concentrations in the CSO fluctuate in response to scouring velocities in the sewer line. Thus, wastewaters during the first flush tend to have a greater proportion of solids of larger size and specific gravity which are more easily removed by the swirl regulator/concentrator.

In order to account for the influences of both flowrate and influent SS concentrations, the performance data of the swirl was statistically fitted using multiple regression analysis to an equation of the form:

$$\text{Percent SS Removal} = K_1 Q^{K2} SS^{K3}$$

where K_1, K_2, K_3 are constants, Q is hydraulic flow or flux to the unit (MGD or gpm/ft^2)(cu m/min or m/hr) and SS is the influent SS concentration (mg/l). The results of the regression analysis are shown in Table 25. The signs associated with the regression coefficients indicate that SS concentration removals generally increased with an increase in influent SS concentrations and decreased with increasing flowrate. Values of "t" associated with Q and SS indicated degrees of confidence of greater than 99 and 95 percent, respectively, and an "F" value for the overall expression of greater than 99 percent.

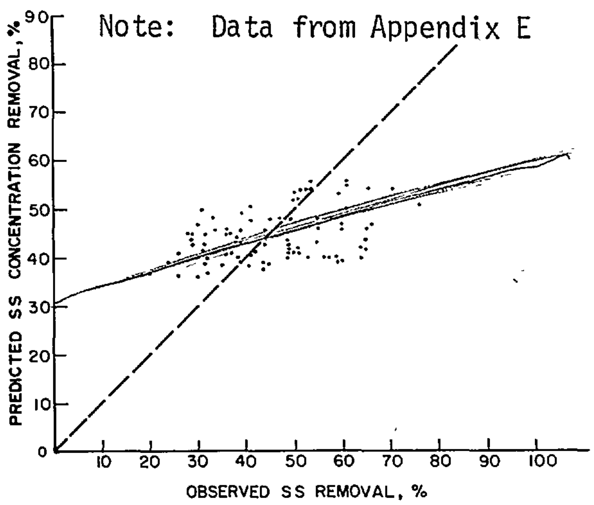


FIGURE 34. Observed vs Predicted SS Removal-Swirl Prototype

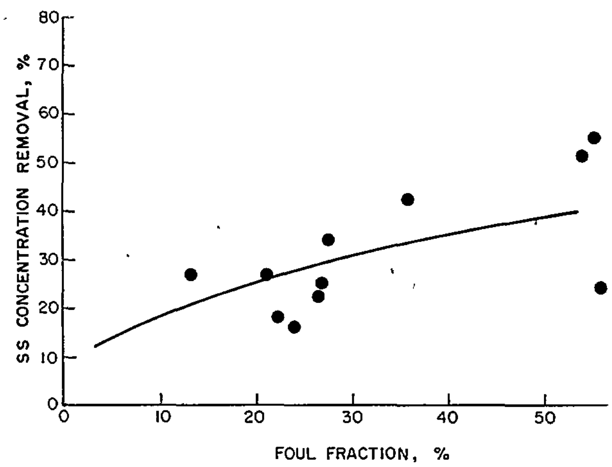


FIGURE 35. Percent Foul Fraction vs SS Concentration Removal - Swirl Prototype

The final regression equation thus obtained was

$$\text{Percent SS Concentration Removal} = 32.4\ Q^{-0.11} SS^{0.07}$$

The trends indicated by the regression equation are presented in Figure 33 where the higher flowrates indicate lower SS removals for given influent SS concentrations. A plot of the observed SS removal versus predicted SS removals is presented in Figure 34.

A regression analysis of SS removal rates versus the foul fraction flow as a percentage of influent flow indicated a slight increase in SS removal efficiency with increased foul fraction. The results are presented in Figure 35 along with measured data of 11 overflow events at the West Newell Street facility, consisting of an average of seven data points per storm, or a total of 79 data points. A possible explanation for this effect is that at increased foul fractions, the SS deposited around the floor gutters and outlet are removed more effectively, and the chances of

TABLE 25. MULTIPLE REGRESSION ANALYSIS FOR SWIRL PROTOTYPE SS REMOVAL

Independent Variable	Mean	Standard Deviation	Correlation X vs Y	Regression Coefficient	Std. Error of Regr. Coef.	Computed t-Value
Log Q, MGD	0.25	0.33	-0.34	-0.11	0.04	-2.97
Log SS, mg/l	2.31	0.36	0.27	0.07	0.04	2.04
Dependent Variable						
Log (Percent SS Removal)	1.65	0.12				
Intercept		1.51				
Multiple Correlation		0.40				
Std. Error of Estimate		0.12				

Analysis of Variance for the Regression

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Squares	V Value
Attributable to Regression	2	0.21	0.11	7.94
Deviation from Regression	84	1.11	0.01	
Total	86	1.33		

resuspension and loss to overflow are lessened.

ORGANICS REMOVAL

Since nonbiodegradable synthesized solids were used, no evaluation of BOD_5 removal was made in the laboratory swirl hydraulic model. Prototype analyses indicated greater than 50 percent BOD_5 removals in terms of mass loading and concentration (Table 26). Total mass loading removals and treatment efficiencies in terms of concentration ranged from 50 to 82 percent and 29 to 79 percent, respectively.

TABLE 26. SWIRL PROTOTYPE BOD_5 REMOVAL

		Average BOD_5 per storm, mg/l			Mass Loading, lb		
Storm No.	Avg. Flow MGD	Influent	Effluent	Percent Removal	Influent	Effluent	Percent Removal
7-74	2.8	314	65	79	610	106	82
1-75	0.5	165	112	32	214	66	69
2-75	1.5	99	70	29	385	189	51
Mean	1.6	193	82	58	403	120	70

Conversion: 1 lbx0.454 = 1 kg
1 MGDx2.63 = 1 cu m/min

Twelve overflow events were monitored for TOC at West Newell Street during this project. Data collected showed removals of TOC by the swirl unit as indicated in Table 27. The TOC removals achieved in individual storms ranged from 5 to 53 percent in terms of concentration. A certain portion of the CSO influent flow is diverted back to the interceptor via the foul sewer outlet. This portion of flow diverted back to the interceptor varies greatly depending upon the severity of the storm and in this study ranged from an average of 14 to 59 percent for individual storms and an overall average of 31 percent. At future CSO treatment sites, the foul sewer fraction may not be as variable depending on the hydraulics at those sites. An overall time-weighted average calculated for all storm data indicates 33 percent removal of TOC in terms of concentration.

TABLE 27. SWIRL PROTOTYPE TOC AND VSS REMOVALS

Storm No.	TOC, mg/l Influent	TOC, mg/l Effluent	TOC, mg/l Percent Removal	VSS, mg/l Influent	VSS, mg/l Effluent	VSS, mg/l Percent Removal	VSS/SS
2-74	166	99	40	304	203	33	57
3-74	56	35	38	135	73	46	74
7-74	139	65	53	68	38	44	62
10-74	83	66	20	65	60	8	28
14-74	48	35	27	96	64	33	60
1-75	128	84	34	94	80	15	25
2-75	44	38	14	173	121	30	51
6-75	62	44	29	130	65	50	38
10-75	202	118	42	289	170	41	41
12-75	39	37	5	132	77	42	45
13-75	30	27	10	-	-	-	-
14-75	25	19	24	46	21	54	29
Time-Weighted Average	80	54	33	132	87	34	46

Removals of VSS for individual storms are also presented in Table 27. The overall time-weighted average removal of VSS at the West Newell Street swirl was 34 percent, with a range for individual storms of 8 to 54 percent. Generally speaking, the removal of VSS paralleled TOC removal rates although there were exceptions. A general tendency of increased VSS removals at the higher VSS/SS ratios was observed as depicted in Figure 36. The data had an R value of 0.54.

9

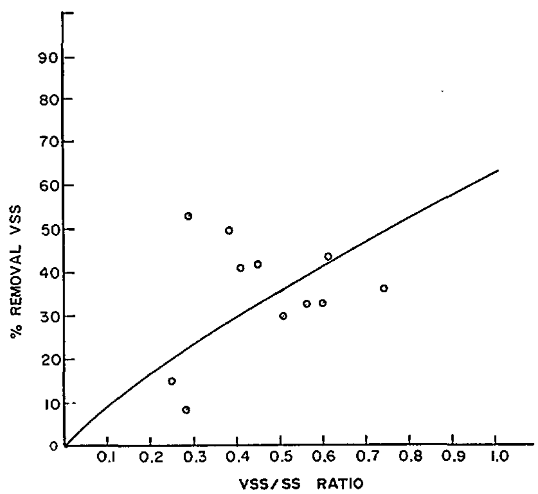


FIGURE 36. VSS/SS Ratio vs VSS Removal - Swirl Prototype

HEAVY METALS REMOVAL

Heavy metals concentrations at this site were generally low except for Fe and Zn. Pb and Cu were measurable and Cr, Cd, and Ni were barely detectable. Table 28 presents a summary of the heavy metals concentrations for all storms. All heavy metals were generally less than 1 mg/l in concentration except for occasional spikes, particularly with regard to Fe. Removal of metals by swirl concentration was erratic as indicated in Table 28. Evaluations for each of these parameters did not yield consistent removals throughout any of the storms. It is interesting to note that the standard deviation of effluent heavy metals concentrations was always less than the standard deviation of influent concentrations. Apparently heavy metal influent spikes are dampened out by passage through the swirl unit. However, removal of heavy metals by swirl concentration would have to be considered incidental to the units more effective role in removal of suspended solids.

COARSE FLOATABLES REMOVAL

The coarse floatables/scum removal mechanism worked satisfactorily. Visual observations during overflow events revealed floatables to be effectively contained by the scum ring in the outer ring of the chamber and forced into the floatables trap (under the weir plate) by the swirl action for subsequent drawn-down and removal to the foul sewer during dry weather. Figure 37 illustrates floatables entrapment during wet-weather operation.

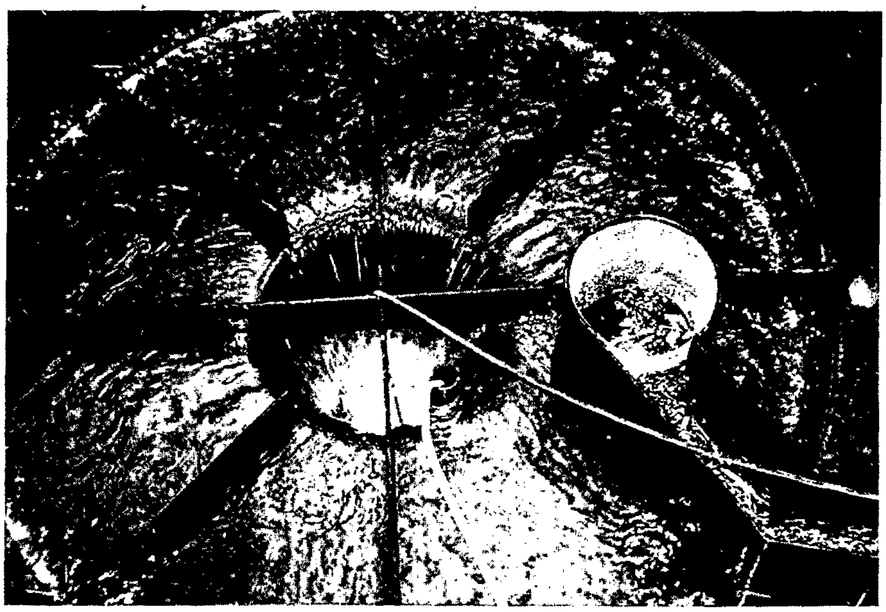


FIGURE 37. Coarse Floatables Removal - Swirl Prototype

TABLE 28 SWIRL PROTOTYPE HEAVY METALS REMOVAL

Heavy Metal	No. Points	Concentration, mg/l Influent Mean	Influent Std Dev.	Effluent Mean	Effluent Std. Dev.	Range of Influent Values, mg/l Low	High
Fe	46	2.26	4.28	2.29	3.74	0.10	19.10
Cr	52	0.00	0.00	0.00	0.00	0.00	0.01
Cu	46	0.05	0.09	0.05	0.05	0.00	0.14
Pb	42	0.01	0.04	0.01	0.03	0.00	0.20
Zn	50	0.55	1.60	0.13	0.07	0.02	0.34
Cd	42	0.00	0.01	0.00	0.00	0.00	0.02
Ni	26	0.00	0.00	0.00	0.00	0.00	0.00

SECTION 10

SOLIDS HANDLING CONSIDERATIONS - RESULTS AND DISCUSSION

GENERAL

A major factor to be considered in design of CSO treatment facilities is disposal of solids. Therefore, an attempt is made in this report to define the impact additional solids loadings to the Syracuse Metropolitan Sewage Treatment Plant (Metro) would have on existing solids handling capabilities and treatment efficiency. From this specific example, inferences may be drawn on the general quantities and impacts of solids in other situations.

In performing these analyses, a number of simplifying assumptions are made concerning the sewage collection and treatment system. First, it is assumed that the main intercepting system has the hydraulic capacity to accept CSO treatment residual wastewater from satellite CSO treatment facilities. In reality, the Syracuse intercepting system has little wet-weather hydraulic capacity, which in itself is a major cause of the overflows. Second, it is assumed that once the CSO residual wastewaters enter the intercepting system, no settling of the heavier particles occurs in the collection system, i.e., all residual wastes entering the collection system arrive at Metro. Third, it is assumed that the flow of CSO residual wastes occurs over a 24 hr period. This assumption implies that a solids concentrate holding tank has been constructed at each of the satellite CSO treatment facilities. In the actual Syracuse CSO system, overflows for the storms selected for examination occured over a time frame of four to six hours. Last, it is assumed that the flow of CSO residuals occurs under average DWF conditions.

Any changes in the assumptions outlined above would require new evaluations as to impact of CSO treatment residuals. These changes should be fully re-examined upon selection of a final CSO residual waste handling alternative.

Three cases have been considered in this section, as follows:

1. Full return of CSO treatment residuals to Metro.
2. Return of dilute CSO treatment residuals to Metro.
3. Direct transmission of CSO treatment residuals to Metro sludge handling facilities.

FULL RETURN OF CSO TREATMENT RESIDUALS TO METRO

Since the characteristics of sludge generated at the Syracuse demonstration sites were not determined in this study, considerable reference to published data is made in the analysis of the impact of CSO sludges on existing treatment facilities (52).

The total sludge loading from the Syracuse CSO system has been projected under four hypothetical conditions, assuming complete treatment by each of the four devices investigated in this program (Zurn, Crane, Sweco, swirl regulator/concentrator).

For comparative purposes, the concentration of SS contained in the effluent from each of the three screening units was estimated by assuming the SS removal efficiency of each unit to be 40 percent at an influent SS concentration of 325 mg/l, the time-weighted average influent SS concentration to the screening units experienced in the demonstration study. Table 29 presents a preliminary estimate of the quantity of sludge solids that would be produced in Syracuse assuming full treatment of all significant CSO. The estimate does not include provisions for increased solids in the sludge resulting from chemical addition to the process wastewaters.

Hydraulic Loading Considerations

Since the volume of sludge which was returned to the interceptor was estimated at 3 percent for the Zurn and Crane microscreens and 25 percent for the Sweco unit during the demonstration study, these figures were used in developing the sludge quantities and volumes produced by the microscreens. The average foul sewer fraction returned to the interceptor by the swirl unit was approximately 30 percent. However, this percentage is considered to be higher than is necessary for proper operation of the unit. For this reason, the foul sewer fraction used in estimating sludge quantities was taken as 3 percent for the swirl regulator/concentrator. The 3 percent value corresponds to the foul fraction investigated in the hydraulic model studies in the LaSalle Hydraulic Laboratory.

Table 29 illustrates the effect of discharges of CSO treatment sludges on the hydraulic loading at Metro for each of the hypothetical treatment systems. Volumes of overflow for the two storm events considered in this analysis are equal to 16.7 MG (63,200 cu m) from an average storm of 0.22 in. (0.56 cm) total rainfall and 83 MG (314,000 cu m) from a 1 year-2 hour storm which has 1.11 in. (2.82 cm) of total rain.

Data presented for the average storm in Table 29 indicates that the hydraulic loading to Metro is increased by about 5 percent when the Sweco unit is used as the CSO treatment device, and by less than 1 percent when the Zurn, Crane, and swirl units are individually considered to be the treatment units. For the CSO treatment conditions presented for the average storm, no hydraulic overload results from CSO treatment residual wastes, since Metro was designed to operate as a secondary

TABLE 29. EFFECT OF CSO TREATMENT SLUDGES ON HYDRAULIC LOADING AT METRO STP

CSO Treatment Process	Average Storm = 0.22 in. Total Rainfall				1 Year-2 Hour Storm = 1.11 in. Total Rainfall			
	Sludge Volume		Sludge Volume Plus Average DWF	Hydraulic Overload	Sludge Volume		Sludge Volume Plus Average DWF	Hydraulic Overload
	Percent of CSO to Metro	MGD*	MGD		Percent of CSO to Metro	MGD*	MGD	
Sweco Unit	25	4.2	84.2	No	25	20.8	100.8	No
Zurn Unit	3	0.5	80.5	No	3	2.5	82.5	No
Crane Unit	3	0.5	80.5	No	3	2.5	82.5	No
Swirl Unit	3	0.5	80.5	No	3	2.5	82.5	No

*Based on total CSO concentrate flow volume returned to Metro STP over 24 hr period.

Notes:

1. DWF = Dry Weather Flow, average of 80 MGD
2. CSO Volume Treated = 16.7 MG (average storm overflow)
   = 83 MG (1 year-2 hour storm overflow)
3. Hydraulic Overload determinations made by comparing Sludge Volume Plus Average DWF with maximum design solids loading of 1.5 x DWF solids loading.
4. Conversions: MGD x 3.785 = cu m/day
   in. x 2.54 = cm

treatment facility up to a hydraulic loading rate of 1.5 times the average DWF hydraulic loading.

Data for the 1 year - 2 hour storm indicate that the hydraulic loading would increase by about 26 percent for the Sweco CSO treatment system and by about 3 percent for the other units considered individually as the CSO treatment devices. No hydraulic overload results from CSO residual wastes for the 1 year - 2 hour storm.

Solids Loading Considerations

A second factor that must be considered when determining the effects of CSO treatment residuals is the impact of increased solids loadings on Metro. For Metro the design solids loadings is 120,000 lb/day (54,500 kg/day) to the primary treatment facilities. For purposes of this analysis, it is assumed that solids loadings up to 1.5 times the design loading could be tolerated for short periods of time without drastically upsetting treatment processes. It is further assumed that only a small percentage of the total SS entering the treatment plant is removed in the aerated grit facilities. Since the CSO bleedback solids concentrations are highly variable in nature, the above assumption is considered sufficient for illustrative purposes.

A solids overload would not result from transmission of CSO treatment residual wastes from the average storm, but a solids overload would result at Metro for the 1 year - 2 hour storm. The unit solids loading to the primary clarifiers at Metro would increase by around 100% during the 1 year -2 hour storm, to an unacceptably high level greater than 180,000 lb/day (81,720 kg/day). The result of the excessive solids loading (greater than 1.5 times the design loading) could be lowered primary effluent quality and overall treatment efficiency. Adverse effects of the solids overload would probably be carried over to the aeration tanks and secondary clarifiers.

Table 30 indicates the effects on Metro of CSO residual solids from treatment by the various treatment units for the average storm and 1 year - 2 hour storm.

Organic Loading Considerations

One of the criteria used in evaluating organic overload is associated with the activated sludge portion of the sewage treatment process. In calculating the BOD_5 characteristics presented in Table 31 for the specific treatment units studied in this demonstration project (Sweco, Zurn, Crane and swirl regulator/concentrator, an initial concentration of BOD_5 in the CSO of 90 mg/l was used.

It is assumed, based on data collected during this demonstration study, that 20 percent of the BOD_5 in CSO is removed by the microscreening units, and 40 percent of the BOD_5 is removed by the swirl regulator/concentrator. Use of these estimates results in BOD_5 concentration in the

TABLE 30. EFFECT OF CSO TREATMENT SLUDGES ON SOLIDS LOADING AT METRO STP

CSO Treatment Process	Average Storm = 0.22 in. Total Rainfall						1 Year-2 Hour Storm = 1.11 in. Total Rainfall					
	Sludge to Metro MGD	Percent Solids	Dry Solids lb/day	Solids Removal Percent	CSO + DWF Solids lb/day	Solids Overload	Sludge to Metro MGD	Percent Solids	Dry Solids lb/day	Solids Removal Percent	CSO + DWF Solids lb/day	Solids Overload
Sweco Unit	4.2	0.075	26,300	40	146,300	No	20.8	0.075	130,000	40	250,000	Yes
Zurn Unit	0.5	0.45	18,800	40	138,800	No	2.5	0.45	93,800	40	213,800	Yes
Crane Unit	0.5	0.45	18,800	40	138,800	No	2.5	0.45	93,800	40	213,800	Yes
Swirl Unit	0.5	0.45	18,800	40	138,800	No	2.5	0.45	93,800	40	213,800	Yes

Notes:

1. DWF = Dry Weather Flow of 80 MGD (average), 120 MGD (maximum design)
2. Average DWF solids loading at Metro of 120,000 lb/day (average)
3. CSO Treated = 16.7 MG (average storm flow)
   = 83.0 MG (1 year-2 hour storm flow)
4. Solids Overload determination made by comparing CSO + DWF Solids with the design solids loading of 1.5 x design solids loading = 180,000 lb/day
5. Conversions: MGD x 3.785 = cu m/day
   lb/day x 0.454 = kg/day
   in. x 2.54 = cm

TABLE 31. ORGANIC CHARACTERISTICS (BOD) OF CSO TREATMENT SLUDGES

		Average Storm = 0.22 in. Total Rainfall				1 Year-2 Hour Storm = 1.11 in. Total Rainfall			
CSO Treatment Process	BOD mg/l	Volume to Metro MGD	BOD to Metro lb/day	BOD Removed By Primary Treatment lb/day	BOD to Activated Sludge lb/day	Volume to Metro MGD	BOD to Metro lb/day	BOD Removed By Primary Treatment lb/day	BOD to Activated Sludge lb/day
Sweco Unit	150	4.2	5250	1050	4200	20.8	26,000	5200	20,800
Zurn Unit	675	0.5	2820	560	2260	2.5	14,100	2820	11,280
Crane Unit	675	0.5	2820	560	2260	2.5	14,100	2820	11,280
Swirl Unit	1250	0.5	5210	1040	4170	2.5	26,000	5200	20,800

Notes:

1. Assumes 20 percent BOD removal by primary treatment at Metro at hydraulic loading rates up to 1050 gpd/ft^2
2. Conversions: MGD x 3.785 = cu m/day
   lb/day x 0.454 = kg/day
   in. x 2.54 = cm

Sweco unit sludge of 150 mg/l, in the Zurn and Crane unit sludges of 675 mg/l, and in the swirl unit sludge of 1250 mg/l. The calculations for Table 31 are based on full treatment of all Syracuse CSO by each of the CSO treatment processes.

In general, the analysis of organic loadings to Metro of CSO treatment residuals indicates that return of residuals to the sewer system during storm events would be acceptable only when the storm occurs during average or less than average DWF periods.

The solids loadings imposed on the secondary treatment units also affects the overall operation of secondary treatment facilities. The secondary clarifiers at Metro are designed for close to the maximum recommended solids loading of 30 $lb/day/ft^2$ (146 $kg/day/m^2$) (52). However, sufficient reserve capacity is available so that, as indicated in Table 32, a solids overload to the secondary clarifiers would not result from return of CSO treatment sludge over a 24 hr period for either the average storm or the 1 year - 2 hr storm.

Significant impact on the sludge handling facilities is not anticipated for the situation where CSO residual bleedback is directed to the head of the treatment plant since the rate of drawoff of the primary and secondary sludges can be limited to prevent hydraulic overloading of the gravity thickeners. The SS loading rate to the gravity thickeners would be increased by as much as 35 percent for the 1 year-2 hour storm and result in decreased efficiency in the thickeners, and higher loadings to the digesters.

RETURN OF DILUTE CSO TREATMENT RESIDUALS TO METRO

An alternative to full return of raw CSO treatment sludges is to return dilute residuals resulting from on-site dewatering of CSO sludges. Dewatered sludges in this case would be transported from the CSO treatment sites to ultimate disposal elsewhere. Although it is probably uneconomical to dewater CSO treatment sludges at individual overflow points, calculations are presented in Table 33 to illustrate the probable impact of on-site dewatering to reduce the quantity of solids sent to Metro from CSO treatment facilities. The data presented in Table 33 are based on the sludge from the Sweco unit at one percent solids and for the Zurn, Crane and Swirl units at 10 percent solids. The concentration of solids in the Sweco unit sludge prior to dewatering is approximately 750 mg/l and it is unlikely that the solids content could be increased to much greater than one percent even with thickening and vacuum filtration.

Table 33 indicates that for the 1 year - 2 hour storm, solids overload at Metro can be prevented when using the Zurn, Crane or swirl unit as the CSO treatment device. However, the upper limit of permissible solids loading of 180,000 lb/day (81,720 kg/day) as defined earlier would be reached if the Sweco unit were used as the CSO treatment device, largely as a result of the large volume of dilute residuals, 20 MGD (75.7 cu m/day), transmitted to Metro from the Sweco unit.

TABLE 32. SOLIDS LOADING TO SECONDARY FACILITIES AT METRO STP

CSO Treatment Process	Average Storm = 0.22 in. Total Rainfall				1 Year-2 Hour Storm = 1.11 in. Total Rainfall			
	Bleedback Solids to Metro lb/day	Solids Removed By Primary Treatment lb/day	Solids to Activated Sludge lb/day	Solids Overload	Solids to Metro lb/day	Solids Removed By Primary Treatment lb/day	Solids to Activated Sludge lb/day	Solids Overload
Sweco Unit	26,300	15,780	10,520	No	130,000	78,000	52,000	No
Zurn Unit	18,800	12,400	6,400	No	93,000	61,900	31,900	No
Crane Unit	18,800	12,400	6,400	No	93,800	61,900	31,900	No
Swirl Unit	18,800	12,400	6,400	No	93,800	16,900	31,900	No

Notes:
1. Bleedback solids obtained from Table 30.
2. Assumed 66 percent SS removed by primary treatment at overflow rates up to 750 gpd/ft^2
   Assumed 60 percent SS removed by primary treatment at overflow rates up to 1050 gpd/ft^2
3. Solids overload established when the CSO solids to the activated sludge system exceeded 30 lb/day/ft^2
4. Conversions: lb/day x 0.454 = kg/day
   in. x 2.54 = cm

TABLE 33. EFFECT OF DILUTE EFFLUENT FROM ON-SITE DEWATERING OF CSO SLUDGES TO METRO STP

	1 Year-2 Hour Storm - 1.11 in. Total Rainfall						
CSO Treatment Process	Dewatered MGD	Dilute Effluent Flow MGD	SS mg/l	Solids lb/day	CSO Effluent + DWF Flow MGD	Solids lb/day	Solids Overload
Sweco	20.8	20.0	380	64,300	100.0	184,300	Slight
Zurn	2.5	2.4	521	10,400	82.4	130,400	No
Crane	2.5	2.4	521	10,400	82.4	130,400	No
Swirl	2.5	2.4	521	10,400	82.4	130,400	No

Notes:

1. Dilute Effluent Solids Concentration based on 1 percent sludge solid content for Sweco unit and 10 percent sludge solids content for the other three units.
2. Dilute Effluent flow volume based on 96.0 percent of influent flow to CSO treatment facilities returned to Metro STP for all units.
3. Calculation of SS concentration in dilute residual bleedback:
   Sweco unit: (20.8 MGD) (750 mg/l)= (0.8 MGD) (10,000 mg/l) + (20.0 MGD)(c) ∴ c = 380 mg/l
   Zurn, Crane, Swirl units: (2.5 MGD)(4500 mg/l) = (0.1 MGD) (100,000 mg/l) + (2.4 MGD)(c) ∴ c = 521 mg/l
4. Conversions: MGD x 3.785 = cu m/day
   lb/day x 0.454 = kg/day
   in. x 2.54 = cm

DIRECT TRANSMISSION OF CSO TREATMENT RESIDUALS TO METRO SLUDGE HANDLING FACILITIES

Although the volume of dry weather residual sludges obtained at a sewage treatment plant is relatively small, usually 2 to 3 percent of the wastewater volume treated, sludge handling and disposal is complex, troublesome, and can represent up to 25 to 50 percent of the capital and operating costs of a typical sewage treatment plant (52).

At Metro, sludge handling for ultimate disposal consists of a series of dewatering steps in which the volume of sludge is progressively reduced by removal of water associated with the sludge solids. The major portion of water removed is accomplished by gravity thickening. Further treatment and sludge volume reduction is obtained by anaerobic digestion in primary and secondary digesters. Final discharge of the digested sludge is to sludge drying beds.

The sludges discharging to the gravity thickeners consist of combined sludges from primary treatment and contact stabilization activated sludge processes. Detention time in the thickeners is about 7 hours. The normal thickened sludge concentration from the gravity thickeners is projected to be 6 percent solids.

Further reduction of sludge volume is achieved by passage of the sludge through high-rate primary digesters and a secondary digester in series. The solids retention time in the primary digesters is designed at 15 days. The primary digester underflow concentration is designed to be 4.4 percent solids (dry basis). The design retention time in the secondary digester is 4.5 days. This is expected to produce an underflow solids concentration of digested sludge of 8 percent (dry basis).

Hydraulic Loading Considerations

The daily design volume of sewage sludge to the sludge handling facilities at Metro is 2.9 MGD (10,970 m^3/day). Shown in Table 29 are projections of CSO treatment sludge volumes. Table 29 indicates that for the average storm, and with the Sweco unit used as the CSO treatment device, the volume of CSO sludge is much higher than the design daily dry-weather sludge anticipated. For the other treatment devices investigated (Zurn, Crane and swirl units), the CSO treatment sludge represents a comparatively minor fraction of the design daily dry-weather sludge. If the CSO sludge were transmitted directly to the Metro sludge handling facilities, solids retention time in the gravity thickeners would be decreased to about 3 hr for the Sweco unit, and to about 6 hr for the other units. For the 1 year - 2 hr storm, the retention time would be decreased to less than 1 hr for the Sweco unit, and to less than 4 hr for the other units.

The effect of transmitting Sweco unit CSO sludge to the Metro sludge handling facilities would be an unacceptable hydraulic overload. Direct transmission of sludge from the other units would result in severe impacts, which might be tolerated for short periods of time. The detention time in the primary and secondary digestors would also be shortened as the result of adding CSO treatment sludge. The hydraulic overload could be expected to result in greater return of incomplete digestion products to the head of the treatment plant, adversely affecting overall plant performance by increasing the total solids loadings entering the primary and secondary treatment units.

Solids Loading Considerations

The daily design loading of dry sewage solids to the Metro sludge handling facilities is 190,000 lb/day (86,300 kg/day). Projected CSO treatment solids (dry weight basis) are presented in Table 33. For the average storm, the solids generated by any of the four treatment units considered would be only a small fraction of the design sewage solids at Metro. However, for the 1 year - 2 hr storm, the CSO treatment solids would be around 50-60 percent of design sewage solids. The total solids loading at the Metro sludge handling facilities for the 1 year - 2 hr storm would be approximately equal to the peak allowable loading. Excess loading rates might be experienced if back-to-back overflow events occurred.

Organic and Inert Solids Considerations

The organic fraction of Metro sludge has been estimated to be 55 percent on a dry solids basis. The design volatile solids loading to the sludge handling facilities is 104,500 lb/day (47,400 kg/day). Projected CSO treatment volatile solids (dry weight basis) are presented in Table 34. The results of analysis indicate that for both the average storm and the 1 year - 2 hr storm, the additional volatile solids loading to the Metro primary digesters resulting from CSO treatment would not cause excessive loading.

TABLE 34. VOLATILE SOLIDS CONTENT OF SLUDGES FROM VARIOUS CSO TREATMENT DEVICES

CSO Treatment Process	Percent Volatile Solids	Average Storm = 0.22 in. Total Rainfall			1 Year-2 Hour Storm = 1.11 in. Total Rainfall		
		Total Solids lb/day	Volatile Solids lb/day	Inert Solids lb/day	Total Solids lb/day	Volatile Solids lb/day	Inert Solids lb/day
Sweco	56	26,300	14,730	11,570	130,000	72,800	57,200
Zurn	61	18,800	11,470	7,330	93,800	57,200	36,600
Crane	61	18,800	11,470	7,330	93,800	57,200	36,600
Swirl	45	18,800	8,460	10,340	93,800	42,200	51,600

Notes:
1. Conversions: lb/day x 0.454 = kg/day
in. x 2.54 = cm

SECTION 11

DISINFECTION-RESULTS AND DISCUSSION

GENERAL

One of the major objectives of this project was to determine the feasibility of high-rate disinfection of CSO following some level of solids removal. Owing to the short distances and limited spaces for construction between overflow structures and the points of discharge, and the high-volume rapid flows associated with the majority of overflows in the City of Syracuse, contact times of one minute were investigated.

Studies had previously been conducted on a bench-scale level to optimize dosages required for adequate disinfection of CSO and to aid in determining the various design parameters for full-scale prototype facilities (7). The results of the bench-scale work are summarized in Section 5 of this report, and a description of the facilities constructed appears in Section 6.

The facilities at both the Maltbie Street and West Newell Street sites included instrumentation for automatic feed of disinfectant, such that a constant dosage of disinfectant would be applied to the treated CSO even under varying flowrates. However, a number of problems arose through both the limitation of storm events (overflow would not occur unless there was a severe storm) and malfunctions of disinfection equipment, which prevented significant disinfection investigations at the West Newell Street site. Notation of these problems is given in Section 6.

All the findings and results presented in this section relate to disinfection performance of Cl_2 and ClO_2 at the Maltbie Street site following screening. The operation schedule for disinfection at Maltbie Street is given in Table 35.

ClO_2 DISINFECTION - 1975 OPERATIONS

Eight storms were evaluated for reduction of fecal coliform (FC) levels by ClO_2 disinfection on the Sweco treatment system and four storms were evaluated on the Zurn system. ClO_2 generating equipment problems on the Zurn process train resulted in four of the eight storms not being evaluated for that unit. Various back pressure valves and pump diaphrams failed, limiting use of this ClO_2 generator. Figures 38 through

TABLE 35. MALTBIE STREET OPERATION SCHEDULE - DISINFECTION

Overflow	Avg. Screen Loading Rate (gpm/ft^2)			Screen Aperture Size (microns)			Cl_2 Dosage (mg/l)			ClO_2 Dosage (mg/l)			Mixing		
	Sweco	Crane	Zurn	Sweco	Crane	Zurn	Tank #1	Tank #2	Tank #3	Tank #1	Tank #2	Tank #3	Tank #1	Tank #2	Tank #3
2-75	40	-	12	105	-	71	0	0	0	1-6	0	0	F	-	-
3-75	36	-	11	105	-	71	0	0	0	0-6	0	0-4	F	-	SF
4-75	32	-	7	105	-	71	0	0	0	0	0	0	-	-	-
5-75	28	-	9	105	-	71	0	0	0	7-8	0	3-5	F	-	SF
6-75	43	-	10	105	-	71	0	0	0	5-7	0	5-7	F	-	SF
7-75	33	-	8	105	-	71	0	0	0	7-8	0	0	F	-	-
8-75	66	-	4	105	-	71	0	0	0	3-4	0	3-10	F	-	SF
10-75	-	-	12	-	-	71	0	0	0	4-9	0	0	F	-	-
11-75	50	-	9	105	-	71	0	0	0	5-6	0	0	F	-	-
16-75	11	2	3	105	23	71	0	0	0	0	0	0	-	-	-
1-76	-	-	no screen	-	-	no screen	-	-	9-24	-	-	0	-	-	SF
2-76	-	-	no screen	-	-	no screen	-	-	12	-	-	0	-	-	SF
3-76	-	8	no screen	-	-	no screen	-	12	0	-	0	11.0	-	F	SF
4-76	-	8	no screen	-	23	no screen	-	0	12	-	3.4	0	-	F	SF
5-76	-	8	no screen	-	23	no screen	-	8	8	-	2	2	-	F	SF
6-76	-	-	no screen	-	-	no screen	-	-	7.3	-	-	0	-	-	SF

F - Single flash mixing
SF - Sequential flash mixing

45 relate to the Sweco disinfection system results and Figures 46 through 49 relate to the Zurn disinfection system results. The disinfection results were plotted on log-normal paper to facilitate evaluation of the bacterial kills for various ClO_2 dosages and one minute detention time.

Figure 38 indicates FC kills of 1 to 3 logs at ClO_2 dosages of 3 to 7 mg/l. The results presented in Figure 39 indicate that at dosages of less than 1 mg/l, no reduction of FC populations were achieved; however, as the dosage increased from less than 1 mg/l to 4 to 6 mg/l, the reduction increased to 1 to 2 logs. Figure 40 indicates 4 log reduction of FC at ClO_2 dosages of 7 to 8 mg/l. During this storm, the actual FC populations were reduced to less than 10 counts/100 ml, well below the desired level of 200 counts/100 ml as discussed in reference (7). Figure 41 exhibits 2 to 5 log reductions at dosages of 5 to 7 mg/l ClO_2. The lower reductions for this storm (in the order of 2 logs) were observed during the first hour, indicating that an increased ClO_2 demand in the first flush adversely affected disinfection. Figure 42 indicates 2 to 3 log reductions of FC achieved at ClO_2 dosages of 3 to 4 mg/l with the higher levels of reduction achieved during the later stages of the overflow. Figure 43 illustrates 2 to 3 log reductions achieved at ClO_2 dosages of 3.5 to 6.0 mg/l ClO_2, with the standard of 200 counts FC/100 ml being achieved during the latter stages of the overflow.

Figure 44 shows 3 to 4 log reductions of FC being achieved throughout the overflow after the first 30 min. Initial dosages of 7.7 to 8.7 mg/l of ClO_2 reduced FC populations to less than 10 counts/100 ml. This level of reduction was maintained through the remainder of the storm at reduced ClO_2 dosages of 4.4 to 5.0 mg/l. Figure 45 exhibits 2 to 3 log reductions of FC at dosages of 5.4 to 5.9 mg/l. Again, the better reductions were achieved during the latter stages of the storm overflow and were sufficient to reach the desired level of 200 counts/100 ml.

Data collected on the disinfection system associated with the Sweco unit indicate that the desired level of 200 counts FC/100 ml in the effluent was reached in all storms when greater than 4 mg/l ClO_2 was injected to the system after the first 30 to 45 min of the overflow. During the first 30 to 45 min, dosages of 7 to 8 mg/l were required to reach the target level of 200 counts/100 ml as indicated in Figure 40. The disinfectant demand is apparently higher at the beginning of the overflow as a result of the first-flush phenomenon, thus requiring higher dosages to achieve a given level of reduction. Also, an initial period of up to 15 min was required for the ClO_2 pumping system to stabilize in its delivery of ClO_2 to the disinfection tank. In part, this may have been due to the time necessary for the ClO_2 piping system to fill upon activation of the ClO_2 generator. In addition, variable back pressure created by the rise and fall of the liquid level in the disinfection tanks at variable CSO flowrates may have resulted in somewhat erratic delivery of the ClO_2. ClO_2 delivery rates would be more consistent under constant CSO flow conditions, resulting in more consistent FC kills. This condition is partially indicated by comparing log reductions of Storms 2 and 3 (Figures 38 and 39 where CSO flowrates are variable) to the remainder of the storms where more constant CSO flowrates were

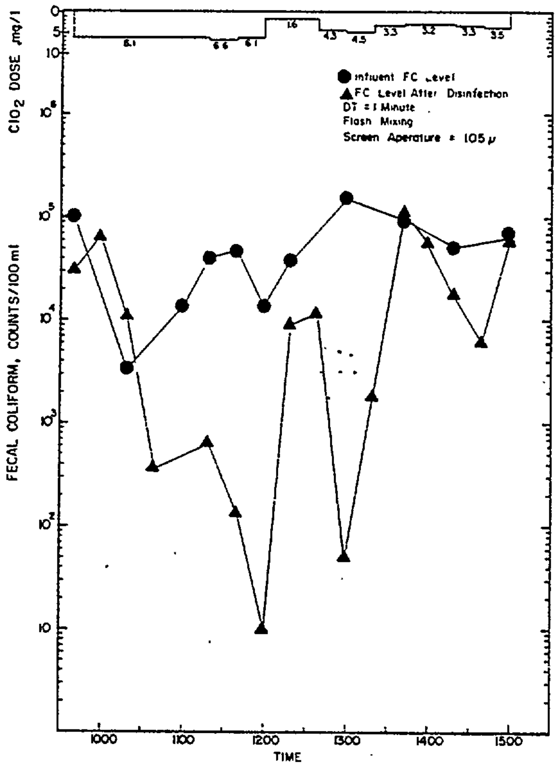


FIGURE 38. ClO2 Disinfection Following Sweco Storm 2, 1975

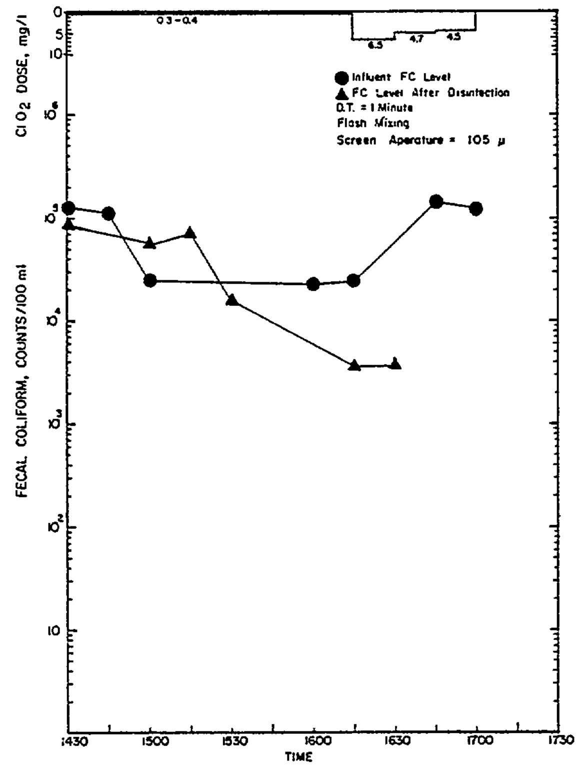


FIGURE 39. ClO2 Disinfection Following Sweco Storm 3, 1975

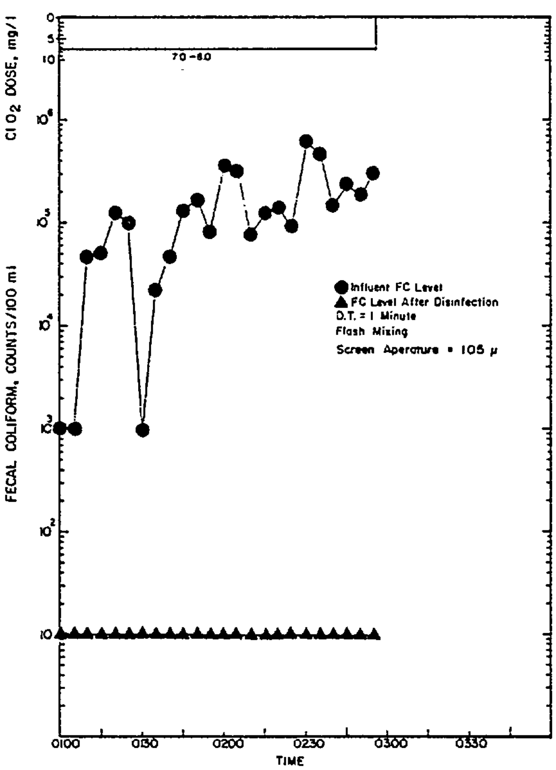


FIGURE 40. ClO2 Disinfection Following Sweco Storm 5, 1975

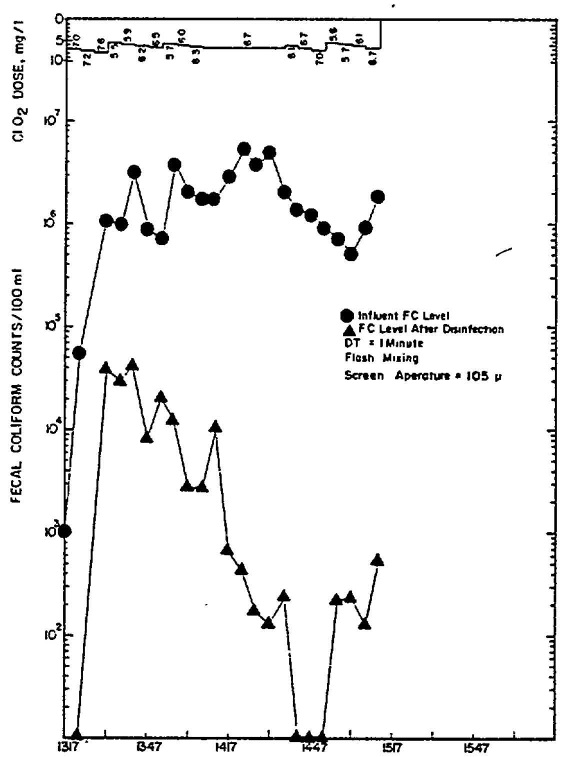


FIGURE 41. ClO2 Disinfection Following Sweco Storm 6, 1975

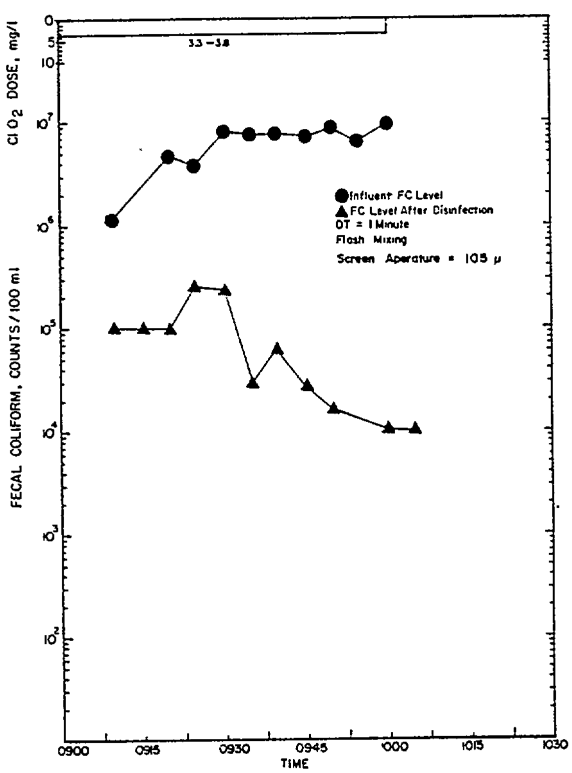


FIGURE 42. ClO_2 Disinfection Following Sweco Storm 8, 1975

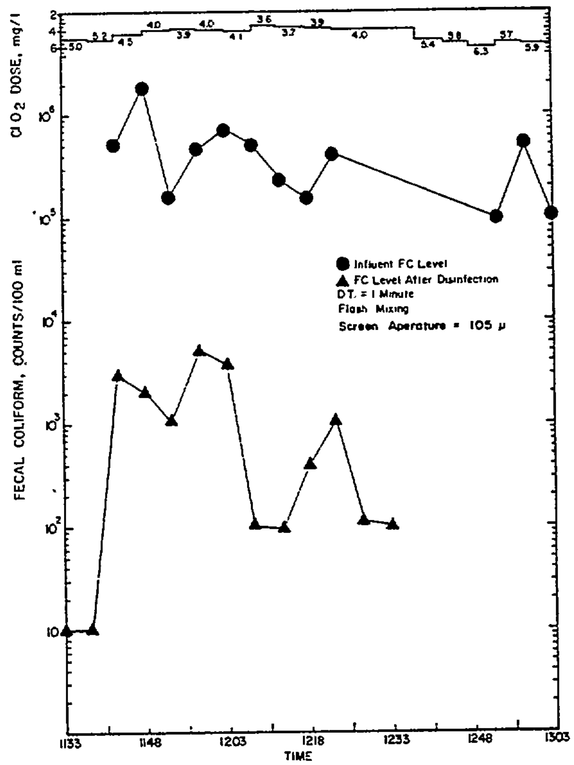


FIGURE 43. ClO_2 Disinfection Following Sweco Storm 9, 1975

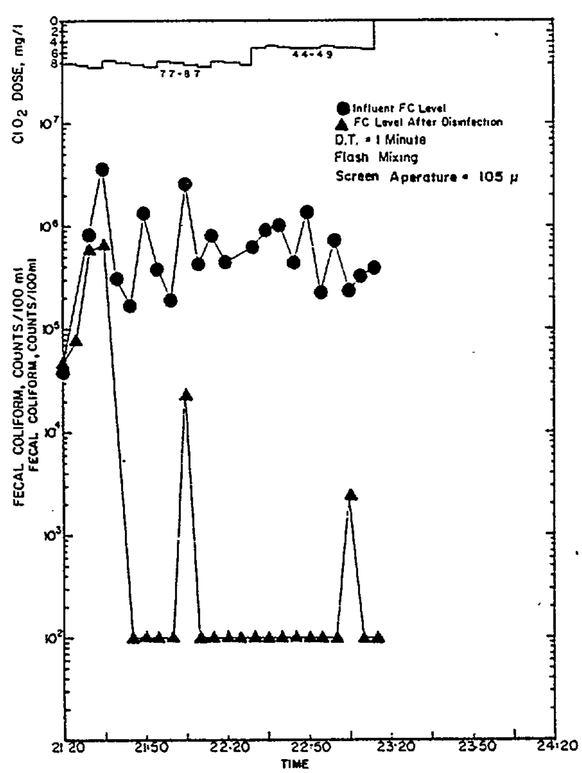


FIGURE 44. ClO_2 Disinfection Following Sweco Storm 10, 1975

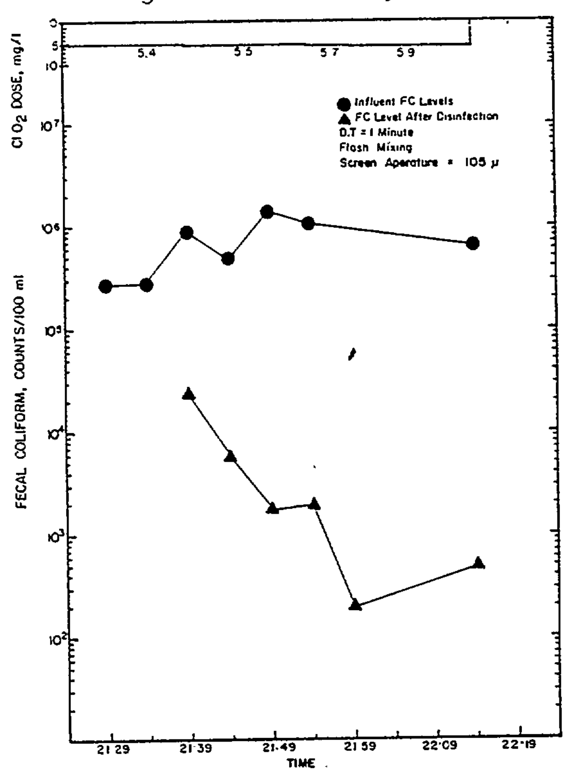


FIGURE 45. ClO_2 Disinfection Following Sweco Storm 11, 1975

applied. FC kills varied throughout Storms 2 and 3 from 0 to 3 logs when flowrates varied from 0.3 to 2.1 MGD (0.8 to 5.5 cu m/min) within individual storms. For the remainder of the storms, flowrates were held relatively constant throughout individual storms. FC kills are seen to be less variable within individual storms as seen in Figures 40 through 45.

The results of the 1975 storms utilizing the disinfection system associated with the Zurn unit are presented in Figures 46 to 49.

Figure 46 indicates no significant reduction of FC populations at ClO_2 dosages of less than 1 mg/l. Reductions of about 1 log were achieved at a dosage of 2.4 mg/l. Figure 47 depicts rather erratic results. During the first hour of the storm flow, over 4 log reductions were achieved at ClO_2 dosages of 3.7 mg/l while the last one and one-half hours shows only 1 to 2 log reductions at a ClO_2 dosage of from 3.7 to 5 mg/l. Since parallel data for the Sweco disinfection system (Figure 39) illustrated lower levels of reduction during initial stages of the storm, the data presented in Figure 47 indicates that contact with the disinfectant may have continued after the samples were drawn.

Figure 48 indicates 4 log reductions of FC achieved at ClO_2 dosages of 6 to 7 mg/l after the first 30 min of the overflow. During the first 30 min dosages near 5.5 mg/l resulted in only 1 to 2 log reductions. This data indicates the time required for ClO_2 delivery rates to stabilize and/or the effects of first-flush effects on disinfection.

Figure 49 illustrates FC kills ranging from 1 to 5 logs at ClO_2 dosages varying from 3 to 10 mg/l; the higher kills being achieved at the higher dosages. Lower ClO_2 dosages of 3 to 6 mg/l applied during the last half of the storm achieved nearly the same log reductions as the higher dosages of 6 to 10 mg/l achieved during the first half of the storm. Again higher pollutant loadings occurring during the initial periods of the storm could account for the higher dosage requirements during initial storm periods.

Overall, data collected during 1975 on the Zurn disinfection system indicate much the same trends as did the Sweco disinfection system. Target levels of 200 counts FC/100 ml were achieved at dosages of 7 to 8 mg/l ClO_2 at the beginning of the overflow period, while lower dosages at later stages of the overflows achieved the same results.

At no time during 1975 was the Crane system investigated with respect to disinfection. This was due to the continuing failure of the Crane Microstrainer to operate properly.

Cl_2 AND ClO_2 DISINFECTION - 1976 OPERATIONS

Description

Six additional overflow events were monitored in 1976 at Maltbie Street to further examine high-rate disinfection of CSO. Since consider-

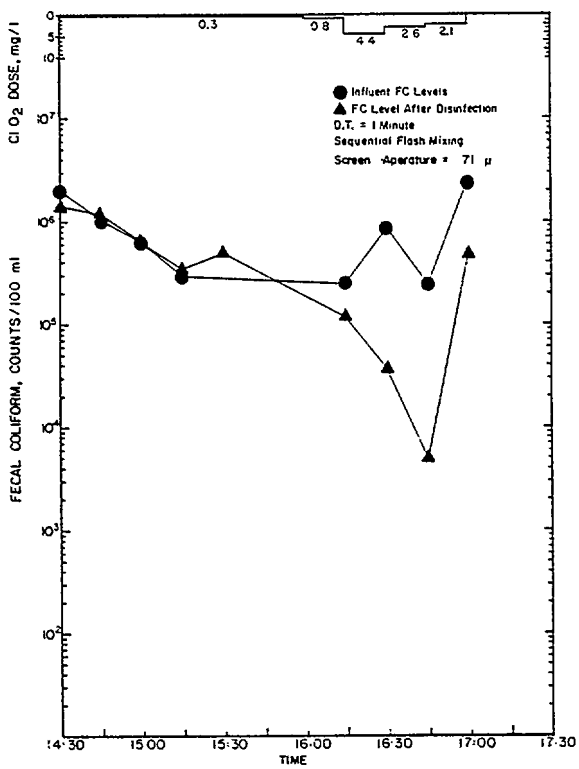


FIGURE 46. ClO_2 Disinfection Following Zurn Storm 3, 1975

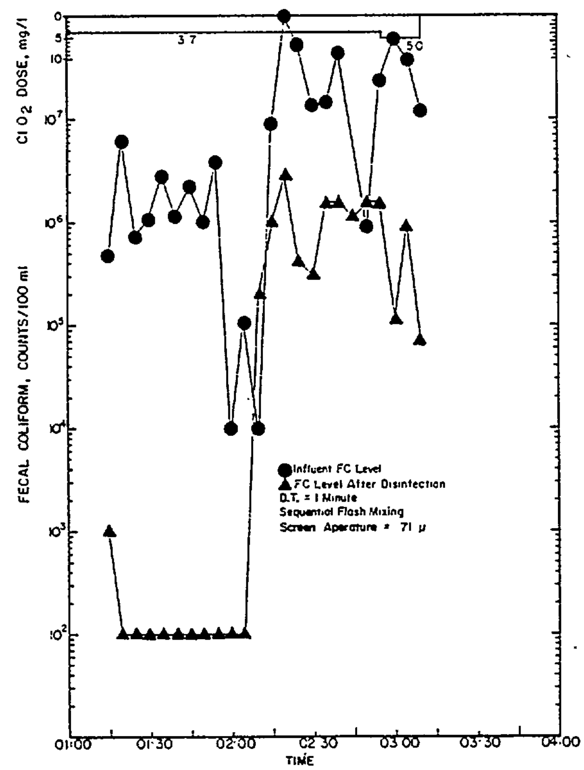


FIGURE 47. ClO_2 Disinfection Following Zurn Storm 5, 1975

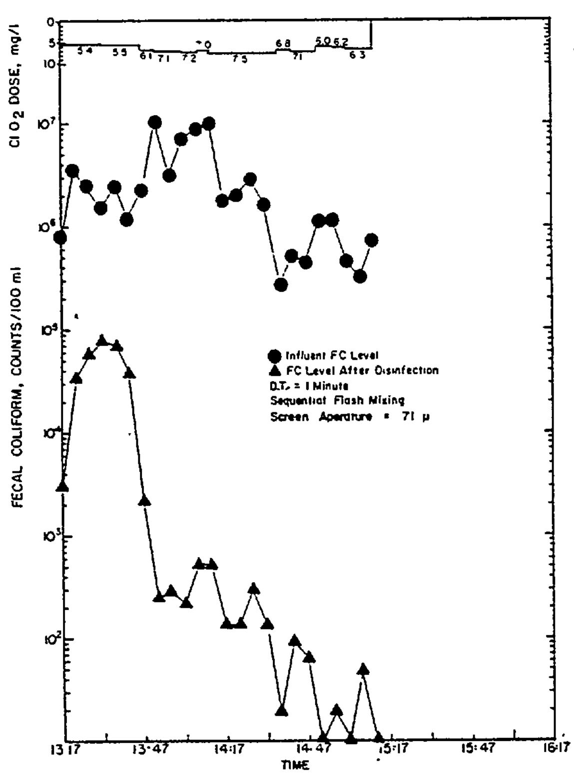


FIGURE 48. ClO_2 Disinfection Following Zurn Storm 6, 1975

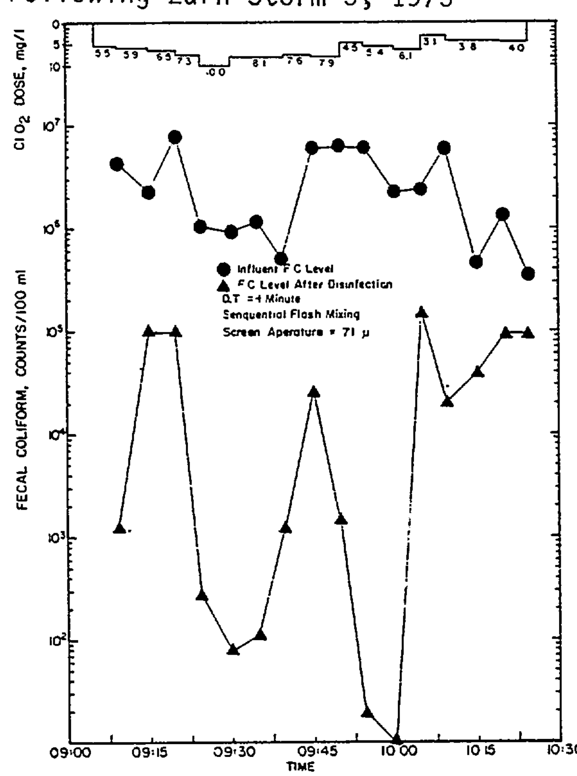


FIGURE 49. ClO_2 Disinfection Following Zurn Storm 8, 1975

able data had been collected relative to the solids removal capabilities of the screening units at this site and since budget constraints limited the number of laboratory analyses to be performed, no effort was directed to assessing the Sweco (105μ) and Zurn (71μ) microscreens during these six events, although solids removals by the Crane (23μ) unit were evaluated for two storms in 1976, to supplement data gathered for one storm in 1975.

Since the main effort of the 1976 program was to further evaluate the feasibility of high-rate disinfection, it was desired to remove as many operational variables as possible to produce a more controlled process train. The most important of these variables was that of flowrate through the treatment processes. Therefore, in all six events monitored in 1976, the flowrates were fixed. Excess CSO above the fixed rate was allowed to bypass directly to Onondaga Creek.

Processes involving unscreened CSO were performed by pumping CSO from the wet well through the Zurn Micromatic, from which all screen panels had been removed. The screened processes were accomplished by passing CSO through the Crane Microstrainer.

During Storms 1,2,3,4 and 6 the disinfectant was injected at the upstream end of each disinfection tank, thus providing a one minute detention time through the tank. During Storm 5, ClO_2 was injected at the upstream end of the disinfection tank and Cl_2 was subsequently injected at the end of the first baffle in the tank. This sequence of injection thus provided a theoretical one minute detention time for ClO_2 and a 40-45 sec detention time for Cl_2. Immediately after injection of ClO_2, the ClO_2 begins to deteriorate into the ClO_2^- ion which is far less potent as a disinfectant than is the ClO_2 molecule. It has been theorized that addition of Cl_2 at some point subsequent to the addition of ClO_2 converts ClO_2^- back to the more potent ClO_2 molecule. By sequential addition of ClO_2 and Cl_2 it was anticipated that the disinfection capability would be enhanced. (7).

Flash mixing was provided at the point of injection of each disinfectant on the unscreened CSO, and flash mixing of only ClO_2 at the point of injection was provided on the screened CSO. Analyses for the reduction of bacterial populations were limited to analysis for FC for the entire duration of each overflow event. However, viral studies were conducted during which grab samples were collected at 2 min time intervals. These grab samples were analyzed for TC, FC and FS. Other parameters analyzed included TOC, SS, TKN, NH_3N and pH. Chlorine demand was determined from laboratory analyses of composite samples.

Results of 1976 Disinfection Tests

Figures 50 through 58 are plots of the results of disinfection for the six storms monitored in 1976.

During Storm 1, as shown in Figure 50, Cl_2 dosages ranged from 0 to 24 mg/l. Sequential flash mixing of the unscreened CSO was provided at the upstream end of the tank and at the end of the first longitudinal baffle. Reductions of FC ranged from 1 to 6 logs.

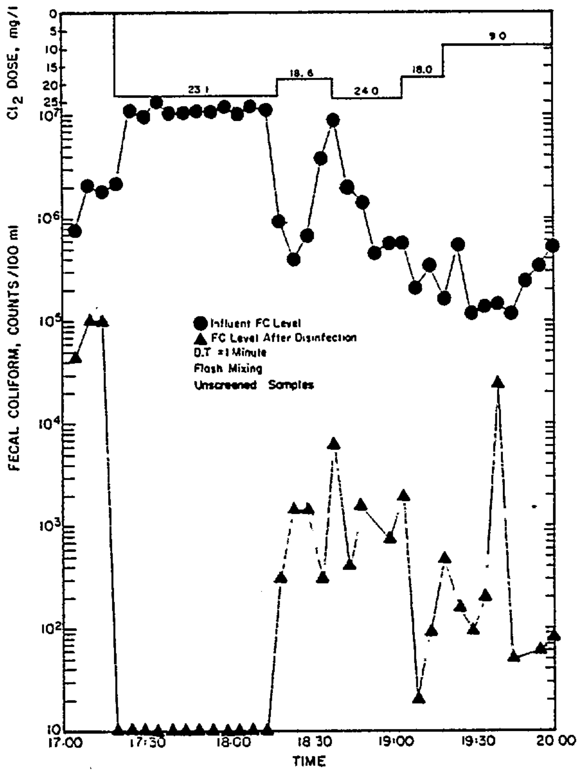


FIGURE 50. Cl_2 Disinfection
Unscreened CSO - Storm 1, 1976

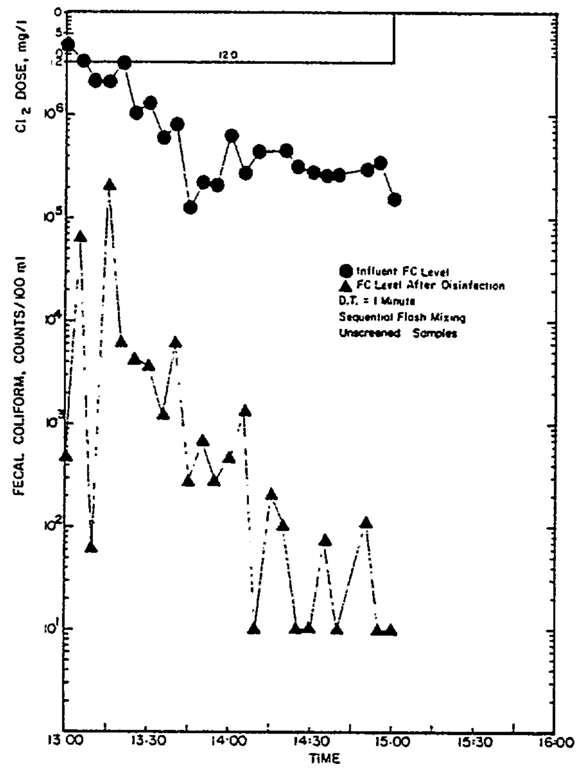


FIGURE 51. Cl_2 Disinfection
Unscreened CSO - Storm 2, 1976

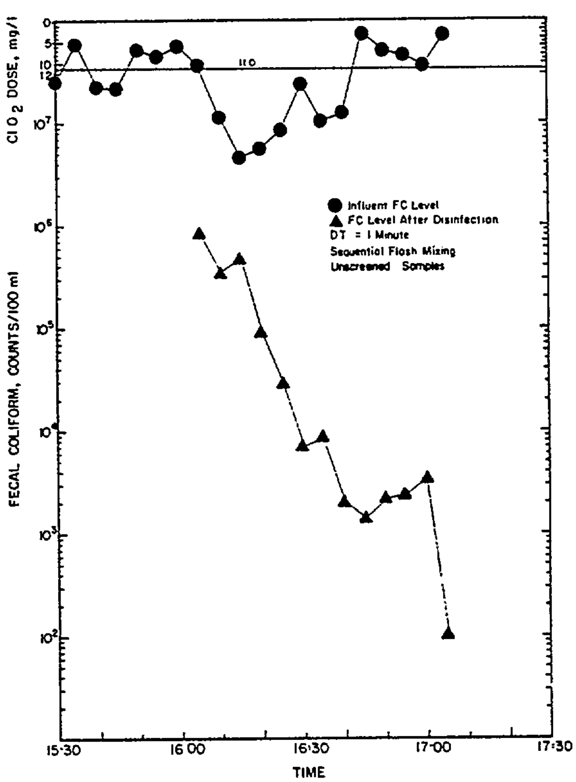


FIGURE 52. ClO_2 Disinfection
Unscreened CSO - Storm 3, 1975

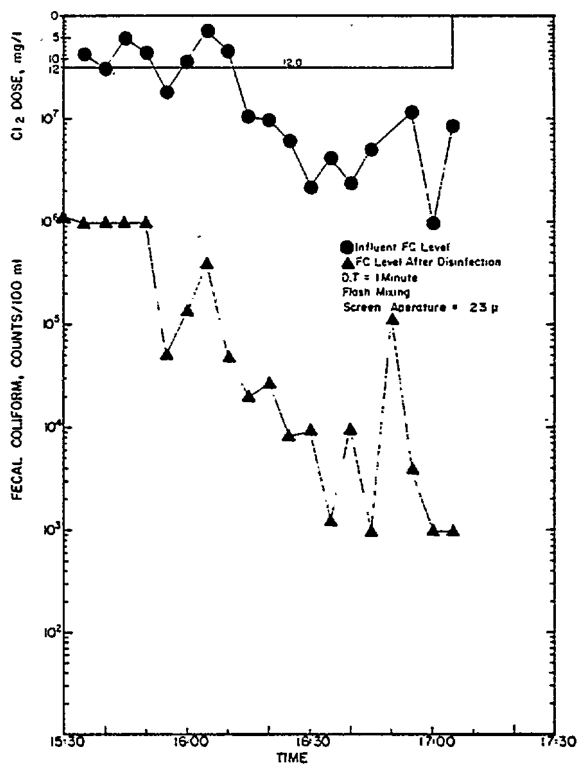


FIGURE 53. Cl_2 Disinfection
Screened CSO (23μ) - Storm 3, 1976

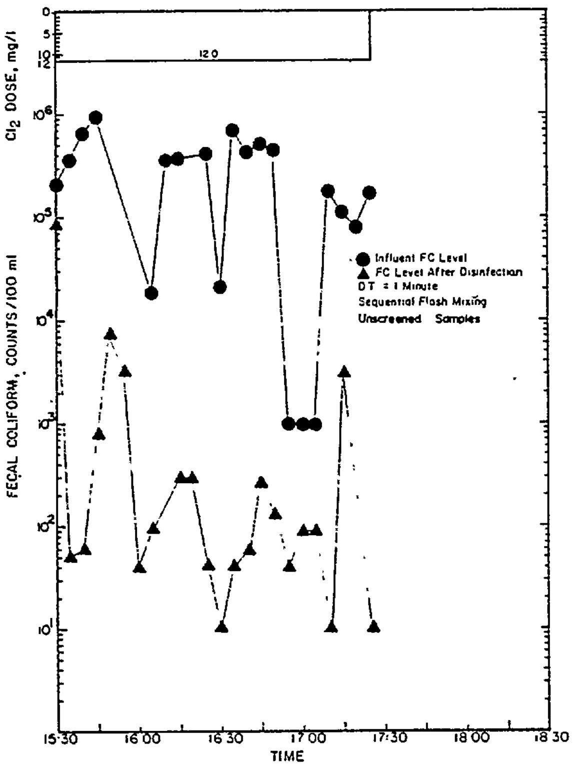


FIGURE 54. Cl_2 Disinfection Unscreened CSO - Storm 4, 1976

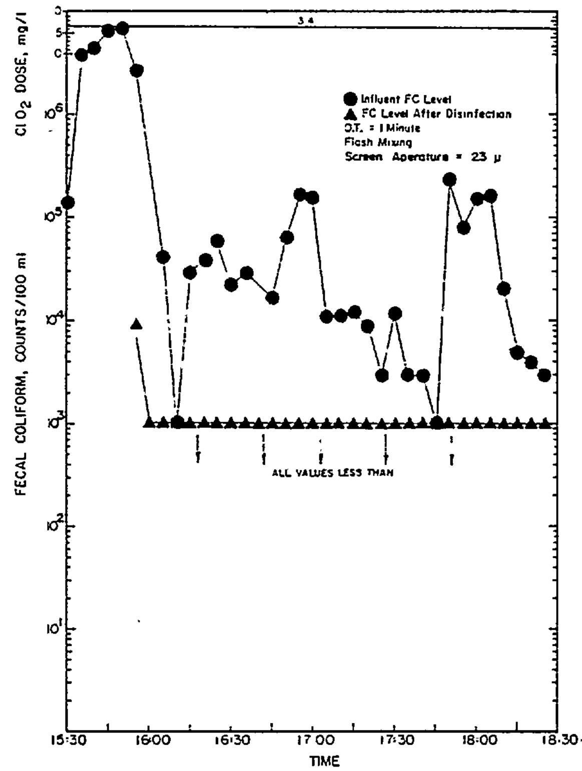


FIGURE 55. ClO_2 Disinfection Screened CSO (23μ) - Storm 4, 1976

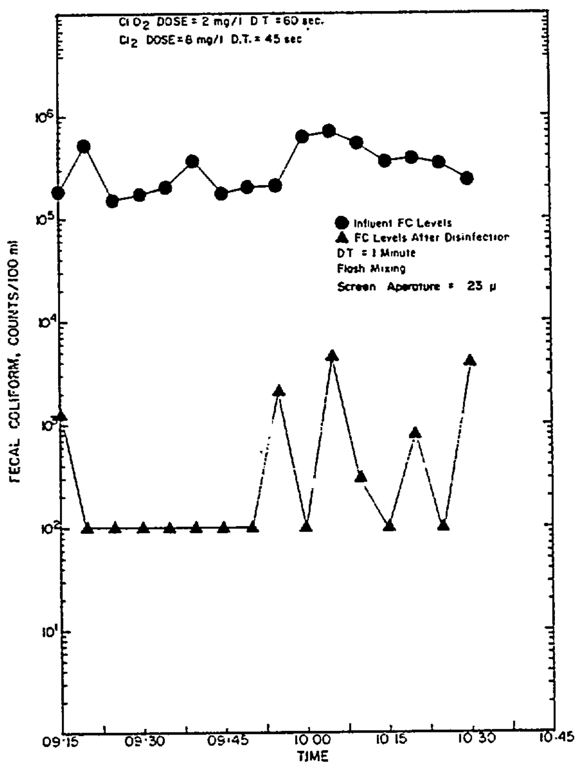


FIGURE 56. ClO_2 & Cl_2 Sequential Disinfection Screened CSO (23μ) - Storm 5, 1976

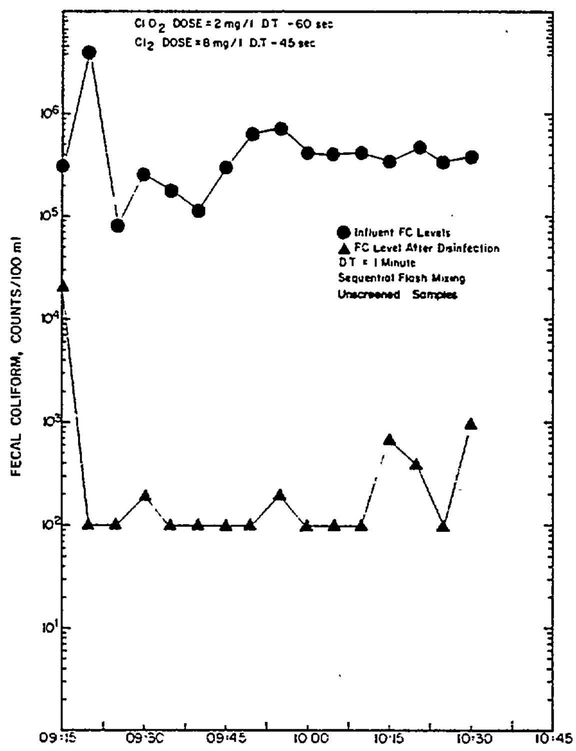


FIGURE 57. ClO_2 & Cl_2 Sequential Disinfection Unscreened CSO - Storm 5, 1976

Figure 51 illustrates the log reductions of FC during Storm 2. The Cl_2 dose during Storm 2 was set at 12 mg/l with resulting Cl_2 residuals ranging from 5.0 to 9.8 mg/l for the one minute of contact time available. Figure 51 indicates a general tendency of increased log reduction of FC with time. A 3 to 4 log reduction of FC for Storm 2 was observed, and the target level of 200 counts/100 ml was attained only during the latter stages of the storm.

Figures 52 and 53 illustrate the results of disinfection tests by ClO_2 on unscreened CSO and Cl_2 on screened (23μ) CSO, respectively, during Storm 3. The ClO_2 test shown in Figure 52 shows an increase in the log reduction of FC with time ranging from 1 log reduction near the beginning of the storm to 6 log reduction at the end of the storm.

Figure 53 illustrates the reduction of FC by application of a 12 mg/l dose of Cl_2 to the screened (23μ) CSO. The log reduction tended to increase slightly with time from 1 to 2 logs at the beginning of the storm to 2 to 3 logs near the end of the storm.

The results of Storm 4 are depicted in Figures 54 and 55. A 12 mg/l dose of Cl_2 to unscreened CSO produced slightly erratic reductions ranging from 2 to 4 logs as shown in Figure 54. The majority of points indicating numbers of FC in the effluent fall below the target level of 200 counts/100 ml. Cl_2 residuals during this storm were measured at 5 to 8 mg/l.

ClO_2 applied at a dose of 3.4 mg/l to the screened (23μ) CSO in Storm 4 (Figure 55) resulted in significant log reductions of FC. All values of FC in the disinfected CSO were measured to be less than 1000 counts/100 ml. The Cl_2 demand for the CSO was determined to be approximately 13 mg/l.

For Storm 5, raw CSO FC populations were determined to be 380,000 counts/100 ml while populations after screening (23μ) were 312,000 counts/ 100 ml. Since this reduction is within the variability of the analysis procedure (estimated to be 20 percent), no conclusion can be reached regarding bacterial reduction as the result of microscreening.

Storm 5 was conducted employing sequential addition of ClO_2 and Cl_2. Two mg/l of ClO_2 was applied at the head of the disinfection tank, and 8 mg/l of Cl_2 applied at the end of the first baffle in the tank. In the case of both screened and unscreened CSO, the level of reduction of FC was in the order of 3 logs as indicated in Figures 56 and 57. The magnitude of the Cl_2 residual measured in each test was 10 to 13 mg/l and 5 to 9 mg/l, respectively.

Figure 58 illustrates the results of Storm 6 where a ClO_2 dose of 7.3 mg/l was injected into unscreened CSO. During the middle portion of the storm, the FC populations were reduced to less than 100 counts/100 ml with a Cl_2 residual of 4.2 to 4.5 mg/l. Reductions of FC were in the order of 3 logs.

MULTIPLE REGRESSION ANALYSIS

To evaluate the effects of variable wastewater quality on the disinfection processes investigated during the demonstration study, mathematical models were developed relating specific water quality parameters to the reduction of FC levels achieved under various disinfectant dosage applications. Multiple regression analysis of the treatment data was performed to statistically fit equations to the experimental data. The final equations fitted to the results take into account the varying levels of FC, SS, pH, etc. that resulted from the microscreening processes.

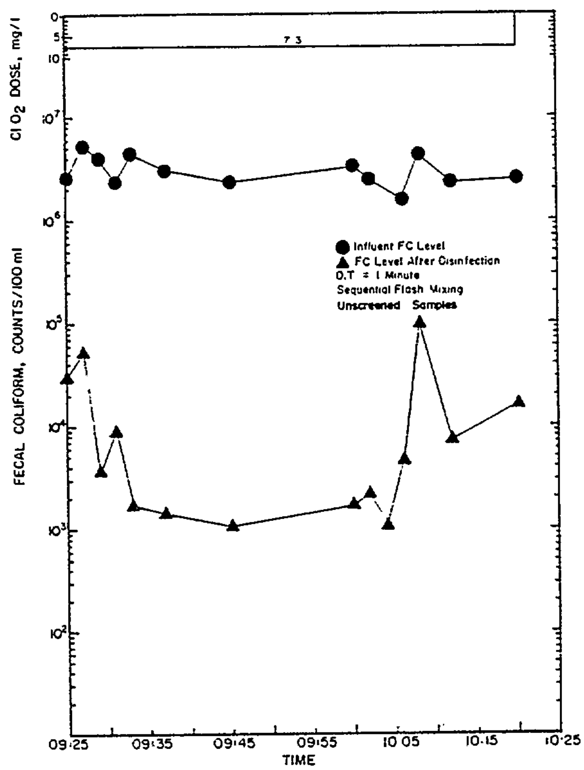


FIGURE 58. ClO_2 Unscreened CSO
Storm 6, 1976

The final form of the regression equation determined to be most representative of the observed data for the disinfection investigations was:

$$\text{logkill} = K_1 C^{K_2} GT^{K_3} SS^{K_4} FC^{K_5} 10^{K_6\,pH}$$

where C = concentration of disinfectant, mg/l

SS = concentration of SS, mg/l

FC = influent level of fecal coliform, counts/100 ml

pH = pH

GT = mixing intensity x detention time in zone of influence

$K_1, K_2, K_3, K_4, K_5, K_6$ = constants from the regression analysis

The velocity gradient G for the flash mixers was calculated from the following equation:

$$G = (550P/V\mu)^{\frac{1}{2}}$$

where P= water horsepower of the mixers, HP
V= volume of the zone of influence of the mixer, ft^3
μ= kinematic viscosity of water at 50°F, 2.73×10^5 lb-sec/ft^2

For the disinfection tank mixing influence, the G value was calculated from the following equation:

$$G = 1730(\mu)^{-\frac{1}{2}} (VS)^{\frac{1}{2}}$$

where μ= viscosity of water at 50°F, = 1.3097cp
V= velocity of flow, fps
S= tank slope, ft/ft

Previous research (53) has indicated that disinfectant concentration is exponentially related to reduction of bacteria. This relationship resulted in the inclusion of the factor C^{K2} in the regression model.

To develop the mathematical relationship between kill and dosage, the SS, FC and pH parameters were included since they indicated statistically significant effects with the disinfection system performance data. Disinfection contact time was held constant at one minute throughout the demonstration study. Therefore, although it is one of the major factors affecting disinfection processes, contact time was not included in the development of the regression equation.

Tables 36 and 37 present the results obtained from the regression analyses for the ClO_2 and Cl_2 disinfection systems, respectively. The regression coefficient values correspond to the exponential K values in the regression equation. The value of K_1 is equal to 10^i where i is the regression intercept value.

The magnitude of the regression coefficient gives some indication of the relative importance of the term in the regression expression; for example, positive coefficients associated with ClO_2 dosage application, mixing intensity (GT), pH and influent FC levels signify that as these values increase, the log kill of FC also increases. The negative coefficient associated with SS indicates that as this value increases, the log kill of FC decreases.

The statistically derived 't' test of significance designates the degree of confidence with which the corresponding regression coefficients can be assumed to be correct. In the ClO_2 regression results, the 't' value for the ClO_2 dose coefficient corresponds to a degree of confidence

TABLE 36. MULTIPLE REGRESSION ANALYSIS RESULTS FOR ClO_2

Independent Variable	Mean	Standard Deviation	Correlation X vs Y	Regression Coefficient	Std. Error of Regr. Coef.	Computed t-Value
Log C	0.72	0.23	0.65	0.68	0.08	8.99
Log FC	5.61	0.89	0.16	0.06	0.03	2.02
Log Gt	3.61	0.25	0.38	0.09	0.08	1.18
Log SS	2.26	0.41	-0.05	-0.07	0.05	- 1.36
pH	6.63	0.50	-0.01	0.02	0.04	0.57
Dependent Variable						
Log (Log kill)	0.39	0.27				

Intercept	-0.71
Multiple Correlation	0.67
Std. Error of Estimate	0.20

Analysis of Variance for the Regression

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Squares	F Value
Attributable to Regression	5	5.21	1.04	25.98
Deviation from Regression	157	6.30	0.04	
Total	162	11.52		

TABLE 37. MULTIPLE REGRESSION ANALYSIS RESULTS FOR Cl_2

Independent Variable	Mean	Standard Deviation	Correlation X vs Y	Regression Coefficient	Std. Error of Regr. Coef.	Computed t-Value
Log C	1.03	0.61	0.86	0.36	0.02	17.29
Log FC	5.88	0.66	0.24	0.02	0.02	0.99
Log Gt	3.53	0.12	0.05	0.42	0.12	3.45
Log SS	2.26	0.39	0.06	-0.07	0.03	-2.05
pH	7.20	0.73	-0.25	-0.03	0.02	-1.47
Dependent Variable						
Log (Log Kill)	0.51	0.25				

Intercept	-1.09
Multiple Correlation	0.90
Std. Error of Estimate	0.11

Analysis of Variance for the Regression

Source of Variation	Degrees of Freedom	Sum of Squares	Mean Squares	F Value
Attributable to Regression	5	3.99	0.80	66.43
Deviation from Regression	75	0.90	0.01	
Total	80	4.89		

greater than 99 percent, while the 't' value for the FC coefficient was greater than 95 percent, GT greater than 75 percent, SS coefficient greater than 80 percent and that for pH greater than 40 percent. The magnitude of the standard error of the regression coefficient and the 't' value associated with the pH indicate that the effect of pH was fairly insignificant.

In the Cl_2 regression results, 't' values for the various parameters indicated a degree of confidence greater than 99 percent for Cl_2 dose and GT coefficients, greater than 95 percent for SS, greater than 85 percent for pH, and greater than 70 percent for FC.

The 'F' value in the multiple regression analysis gives an indication of the validity of the entire regression equation. The statistically derived 'F' test for equality of data variances conducted for each of the ClO_2 and Cl_2 equations represented a degree of confidence greater than 99 percent. The final regression equations obtained were as follows:

$$ClO_2: \text{Logkill} = 0.19\ C^{0.68}\ GT^{0.90}\ SS^{-0.07}\ FC^{0.06}\ 10^{-0.02pH}$$

$$Cl_2: \text{Logkill} = 0.08\ C^{0.36}\ GT^{0.42}\ SS^{-0.07}\ FC^{0.02}\ 10^{-0.03pH}$$

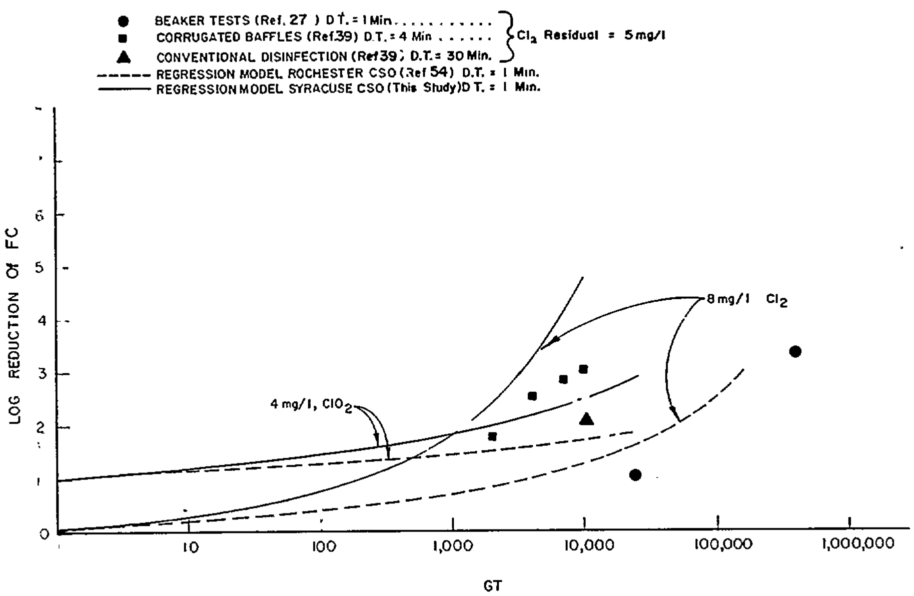


FIGURE 59. Regression Model Results-GT vs FC Reduction

Illustrative Trends of the Regression Models

The separate effects of the independent variables on the disinfection unit performance were examined by use of the regression models. Variations in performance with respect to mixing intensity as supplied by the flash mixers and detention time were plotted for both Cl_2 and ClO_2. An effort to compare the Syracuse model results with the results Glover obtained (39) using a Cl_2 residual of 5 mg/l was attempted by applying a Cl_2 dosage of 8 mg/l and a ClO_2 dosage of 4 mg/l in the regression models. Results were also compared to the results obtained in a similar study in Rochester, (54) using equivalent dosage applications. Values for the parameters contained in the regression models were the averages experienced in the Syracuse study. The effects of flash mixing of disinfectants with the wastewater is reflected in the magnitude of the mixing intensity, GT.

Plots of Glover's results and the regression model results are presented in Figure 59. A comparison of the curves shows similar trends. The slope of the curves indicates that disinfection with Cl_2 is greatly enhanced with an increase in mixing intensity, while the curves for ClO_2 indicate that increased mixing intensities do not result in as pronounced an increase in bacterial reductions as observed for Cl_2. Figure 59 also suggests that at low mixing intensitites and short contact times, ClO_2 is more effective than Cl_2 in reducing bacterial populations.

Figure 60 is a plot of performance versus GT for different dosages of Cl_2 using the average parameter values from the Syracuse data. The effects of mixing intensity on Cl_2 disinfection effectiveness are apparent. The slope of the curves indicate that the effect on performance is more pronounced when dosages are varied at higher mixing intensities.

A plot of performance versus GT for various ClO_2 dosages is presented in Figure 61. The slopes of the curves indicate that mixing intensity does not affect bacterial reduction when ClO_2 is used as significantly as mixing affects reduction of bacteria when using Cl_2. However, as in the case of Cl_2, the effects of mixing on performance are more pronounced when the ClO_2 dosage is varied at the higher mixing intensities. Comparison of Figures 60 and 61 shows ClO_2 to be a better disinfectant than Cl_2 at the lower mixing intensitites.

The effect of varying SS levels was evaluated using the regression models. Figure 62 presents plots of performance versus disinfectant dosage for both Cl_2 and ClO_2. Comparison of the plots show that variations in the SS levels of the applied wastewater produce relatively minor improvement in the disinfection effectiveness of both Cl_2 and ClO_2.

A similar sensitivity analysis was conducted for the FC levels with results presented in Figure 63. The set of curves indicate that the effect of FC levels in the applied wastewater on the disinfection effectiveness of Cl_2 is relatively insignificant, while the effect is slightly more pronounced for ClO_2. The effect on ClO_2 is probably insignificant from a practical standpoint since FC levels in CSO are normally in the range of 10^5 to 10^6 counts/100 ml with only occasional FC levels in the order of 10^7 counts/100 ml.

Sensitivity analysis of the effects of pH as performed with the regression models and illustrated in Figure 64, indicate that pH variation is insignificant when ClO_2 is used, while use of Cl_2 results in a

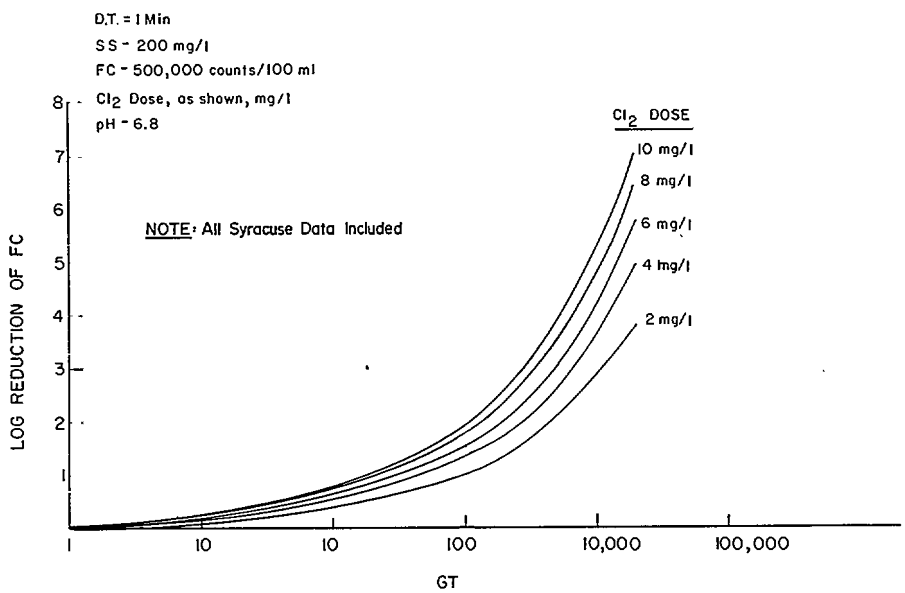


FIGURE 60. Effect of Cl_2 Dosage - GT vs FC Reduction

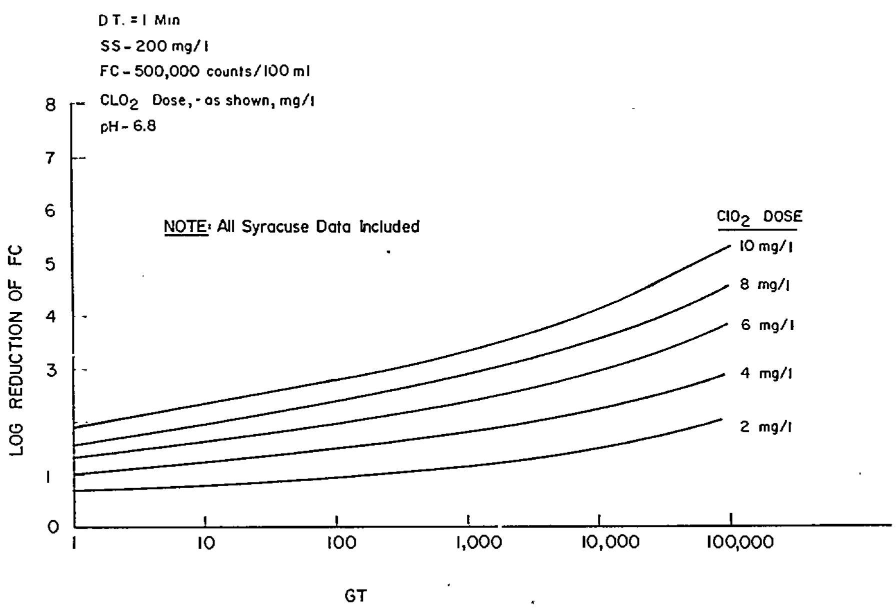


FIGURE 61. Effect of ClO_2 Dosage - GT vs FC Reduction

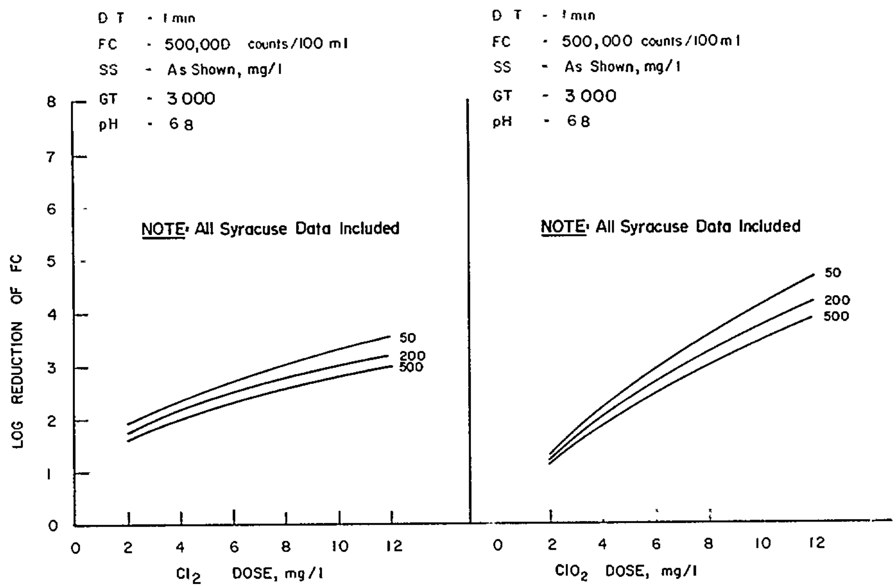


FIGURE 62. Effect of SS on Cl_2 & ClO_2 -Dosage vs FC Reduction

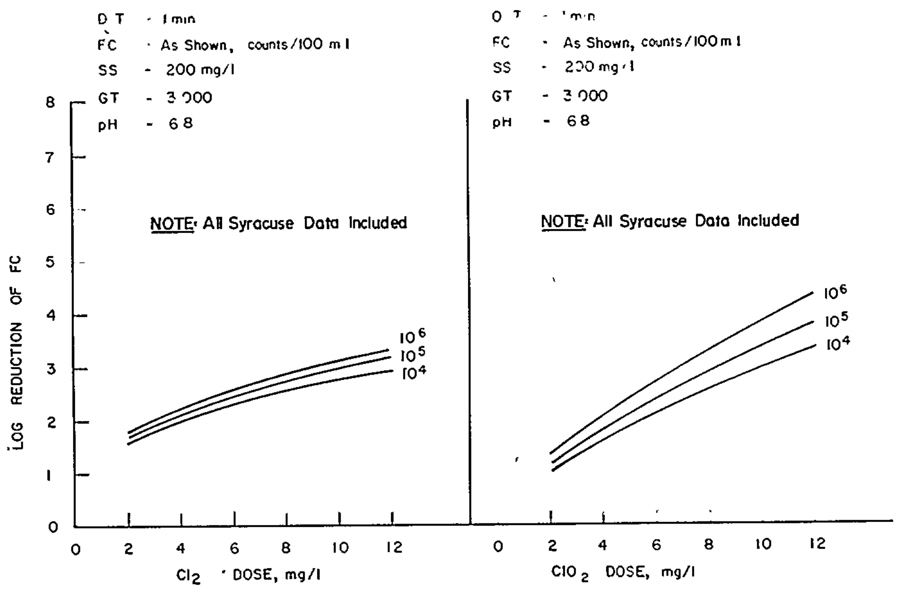


FIGURE 63. Effect of FC on Cl_2 & ClO_2 - Dosage vs FC Reduction

slightly more pronounced effect on log kill with varying pH. This observation is partially supported by the fact that a greater proportion of the Cl_2 disinfectant exists as the more potent HOCl at lower values of pH (55). At higher pH, more of the Cl_2 exists as the less potent OCl^-. With ClO_2, the most potent form of the disinfectant is ClO_2 with the less potent ClO_2^- formed as the result of reactions with reducing agents, which are much less dependent on pH.

Figures 65 and 66 indicate the correlation between observed data and performance predicted by the regression models.

Sequential Addition of ClO_2 and Cl_2

During Storm 5 in 1976, investigations were conducted into the feasibility of disinfecting CSO by a process of sequential addition of ClO_2 and Cl_2. For these tests, ClO_2 was injected into the CSO at the upstream end of each of two disinfection tanks at a dosage of 2 mg/l. Cl_2 was injected at a dosage of 8 mg/l at the end of the first baffle in each tank. The wastewater was thus subjected to a ClO_2 dosage alone for a detention time of 15 seconds and to a combination of ClO_2 and Cl_2 for a detention time of 45 seconds. It had been suggested that Cl_2 added 15 to 30 seconds after injection of ClO_2 would enhance disinfection. ClO_2 is oxidized by various reducing agents to ClO_2^- during the disinfection process. It was suggested that addition of Cl_2 would, to some degree, reduce ClO_2^- back to ClO_2 to prolong the existence of the more potent disinfectant ClO_2, and thus enhance disinfection beyond that expected by the sum of the respective concentrations of Cl_2 and ClO_2.

Effluent from the Crane Microstrainer was discharged to one of the disinfection tanks, while unscreened CSO was discharged to the second tank. Mixing of the screened CSO was accomplished by flash mixing at the ClO_2 injection point, and mixing of the unscreened CSO was accomplished by flash mixing at both disinfectant injection points.

From multiple regression analysis, a mathematical expression was developed to evaluate the effects of sequential addition of disinfectants for the specific combination of Cl_2 and ClO_2 that was applied. The final form of the regression model was

$$\log kill = 0.84 FC^{0.10} GT^{0.008}$$

The 't' values associated with the regression coefficients corresponded to a degree of confidence for the FC coefficient of greater than 99 percent, and for the GT coefficient slightly over 10 percent. The 'F' value indicating the overall statistical significance of the regression equation resulted in a degree of confidence greater than 99 percent. It would be expected that with additional testing over a range of dosage applications, GT values and ClO_2: Cl_2 ratios, a more precise regression equation would result.

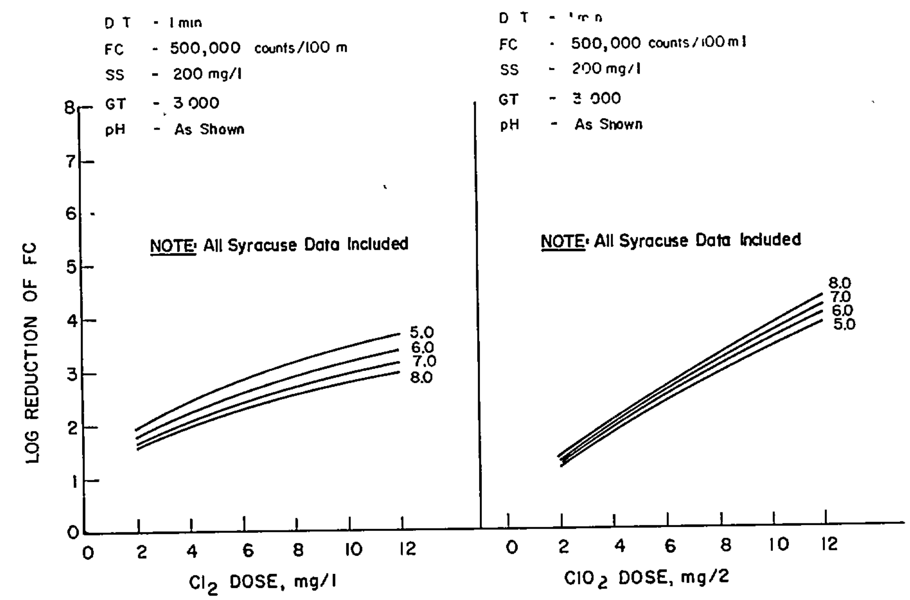


FIGURE 64. Effect of pH on Cl_2 & ClO_2 - Dosage vs FC Reduction

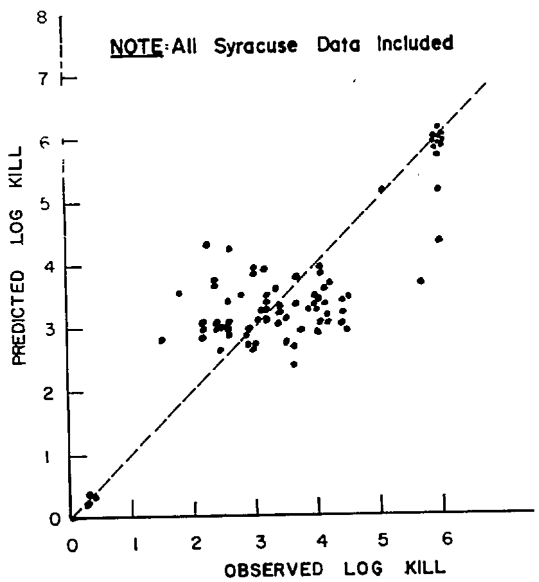


FIGURE 65. Cl_2 Disinfection - Observed vs Predicted FC Kill

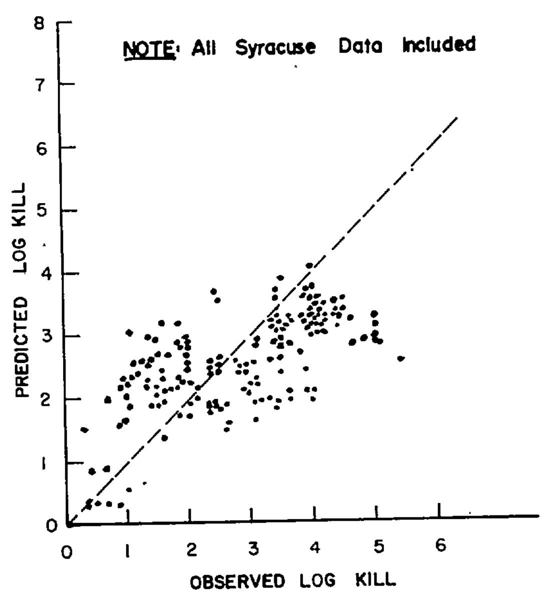


FIGURE 66. ClO_2 Disinfection - Observed vs Predicted FC Kill

A range of FC levels from 10^4 to 10^7 counts/100 ml could affect the reduction of FC by as much as 2 logs, as indicated in Figure 67. Figure 68 indicates that the mixing intensity had an insignificant effect on the reduction of FC. Analysis of SS and pH data indicated these two parameters to be statistically insignificant to the regression model and therefore were not included in the final equation.

The log reduction of FC attained during the sequential addition tests in the Syracuse demonstration study was compared to the results obtained during similar studies in Rochester (54). Figure 69 is a reproduction of a plot of iso-kill curves obtained in Rochester for sequential addition of Cl_2 and ClO_2, where Cl_2 was added first. Superimposed on that plot is the overall average log reduction for the Syracuse studies of sequential addition. Note that in Syracuse the order of addition was ClO_2 first. However, an approximate comparison of the results from the two studies is possible since the Rochester studies indicated only slightly higher bacterial kills were obtained when ClO_2 was introduced prior to the addition of Cl2. The Syracuse results tend to support the Rochester findings as indicated in Figure 69.

DISCUSSION OF DISINFECTION RESULTS

The results of the study at Maltbie Street indicate that application of high-rate disinfection processes can result in significant reduction of bacterial populations in CSO. ClO_2 dosages in the order of 6 to 12 mg/l applied in the initial stages of overflows reduced FC levels to 200 counts/100 ml. Applied dosages of 4 mg/l after first-flush loadings had passed through the treatment system, maintained the 200 counts FC/100 ml level in the majority of the samples collected. Application of Cl_2 at dosages of from 12 to 24 mg/l during the initial stages of overflow also were able to achieve 3 to 4 log reductions of FC, while lower dosages (12 mg/l) produced similar reductions after the first 30 to 45 min. Sequential addition of disinfectants (2 mg/l ClO_2 followed by 8 mg/l Cl_2 after 15 sec) at a total contact time of 1 min produced 3 to 4 log reductions of FC. The limited data obtained in the sequential addition tests precludes a comparison of this method of disinfection with the application of Cl_2 or ClO_2 separately.

Regression analyses of the disinfection data collected indicated that removal of SS would improve the reduction of bacterial populations by the disinfection processes. The effects of solids removal are slightly more pronounced for ClO_2 than for Cl_2 disinfection with both exhibiting improved FC kills of about one-half to one order of magnitude.

ClO_2 residuals in the treated effluent were not measured during the 1975 testing period. However, ClO_2 and Cl_2 residuals (determined as Cl_2) measured in the 1976 tests indicated that high disinfectant residuals could be expected in the effluent after a contact time of only one minute. No attempts to determine the ClO_2 residual in the form of ClO_2 in the effluent were made during the demonstration study. Residuals of disinfectants are important due to their potential impact on receiving

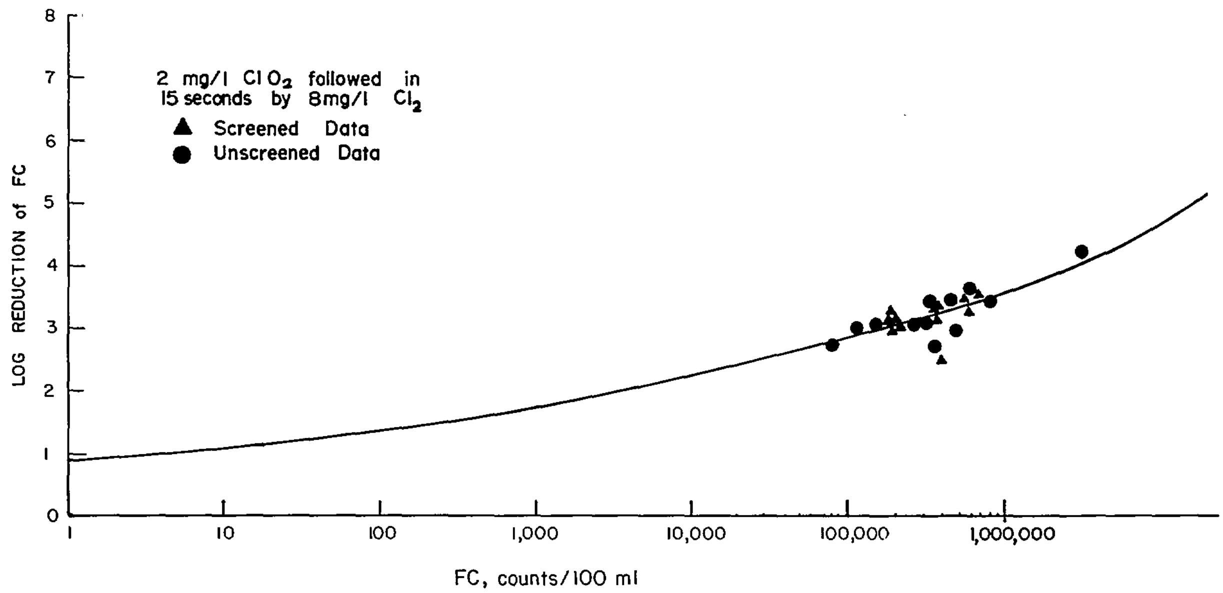


FIGURE 67. Sequential Addition of ClO_2 & Cl_2 - FC Count vs FC Reduction

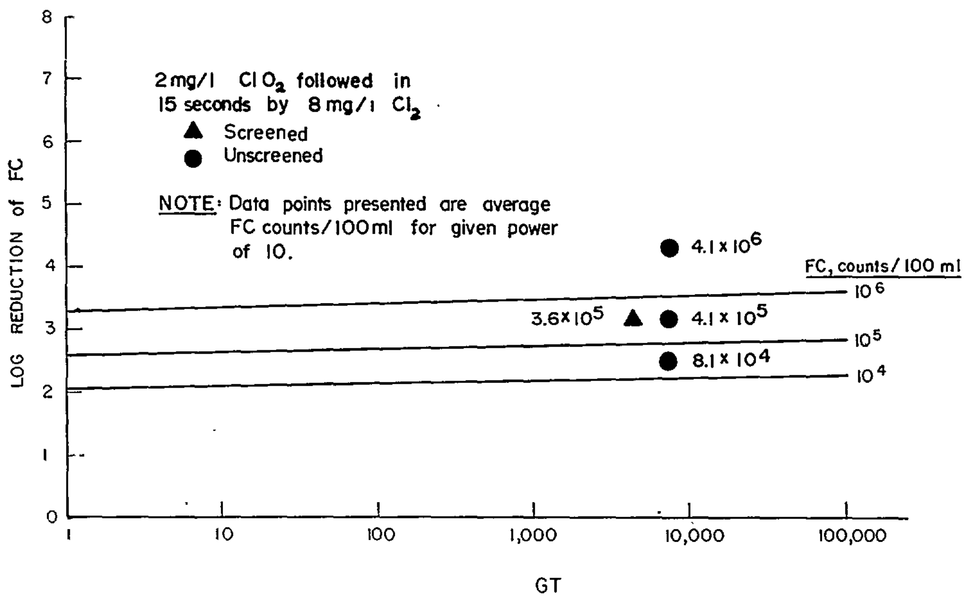


FIGURE 68. Sequential Addition of ClO_2 & Cl_2 - GT vs FC Reduction

water. This aspect of disinfection should be further addressed in subsequent studies and facilities planning.

The demonstration study also indicated that further research of on-site ClO_2 generating equipment is necessary. The generator used in this study could not be operated unattended because of continuous mechanical malfunctions. It is believed, however, that the potential exists for development of a reliable ClO_2 generator. In this study, the chlorination equipment functioned safely and reliably even though the facilities were operated intermittantly. For full-scale CSO treatment applications, consideration should be given to use of one ton cylinders rather than 150 lb Cl_2 cylinders to reduce operation and maintenance costs. The installation of a weighing mechanism would also provide a more accurate record of Cl_2 usage.

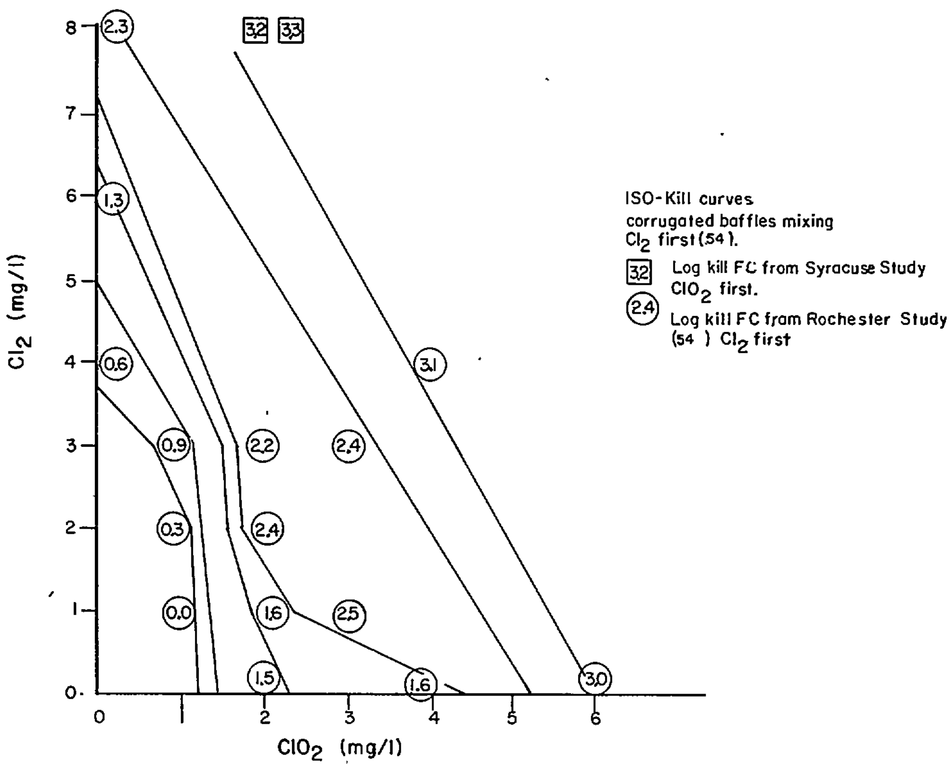


FIGURE 69. Iso-Kill Curves - Syracuse and Rochester Studies

SECTION 12

CAPITAL AND OPERATING COST ESTIMATES

GENERAL

Generalized capital and operating costs have been developed, based on analysis of the actual cost of construction and operation of the CSO demonstration treatment facilities at both the Maltbie and West Newell Street sites. The various cost elements directly attributable to construction and operation of the facilities are tabulated. The costs incurred as a result of additional peripheral equipment required for treatment evaluations and data collection that ordinarily would not be included in full-scale CSO treatment is subtracted from the total project costs and compared to cost estimates presented in a previous report (56). For the latter comparison, costs resulting from site-specific parameters, such as site work, excavation, pumping requirements, etc., are also subtracted from total costs, as these factors will vary within each region and also on a site-by-site basis.

Capital costs include structural, mechanical, piping, housing, labor, contingency, electrical and instrumentation expenses. The capital costs do not include fees associated with land and site work, engineering, legal and administrative services, fiscal concerns, or interest during construction. Operating and maintenance costs include labor, power, chemicals, miscellaneous supplies, repair and replacement parts, administration costs, laboratory and sampling costs, and yard maintenance. Final cost estimates are adjusted to September, 1978 according to the ENR Construction Cost Index of 2861.

ACTUAL CAPITAL COSTS

Presented in Table 38 are the actual capital costs incurred in construction of the Maltbie Street CSO screening facilities. The total cost including construction bids, change orders and contingent work was $509,514. Adjusting the costs of the items presented from June 1974 (ENR=2000) to September 1978 (ENR 2861) results in a total capital cost in 1978 of $728,830, or $6340/acre ($15,670/ha) or $48,590/MGD ($12.84/cu m/day). The third column of Table 38 represents those costs attributable to the screening facilities exclusive of pumping station, site work, disinfection equipment, flow metering equipment, samplers and telemetering facilities. Since much of the equipment installed at the Maltbie Street site was for demonstration purposes and might not be required for a normal facility of this type, it has been estimated that only 50 percent of the total costs of valves and piping, electrical and instrumentation, and 75 percent of the miscellaneous costs are directly attributable to the screening facilities. Summation of the costs of items attributable to screening facilities listed in Table 38 result in a total cost of $366,960 for CSO screening facilities exclusive of disinfection equipment and site-specific factors. The pumping station costs were omitted at this time since site-specific factors will dictate the pumping requirements and size of pumping facilities, as well as the piping requirements.

TABLE 38. ACTUAL COSTS* FOR MALTBIE STREET SCREENING FACILITIES

Cost Component	Actual Cost	Attributable To Screening Facilities
Pumping Station	172,620	
Screen Housing	137,520	137,520
Vertical Shaft Screen	53,640	53,640
Horizontal Shaft Screen (2)	104,430	104,430
Valves and Piping	74,560	37,280
Electrical Instrumentation	44,760	22,380
Miscellaneous	15,610	11,710
Site Work	31,720	
Chlorination Equipment	20,740	
Chlorine Dioxide Generators	26,300	
Flow Measurement	22,890	
Samplers	18,470	
Telemetering	5,570	
Total	728,830	366,960

*Costs adjusted to ENR 2861, September 1978

Comparison of $366,960 for the CSO screening facilities in Syracuse with estimated costs of similar facilities (56) adjusted to 1978 costs ($326,250), illustrates that the capital costs are comparable. The Syracuse project results in a value 12 percent higher than estimated by previous cost curves (56). Therefore, projections of total cost for CSO treatment facilities required for full treatment of Syracuse overflows have been determined from previously published data (56).

Table 39 presents the actual capital costs incurred in construction of the West Newell Street swirl regulator/concentrator. The total cost including construction bids, change orders, and contingent work was $112,516 in 1974, or $160,930 when adjusted to September 1978, or $2980/acre ($7360/ha) or $23,670/MGD ($6.25/cu m/day). Column 3 of Table 39 lists the capital costs attributable to only the swirl unit where 50 percent of the miscellaneous items and 100 percent of the electrical costs are considered directly attributable to the swirl. The resulting total cost of $38,300 for the swirl regulator/concentrator is comparable to the adjusted cost of $35,500 of a similar unit estimated from previous data (56). The cost of the Syracuse swirl prototype is approximately 8 percent more than reference

56 estimates. Therefore, projections of total cost for full CSO treatment by swirl regulation/concentrators from previous data are applicable.

ACTUAL OPERATING COSTS

The actual operating and maintenance costs for the Syracuse demonstration facilities were difficult to establish since such costs included considerable charges for utilities and manpower which ordinarily would not be included in actual CSO treatment facilities. Items such as yard maintenance, cleaning of blocked and/or silted sewer lines and siphons associated with the demonstration treatment facilities were performed by personnel from various city and county departments. Under these circumstances, the costs for this work were not available but recognized as a real expense for the proper operation of the demonstration facilities. Therefore, the operation and maintenance costs shown in this report are based on estimates as previously published (56).

The operating costs include the following items: (1) operating and maintenance labor, (2) power, (3) chemicals, (4) miscellaneous supplies, (5) administrative costs, (6) laboratory and sampling costs, and (7) yard maintenance. The following is a brief discussion of each of the items:

1. Operating and Maintenance Labor - It is assumed that the CSO treatment facility will operate automatically and personnel will not be required during operation unless equipment malfunction occurs. Dewatering and general clean-up of the facility after a storm event is provided for in the cost analysis. Labor for routine visits and maintenance is included regardless of plant operation while a variable amount of labor is provided for the number of times and duration of overflow events. This labor is presented in terms of manhours per year to accommodate varying wage scales.

TABLE 39. ACTUAL COSTS* FOR WEST NEWELL STREET SWIRL REGULATOR/CONCENTRATOR

Cost Component	Actual Cost	Attributable To Swirl
Site Work	44,240	
Piping	28,210	
Swirl Chamber	28,180	28,180
Electrical	6,910	6,930
Miscellaneous	6,390	3,190
Cl02 Generator	8,770	
Samplers	6,150	
Pumping	14,630	
Flow Measurement	14,020	
Telemetering	3,430	
Total	160,930	38,300

*Costs adjusted to ENR 2861, September 1978

(5310 / 2861) 38,300 / ~~6.8~~ 8.9 mgd = $8,000 ~~10,500~~ / MGD

1,856 (38,300) = 71,085 / 6.8

2. Power - Power is presented in terms of kilowatt hours/year (KWH/yr) for the time of operation. The average power usage is assumed to be based on an average flowrate of 45 percent of the rated capacity of the plant, since most overflow rates do not reach the plant capacity.

3. Chemicals - The chemical requirements are basically a function of the flowrate and dosage. The amount of chemical usage is based on the average flow treated. For estimating purposes, it is assumed that the mixing intensity GT for the disinfection process is between 4,000-6,000. A Cl_2 dosage of 12 mg/l, a ClO_2 dosage of 6 mg/l, or combination of 2 mg/l ClO_2, followed by 8 mg/l of Cl_2 is assumed to provide a reduction of bacterial levels sufficient to achieve 1000 counts TC/100 ml and 200 counts FC/100 ml in the plant effluent. The maximum pumping rate at the Maltbie Street facility is 15 MGD (56,800 cu m/day). However, the average flowrate used in the cost estimate is 45 percent of the rated capacity, or 6.7 MGD (25,400 cu m/day). The duration of overflow is also assumed to be four hours for the purpose of calculating chemical requirements.

4. Miscellaneous Supplies - Miscellaneous supplies include spare parts, tools, insurance, gas, oil, contracted maintenance work allowances, and other consumable products not specifically accounted for elsewhere. These costs are less than those associated with a continuously operated plant but are essentially independent of the actual hours of operation of the plant.

5. Administrative Costs - The administrative costs associated with the CSO treatment facilities are assumed to represent a total of 5 percent of the overall administrative requirements for the agency responsible for the dry-and wet-weather treatment facilities. Similarly, material and supply costs associated with administrative requirements are included under this cost category.

6. Laboratory and Sampling Costs - This cost category is primarily a function of the number of samples and the types of analysis performed on each sample. For the estimates presented here, it is assumed that the number of samples collected is 4 per day per overflow, and a total of 60 overflows per year is assumed to occur. The cost of laboratory materials and supplies are included in the estimate.

7. Yard Maintenance - The requirements for yard maintenance are basically independent of the flow capacity of the plant. Guidelines which relate yard maintenance to area of site have been presented in the Dodge Guide (57) and reference (56), and are used as a basis for estimates.

PROJECTED COSTS FOR FULL CSO TREATMENT

In projecting the costs for treatment of CSO in the Syracuse area, it is assumed that the treatment devices investigated in this demonstration study will be utilized, i.e., microscreens or the swirl regulator/concentrator. Since evaluations of various storage and treatment combinations and specific sewerage conveyance capacities would require a complete Facilities Plan, these latter factors are not addressed in this report. Instead, it is assumed that the design flow to a specific CSO treatment facility is 15 MGD (56,800 cu m/day) and a design drainage area of 115 acres (46.5 ha) is utilized. Costs are developed on a cost/acre (cost/ha) basis and extrapolated to reflect the entire CSO drainage area in Syracuse of 9000 acres (3630 ha). Table 40 presents the design parameters for the CSO treatment facilities projected for treatment of CSO in the Syracuse area. A swirl regulator/ concentrator required to handle the design flow of 15 MGD (56,800 cu m/day), would be 18 ft (5.5 m) in diameter (13). Capital costs for all treatment devices were developed from previously published data (56). The same cost reference was applied to the disinfection equipment. However, the disinfectant feedrates were based on dosages of 12 mg/l Cl_2 and 6 mg/l ClO_2 for the Cl_2 and ClO_2 disinfection systems, respectively, in order to achieve desired bacterial levels as demonstrated in this study. The site work and pumping costs could be significant but will be variable for specific CSO treatment plant locations. An estimate of pumping and site work costs is provided later in this section.

TABLE 40. DESIGN PARAMETERS* - CSO TREATMENT FACILITIES

Design Parameter	CSO Treatment Device Sweco Unit	Zurn Unit	Crane Unit	Swirl Unit
Design Flowrate, MGD	15	15	15	15
Capacity/Unit, MGD	2.2	3.1	5.0	15
No. of Units Required	7	5	3	1
Cl_2 Dosage, mg/l	12	12	12	12
ClO_2 Dosage, mg/l	6	6	6	6
Cl_2 Feedrate, lb/day	1500	1500	1500	1500
ClO_2 Feedrate, lb/day	750	750	750	750
Design Drainage Area, acres	115	115	115	115
Total Drainage Area, acres	9000	9000	9000	9000

*Exclusive of Site Work and Wastewater Pumping Facilities
Notes: 1. Conversion: MGD x 3785 = cu m/day
lb/day x 0.454 = kg/day
acres x 0.4047 = hectares

Presented in Table 41 is a summary of the capital costs for identified Syracuse CSO treatment facilities handling 15 MGD (56,800 cu m/day). The capital costs for the entire CSO system ($/Total System) were derived by multiplying the cost per acre by 9000 acres (3640 ha), which represents the total acreage of the Syracuse combined sewer service area. The costs are presented to reflect the capital costs associated with disinfection by Cl_2 and ClO_2, respectively. From Table 41 it is apparent that the capital costs are significantly higher for microscreening than for a swirl regulator/concentrator. Using the swirl unit capital cost as a base, the Sweco unit would be five times higher, while the Zurn and Crane units would be 2.7 times and 1.8 times higher in cost, respectively, than the swirl facility. Although site work, connection piping and CSO pumping facility capital costs are not included in the analysis, the additional expense would be proportionately higher for the microscreens than for the swirl unit based on site work, interconnecting piping requirements, and equipment housing. Pumping facility requirements would be anticipated to be similar. The actual cost for site work and pumping for the microscreening installation was \$9,522/MG (\$2.50/cu m) as opposed to \$5,413/MG (\$1.33/cu m) for the swirl installation. However, the major portion of cost differential (pumping) is related to site-specific factors.

Table 42 presents a summary of the annual O&M costs projected for the CSO treatment facilities. The swirl unit is shown to be less expensive to operate and maintain than the microscreens. When compared to the swirl unit, O&M costs for the Sweco, Zurn and Crane units are projected to be 1.8, 1.6, and 1.6 times higher than the swirl, respectively. (Note: In addition to cost comparisons between the microscreens and swirl regulator/concentrator, consideration must be given to the solids removal effectiveness of the treatment devices. Higher removal requirements may dictate that microscreens be utilized dispite the cost advantage of the swirl regulator/concentrator. At the 15 MGD (56,800 cu m/day) design flowrate, the Crane would remove approximately 45 percent of the SS in terms of concentration, and the Zurn, Sweco and swirl units would remove approximately 30, 25 and 35 percent, respectively, as indicated in Figure 26).

Examination of Tables 41 and 42 also indicate that capital and O&M costs are projected to be lower for CSO treatment facilities which utilize Cl_2 as the disinfectant rather than ClO_2. This projection is a direct result of the higher cost for generation of ClO_2 at \$0.54/lb (\$1.18 kg) when compared to \$0.11/lb (\$0.24/kg) for Cl_2.

TABLE 41. SUMMARY OF CAPITAL COSTS - SYRACUSE CSO TREATMENT FACILITIES*

Treatment	Cl_2 Disinfection			ClO_2 Disinfection		
Device	\$/Acre	\$/MGD	\$/Total System	\$/Acre	\$/MGD	\$/Total System
Sweco Unit	11,120	82,250	100,080,000	11,420	87,500	102,780,000
Zurn Unit	5,860	44,960	52,740,000	6,150	47,210	55,350,000
Crane Unit	3,700	28,350	33,300,000	4,000	30,610	36,000,000
Swirl Unit	1,950	14,950	17,550,000	2,240	17,200	20,160,000

*Exclusive of Site Work, Connection Piping, and CSO Pumping Facilities
Notes: acres x 0.4047 = hectares
MGD x 3875 = cu m/day

TABLE 42. SUMMARY OF ANNUAL O&M COSTS* - SYRACUSE CSO TREATMENT FACILITIES

Treatment	Cl_2 Disinfection			ClO_2 Disinfection		
Device	\$/Acre	\$/MGD	\$/Total System	\$/Acre	\$/MGD	\$/Total System
Sweco Unit	290	2,220	2,610,000	310	2,350	2,790,000
Zurn Unit	240	1,830	2,160,000	260	1,950	2,340,000
Crane Unit	230	1,780	2,070,000	250	1,910	2,250,000
Swirl Unit	140	1,090	1,260,000	160	1,210	1,440,000

*Exclusive of Power Requirements for CSO Pumping Facilities
Notes: acres x 0.4047 = hectares
MGD x 3785 = cu m/day

An estimate of projected capital costs for CSO pumping facilities and site work is presented in Table 43. The basis of this estimate assumes that the costs incurred for the demonstration facilities would be applicable for all CSO treatment facilities throughout the Syracuse combined sewer system. It is recognized, however, that each site should be evaluated with respect to hydraulics. Table 43 includes an additional estimated annual cost for pumping and site work of $1,230,000 for the CSO treatment facilities if microscreening were employed, and an increased annual cost of $561,000 if the swirl regulator/concentrator were employed. When the pumping and site work costs are added to the projected annual costs, the resulting total annual costs would be as presented in Table 44.

Table 44 presents projected total annual costs for construction and operation of CSO treatment facilities in Syracuse. The analysis indicates that utilization of the swirl regulator/concentrator with application of Cl_2 as the disinfectant would be the least expensive method for treating CSO. Use of microscreens would cost at least 1.7 times more in construction, operation and maintenance than use of swirl regulator/concentrators. However, the solids removal requirements of the treatment devices must be considered as well as cost when the specific treatment device is to be selected.

In Section 1 of this report it was noted that the 1968 Comprehensive Sewerage Study had estimated the capital cost of centralized CSO treatment adjacent to Metro at $330,000,000 and a capital cost of point-source treatment at individual overflow sites at $485,000,000 (1978 dollars). The cost estimate presented in this report for solids removal and high-rate disinfection treatment processes have projected a capital cost of $33,000,000 to $103,000,000 for microscreening, and approximately $20,000,000 for swirl regulator/concentrators.

TABLE 43. PROJECTED CAPITAL COSTS OF PUMPING AND SITE WORK - CSO TREATMENT FACILITIES

Treatment Device	$/Acre	$/MG	$/Total System	Annual Cost
Sweco	1330	10,200	11,970,000	1,130,000
Zurn	1330	10,200	11,970,000	1,130,000
Crane	1330	10,200	11,970,000	1,130,000
Swirl	660	5,410	5,940,000	561,000

*Capital Costs amortized at 7 percent interest for 20 years.
Notes: acres x 0.4047 = hectares
MGD x 3875 = cu m/day

TABLE 44. PROJECTED ANNUAL COSTS* - SYRACUSE CSO TREATMENT FACILITIES

Treatment Device	Cl_2 Disinfection	ClO_2 Disinfection
Sweco	$13,189,000	$ 13,621,000
Zurn	$ 8,268,000	$ 8,694,000
Crane	$ 6,343,000	$ 6,778,000
Swirl	$ 3,478,000	$ 3,904,000

*Capital Costs amortized at 7 percent interest for 20 years.

The projected cost estimates developed under the CSO demonstration program for Syracuse indicate that a substantial reduction in costs could be attained through high-rate treatment application at point-source locations. It has also been demonstrated that further cost savings could be accomplished through the utilization of swirl regulator/concentrators in comparison to mechanical screening. Although the results of this program are based on a site-specific application, it does provide a viable alternative in evaluating overall abatement alternatives for the handling and treatment of CSO.

The reader is reminded that the design criteria used for development of costs for CSO treatment in Syracuse were extrapolated from actual costs incurred for construction and operation of the demonstration facilities. Therefore, consideration has been given to site-specific factors such as drainage area size and characteristics, population density, runoff coefficient, land use distribution, trunk sewer and intercepting sewer conveyance capacities, runoff pollutant characteristics, and rainfall patterns. Before utilizing cost data presented in this report, the reader should consult a USEPA report (107) published in 1976 which presents a methodology for assessing intermittent urban point-source loads such as stormwater and CSO. That study discusses in detail the importance of such site-specific factors as mentioned above, as well as describes supplementary data desired, levels of accuracy and spatial detail required in storm load characterization, and the effect that existing conveyance and treatment facilities may have on the storm load contribution. Methodologies for evaluating collection system control techniques, storage/treatment options, and flow regulation measures are also presented. Reference to that report is recommended to the reader to assist him in determining the level of effort and methodology of approach to address his specific CSO abatement problems.

SECTION 13

VIRUS STUDIES - MALTBIE STREET FACILITY

GENERAL

This section presents a summary description of the experimental procedures utilized in the collection, handling and analysis of virus organisms investigated in the Syracuse demonstration study. The testing program was divided into two phases, as described in Section 4. The results and discussion for each of the phases are presented separately following the description of experimental procedures utilized in each phase.

PHASE I PROGRAM

In the Phase I virus program, samples of CSO were taken before and after treatment, to determine the efficiency of various treatment and disinfection techniques for reduction of virus organisms in CSO. The Aquella Concentrator was used to concentrate numbers of organisms per unit volume of sample, to facilitate more reliable analysis. It was found that in general naturally occurring viral organisms in CSO are too few in number to permit satisfactory analysis, even with high degrees of post-sampling concentration.

Experimental Procedures (Phase I)

The experimental procedures used in the Phase I Program involved three separate steps:

1. Demonstration of and familiarization with the Aquella Virus Concentrator (Carborundum Company, Buffalo, N.Y.).

2. Actual sampling of flows during storm events.

3. Laboratory processing of samples.

Aquella Virus Concentrator--

The virus concentrator basically consists of a centrifugal pump, followed by three clarifying filters, a wound fiberglass adsorbing filter and a membrane type adsorbing filter. Accessories to these functions include a flow meter, by-pass valves, a proportioning pump for the addition of acid and cationic reagents, and an air compressor for emptying the liquid from the system. The filters have vents to release trapped air as they fill with water. In principle the orlon clarifying filters serve two functions: to remove silt, bacteria and other suspended solids and to remove membrane-coating organic materials which interfere with the adherence of virions to the adsorption filters. The viruses are preferentially adsorbed by the final two filters in the system. The first, a wound fiberglass filter of 1 nanometer porosity, adsorbs 50 to 95 percent of the virions. The second, a plate-filter holder with epoxy-fiberglass with 5.0, 2.0 and 0.45 micron membranes in series, concentrates the small number of viruses which pass the fiberglass filter.

Adsorption of virus particles to these filters is greatly enhanced by an acid pH and either aluminum or magnesium ions (22). In the absence of either magnesium or aluminum the adherence of poliovirus was very irregular which was probably due to trace contaminants of various divalent and trivalent ions. It was found that optimum recoveries of adenovirus were obtained at pH 4.5 rather than 3.5 when glycine buffer was used as the eluent. Aluminum ion at pH 4.5 and beef extract plus magnesium at pH 5.5 or 6.5 yielded equal numbers of viruses although both were less than expected. It has previously been stated that pH 4.5 is near the upper limit for the most efficient adsorption of enteroviruses (21). However, the recovery of poliovirus appeared slightly better at pH 4.5 than at 3.5, which was originally recommended for the Aquella (22).

Although the use of 3 percent beef extract, pH 8 to 9, was proposed as a nondestructive means of eluting viruses from membrane filters (58), no appreciable advantage of beef extract over rapid elution by pH 11.5 glycine followed by nearly instantaneous neutralization of the filtrate was found.

Phase I Sampling --

Storm overflows were collected in 55 gal (208 l) drums, trucked to the laboratory and refrigerated at 39.2°F (4°C). They were usually concentrated the following day. In the case of disinfected overflow samples, sodium thiosulfate was added to a level of 300 mg/l to neutralize disinfectant residuals. At 39.2°F (4°C) the titers of enteroviruses in storm overflows remained constant for more than a week. Repeated isolations of wild adenoviruses from sewage also indicated they were not inactivated by storage for several days at 39.2°F (4°C).

Phase I Laboratory Procedures --

After prefiltration to remove inorganic solids, bacteria and membrane-coating materials, the samples were split and concentrated on wound fiberglass and/or Cox membrane filters either at pH 3.5 (with Al+++) or pH 4.5 (with Mg++). The concentrates were frozen after neutralization and addition of nutrients.

Enteroviruses, adsorbed at pH 3.5, were cultured. Adenoviruses had previously been isolated in Syracuse sewage samples in the laboratory (but not in CSO) by a technique suggested by T.G. Metcalf (59), i.e., prefilters were treated to reduce adsorption losses and the viruses were adsorbed at pH 4.5 plus 0.05M $MgCl_2$. The virions were eluted with 3 percent beef extract and quantitated by plaque assays.

The grouping of the viruses from pH 3.5 adsorption was performed by using pooled antisera against polioviruses and coxsackie B viruses to remove one or both groups selectively. It was assumed that only the echoviruses would break through neutralization by the combined antisera. Presence of hemagglutinin for human type O cells and "ragged" plaque morphology helps confirm the identification of many putative echoviruses (16). Acridine orange staining of infected cell monolayers in microtiter plates quickly differentiated the occasional adenovirus or reovirus which survived adsorption at pH 3.5, since both groups exhibit green fluorescent emission. Sensitivity to low bicarbonate concentration in the agar overlay and ability to grow at 104°F (40°C) were used to determine whether the polio strains were vaccine or wild isolates.

Three types of assays were employed for the animal viruses: plaque formation (16), 50 percent tissue culture infective dose assays ($TC1D_{50}$) (17) and minimum probable number (MPN) estimates (18). The plaque forming units (PFU) were determined in conventional fashion by neutral red overlays in 60 mm plates. The $TC1D_{50}$ and MPN estimates were performed in Mictrotest II plates with 1 cm diameter wells (17). The wells were seeded with growth medium, an operation which was conducted in a laminar flow isolation hood. The seeded plates were incubated 48 hr, 98.6°F (37°C) 5 percent CO_2 atmosphere, 85 percent relative humidity. The plates were not sealed individually during the incubation period. Plates were inspected for cytopathic effects at 48 to 96 hr for enteroviruses; adenovirus plates were maintained 7 to 10 days with refeeding. The use of L-15 medium (20) and a higher pH, 7.4 to 7.6, greatly improved the maintenance of Microtest II cultures for these extended periods of observation.

Polioviruses were neutralized in samples of concentrated viruses by addition of pooled human gamma globulins or pooled hyperimmune rabbit antisera. Adenovirus isolates were confirmed in part through the use of pooled rabbit antisera. Pooled rabbit antisera were used to neutralize coxsackie B viruses and pooled human gamma globulin was found which had anticoxsackie B activity as well as activity against the polioviruses.

Results and Discussion (Phase I)

Two general types of filtration procedures were used to concentrate viruses from CSO: a two-step, discontinuous batch process and a continuous process. In the batch process a water sample was clarified and pumped into a reservoir, after which Al++++ or Mg++ was added, the pH was adjusted and the sample was pumped through the adsorbing filters. In the continuous process the water was pumped through the clarifying filters, a proportioning pump added inorganic ion and adjusted the pH to a previously determined ratio, and the water then passed through the adsorbing filters. In both cases, the flowrate averaged approximately 40 gal/min (150 1/min) although it was reduced in very turbid water.

There were marked differences in efficiency of virus recovery and reproducibility for the two processes. Batch type recoveries in volumes varying from 0.5 to 20 gal (1.9 to 75.6 l) were about 55 percent of adenovirus and 100 percent of enteroviruses, such as poliovirus 1 (PV1). With the continuous process low recoveries were experienced and there was no apparent correlation between the size of the inoculum, the volume of water or the type of water.

Recovery of Wild Viruses From CSO --

Viruses have been found in all sewage samples from both Syracuse and Baltimore (60) suggesting that urban stormwater and CSO have a high probability of carrying viruses.

Direct isolation of virus pathogens was demonstrated in the August 14, 1974 Syracuse CSO samples collected at the Maltbie Street facility. Poliovaccine virus probably constituted much of the isolation but other viruses broke through the poliovirus antisera in half the positive samples. Despite the obvious differences in sample sizes, Syracuse CSO and Baltimore stormwater did not differ greatly in the detection frequency. Whether or not this is fortuitous remains to be seen after a larger number of samples have been analyzed.

A limited number of CSO at Maltbie Street in the summer and fall of 1974 were analyzed in some detail. Viruses were detected in the overflows with one sample yielding quite high levels of viruses. Although full identification of the viruses in the "other" category was not made, plaque morphology and staining characteristics suggested that the viruses were not typical adenoviruses.

No clear-cut pattern emerged from the Maltbie Street results concerning the likelihood of detecting virus in early samples versus late samples during the course of a storm overflow. Viruses were observed in both early and late samples. Likewise there was no convincing evidence from these data of a distinct seasonal variation or a correlation between virus numbers and total volume of flow.

Disinfection of Viruses in CSO --

Storm overflows filtered by the Sweco microscreening unit were assayed before and after treatment with 4 mg/l ClO_2. Approximately a 1 min contact time was provided, after which the wastewater was pumped into barrels containing excess sodium thiosulfate to halt residual disinfection. Treatment with ClO_2 at this level provided no significant reduction in titer of these wild viruses. It must be noted that at the times of the highest level of viruses surviving ClO_2 disinfection sampling methods had not been fully perfected and lapses of 1.5 hr occurred between the chlorinated and unchlorinated samples. Without nearly simultaneous pretreatment controls it cannot be concluded unequivocally that no knockdown of wild virus was achieved--although it seems unlikely that levels of poliovirus as high as 230 PFU/gal (61 PFU/l) would be the remnant of a much larger virus concentration in CSO.

In general it appears that the low levels and random appearances of wild viruses will obfuscate any attempt to evaluate viricidal effectiveness of the treatment process directly.

Feasibility and Operating Experiences With the Aquella Virus Concentrator--

In wastewater samples with high organic content and/or silt content, a problem was encountered with the formation of a dense precipitate which clogged the adsorption filters at pH 3.5 to 4.5 and greatly impeded reconcentration of viruses from the 267, 125 and 47 nanometer membrane filters. Virus filtrates from 55 gal (208 l) samples frequently could not be recovered entirely on the 47 mm filter because of the low flowrate. The precipitates formed in creek water, sewage, pond water, stormwater and tap water. Attempts to eliminate the precipitate by various pH manipulations, addition of EDTA and substitution of other buffers for the glycine failed. The working solution was to concentrate the virions on the 125 nanometer filters only and to elute the virus with 3 percent beef extract, pH 9, overnight, 39.2°F (4°C). This eliminated long exposures of the viruses to pH 3.5 during reconcentrations.

Attempts to Develop f2 Phage as an Indicator Virus for Disinfection Studies --

The application of the Aquella virus concentrator to different water sources requires considerable expertise and necessitates expensive time consuming preliminary studies to determine the effect of a particular environment on the efficiency of virus recovery. Where water conditions change continuously, such as in CSO, very elaborate controls for adsorption efficiency must be instituted.

An attempt was made to determine if conditions for the adsorption and elution of enteroviruses from depth filters and membrane filters in the Aquella system would also concentrate f2 phages. This seemed feasible since these phages resemble picornaviruses superficially. Three concentrations of viruses in CSO were studied and four points in the Aquella system were titered to determine the amount of unadsorbed f2: 1) after the three orlon clarifying prefilters, 2) after the addition of glycine-HCl and Al+++ and before the depth filter, 3) after passage through the depth filter and the Cox filter, and 4) after elution with glycine-NaOH buffer, pH 11.5 and adjustment to pH 7.0.

It was found that both the adsorption and elution characteristics of f2 phage in CSO are different from those of the enteroviruses. Addition of Al+++ at pH 3.5 did not improve removal. Attempts to recover the f2 phage from the membrane filters by elution using several techniques failed as well. The efficiency of adsorption did not appear to be related to the initial virus load.

Feasibility and Operation Experiences --

The Aquella virus concentrator and its system of selective adsorption of viruses by filter media are reasonably well suited for study of CSO. The basic instrument met most of the manufacturer's claims. The system of prefiltration functioned well for stormwaters despite their high turbidity and solids contents. An overly fragile centrifugal pump and chronically leaky hose connections were both a nuisance and a potential hazard for the operating crew. Although the Aquella is a movable instrument, it is not very portable, particularly over broken ground, and is nearly impossible to move to difficult sampling sites. Field work with the concentrator requires rather elaborate supportive facilities including power generators, covered working space, refrigerated sample storage, etc.

The design oversight which permitted stratification of the acid $AlCl_3$ solution and hence poor pH control invalidated much of the first three months work. However, the recent addition of an efficient mixing chamber seems to have corrected the problem and pH can be maintained indefinitely in most CSO.

The principal drawback with the Aquella was the limited numbers of large samples which can be processed per working day. Even with hard-working, trained crews, a maximum of two 55 gal (208 l) samples per day could be concentrated to 13 ml. Flowrates rarely exceeded 40 gph (2.5 l/min) and usually fell to about 25 gph (1.6 l/min) halfway through the primary isolation, apparently due to the aluminum precipitate and colloidal materials which passed through the 1 micron prefilters and clogged the 257 nanometer Cox filters. Prospects for using the Aquella concentrator for frequent or <u>continuous monitoring</u> of viruses in CSO seem poor.

The levels of virus observed in the Phase I program by Aquella concentration techniques underscored two important considerations: 1) recoveries of seed viruses are very efficient so that an indicator vaccine virus(es) introduced into CSO before disinfection treatment and isolated after some standard retention time could be expected to reflect the efficiency of the viricide (dilution phenomena could be estimated by simultaneous introduction of a fluorescent dye indicator), and 2) the population of wild viruses in CSO is at a very low level, and the sample variation is very high. Thus it seems questionable whether a meaningful measure of disinfection effectiveness can be made on the basis of observed reductions in the wild viruses in CSO. Certainly a most important need is to determine the statistical constraints on virus sampling necessary to show any significant germicidal activity of the treatments. It is presently unknown whether virus pathogens in CSO approach anything like the steady-state populations of coliforms which occur in sewage. Closely spaced fluctuations in virus populations have not been measured. Thus there is no rationale with which to defend grab sampling as a means for evaluating any viral disinfection of CSO.

The degree to which CSO and Onondaga Creek contaminate Onondaga Lake with viruses has not yet been determined. It is probably a fraction of the virus input from the effluent of the Syracuse Metropolitan Sewage Treatment Plant which, when upgraded to tertiary treatment, will still discharge perhaps as much as 1 to 7 virus units per 100 ml if it performs like similar advanced treatment facilities (61). However, even if Metro could eliminate viruses in its effluent, the CSO would maintain Onondaga Lake and a portion of the Seneca River at an unacceptable level of virus content for fishing and bathing, since most virologists believe that 1 PFU can constitute an infectious dose (23, 62, 63). This hazard must be judged marginal but real. At present no virus standards for recreational (or potable) water supplies exist. Suggestions of virus levels at 1/1 or 5/1 have been made for drinking purposes (62) although this will probably be revised upward now that more efficient concentration methods such as the Aquella technique are available.

PHASE II PROGRAM

In the Phase II virus program, CSO flows to treatment units were deliberately seeded with relatively high levels of viral indicator organisms in order to insure more reliable analysis of numbers of organisms present, and of degrees of viral reduction with various disinfectant dosages. It was presumed, based on previous work by other investigators (64), that the log reduction in viral populations is largely independent of the initial size of population.

Experimental Procedures (Phase II)

The basic test system for the high-rate virus disinfection studies in the Phase II program employed 3 viruses as indicators which could be used to monitor inactivation of viruses in CSO: coliphages f2 and ØX174, and poliovirus 1 (Sabin, K1) oral vaccine strain. Phage levels were measured prior to and subsequent to disinfection injection. Eight grab samples were taken at each sampling point over a 30 min interval. Poliovirus 1 (PV1) was limited in most experiments to a single 55 gal (208 l) grab sample upstream and downstream from the point of disinfection; the variability due to dilution and uneven mixing was controlled by the addition of sodium fluorescein to the inoculum. Thus the base line was the ratio of plaque forming units (PFU) of virus to fluorescence units.

Wastewater Sampling and Inoculation --
Storm overflows collected in 55 gal (208 l) drums were trucked to the laboratory and refrigerated at 39.2°F (4°C). Sodium thiosulfate was placed in the drums before the samples were collected; the final concentration of thiosulfate was 300 mg/l, sufficient to neutralize any disinfectant residuals. At 39.2°F (4°C) the enterovirus titers remained constant for more than a week; adenoviruses were not inactivated by storage for several days at 39.2°F (4°C).

The virus inoculum (0.93 gal (3.5 l) containing 4 g disodium flourescein) was added to the influent of the Zurn Micromatic fine mesh (71µ) drum screen. This method of inoculum addition allowed complete mixing of the inoculum and CSO during the screening process. Grab samples were collected at two points--at the effluent from the Zurn unit just prior to discharge to the disinfection tank, and at the outlet weir of the disinfection tank after flash mixing and a one min contact time.

The bacteriophages were sampled in 10 ml aliquots at intervals of 0, 2, 4, 6, 8, 12, 20 and 30 min after addition of the inoculum. The 10 ml precalibrated tubes contained 0.5 ml sodium thiosulfate to halt the action of residual disinfectant. The samples were sterilized with chloroform and divided. One part was used for plaque assays after storage at 39.2°F (4°C); the other was frozen at -40°F (-40°C) and analyzed for fluorescein content later.

The 55 gal (208 l) samples were collected 4 min after inoculum injection on the Zurn effluent and after 6 min at the disinfection tank outlet weir. Ten ml samples were removed from each barrel for analysis of fluorescein tracer and bacteriophage survival; the animal viruses were concentrated from 50 gal (190 l) to 20 ml.

Results and Discussion (Phase II)

Grab Sampling and Equilibrium --

Two methods were used to determine the dwell time of the CSO between the pre-disinfection and post-disinfection sampling points-- disodium florescein concentration and phage titers. Fluorescein tracing tests indicated that dye appeared at the disinfection tank outlet less than 2 min after it was detected in the Zurn effluent. A peak of activity was observed between 2 and 6 min in nearly every case.

However, the peak activity showed large variations, apparently because thorough mixing of the inoculum injected in the influent to the Zurn unit contact tank (receiving the Zurn screened effluent) required several minutes to stabilize.

Typical plots of influent and effluent viral indicator organism levels are shown in Figures 70 through 73. Runs were made in which no disinfectant was added to seeded CSO, and concentrations of viruses at the two sampling points were approximately equal 8 min after addition of the virus inoculum. Nevertheless, large fluctuations were observed occasionally on the influent side for up to 12 min. Because of the propeller-type flash mixer, the effluent had much less variation. At 20 min the untreated samples differed by less than 2 min (dwell time) and were nearly equal.

Originally it had been planned to use the declining ratio of phage to fluorescein as a measure of relative activity of the disinfectant. Unfortunately, florescein is bleached by Cl_2 and ClO_2 and is not a suitable marker at concentrations above 5 mg/l Cl_2 and 2 mg/l ClO_2.

Table 45 presents the results of the 1976 disinfection investigations as related to the inactivation of viruses and enterobacteria. Each of the six storm events are discussed individually in the following paragraphs.

Storm 1. A relatively massive dose of chlorine (24 mg/l with a chlorine residual 1.0-1.4 mg/l) virtually sterilized the CSO. A clear-cut equilibrium was achieved on the influent side, but high level, constant dosing on the effluent side prevented any sort of steady-state from being established between the replacement viruses and the flash-mixing pool. The rate of kill exceeded the replacement rate.

Storm 2. Two serial runs were made on this storm without allowing an interval between the experiments for the Zurn screening unit to purge itself of the phage inoculum after the first serial run. The first run indicated more or less typical declines of a virus inoculum being killed by 12 mg/l Cl_2 at the same time that it was being slowly diluted. A

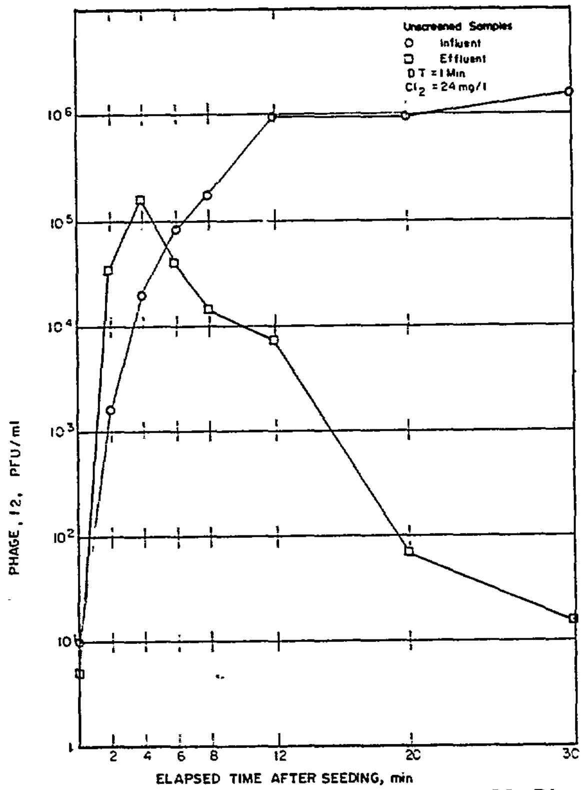


FIGURE 70. Inactivation of f2 Phage - Storm 1

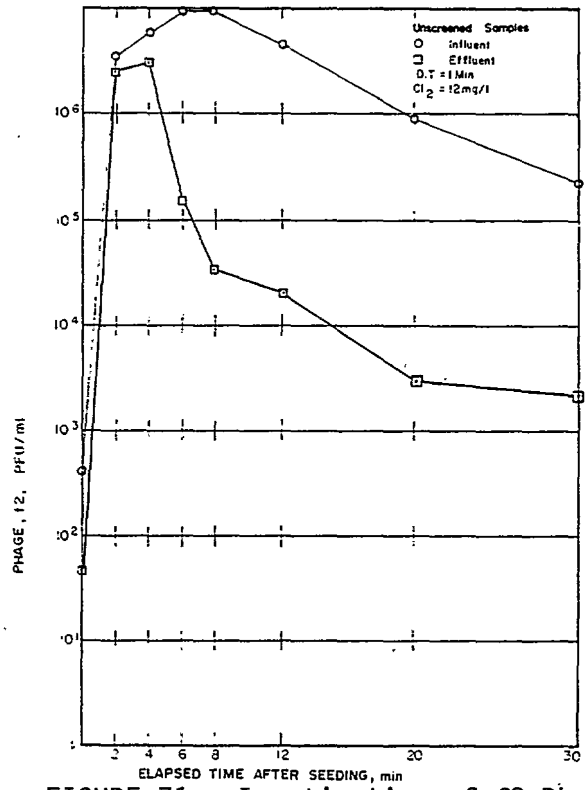


FIGURE 71. Inactivation of f2 Phage - Storm 4

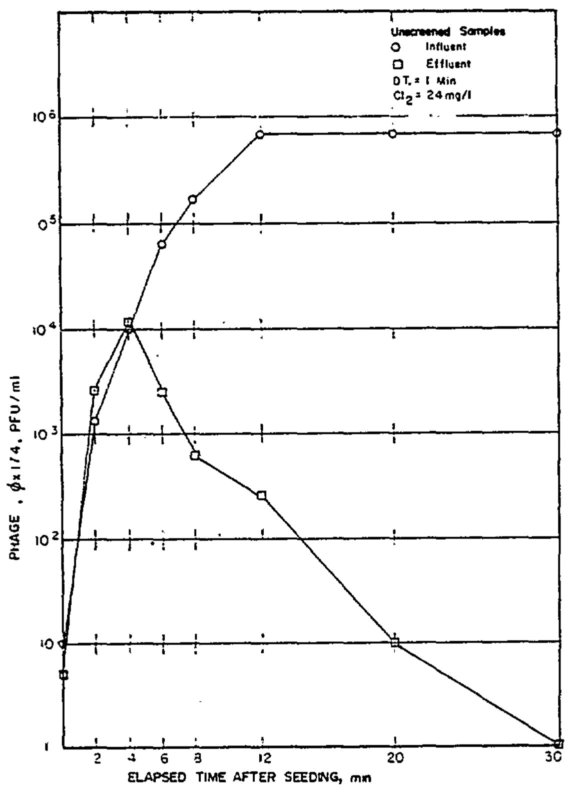


FIGURE 72. Inactivation of ØX174 Phage - Storm 1

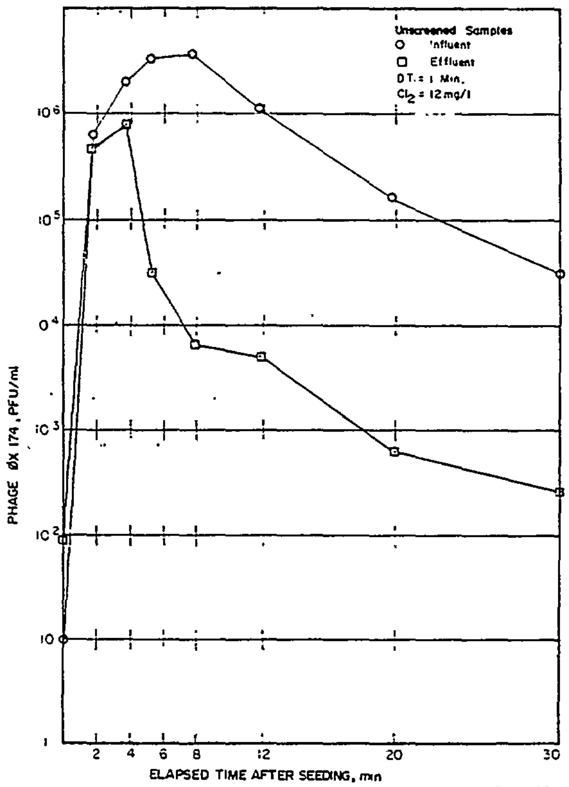


FIGURE 73. Inactivation of ØX174 - Storm 4

TABLE 45. COMPARATIVE INACTIVATION OF VIRUSES AND ENTEROBACTERIA

Storm	Date	Process[a]	Disinfectant Dose, mg/l Cl_2	ClO_2	Mixing[b]	Log Reduction f2	ØX174	Polio[c]	TC	FC	FS	Elapsed Time After Seeding, min
1	6/16/76	U	24		SF	4.3	4.7			5.3		20
2	6/30/76	U	12		SF	1.4	0.9		5.9	4.4		20
		U	12		SF	1.1[d]	0.9[d]		5.3	4.1		20
3	7/8/76	U		11	SF	0.3	0.1	4.4				20
		S	12		F	1.2[c]	0.2	3.1				20
4	7/29/76	U	12		SF	2.5	2.7	5.3		3.0		20
		S		3.4	F	2.1	2.4	5.2		1.5		20
5	9/10/76	U	8	2	SF	0.6	0.4	3.5	1.0	0.9		20
		S	8	2	F	0.0[e]	0.2	4.8	2.7	2.3		20
6	10/8/76	U		7.3	SF	0.2	0.3	2.1	1.4		2.3	20
		U		7.3	SF	0.3	0.2	2.3	0.7		2.1	20

[a] U = unscreened samples; S = screened (23μ) samples

[b] SF = sequential flash mixing; F = single flash mixing

[c] Elapsed time after seeding = 8 min

[d] Elapsed time after seeding = 30 min

[e] Elapsed time after seeding = 12 min

second run made in the last few minutes of the storm overflow gave an apparent confused pattern of mixing between 6 and 20 min. Interpretations of this anomaly can only be conjecture, but the reduced virus flow at these points may reflect the low level of water in the pump well at Maltbie Street as the storm abated. The 30 min samples may be more representative of the disinfection process in the second run.

Storm 3. The inoculum in Storm 3 contained not only phages f2 and ØX174 but poliovirus as well. Run 1 indicates the effect of a relatively heavy dose (11.0 mg/l) of ClO_2. Inactivation of phage at this level was minimal, being less than 1 log reduction in titer. The storm overflow ended before a complete second run with Cl_2 (12 mg/l) could be achieved. Only a low degree of inactivation appeared to be underway despite this relatively high level of Cl_2.

Both Cl_2 and ClO_2 resulted in very high levels of inactivation for PV1 (Table 46), which is not surprising for Cl_2 in view of the fact that at this high dosage, the free Cl_2 residuals are likely to be as large as 1-2 mg/l, a highly lethal dose of HOCl (19, 20). A killing efficiency of this order of magnitude for PV1 is greater than has been observed before for virus in wastewater and should be viewed with some skepticism until the experiment is repeated several times. That ClO_2 would show no effect would not be anticipated since other experiments have demonstrated the capacity of ClO_2 to be viricidal at both greater and lesser concentrations.

Storm 4. Treatment of the CSO was essentially a repetition of the schedule for Storm 3 with the exception of sequential vs single-flash mixing (Table 47). It is not clear how a reduction in the concentration of ClO_2 from 11.0 mg/l (Storm 3) to 3.4 mg/l (Storm 4) could permit an observed increase of 10-14 times the kill. Bacterial kills in Storms 3 and 4 showed a dependency on the time at which the samples were taken. In Storm 3 the reduction of FC ranged from 1 log near the beginning of the storm to 6 logs near its end; in Storm 4 the reductions varied from 1 to 2 logs at the storm beginning to 2 to 3 logs near the storm's end. However, the changes in CSO composition which affected the kill rate of bacteria so drastically had very little influence on the phages and the poliovirus.

It appears that the inactivation of viruses was nearly identical for ClO_2 and Cl_2 for the same storm and largely independent of the disinfectants' initial concentration. The disinfectants were probably in excess but an unknown secondary factor controlled the total kill.

Storm 5. This test attempted to examine the effectiveness of the disinfection procedure by sequential disinfection with 2 mg/l ClO_2 followed by 8 mg/l Cl_2 after 15 sec. There was no dramatic increase in viral kill over the previous storms in which the disinfectants were added separately. The total kill of PV1 (Table 48) remained at about the same level as observed in Storms 3 and 4. The second run produced killing curves more or less identical to the first run.

TABLE 46. DISINFECTION OF CSO SEEDED WITH PV1 - STORM 3

Treatment of CSO+	PFU/ml concentrate Cl_2*	ClO_2**
Before disinfection	$23x10^6$	$66x10^5$
After disinfection	$64x10^2$	$83x10^2$
Log_{10} reduction	4.44	3.09

**Unscreened, ClO_2 11.0 mg/l
*Screened, Cl_2 12 mg/l
+CSO was seeded; unseeded CSO titer was less than 1 PFU/ml

TABLE 47. DISINFECTION OF CSO SEEDED WITH PV1 - STORM 4

Treatment of CSO+	PFU/ml concentrate Cl_2*	ClO_2**
Before seeding	0	0
After seeding	$133x20^3$	$160x10^3$
After disinfection	$42x10^1$	0
Log_{10} reduction	5.13	5.20

**Screened, ClO_2 3.4 mg/l
*Unscreened, Cl_2 12 mg/l
+CSO was seeded; unseeded CSO titer was less than 1 PFU/ml.

Storm 6. Continuous addition of ClO_2 throughout the storm allowed a large degree of stabilization of disinfectant dosage. The previous disinfection patterns of phages f2 and ØX174 reduction were obtained on both the first run and the second run. Phage reductions were very low or nonexistent; PV1 recoveries (Table 49) indicated reduced activity from that obtained with the quantities of ClO_2 in Storms 3, 4 and 5. Little indication of significant sampling anomalies in the inactivation curves of the phages was evident.

TABLE 48. DISINFECTION OF CSO SEEDED WITH PV1. STORM 5

Treatment of CSO+	PFU/ml concentrate Run 1*	Run 2**
Before disinfection	57×10^4	6×10^5
After disinfection	16×10^2	96×10^1
Log_{10} reduction	3.45	4.78

*Unscreened, ClO_2 (2 mg/l) followed by Cl_2 (8 mg/l) after 15 sec
**Screened, ClO_2 (2 mg/l) followed by Cl_2 (8 mg/l) after 15 sec
+CSO was seeded; unseeded CSO titer was less than 1 PFU/ml

TABLE 49. DISINFECTION OF CSO SEEDED WITH PV1. STORM 6

Treatment of CSO+	PFU/ml concentrate Run 1*	Run 2*
Before disinfection	37×10^4	25×10^5
After disinfection	48×10^2	55×10^3
Log_{10} reduction	2.11	2.34

*Unscreened, continuous ClO_2 dose, 7.3 mg/l.
+CSO was seeded; unseeded CSO titer was less than 1 PFU/ml

Effect of Cl_2 and ClO_2 on Naturally Occurring, Wild Viruses in CSO

Isolations of viruses from untreated water showed a background titer of about 40 PFU/gal (10 PFU/l). These appeared to be mostly poliovirus of unknown origin since they were neutralized by antipolio sera. It is possible that some of the viruses originated from earlier polio seeding experiments. Although adenoviruses occasionally appear in sewage, none were isolated from any of the Syracuse CSO. Flows from Storms 4 and 5 were seeded with 3780 PFU Adenovirus 31/gal CSO and treated with 12 mg/l Cl_2 and with 4 mg/l ClO_2 for 6 min each. No virus was recovered from either treatment when 0.52 gal (2.0 l) were concentrated and the equivalent of 400 PFU were inoculated into tissue culture flasks.

SUMMARY OF VIRUS STUDIES

The data indicate that several novel observations were made during the course of the study and that after treatment of CSO with Cl_2 and/or ClO_2 the indicator viruses behaved somewhat differently than had been predicted from the bench-scale study (7). Phage ØX174, which was shown to be much more sensitive to ClO_2 than phage f2 (under conditions of low ClO_2 demand), exhibited almost identical sensitivity to ClO_2 under the conditions of these experiments. Indicator viruses showed minimal response-- and sometimes no response-- to either the chemical form or the concentration of the disinfectant. For example, in three different experiments 12 mg/l Cl_2, 11 mg/l ClO_2 and 3.4 mg/l ClO_2 achieved essentially identical kills.

The prediction that f2 phage would make a good simulant for enteroviruses in CSO and would be useful for monitoring treatment plant effluent did not prove to be true. Polio virus in CSO was 2 to 3 orders of magnitude more sensitive to Cl_2 and ClO_2 than f2 phage. Neither the concentration of suspended solids nor the chlorine demand of the water proved to be a reliable predictor for the efficiency of disinfection. Some of the most effective disinfection occurred when SS were highest.

Screening had no influence on virus inactivation in CSO. This confirmed earlier observations from the bench-scale study that the enteroviruses and phages occurred as single, free virions and were several orders of magnitude smaller than the finest microscreen.

The use of a fixed flowrate for the experiment improved the reproducibility of the disinfection methods within a storm. However, it had no obvious effect on inter-storm variability. Flash mixing provided evenly distributed samples and supplied a more predictable microbial population as targets for disinfection. The even distribution of fluorescein label measured immediately after flash mixing showed that mixing of the samples and disinfectant was complete within a few seconds.

Simultaneous reductions in bacterial and viral titers were common but there was no direct proportional relationship between them. Samples were obtained in which (1) both bacteria and viruses were reduced; (2) bacteria were reduced several logs and viruses only slightly; and (3) viruses were reduced several logs and bacteria only slightly.

Previous experiments reported elevated kills as a result of episodic additions of small doses of two disinfectants (Cl_2 and ClO_2) rather than a large dose of one. The field trials at Maltbie Street failed to confirm such enhanced kills. The explanation is fairly direct. The multiple additions in the bench scale studies were made to still suspensions of viruses followed by stirring. Since multiple small additions resulted in prolonged stirring, the kills were improved because of greater contact time between the virus and the disinfectants (shorter diffusion path) before the latter decayed. In the demonstration treatment plant maximum contact was achieved in all schedules by flash mixing. Thus no improvement was seen by multiple additions to the flash mixer. In fact, reduced total kills resulted probably because the flash-mixing of multiple small doses failed to achieve free residual chlorine levels for as long as one large flash-mixed dose.

The use of either Cl_2 or ClO_2 can be justified on the basis of their ability to reduce titers of enterovirus (PV1) in CSO by 2.1 to 4.7 logs. Naturally occurring wild enteroviruses (1600 PFU/40 gal) were eliminated by treatment with 12 mg/l Cl_2 for 1 min and by 3.4 mg/l ClO_2 for 1 min.

On the basis of reductions of seeded PV1 samples it appears that the minimum dose of ClO_2 should be 8 mg/l, and the minimum dose of Cl_2 should be 12 mg/l.

Mild acidification (pH 3.5) followed by chlorination would vastly increase the efficiency of Cl_2 or ClO_2. If the receiving water of the creek could not tolerate this pH change, it might be feasible to pump the effluent over a bed of crushed limestone to neutralize the acid.

SECTION 14 SPECIAL CONSIDERATIONS

GENERAL

In addition to the basic objectives of the Syracuse CSO demonstration study, certain peripheral investigations and studies were performed either on related research and demonstration work which would enhance present and future studies on CSO, or field testing of specific applications.

The areas covered under this section include special analyses for formation of Cl_2 and ClO_2 organic species formed during disinfection, and ClO_2 sensitivity; special investigations for ultraviolet disinfection, and adenosin triphosphate (ATP) assay for disinfection control; and special instrumentation applications of telemetering, a total organic carbon (TOC) monitor, and a suspended solids (SS) meter.

SPECIAL ANALYSES

Interaction of Cl_2 and ClO_2 with Organic Species in CSO Wastewaters

Chlorinated Hydrocarbon/Pesticide Scan--

In 1976, analyses of trace organic compounds by gas chromatography scans (the concurrent determination of several related compounds by a single procedure) were conducted on 13 CSO samples. The samples were collected from the Maltbie Street treatment processes prior to and subsequent to disinfection with Cl_2, ClO_2, or sequential addition of ClO_2 and Cl_2. Samples were analyzed for non-volatile chlorinated hydrocarbons, which include PCBs and several common pesticides. The analytical technique employed was hexane extraction of hydrocarbons from a sample followed by gas chromatography analysis as recommended by EPA in 40 CFR 136. The chromatographic conditions were:

Column: 3 percent OV-1 on Chromosorb W-HP 80/100

Carrier gas: nitrogen at 0.013 gpm (50 ml/min)

Column temperature: 392°F (200°C)

Detector: electron capture

The objective of this trace organic scan was to provide a first-cut screening of CSO to determine the general magnitude of chlorinated hydrocarbons rather than to quantify or identify specific compounds.

The results of the gas chromatography as presented in Table 50 indicated a lack of correlation of chlorinated organics formation upon addition of either Cl_2 or ClO_2 or upon sequential addition of ClO_2 and Cl_2. The change in chlorinated hydrocarbon content ranged from -8.5 to +123.7 percent for Cl_2, from -7.5 to +108.3 percent for ClO_2, and from -2.2 to +200.0 percent for the sequential addition case. Concentrations of chlorinated hydrocarbons of untreated CSO ranged from 1.2 to 59 µg/l and disinfected samples contained from 2.5 to 54.0 µg/l. However, the majority of samples (5 of 8) indicated significantly increased levels of chlorinated organics after addition of disinfectants.

The limited analyses conducted on untreated and treated CSO in the Syracuse demonstration study indicated that chlorinated hydrocarbons are formed during disinfection processes utilizing Cl_2 and/or ClO_2 as disinfectants. However, significantly more data points are required to adequately quantify the amounts of chlorinated organics formed.

Volatile Chlorinated Organics Formation - Bench Scale Studies--

If both Cl_2 and ClO_2 are to be used effectively as wastewater disinfectants, the chemical processes associated with disinfection must be understood. It has been fairly well established that organic compounds in CSO compete with pathogens for disinfectants. This has been established in studies of natural water systems containing synthetic organic chemicals. Chloroform, bromoform, and their intermediates have been identified after chlorination of Rhine River water for the water supply of Rotterdam (65) and Mississippi River water for the water supply of New Orleans (66).

In order to review the potential interaction of Cl_2 and ClO_2 with organic matter present in CSO, the aqueous chemistry associated with Cl_2 and ClO_2 must be understood. In aqueous solutions of Cl_2 five reversible half-reactions are necessary to describe the rapid equilibria which describe the distribution of hypochlorous acid (HOCl), hypochlorite ion (OCl^-), chlorine monoxide (Cl_2O), trichloride ion (Cl_3^-), the hypochlorous acidium ion (H_2OCl^+) and of course undissociated chlorine (Cl_2). The chlorine used in wastewater disinfection also contains traces of bromine and subsequently small quantities of the respective bromooxy acids.

At low pH's the chlorate ion interacts with hydrochloric acid to form Cl_2 and ClO_2, ClO_2 being a relatively stable and water-soluble free radical. Since this process is one of the fundamental methods of producing ClO_2, it should be noted that the reaction forming ClO_2 results in the production of significant quantities of Cl_2 via the following mechanism:

$$4H^+ + 2Cl^- + 2ClO_3^- \rightarrow Cl_2 + 2ClO_2 + 2H_2O$$

Another method of producing ClO_2 involves the reaction of HOCl with sodium chlorite to produce Cl_2O_2 and its subsequent breakdown into ClO_2 and Cl_2.(67)

Therefore unless extraordinary measures are undertaken to separate the Cl_2 and subsequent oxychlorine species, a pure solution of ClO_2 is not

TABLE 50. CHLORINATED HYDROCARBON/PESTICIDE SCAN DURING DISINFECTION TESTS

Storm No.	Date	Sample Location	Disinfection Dose	CHP µg/l	Change Upon Disinfectant Addition: Absolute Volume	Change Upon Disinfectant Addition: Percent Change
1	6-16-76	Raw Influent		59.0		
		Unscreened Effluent	Cl_2 = 20 mg/l	54.0	- 5.0	- 8.5
3	7-08-76	Raw Influent				
		Crane Effluent	Cl_2 = 12 mg/l	4.1	+ 0.1	+ 2.5
		Unscreened Effluent	ClO_2 = 11 mg/l	3.7	- 0.3	
4	7-29-76	Raw Influent		17.7		
		Crane Effluent	ClO_2 = 3.4 mg/l	36.6	+18.9	+106.8
		Unscreened Effluent	Cl_2 = 12 mg/l	39.6	+21.9	+123.7
5	9-10-76	Raw Influent		4.6		
		Crane Effluent	ClO_2 & Cl_2 *	13.8	+ 9.2	+200.0
		Unscreened Effluent	ClO_2 & Cl_2 *	4.5	- 0.1	- 2.2
6	10-08-76	Raw Influent		1.2		
		Unscreened Effluent	ClO_2 = 7.3 mg/l	2.5	+ 1.3	+108.3

*Disinfectant dosages applied as follows: 2 mg/l ClO_2 followed by 8 mg/l Cl_2 after 15 sec.

TABLE 51. VOLATILE CHLORINATED ORGANIC CONCENTRATIONS* PRODUCED FROM VARIOUS DISINFECTION SCHEMES APPLIED TO SIMULATED CSO

Disinfection Scheme	Contact Time	$CHCl_3$	$BrCl_2CH$	$ClBr_2CH$	$CHBr_3$	Cl_3CCH_3	CCl_4	Cl_3C_2H	Cl_4C_2
8 mg/l ClO_2	1 min	5.1	<1.0	<1.0	<1.0	<1.0	<1.0	3.4	8.1
	5 min	9.5	<9.5	<1.0	<1.0	1.9	<1.0	2.7	6.8
	10 min	5.0	<1.0	<1.0	<1.0	<1.0	<1.0	2.8	6.0
12 mg/l Cl_2	1 min	9.3	<1.3	1.8	<1.2	2.1	<1.2	6.5	12.0
	5 min	6.7	<1.2	1.8	<1.0	1.6	<1.0	4.1	4.1
	10 min	6.7	<1.0	<1.0	<1.8	2.1	<1.8	6.1	18.0
2 mg/l ClO_2	1 min	6.3	<1.0	<1.0	<1.0	1.4	<1.0	2.2	3.9
followed by									
8.0 mg/l Cl_2	5 min	5.2	<1.0	<1.0	<1.0	1.6	<1.0	2.1	3.9
	10 min	5.9	<2.0	<2.0	<2.0	2.0	<2.0	6.4	20.0
Control		5.1	<1.0	<1.0	<1.0	<1.0	<1.0	3.4	8.1
Reagent Blank		<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0

* All analytical results are presented in μg/l.

produced. This will have a significant bearing when evaluating the interaction of both Cl_2 and ClO_2 with organic substrates characteristic of CSO.

Chlorine and oxychlorine species react with organics and organo-nitrogen compounds via a wide range of mechanisms involving electrophilic substitution, nucleophilic substitution, oxychlorine-catalyzed hydrolysis and free radical mechanisms. Some information exists which indicates that in certain oxychlorine solutions where ClO_2 is dominant, ClO_2 will react with various organic constituents by an "oxidative hydrolysis" reaction mechanism whereas Cl_2 results in chlorination via the chlorine-free radical mechanism.

In an effort to determine the relative formation of volatile chlorinated organic compounds as a result of the Cl_2 or ClO_2 disinfection of CSO, several bench scale tests were conducted. CSO was simulated by mixing equal parts of the Syracuse Metropolitan Sewage Treatment Plant influent with distilled water and then subjecting equal aliquots with 12 mg/l Cl_2, 8 mg/l ClO_2 and a sequential addition system of 2 mg/l ClO_2 with 8 mg/l Cl_2 added after 25 percent of the total detention time. Samples were analyzed after 1, 5 and 10 min detention times via gas chromatography utilizing a conventional purge and trap technique. Each sample was analyzed for the following volatile halogenated organics:

chloroform ($CHCl_3$)	carbon tetrachloride (CCl_4)
bromodichloromethane (BCl_2CH)	tetrachloroethylene (C_2Cl_4)
chlorodibromomethane ($ClBr_2CH$)	1,1,1 trichloroethane (Cl_3CCH_3)
bromoform ($CHBr_3$)	1,1,2 trichloroethylene (Cl_3C_2H)

The volatile halogenated organic analytical results are presented in Table 51 for each evaluated disinfection scheme and contact time. The results indicate that tetrachloroethylene is the most significant volatile chlorinated organic produced by application of either ClO_2 or Cl_2. The second most significant volatile chlorinated organic produced in the course of disinfection is chloroform. The concentrations of both chloroform and tetrachloroethylene in the CSO control are significant with approximately 5.1 µg/l of $CHCl_3$ and 8.1 µg/l of C_2Cl_4.

If the control measured concentrations of each of the volatile chlorinated organics are subtracted from the concentrations measured on each of the disinfection systems, it can be seen that only low levels of volatile chlorinated organics are produced in the ClO_2 system. The production of chlorinated organics in the Cl_2 and ClO_2/Cl_2 systems is more significant with higher concentrations measured in those samples reflecting longer detention times.

The results cannot be considered conclusive; however, they do tend to indicate advantages in the use of ClO_2 as a disinfectant relative to reduced volatile chlorinated organic production. It is important to note that Cl_2 and oxychlorine aqueous systems have very complex equilibria systems which result in measurable concentrations of ClO_2 being present in Cl_2 solutions as well as measurable concentrations of Cl_2 present in ClO_2 solutions. Because of this latter phenomena and the variability in precursor concentrations in wastewater samples, it is difficult to conclude that the application of ClO_2 to CSO disinfection would be without volatile chlorinated organic byproduct formation.

ClO_2 Sensitivity

The ClO_2 used for conducting many of the disinfection tests at both Syracuse demonstration sites was generated on-site by means of a Nitrosyl Chloride system (U.S. Patent 3754079, Chemical Generators, Inc., Rochester, New York). The process consisted of pumping two batches, sodium chlorate-sodium nitrite slurry and nitric acid, into a specially designed lucite chamber where the chemicals were mixed. The resulting reaction was expected to produce a 12 percent solution of ClO_2 to be fed directly to the microscreened CSO.

The two solutions of chemical reagents were pumped into the specially designed mix chamber by duplex pumps mounted on the base of each generator. The strengths of the reagent solutions were such that volumes had to be mixed at a 1:1 ratio to produce the 12 percent solution of ClO_2. The 12 mg/l dosage feed to the microscreened CSO was to be held constant regardless of total effluent flow by controlling the pump motor rotational velocities with 4-20 ma DC signals from each screening system flowmeter. The micrometer setting on each chemical feed pump could be adjusted to provide a smaller ClO_2 dose to the microscreened CSO, if desired.

At the start of the project, the duplex pumps for each of the generating systems were calibrated to determine the exact volumes of liquid pumped to the microscreened CSO. The pumps were adjusted to deliver equal volumes of reagent to the mix chamber to meet the 1:1 ratio mix criteria. The pump calibration curves were checked periodically throughout the project. Subsequent to mixing of the two reagents, the generated ClO_2 flowed by gravity to the disinfection tank.

Several problems were encountered during the project requiring constant attention. The first major problem was failure of the duplex pumps to function properly. Minor malfunctions such as the plugging of check valves and leaks in the piping arrangements prevented operation of the units during the first stages of the project. In addition, major electrical problems occurred which made pacing of the ClO_2 generators extremely difficult and forced abandonment of automatic pacing. Instead, the units were operated manually and the resulting ClO_2 feed dosages were determined for the varying CSO flowrates.

Serious questions arose as to the strength of the ClO_2 being generated as a result of the frequent, short duration operation of the facilities and subsequent reagent pump warm-up time, corrosion of the lucite chambers, and the containment time in the pumping system after formation of the ClO_2. Since immediate response of the facilities was necessary when an overflow event occurred, warm-up time of the reagent pumps became a critical factor in producing ClO_2. Approximately one-half hour was necessary for warm-up during which time the 1:1 ratio was not being achieved and the strength of ClO_2 became less than optimal.

In addition, a more serious question of ClO_2 strength at the point of disinfectant injection to the microscreened CSO became evident. Depending on the CSO flowrate, the volume of ClO_2 being fed to each system varied from a mean low of 20 to 40 gpd (75 to 150 l/day) to a high of 450 gpd (1700 l/day). These flowrates resulted in containment times in the disinfection piping system of from 30 min to 2 min, respectively. A study of the effects of the stability of ClO_2 with time had been conducted in another study (7) and results indicated that ClO_2 deteriorated by as much as 40 percent during the first 5 min after generation. Thus, containment times from the point of generation to point of injection became very significant.

Since the containment time in the initial ClO_2 generation piping was significant, a technique for more rapid delivery of disinfectant to the treated CSO was considered necessary. After consultation with Chemical Generators, Inc., a new mixing chamber was installed on each of the four ClO_2 generators at the two demonstration sites. The lucite chamber was replaced by a 4 in. (10 cm) diameter glass tube affixed to the front of the ClO_2 generator panel. The two reagents were pumped into one end of the glass tube at the desired 1:1 ratio. In addition, water was pumped into that same end of the glass tube to increase the flowrate from the tube to the point of injection. A Jabsco pump at a rated capacity of 3 gpm (11.3 l/min) pumped water out of a small storage box constantly refilled from the city water supply by a mechanical float control mounted inside the box. The 3 gpm (11.3 l/min) pump delivered approximately 1 gpm (3.8 l/min) to each of the three ClO_2 units at Maltbie Street and a total capacity of 3 gpm (11.3 l/min) at West Newell Street. Containment times for various flowrates were thus reduced from 2 to 30 min to 30 to 40 sec at Maltbie Street. At West Newell Street the containment time was reduced to 10 to 15 sec.

In addition, for a portion of the study, the ClO_2 was generated at a strength of only 4.2 percent instead of the desired 12 percent. It is theorized that the effects of sunlight on the reagent storage tanks may have resulted in a loss of strength on either or both of the reagents thus resulting in decreased production of ClO_2.

Two methods of assessing ClO_2 presence were considered in this project. The first is the DPD technique (N-N-diethyl-p-phenylene diamine) (68) which has the capability of determining specific concentrations of ClO_2, chloramines, ClO_2, or Cl_2. However, the concentrations of such constituents must be below 2 to 4 mg/l to consider the analysis accurate. Since the ClO_2 strength desired in the field tests was supposed to be a 12 percent solution or 120,000 mg/l, the DPD technique was not considered suitable for accurate use since a lengthy procedure of diluting the original sample to 2 to 4 mg/l would have to be developed. Such a procedure is difficult in the field where conditions are rapidly changing and thus not conducive to good accuracy.

The second method of analyzing ClO_2 strength in the field is the starch-iodide titration method (68). This technique has the disadvantage of measuring ClO_2, chloramines, and Cl_2 as a group. Thus no direct determination of ClO_2 strength is made with this technique. The most optimal solution to the measurement of ClO_2 strength as produced in the field would be a DPD titration to determine if the generated solution is predominately ClO_2 followed by the starch-iodide technique to determine the actual strength of the solution.

Field analyses of the ClO_2 generated at the Maltbie Street and West Newell Street sites were based on the starch-iodide titration method. Since the solution produced was expected to be predominantly ClO_2, the measurement by starch-iodide titration was assumed to yield a fair indication of the ClO_2 strength. The field analyses for ClO_2 strength followed the procedure for the starch-iodide method outlined in Standard Methods. Field samples analyzed were collected at a point just prior to injection into the CSO at the contact tank. Although considerable evaluation and confirmation of ClO_2 activity in simulated CSO had been conducted on a bench-scale level, using electron spin resonance techniques (7), confirmation of ClO_2 dosages applied during the full-scale demonstration study was not attempted due to the sophistication and expense of the electron spin resonance equipment.

SPECIAL INVESTIGATIONS

Ultraviolet Disinfection

General--

At CSO treatment facilities, where long Cl_2 detention times are not feasible for adequate disinfection, a bactericidal method or combination of methods requiring minimum retention time is desired. The bactericidal properties of ultraviolet radiation for certain applications are well-established, and there are several advantages of ultraviolet (UV) disinfection over chlorination that make UV disinfection very attractive. The major advantage of UV radiation is that disinfection occurs without addition of any chemicals to the waste stream that may result in harmful residual compounds.

A series of experiments were performed to determine the feasibility of disinfecting simulated CSO using UV radiation alone and in conjunction with Cl_2. The various combinations of disinfection tried were:

1) UV alone
2) Cl_2 alone
3) UV followed by Cl_2
4) Cl_2 followed by UV
5) Simultaneous UV and Cl_2

The parameters that were varied were:

1) Cl_2 dosage
2) Cl_2 contact time

3) UV radiation time which affects power usage
4) Sample volume

Bench Scale Program--

The samples used were raw sewage samples collected each Monday between 8:30 and 9:30 AM at Metro. Each sample was screened through a 74 micron mesh screen and diluted with an equal volume of deionized water. Portions of sample which were left over from the Monday experiments were refrigerated; before reuse they were brought back to room temperature.

The UV source was a PCQ 9 G-1,7 in.(17.8 cm) photochemical immersion lamp manufactured by Ultraviolet Products, Inc., San Gabriel, California. The intensity was calculated to be approximately 5,800 $\mu W/cm^2$.

Chlorine solutions were prepared by diluting chlorox 1:10 to yield a concentration of 5.5 mg/l Cl_2.

One liter samples were irradiated in a bell jar reaction vessel which was designed so that the walls of the flask were equidistant (2.25 in.) (5.7 cm) from the UV lamp. One half liter samples were irradiated in a plastic one liter graduate cylinder which was modified to accept the UV lamp. 100 ml samples were irradiated in a 100 ml glass graduated cylinder equipped with a ground glass joint. Each vessel was covered with aluminum foil to increase the intensity of the UV radiation. Figure 74 presents a schematic of the UV disinfection test apparatus.

The prepared sample was introduced into the reaction vessel and stirred via magnetic stirring bar for one minute prior to any treatment. The action of the Cl_2 was arrested by adding, at the appropriate time, sodium thiosulfate directly to the reaction vessel or by introducing the sample to a sterile collection bottle containing the thiosulfate. The UV lamp was used with no prior warm-up and was controlled by a switch.

Samples were withdrawn from the reaction vessels via sterile 10 ml pipettes and collected in sterile bacteria bottles. In some instances, 1 ml samples were withdrawn and directly pipetted into bacteria dilution bottles. The solution was continuously stirred during the collection of the sample.

Disinfection Results--

UV Only-- UV disinfection of wastewater is a function of the lamp intensity, measured as microwatts per square centimeter ($\mu W/cm^2$), contact time, measured in seconds and distance from the UV source. The product of the lamp intensity and contact time is expressed as $\mu W\text{-}sec/cm^2$.

The UV source used in these bench scale studies was approximately one-sixth as powerful as the lamps used in commercial UV disinfection units. For that reason, the percent kills of bacteria were related to $\mu W\text{-}sec/cm^2$ rather than to contact time. The experimental data was plotted as Number of Surviving Colonies per 100 ml vs $\mu W\text{-}sec/cm^2$ and the curve was extrapolated to determine the power required to disinfect a one liter sample to a target level of less than 2,400 counts TC/100 ml, (see Figure 75).

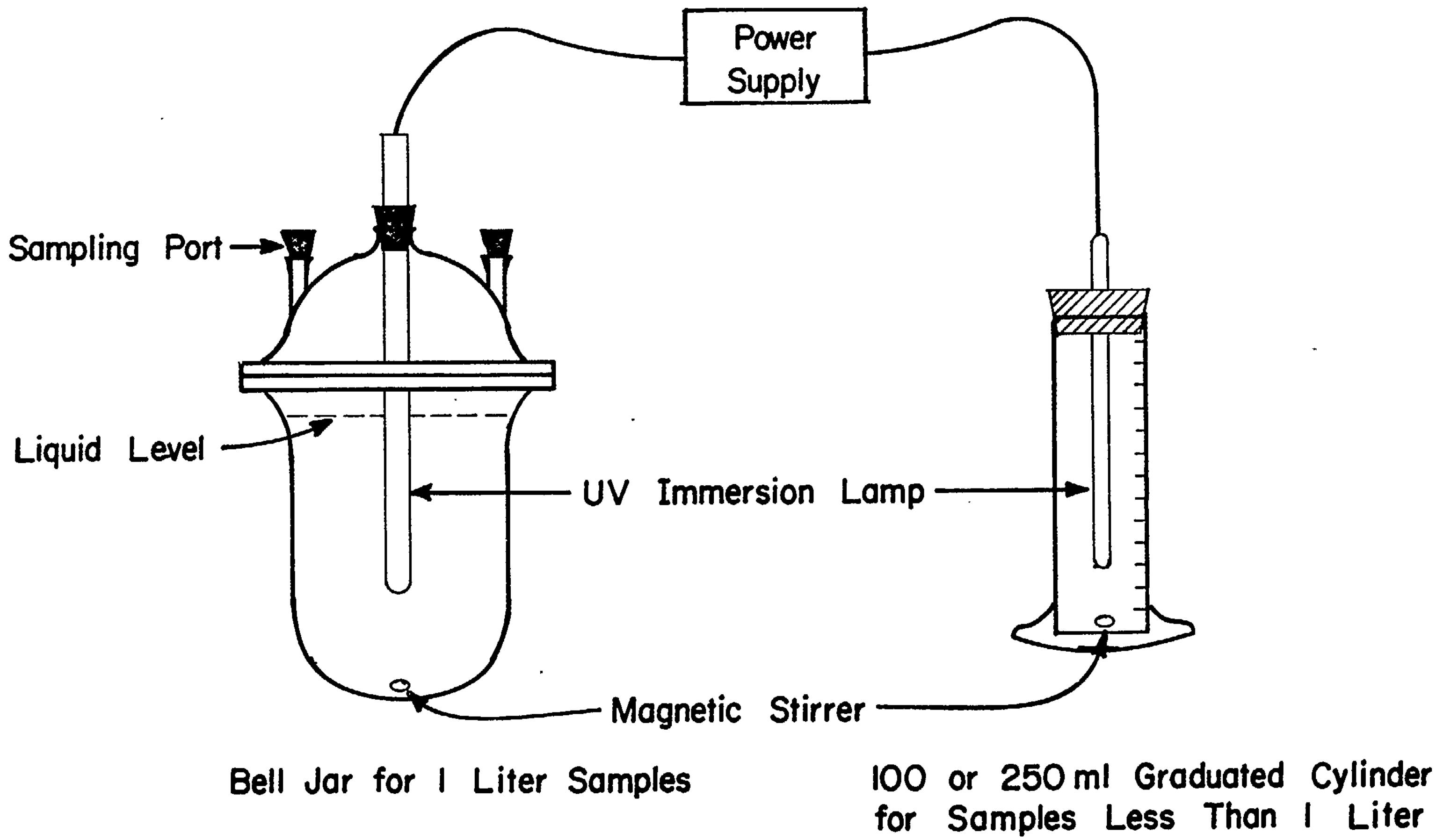


FIGURE 74. Schematic of Bench-Scale UV Disinfection Testing Apparatus

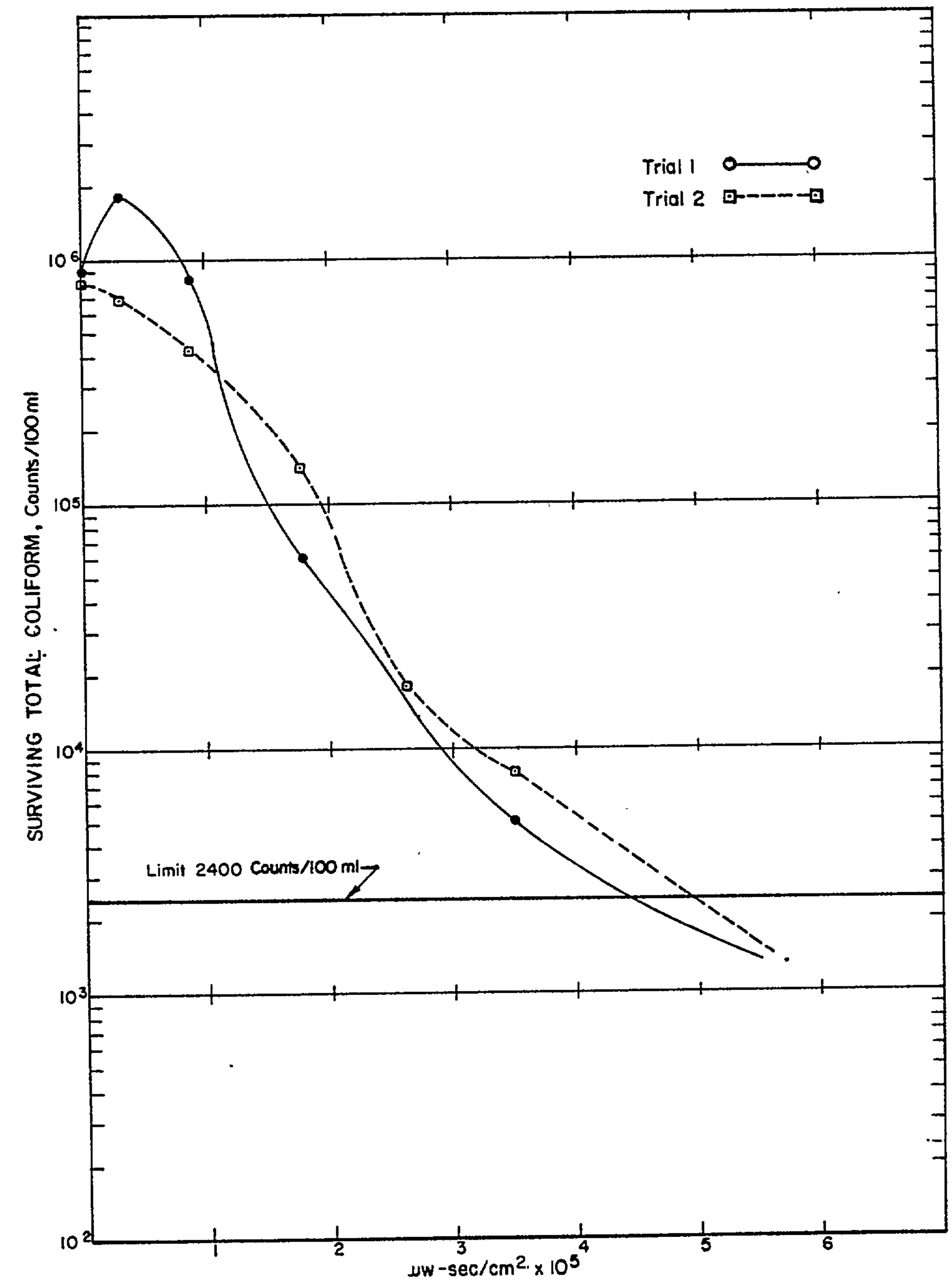


FIGURE 75. Number of Surviving TC vs μW-sec/cm^2

A value of approximately 500,000 per liter (equivalent to $2.3X10^{12}$ μW-sec/cm^2/MG {$8.7X10^{15}$ μW-sec/cm^2/cu m}) was obtained.

Although 500,000 μW-sec/cm^2 per liter are required for the desired level of disinfection, a plot of percent inactivation vs μW-sec/cm^2 for 1 liter samples (Figure 76) reveals that 50 percent inactivation is obtained with only 58,000 μW-sec/cm^2 and 75 percent inactivation is obtained with 116,000 μW-sec/cm^2. To achieve an additional 24.9 percent inactivation required greater than 350,000 μW-sec/cm^2. This data suggests that the disinfection of TC bacteria with UV radiation, under static conditions, is dependent only on the concentration of the bacteria, and is, therefore, a first order process. However, it was not in the scope of these bench-scale tests to evaluate the effects of such parameters as color, turbidity, and solids levels which would affect UV disinfection performance.

The effect of contact time and lamp intensity for the experimental apparatus is presented in Figure 77. The analysis of surviving TC as a function of radiation time indicates a logarithmic relationship. Similarly, a logarithmic relationship is observed between lamp intensity and surviving TC.

<u>Cl_2 Only</u>-- Disinfection with Cl_2 was performed only to serve as a comparison to UV disinfection and to combined UV and Cl_2 disinfection. Results of Cl_2 disinfection are discussed in the bench-scale study report (7).

<u>UV Followed by Cl_2</u>-- Irradiating the sewage mixture for 5 sec (29,000 μW-sec/cm^2)with UV light resulted in an average TC inactivation of 46 percent. Chlorination of the UV radiated mixture with 5.5 mg/l Cl_2 for 10 sec inactivated 8 percent of those bacteria which survived the UV irradiation, whereas a 30 sec contact time inactivated 60 percent of the surviving bacteria.

Doubling the UV radiation time, while keeping the Cl_2 dosage and Cl_2 contact time constant at 5.5 mg/l and 30 sec respectively, increased the Cl_2 efficiency by only 2 percent (Figure 78). On the other hand, doubling the Cl_2 concentration to 11.0 mg/1, while keeping the Cl_2 contact time and the UV radiation time constant, increased the percent inactivation of UV surviving bacteria from 60 to 90 percent (see Figure 79).

This suggests that extending the UV radiation time does not significantly enhance the efficiency of Cl_2 at concentration levels such as 5.5 mg/l, and that following UV irradiation the percent inactivation is more dependent on the subsequent Cl_2 concentration rather than on the UV radiation time.

<u>Chlorination Followed by UV</u>--This series of experiments determined the effect of UV irradiation after a sample had been chlorinated. During the experiments, the action of the Cl_2 was not neutralized by addition of thiosulfate when the UV radiation began; therefore, the Cl_2 and UV were acting on the sample simultaneously.

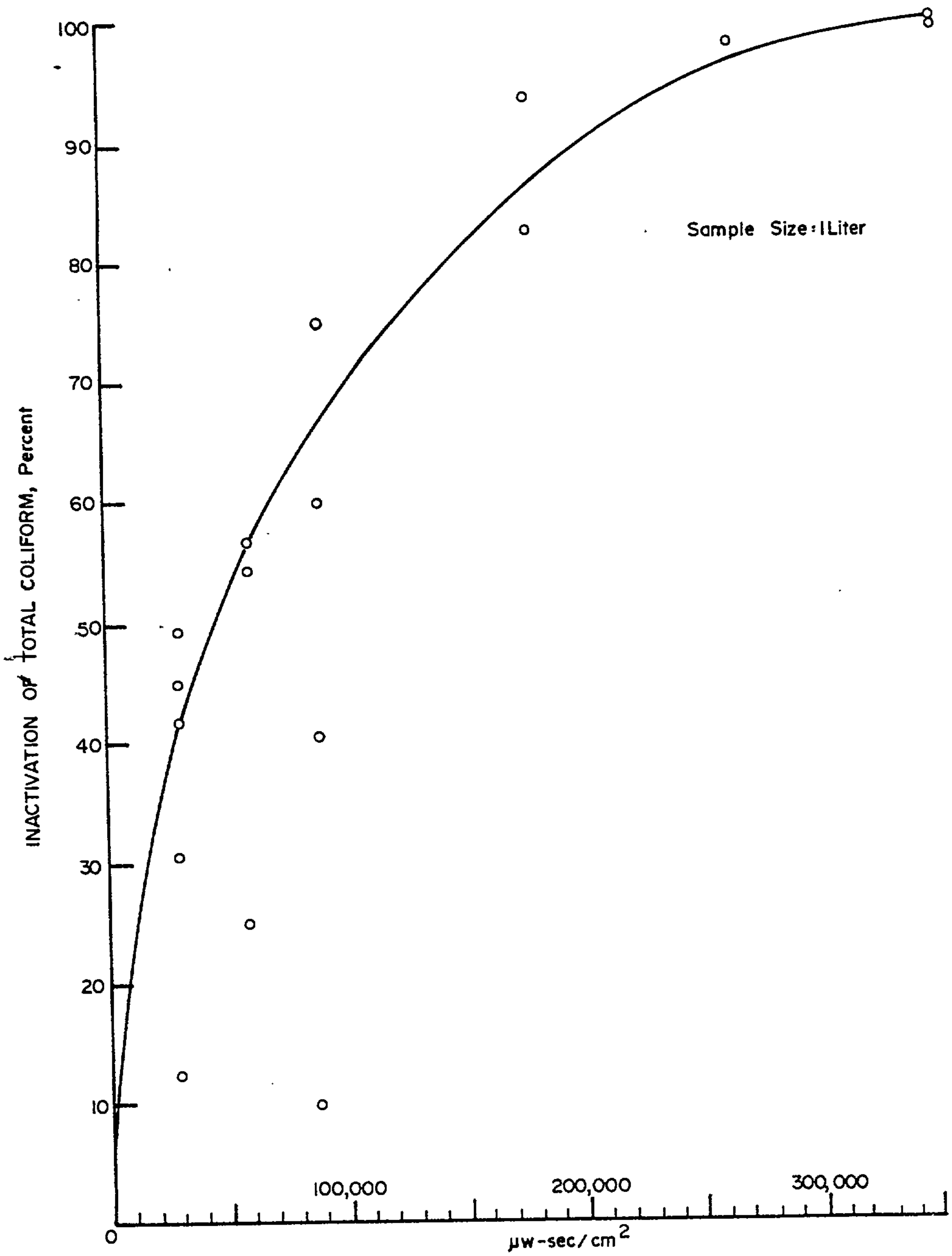


FIGURE 76. Percent Inactivation of TC vs $\mu W\text{-}sec/cm^2$

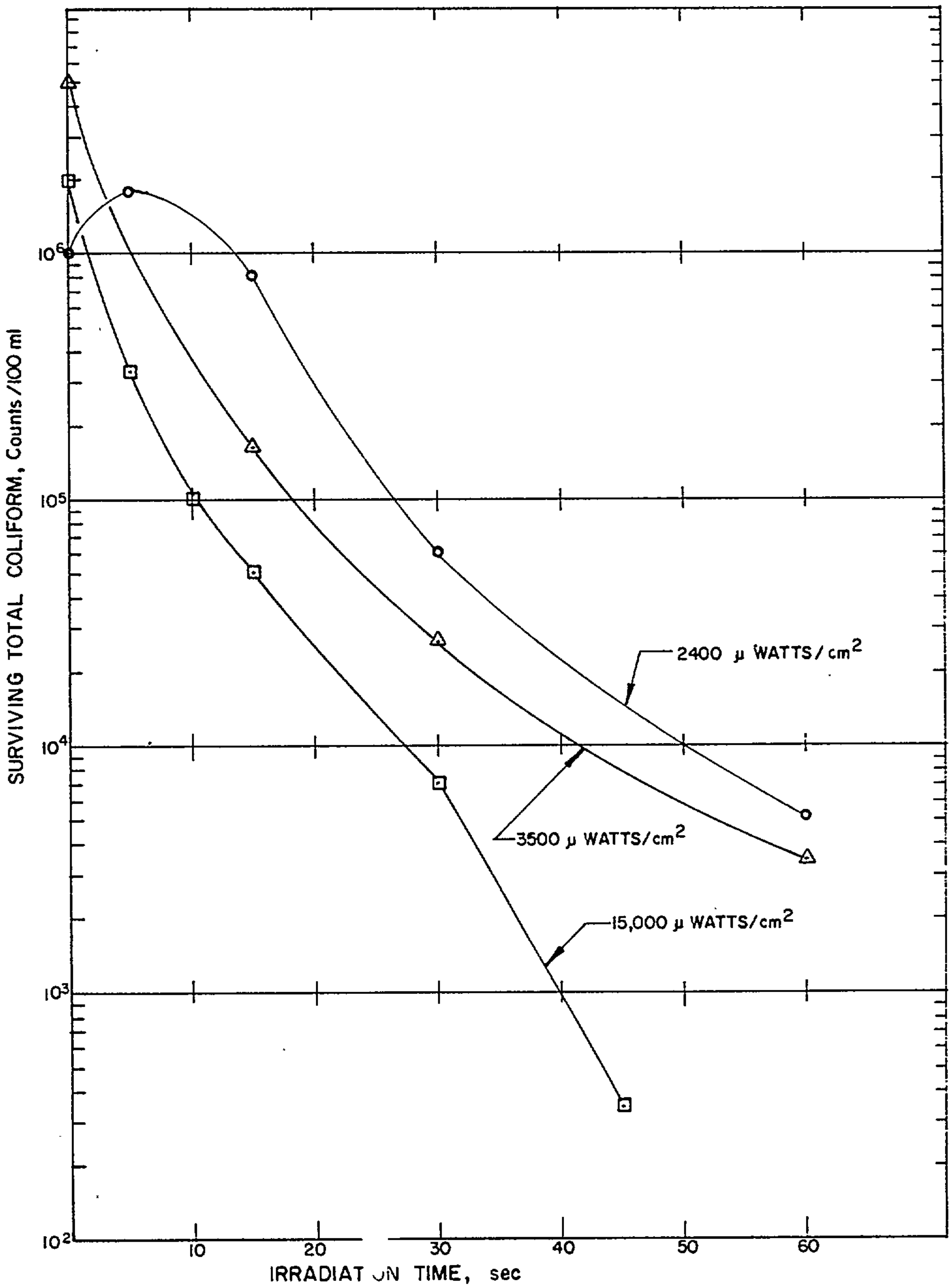


FIGURE 77. Effect of Ultraviolet Light Intensity on Disinfection

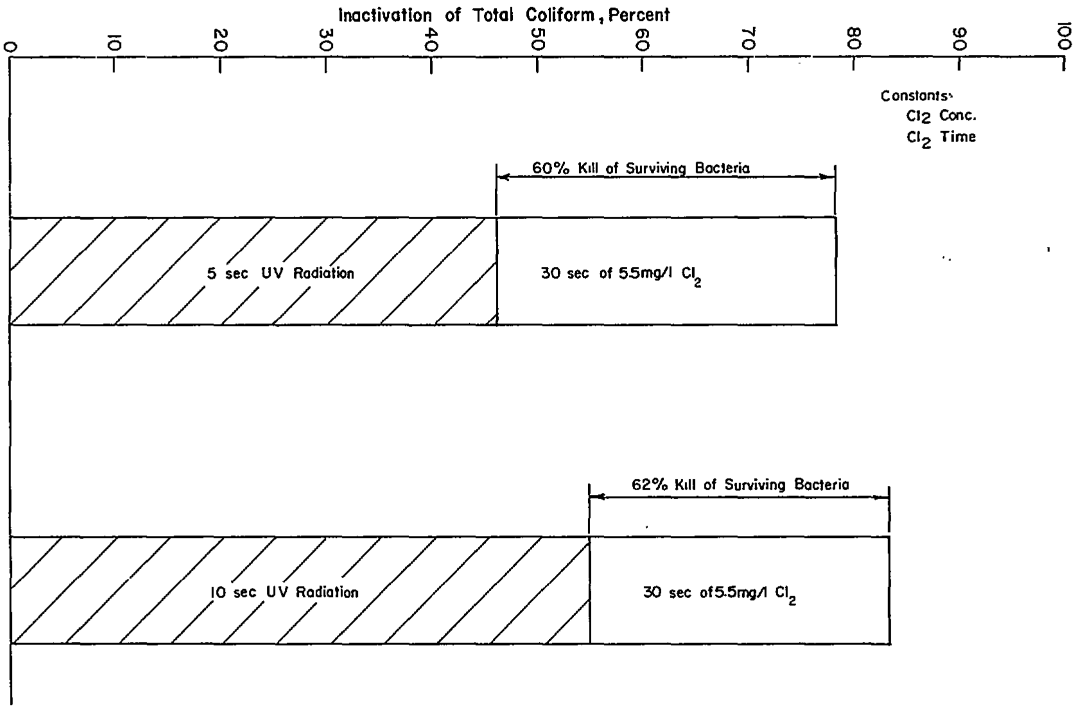


FIGURE 78. Effect of UV Exposure on Cl_2 Disinfection

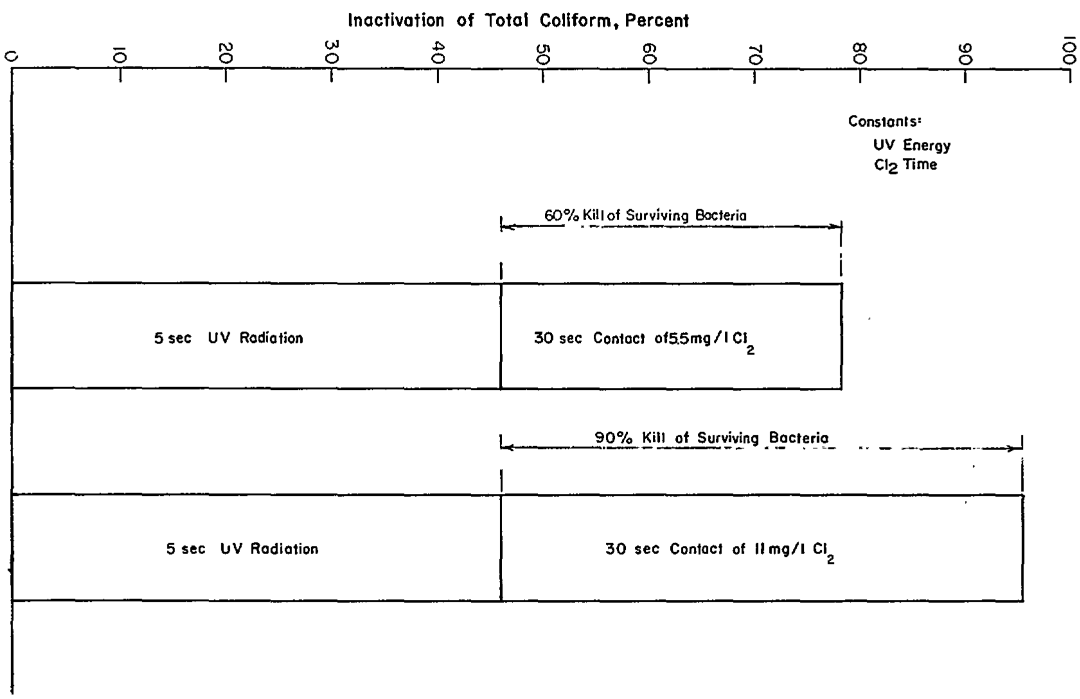


FIGURE 79. Effect of Cl_2 Dose on Disinfection Following Ultraviolet Light Exposure

Experimental data showed that higher levels of inactivation were obtained when, regardless of the Cl_2 concentration, longer Cl_2 contact times were in effect prior to UV irradiation. It was also noted that the percent inactivation was more dependent on the Cl_2 concentration than on the UV energy supplied; in essence, greater kills were achieved by increasing the Cl_2 dosage rather than increasing the UV radiation time.

Simultaneous UV and Cl_2-- The results of simultaneous disinfection of the combined wastewater with Cl_2 and UV was also evaluated. Figure 80 presents a comparison of the results of the effectiveness of Cl_2 versus simultaneous UV and Cl_2 disinfection as a function of contact time. It can be seen that the effectiveness of Cl_2 as a disinfectant is significantly enhanced by the application of UV radiation. Simultaneous disinfection was more effective in that it resulted in a higher rate of disinfection, producing slightly higher levels of inactivation at lower contact times than the corresponding single disinfecting components as indicated by increases in the slopes of the curves presented in Figure 80.

Review of UV Disinfection Equipment--

The most recent UV disinfection modules consist of sterilizing chambers which house the high intensity UV lamps. Depending on the manufacturer these lamps may be a low pressure mercury vapor lamp, or a hot cathode tungsten filament lamp. These lamps do not actually come in contact with the water since they are housed in specially constructed quartz sleeves. The purpose of the quartz sleeve is to act as a temperature buffer, so that the lamp may operate at approximately 105°F (41°C) regardless of the water temperature. A second feature of the quartz sleeve is that it protects the lamp from a buildup of solids. Many units on the market are equipped with a device which automatically wipes the quartz jacket clean in the event of any debris buildup.

The water enters the module and, by means of baffles and/or helical discs, is made to travel around and over the quartz sleeves, thereby, increasing the time in contact with the UV radiation. These baffles and discs preclude the possibility of water entering and leaving the module on a plug flow basis.

Most commercial UV disinfection units provide in excess of 25,000 µW-sec of ultraviolet radiation at a wavelength of 2540Å, whereas, most bacteria require between 6000 to 13000 µW-sec of UV light at 2540Å for complete destruction. According to manufacturer's information, lamp life is approximately 7500 hr assuming a lamp on-time of 8 hr/day.

More intermittent operation reduces the lamp life due to the degradation of the lamp caused by startup and shutdown. Based on an average 12 hr/day operation, a UV lamp would need to be replaced in approximately 24 months. Two years is the maximum time recommended for a lamp to be used in intermittent operation because of various deterioration and degradation factors which take place. After extensive use of the UV lamp, the glass solarizes and starts absorbing some of the 2540Å light, thus reducing the amount of light available for disinfection, and reducing its efficiency. Replacement lamps are available from $50 to $200 depending on the size.

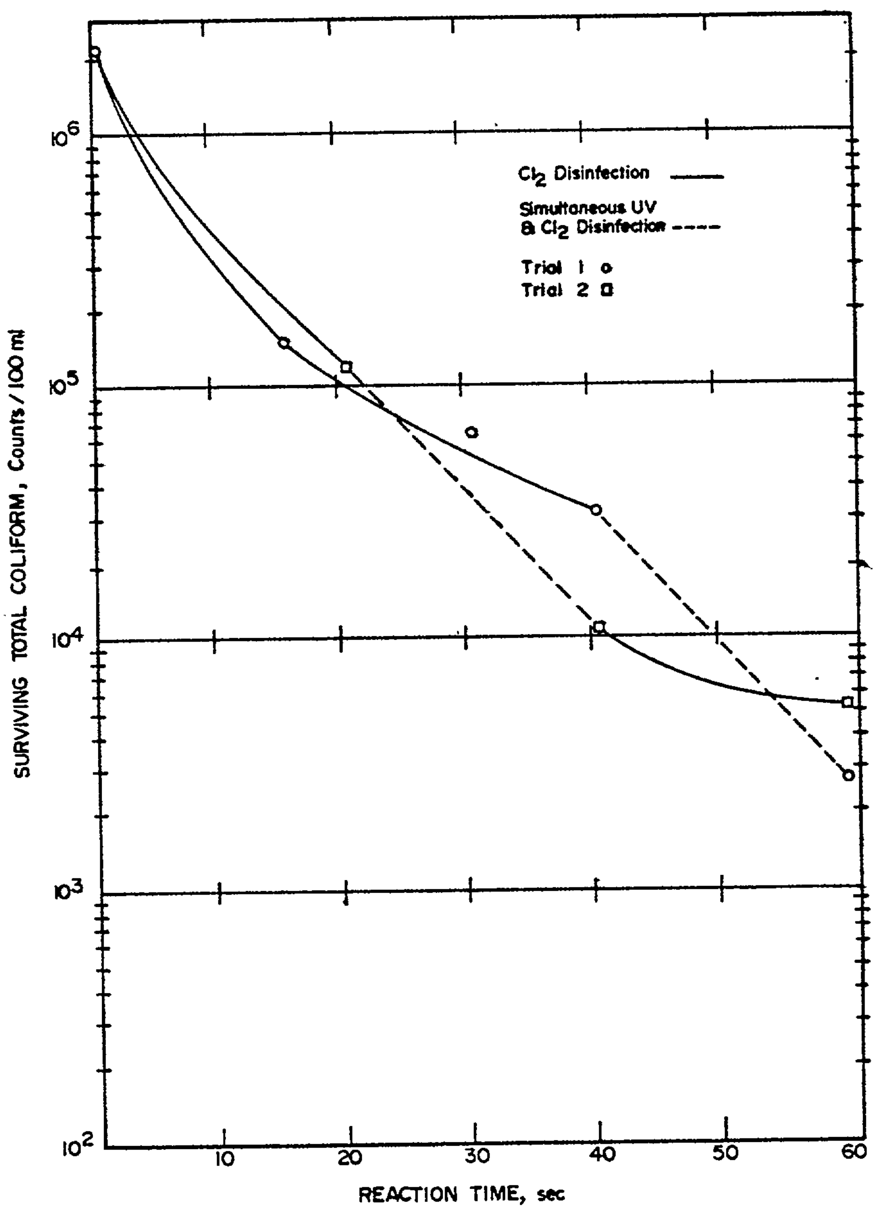


FIGURE 80. Surviving TC vs Reaction Time for Simultaneous Disinfection by UV and Cl_2

The efficiency of UV disinfection is reduced as the concentration of UV absorbing material in the water is increased. Suspended and colloidal matter will interfere with disinfection by absorbing the UV light. Also dissolved matter, such as organic matter and iron will absorb UV radiation. Dissolved alkali metals are known to absorb UV light but to a much lesser degree than iron salts.

To eliminate any of the adverse effects of extraneous matter found in CSO, the influent to the UV disinfection system should be pretreated by chemical coagulation and filtration or some reasonable equivalent.

Comparative Cost Analysis of Disinfection Systems--

Based on the results of the bench scale investigations a cost estimate was developed for UV and combined Cl_2 and UV disinfection systems. Estimates were also prepared for Cl_2 and combined Cl_2 and ClO_2 disinfection systems based on results presented in Sections 11 and 12. The capital and operating costs for all systems are presented in Tables 52-55, and have been developed for a design flow of 15 MGD in accordance with comparative cost relationships established in Section 12. A summary of costs for all disinfection systems is presented in Table 56.

Annual costs were not developed since these investigations represented bench-scale applications, and the purpose of presenting capital and operating costs is merely for basic cost comparison. Although it appears that disinfection by UV is not cost-effective when compared to Cl_2, consideration has not been given to the toxicity effects from Cl_2 residuals which may require dechlorination and consequently would increase the cost of Cl_2 application.

Adenosine Triphosphate Assay for Disinfection Control

The present methods of determining the efficiency of wastewater disinfection processes and controlling the addition of disinfectants may not be adequate to meet recently enacted standards of 200 FC/100 ml of sampled water (69) without the risks involved in the presence of excess disinfectants (70). The traditional approach to the assessment of disinfection efficiency has been the enumeration of the coliform bacteria, especially fecal coliforms, which are generally accepted as an indication of contamination from human or animal sources and, consequently, the presence of pathogenic bacteria and viruses (71). The principal objection to this approach is that one of the most common and rapid methods of bacterial enumeration, the membrane filter technique, requires a 24 hr incubation period, during which incomplete disinfection or the addition of excess disinfectants may occur in the system. A second objection is that coliforms may not be as resistant to disinfection as some pathogenic microorganisms, therefore an inadequate measure of disinfection efficiency (72).

The most common method of controlling disinfectant dosages is the monitoring of residual disinfectant to maintain a fixed level in the treated effluent (73), a method that admittedly has its limitations (74). Other methods may include addition of disinfectant based on historical flow patterns

TABLE 52. ULTRAVIOLET SYSTEM COSTS - 15 MGD CAPACITY

Capital Cost	
a) American UV Unit*	
1.7 MGD unit @	$100,000
9 units required for 15 MGD	x 9
Total	$900,000
b) Sterilaire Unit**	
0.22 MGD unit @ $2,500 extrapolated to 15 MGD	$170,000
Compensation for solids and turbidity (Number of units tripled)	x 3
Total	$510,000
Operating Costs/Day	
a) American UV Unit*	
(9 units)(1,280W/unit)$\left(\frac{24\text{ hrs}}{\text{day}}\right)$($0.02/KWHR)	$5.55
Total	$5.55
b) Sterilaire Unit**	
78.3 MG treated @ cost of $105.00 extrapolated to 15 MG	$20.20
Total	$20.20

* American Ultraviolet Company, Chatham, New Jersey
**Ultraviolet Products, Inc., San Gabriel, California

TABLE 53. COMBINED Cl_2 & UV SYSTEM COSTS - 15 MGD CAPACITY

Capital Cost	
UV component*: (35 units) x ($1,600/unit)	$ 56,000
Chlorination component:	37,000
Total	$ 93,000
Operating Costs/Day	
UV power: (40 units)(280 W/unit)(24 hrs/day)($0.02/KWHR)	$ 5.38
Chlorine: (8 mg/l)(15 MGD)(8.4)($0.10/lb)	100.80
Total	$ 106.18

*Sanitron Modular UV Disinfection Unit (Atlantic Ultraviolet Corp., Bay Shore, N.Y.)

Price: $1,600
Capacity: 5,000 gph
Power Requirements 280 W
Contact Time 13 sec
Intensity 30,000 $\mu W/cm^2$
Disinfecting Power**(13 sec)(30,000 $\mu W/cm^2$) = 390,000 $\mu W\text{-}sec/cm^2$

**Bench scale studies revealed that 87,000 $\mu W\text{-}sec/cm^2$ (15 sec x 5800 $\mu W/cm^2$) of UV light alone resulted in a 95 percent kill of TC bacteria. Since the UV would be used as a polishing step following chlorination, it is felt that a disinfecting power of 100,000 $\mu W\text{-}sec/cm^2$ is sufficient to achieve the desired TC kills and the capacity of this unit, which produces 390,000 $\mu W\text{-}sec/cm^2$, may be increased by a factor of 3.9, (available power/ required power). Therefore, 35 such modular units would be necessary to handle 15 MGD (39.5 cu m/min).

TABLE 54. CHLORINE SYSTEM COSTS - 15 MGD CAPACITY

Capital Costs:	
Chlorine Equipment	$15,000
Holding Tank	7,000
Appurtenances	15,000
Total	$37,000
Operating Costs/Day:	
Chlorine: (8 mg/l) (15 MGD) (8.4) ($0.10/lb) =	$100.80
Electricity =	2.20
Total	$103.00

TABLE 55. COMBINED Cl_2 AND ClO_2 SYSTEM COSTS - 15 MGD CAPACITY

Capital Cost:	$37,000
Chlorine Dioxide & Chemical Generator	5,000
Total	$42,000
Operating Cost/Day	
Chlorine (15 MGD)(8 mg/l)(8.4)($0.10/lb) =	$100.80
Chlorine Dioxide: (15 MGD) (2 mg/l) (8.4) ($.50/lb) =	126.00
Electricity	4.20
Total	$231.00

TABLE 56. SUMMARY OF COSTS FOR DISINFECTION SYSTEMS - 15 MGD CAPACITY

System	Capital Cost	Operating Cost* Per 15 MGD Flow
Chlorine	$37,000	$103.00
Combined Chlorine & Chlorine Dioxide	42,000	231.00
Ultraviolet	900,000 High 510,000 Low	20.20 5.54
Combined Chlorine & Ultraviolet	93,000	106.18

*Operating Costs Based on:

Chlorine 8 mg/1 conc. and @ $0.10/lb
Chlorine Dioxide 2 mg/1 conc. and @ $0.50/lb
UV Power Costs @ $0.02/KWHR

and periodic bacterial counts. These methods may be acceptable for domestic wastes in which the microbial variations are somewhat predictable, but totally unacceptable for wastes such as storm and CSO in which these variations are large and unpredictable. The objection is that simply maintaining a fixed volumetric residual does not guarantee that bacterial and viral populations have been reduced to a safe level. A more direct reflection of microbial activity than residual disinfectants is needed.

The quantitative determination of adenosine triphosphate (ATP), using the bioluminescent reaction characteristic of fireflies, offers a potentially rapid alternative to bacterial measurements as an indicator of microbial content of a water sample (75).

McElroy (76) initially reported that an unreactive preparation of two extracts from firefly lanterns, one presumably containing luciferin, the other containing the enzyme luciferase,could be made to luminesce in the presence of ATP. This finding laid the foundation for the development of an ATP assay which can be performed with relative ease and accuracy, and may be utilized for the enumeration of microbial populations (77) or as a measure of biomass (78).

ATP is present as the driving force of bioenergetic reactions in all living cells (79). It is the primary phosphorylating agent for most biochemical enzymatic reactions and, therefore, the primary energy source in cellular metabolsim. Only under rare conditions is ATP found in nonbiological systems (80, 81). ATP that is released by dying microorganisms is rapidly acted on by other organisms or the surrounding environment and converted to dissimilar phosphate forms. Therefore, an ATP determination can be considered a measure of only the living organisms within a system (82).

This can be seen as a considerable advantage over the assay of proteins, nucleic acids, organic nitrogen, or nitrogen/phosphorus ratios, all of which are relatively unaffected by the inactivation of the organisms.

The chemistry of luciferin-luciferase bioluminescence has been elucidated (83, 84, 85) and divided into four stepwise reactions where:

E = luciferase (enzyme)

LH = luciferin (substrate)

AMP = adenylic acid

E-LH-AMP = enzyme bound luciferyl adenylate

PP= pyrophosphate

L = dehydroluciferin

CoA = coenzyme A

E-L-AMP = dehydroluciferyl adenylate

L-CoA = dehydroluciferyl coenzyme A

$$E + ATP + LH_2 \xrightarrow{Mg^{2+}} E\text{-}LH\text{-}AMP + PP$$

In the above equation the initiating rate-limiting step of the entire reaction, luciferin must first react with ATP before it can be oxidized with light production. The ATP does not act as an electron donor for light emission, but performs an unknown catalytic function, in some way altering the excited state of luciferin (86).

$$E\text{-}LH\text{-}AMP + O_2 \rightarrow E\text{-}L\text{-}AMP + \text{Light} + ?$$

In the presence of oxygen, the enzyme-bound luciferyl adenylate rapidly reacts to produce the excited intermediate which subsequently emits light. One quantum of light is emitted for each luciferin molecule oxidized, while one molecule of oxygen is used in the reaction (87). The identity of the additional products formed is unknown. The E-L-AMP is a relatively stable complex and does not spontaneously disassociate to regenerate luciferase.

The enzyme may be regenerated by either or both of two reactions:

$$E\text{-}L\text{-}AMP + CoA \rightarrow E + AMP + L\text{-}CoA$$

$$E\text{-}L\text{-}AMP + PP \rightarrow L + E + ATP$$

The luciferin-luciferase ATP assay is a useful procedure in a variety of fields: the determination of biomass in limnology (88), oceanography (89), and activated sludge control (90); the direct enumeration of bacterial contaminants (82); in laboratory determination of microbial ATP levels, particularly in growth studies (91) since changes in metabolic activity are frequently characterized by accompanying variations in the ATP concentration in the organism; and in the control of wastewater treatment (92).

The hypothesis tested in this study was that ATP determinations yield information similar to the measurement of traditional bacterial indicators of microbial contamination with respect to disinfection efficiency and control.

Experimental--

All samples were blended for a minimum of 6 sec in a Hamilton Beach Model 50 blender to expose bacteria which may have been harbored within particulate matter and unavailable for subsequent recovery and enumeration (7). Bacterial analyses for TC, FC and FS were conducted using the standard membrane filter technique (71).

The natural virus levels in CSO were sufficiently low that the effects of disinfection could not be easily observed. Therefore, samples were seeded with the desired test virus before disinfection to raise initial levels. Viral titers were determined by the plaque assay technique after concentration on a 0.45μ pore size membrane filter (93).

The Du Pont Model 760 luminescence biometer was used for the detection and quantitation of the light reaction (94). All reagents and supplies were obtained from DuPont with the exception of morpholino-propane sulfonic acid (MOPS buffer)(Aldrich Chemical Co.).

The biometer was calibrated using ATP standards prepared following detailed instructions in the instrument operations manual (94). ATP was extracted from wastewater by filtering 5 ml of sample through a 0.45m pore diameter membrane filter (Millipore Corp.), discarding the filtrate. One ml of a mixture of 1-butanol (80 percent) (Fisher Scientific) and MOPS buffer (20 percent, 0.01 M) was added to the filter to lyse the cells. This was followed by two 2.5 ml rinses of MOPS buffer to elute the released ATP. After 20 samples had been extracted by this procedure and stored on ice, the ATP was assayed. The assay technique involves the injection of 10 μl of the ATP, either as a standard or unknown, into a light-protected cuvette containing $MgSO_4$ (0.01 M) buffered luciferin-luciferase mixture (100 μl). The magnitude of the resulting light reaction is measured by a photomultiplier and indicated as a digital readout. The instrument was calibrated before and after each set of assays to assess stability.

Mean values of ATP per cell for a variety of bacterial species and other microorganisms have been determined in prior studies (Table 57), and an average value of 5.02 X 10^{10} μg ATP/bacterial cell may be assumed based on this data (82, 94, 95).

Correlations between bacteria, viruses, and ATP were determined to investigate the feasibility of using ATP as a substitute for bacterial enumeration and the measurement of residual disinfectant concentrations.

TABLE 57. ATP CONTENT OF BACTERIA

Organism	μg ATP X 10^{10}/cell	Ref
Aerobacter aerogenes	2.4	94
Aerobacter aerogenes	0.28	95
Bacillus cereus	6.4	94
Bacillus cereus	1.1	95
Bacillus coagulans	1.7	95
Bacillus subtilis	24.	82
Bacillus subtilis	9.9	94
Clostridium sporogenes	2.1	82
Corynebacterium striatum	49.	82
Erwina carotovora	0.44	95
Escherichia coli	5.8	82
Escherichia coli	4.1	94
Escherichia coli	1.0	95
Flavobacterium arborescens	1.5	95
Klebsiella pneumoniae	5.0	95
Micrococcus lysodeikticus	1.3	95
Mycobacterium phlei	1.9	95
Proteus vulgaris	3.0	82
Proteus vulgaris	1.8	95
Pseudomonas aeruginosa	1.0	95
Pseudomonas fluorescens	3.9	82
Pseudomonas fluorescens	3.1	95
Sarcina lutea	2.2	82
Sarcina lutea	0.37	95
Serratia marcescens	1.0	95
Staphylococcus albus	3.1	94
Staphylococcus aureus	0.64	95
Staphylococcus epidermidis	2.2	82
Streptococcus faecalis	4.9	94
Streptococcus salivarius	5.7	82

Samples from the bench-scale studies were collected after the appropriate contact time, and the residual disinfectant was immediately neutralized with sodium thiosulfate (71). Samples from the full-scale facilities were collected automatically by sequential, refrigerated samplers (Sigmamotor, Inc., Middleport, N.Y.). Sample bottles were thoroughly cleaned, and an excess of sodium thiosulfate was added before use.

Results and Discussion--

For ATP to serve as a measure of disinfection, the disinfecting agents, in this case Cl_2 and ClO_2, must not interfere with the bioluminescent reaction. To study this possibility, Cl_2 and ClO_2 were added to a standard ATP solution to give disinfectant concentrations from 0.0 to 100.0 mg/l; a range designed to include bactericidal doses at rapid contact times. The ATP content of each solution was measured after 60 and 300 sec contact time followed by the application of sodium thiosulfate to eliminate residual disinfectant.

The results, presented in Tables 58 and 59 indicate a definite interaction between the reaction mixture and disinfectants, particularly Cl_2. At the doses of ClO_2 used for normal or high-rate disinfection, 25 mg/l or less, there would appear to be only a slight interference for 60 sec exposure (Table 59). The corresponding interference caused by Cl_2 is considerably greater. In the analysis of wastewater samples, this effect would be less than shown because the ATP becomes exposed only to the residual Cl_2 and ClO_2 remaining after the Cl_2 or ClO_2 demands of the wastewater are met. However, to eliminate up to 100 mg/l of residual disinfectant, sufficient sodium thiosulfate (2500 mg/l) should be added to the MOPS buffer used in all stages of the analysis. This is common practice to neutralize disinfectant reactions prior to bacterial analysis, preventing inaccurate bacterial kills by residual disinfectant. To test the effect of sodium thiosulfate on the reaction mixture, various amounts were added to an ATP standard solution and the ATP concentration assayed after 60 and 300 sec. The results in Table 60 show that the effect of sodium thiosulfate on the ATP assay system can be considered negligible. This procedure was routinely adopted for ATP measurements of disinfected waters.

An attempt was made to correlate measured ATP with the presence of test bacteria. This was performed in two stages; first, on untreated samples from CSO, and second, on disinfected samples. Samples of untreated CSO from different times and locations were analyzed for ATP, TC, FC, and FS. The correlation coefficients (Table 61) were derived from a least-squares fit of over 250 points assuming a linear relationship, and indicated little correlation between ATP and any of the bacterial parameters. The results are not unexpected. The lack of correlation is attributable to the presence of ATP sources in the wastewater other than the three measured bacteria. The results are confirmation of the small contribution the test bacteria make to the total ATP pools of the individual wastewater samples. Future work in this direction should include the correlation between ATP and total bacterial levels.

The second stage involved the measurement of ATP and test bacteria following disinfection. The study was conducted on a bench scale under a variety of conditions. Data were derived from a single-stage disinfection

TABLE 58. EFFECT OF Cl_2 on ATP ASSAY MIXTURE

Cl_2 dose, mg/l	Contact time, Sec: 0		60		300	
	ATP, µg/l	% Recovery	ATP, µg/l	% Recovery	ATP, µg/l	% Recovery
0	17.1	100.0	16.80	98.2	17.00	99.4
5	17.1	100.0	13.60	79.5	10.50	61.4
10	17.1	100.0	9.80	57.3	9.00	52.6
15	17.1	100.0	5.17	30.2	3.18	18.6
25	17.1	100.0	1.04	6.1	0.63	3.7
50	17.1	100.0	0.84	4.9	0.97	5.7
100	17.1	100.0	0.25	1.5	0.15	0.9

TABLE 59. EFFECT OF ClO_2 on ATP ASSAY MIXTURE

Cl_2 dose, mg/l	Contact time, Sec: 0		60		300	
	ATP, µg/l	% Recovery	ATP, µg/l	% Recovery	ATP, µg/l	% Recovery
0	10.10	100.0	9.63	95.3	10.60	105.0
5	10.10	100.0	8.87	87.8	8.86	87.7
10	10.10	100.0	9.95	98.5	9.25	91.6
15	10.10	100.0	9.50	94.1	5.74	56.8
25	10.10	100.0	9.40	93.1	4.82	47.7
50	10.10	100.0	7.11	70.4	2.36	23.4
100	10.10	100.0	6.53	64.7	2.54	25.1

TABLE 60. EFFECT OF SODIUM THIOSULFATE ON ATP ASSAY MIXTURE

Sodium thiosulfate dose, mg/l	Contact time, Sec: 0		60		300	
	ATP, µg/l	% Recovery	ATP, µg/l	% Recovery	ATP, µg/l	% Recovery
0	15.4	100.0	14.8	96.1	12.2	79.2
500	14.1	100.0	13.9	98.6	11.6	82.3
1000	16.2	100.0	14.6	90.1	12.0	74.1
2500	15.6	100.0	14.4	92.3	12.4	79.5
5000	16.0	100.0	14.9	93.1	12.1	75.6

TABLE 61. LINEAR REGRESSION CORELATION COEFFICIENTS OF ATP vs TEST BACTERIA

Range of ATP, µg/l	Total Coliforms		Fecal Coliforms		Fecal Streptococci	
	Raw	Disinfected	Raw	Disinfected	Raw	Disinfected
0.00-0.10	0.468	0.387	0.429	0.346	0.364	...
0.00-1.50	0.237	0.842	0.030	0.794	0.237	...
0.00-5.00	0.033	0.700	0.035	0.759	0.034	...
1.51-5.00	0.005	0.206	0.057	0.515	0.173	...

procedure (disinfection utilizing a single-stage disinfectant, in this case either Cl_2 or ClO_2) or by a two-stage procedure (the sequential addition of a single disinfectant, or two disinfectants). Discrete data is presented in Appendix G .

Table 61 describes the resultant correlation after linear regression analysis between ATP and two bacterial indicators, TC and FC , upon disinfection. The higher correlations found upon disinfection (than in untreated samples) indicate that the fraction of the total wastewater microorganisms represented by the coliforms may increase with disinfection. The absence of this increase in the ATP range of 0.00 to 0.10 μg/l is due to the small number of data pairs in that sampling range, and the results are probably not significant at that level. The coliforms show greater resistance to disinfection that that fraction of the total microorganisms present which contributes the greater portion of the total ATP measured. Thus, an ATP level selected as equivalent to the ATP contained in the highest limit allowable of indicator bacteria could more closely represent the presence of such indicators, or more resistant pathogens, than initially believed. While they are necessarily vague in an absolute sense due to the highly variable range of ATP from diverse sources in wastewater, the correlations do indicate that the contribution of ATP by microbial sources of nonsanitary significance decreases measurably upon disinfection.

The results from the operation of the prototype full-scale facilities are given in Table 62. The CSO was diverted through a 71μ aperture screening unit prior to application of 7.0 mg/l ClO_2 for 1 min contact time at a flow rate of 1 MGD.

When the FC count was below the limit of 200 colonies per 100 ml, the ATP level was lower than 0.1 μg/l.

The data described to this point must be evaluated carefully. The primary concern is that ATP can be used to differentiate between the high bacterial counts encountered in untreated overflows and the lower levels required by effluent limitations. Since the coliform group, the present indicator of sanitary quality, represents only a small fraction of the microorganisms found in untreated CSO, poor correlations with ATP were expected and found. However, if the ATP concentration of the treated overflow is maintained at a level equivalent to the ATP concentration found at the maximum permissible indicator bacteria levels, any decisions regarding the water quality would be at least as valid as those based on the present indicator, the coliforms.

To verify the inactivation of pathogenic viruses using the disinfection procedures, poliovirus Sabin Type 1 was treated with ClO_2. The results of the study (Figure 81) show that the levels of ClO_2 used and the time limits imposed by the high-rate disinfection procedure still allow a substantial inactivation of viral pathogens. Thus, reduction of ATP values observed upon disinfection at test dosages indicates not only a reduction in bacterial levels, but in the levels of pathogenic viruses. Again, this reduction should not be construed as a 1:1 relationship between disinfectant dosage and reduction of ATP levels, nor of ATP concentration and any microbial specie

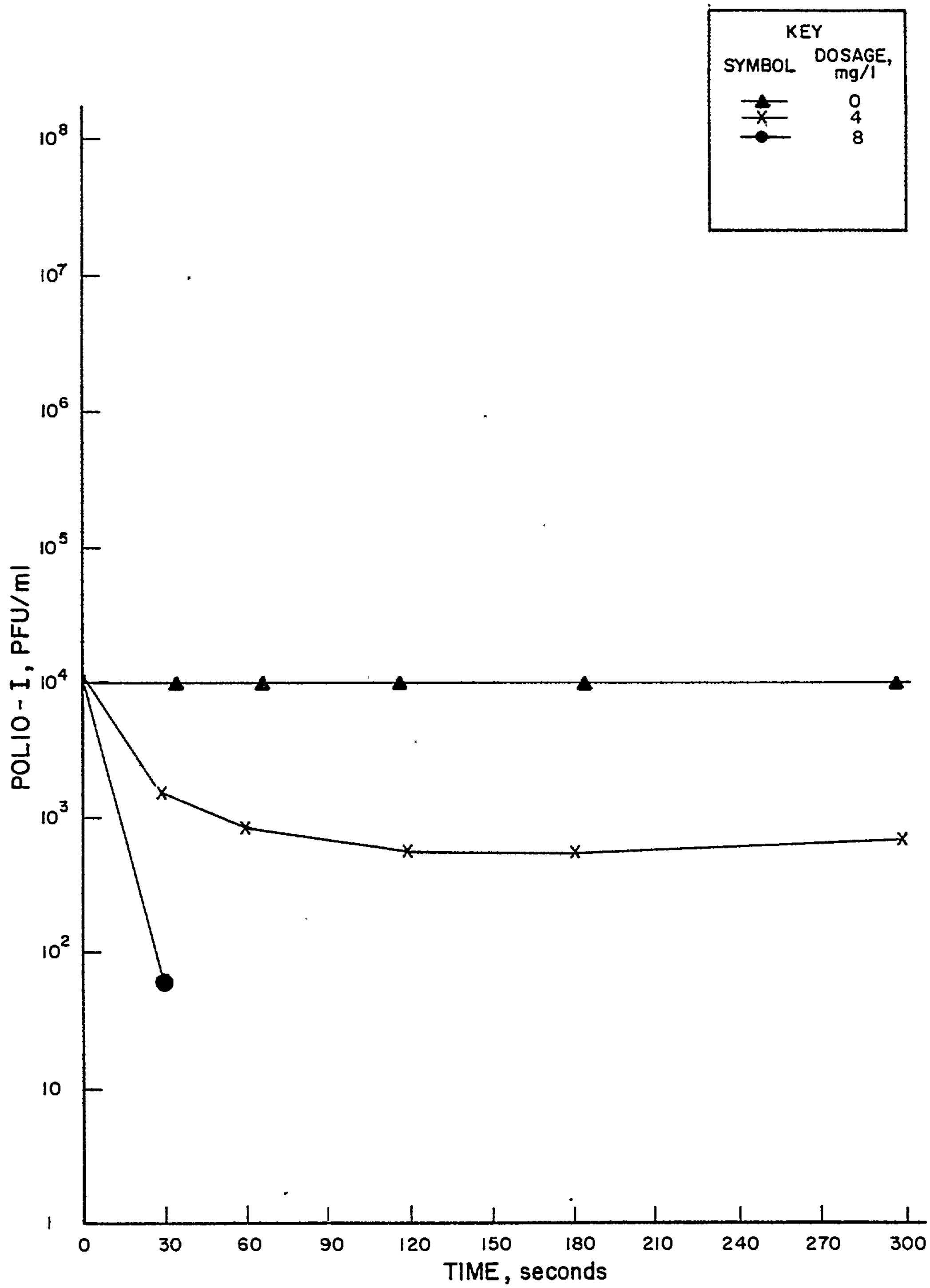


FIGURE 81. Single-Stage Disinfection of Poliovirus Sabin Type I with ClO_2

upon disinfection. The data observed do describe the utility of the assay in measuring the large differences between influent and effluent populations.

Before the development and testing of an in-line ATP monitor can be considered realistically, the stability of reagents, adequate sampling technique, availability of the necessary electronics, and the automation of the chemistry must be investigated. The feasibility of such a system has already been mentioned (92, 96, 97, 98). A continuous flow monitor is envisioned with periodic switching from overflow samples to ATP for standardization. The control signal for addition of disinfectants could be generated from an ATP monitor located at a point following disinfection. This arrangement would require that the ATP concentration be kept below some predetermined safe level. Another possibility would be to measure ATP before and after disinfection to detect sudden increases in the degree of contamination. The electronics necessary to continuously record the light intensity from the bioluminescent reaction have already been developed (99). The procedures for a continuous, automated extraction are already in use in several of the Technicon Methodologies (100). The expense of the reagents is likely to be quite significant because of the high cost of the luciferin-luciferase reaction mixture derived from the firefly lanterns and should most definitely be studied in greater detail before attempting to develop a monitor.

Summary of Results--

Concurrent determinations of ATP, utilizing the luciferin-luciferase bioluminescent assay system, and indicator bacteria from CSO have indicated the feasibility of using ATP as a reliable and rapid indicator to control the disinfection process.

TABLE 62. DISINFECTION OF COMBINED SEWER OVERFLOWS IN FULL-SCALE FACILITY

	ATP, μg/l		FC Counts/100 ml	
Time	Influent	Effluent	Influent	Effluent
1730	2.50	0.260	315 000	615
1750	1.25	0.300	460 000	1095
1810	1.44	0.066	190 000	20
1850	1.32	0.101	84 667	10
1930	1.79	0.036	83 500	10
1950	1.44	0.051	62 000	20

In the dosages required for high-rate disinfection, Cl_2, ClO_2, and sodium thiosulfate did not significantly interfere with the ATP assay.

The development of an instrument to monitor ATP in unattended operation is a realistic venture primarily contingent upon the stability and cost of the enzyme reagent.

SPECIAL INSTRUMENTATION

Telemetering

The flow data that was generated at each of the demonstration sites was telemetered to a central location, identified, and stored on punch tape. The tapes were then processed and the information permanently stored, along with coinciding analytical and rainfall data, on computer disc files. The purpose of the telemetry and computer system was to minimize manhours in data handling and to establish a basis for a central control station for the overall CSO abatement program.

TOC Monitor

General--

During the demonstration phase of the project, an investigation was made at Maltbie Street of a previously developed total organic carbon (TOC) analyzer (101) to monitor CSO *in situ* and on a continuous, rapid-response basis. The device was developed by Raytheon for evaluation under EPA sponsorship. Testing of the unit was conducted for a two month period.

The basic method of operation of the TOC monitor required that a constant volume of the overflow be pumped to a homogenizer prior to entry into the TOC unit. This homogenizer was included to provide complete maceration of solids in the sample stream and thus minimize the possibility of plugging problems in the TOC unit. A finite volume of the sample stream was delivered by a small pump in the TOC unit to a 1812^oF (950^oC) oven which provided complete combustion of the sample to form CO_2 from all carbon compounds in the sample stream. The remainder of the sample stream from the homogenizer was discharged to a drain. The values of TOC as measured by the unit were recorded on a panel-mounted continuous recorder for visual observation during a storm. A back-up recorder was also installed inside the rear of the cabinet for use in case of failure in the panel-mounted recorder. The TOC unit was calibrated by a standard solution of common sugar.

The TOC monitor was equipped with an automatic zero and span exercise, which was activated automatically when an overflow occurred. A supply of CO_2-free oxygen was delivered to the analyzer to establish a baseline zero. Next, a gas containing a known concentration of CO_2 was delivered to the analyzer and the recorder scaled to this known concentration of CO_2. The zero gas used was obtained from the oxygen supply while the span gas was purchased to specification.

Field Program--

During the initial startup period, 5 min was required to automatically zero and span the TOC instrument. After that time period, data was collected continuously on CSO samples delivered to the homogenizer.

Calibration procedures were carried out about every third day to assure proper operation. Generally it was observed that the unit would not hold to zero calibration level for an extended period of inactivity (greater than two days). This problem made it difficult to determine actual TOC values when an overflow event was monitored.

Figure 82 illustrates as three examples the failure of the TOC unit to return to the zero level after exercising. Had a storm occurred when the calibration was off, erroneous TOC readings would have resulted. An effort to determine the percent error that would have occurred was attempted. The TOC value above the zero point in each case (A) was subtracted from the actual recording to determine the TOC level measured. The latter values were compared to the TOC value that should have been registered on the chart recorder. For example, for data of June 20, 1975, a sucrose standard of 276 mg/l was injected into the TOC unit. A resulting TOC value of 230 mg/l was recorded on the chart. Once the baseline TOC value of 25 mg/l was subtracted from the recorded value of 230 mg/l the resulting value of 205 mg/l indicates the TOC measured in the unit. The difference between the standard of 276 mg/l and the 205 mg/l measured was compared to the standard (276 mg/l) to yield a percent of error for the three illustrations presented here. The variability of percent error indicated in Table 63 from 9.4 to 25.7 percent indicates the unreliability of absolute values being measured.

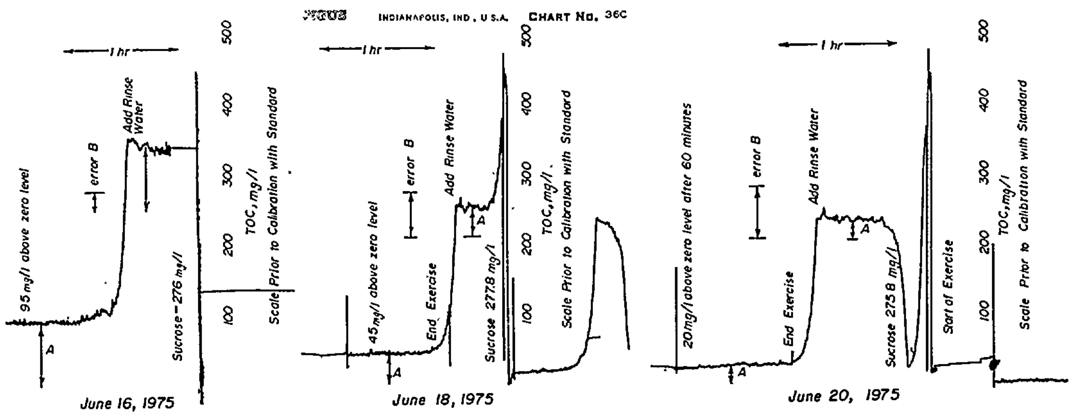


FIGURE 82. TOC Monitor Data Obtained During Calibration Procedures

Results--

As a result of several mechanical failures to the TOC monitor during the two-month investigation (including replacement of the 1812°F (950°C) furnace, replacement of tubing, switches and hose lines, failure of the recorder clock, and required replacement of a solenoid valve) only data collected from one of the five storm events occurring during this period was considered to be a reliable example of the TOC monitor's capability. A plot of the TOC as monitored vs results from laboratory analysis of the actual overflow is shown in Figure 83. As can be seen from the plot, the Raytheon unit only roughly followed the trend of TOC changes in the overflow. Through the method of linear regression, the degree of association between the random points as expressed by the correlation coefficient indicated no reliable association existed (r = 0.10).

Data observed during one storm event indicates that the technology exists for developing a reliable TOC monitor, although the Raytheon unit needs further refinement to provide a unit more reliable in terms of operational response and calibration tracking.

TABLE 63. DEVIATION OF MEASURED TOC FROM KNOWN TOC STANDARD

Date	(1) TOC Reading on Full-Scale Chart	(2) Deviation From Zero Scale	(3) Actual TOC Measured (1)-(2)	(4) TOC OF Standard	Percent Error (4)-(3)/(3)
June 16	345	95	250	276	9.4
June 18	255	45	210	278	24.4
June 20	230	20	205	276	25.7

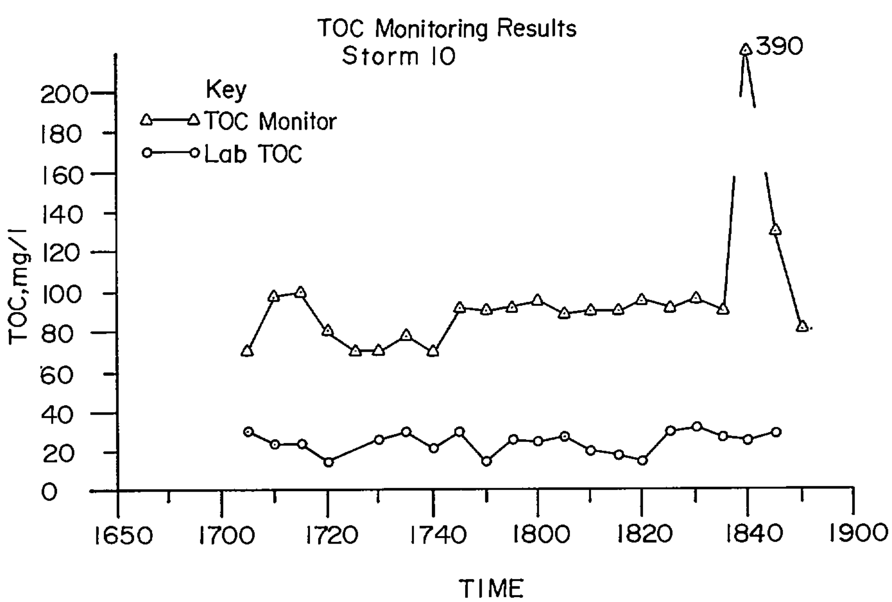


FIGURE 83. Comparison of TOC Monitor Results with Laboratory Results

Suspended Solids (SS) Meter

Introduction--

The increasing emphasis on water quality has brought with it many demands for improved information not the least of which has been for a more accurate method of on-line suspended solids monitoring. Present state-of-the-art units can be broadly classified as those which use gravimetric methods and those which use transmission, refraction or reflection of light. In almost all non-gravimetric designs, the light source is non-polarized white light, and the detector senses the intensity of received light, some ratio of received to transmitted light or the ratio of intensities at two or more receivers.

All of these devices are sensitive in varying degrees to many interfering parameters. Those parameters which are most influential are the absorption coefficient, scattering coefficient, refractive index, particle size, particle shape, particle density and dissolved solids concentration. Since each of the devices is sensitive to different parameters in differing degrees, they tend to have calibration curves for a given suspended solid which differ (often quite significantly) from device to device. Even more alarming is that the calibration curve for a given device usually varies significantly from one type of suspended solid to another. Thus, unless a particular unit is calibrated specifically for a particular suspended solid, it may have very limited use as a monitoring device. Most, if not all existing monitors exhibit little promise as highly accurate monitors of suspensions where the parameters mentioned earlier are apt to change significantly.

Background--

The initial feasibility study of the technique of using the depolarization of backscattered polarized light to determine suspended solids levels was undertaken by American Standard's Research and Development Center. The empirical method was verified by Liskowitz (103) using standard laboratory equipment such as beakers, cuvettes, etc. Encouraged by the initial results, and under a separate contract (104) a more elaborate test device was fabricated. The sensor element alone measured roughly 12 in. on each side. This device included a quartz window, tungsten lamp with focusing apparatus and an optical beam-splitter receiver. The device received only limited testing during the initial program and was not suited to long term unattended service in the sewer environment. The results of these early tests were reported by Liskowitz and Franey (105).

When American Standard discontinued its R&D program on the solids monitor, Badger Meter secured all patents, design, construction and test data from American Standard and obtained the feasibility model from the EPA. Subsequently, Badger has redesigned the instrument to simplify its optics and electronics, and to suit it to unattended sewer installations.

Application--

As part of this demonstration study, Badger meter has constructed a revised prototype model incorporating the simplified design which can be mounted in either open channels or in pressure lines as small as

12 in. diameter. Badger obtained laboratory verification of the prototype operation, provided the prototype, calibrated the unit in kaolin-equivalent mg/l units and provided the prototype to O'Brien & Gere for field data collection.

Theoretical Basis--

Each particle in solution is a scattering site, slightly depolarizing the incident polarized light and acting as a new source of partially polarized light. The existence of many such particles results in multiple scattering which more completely depolarizes the emitted light.

The emitted light is assumed to be 100 percent plane polarized and is referred to as the polarized (p) component.

At any point in the medium the light exhibits two components: That which is still polarized - the remaining 'p' component (with a reference of 0 degrees) and that which is depolarized - the 'd' component (uniformly distributed from 0-360 degrees).

Clearly, by using the backscatter radiation, not only is the density of particles a factor, but so is the mean effective backscatter path length. As the density increases, the effective path length decreases (except for a beam of 0 degree solid angle and receptor with a 0 degree solid angle of acceptance).

The backscattered radiation perceived by the receptor has the two components previously described - the 'p' and 'd' components. If the emitter could be co-located with the receptor window so as to have 0^{o} included angle between the emitted beam axis and the receptor's acceptance axis, then most backscattered radiation would be singly scattered at higher densities resulting in little depolarization. If there is a small included angle with axes intercepting at some distance into the medium, then reception of multiple backscatter is enhanced at the higher densities.

The radiation perceived by the receptor system is split into two beams separated by 90^{o} (Refer to Figure 84). It is assumed that the split is 50-50 in intensity and chromatically balanced. One beam passes through a polarizer whose polarization axis is parallel to that of the emitter, and the other beam passes through a polarizer whose polarization axis is perpendicular to that of the emitter. It is assumed that these polarizers are capable of complete polarization.

Light polarized at an angle other than that passed by the polarizer can be viewed as comprised of two components: One parallel (0^{o}) the other perpendicular (90^{o}) using the trigonometric relationships for a right triangle. As such, uniformly depolarized radiation would be passed at half its intensity by any polarizer plate (assuming ideal transmission characteristics). Thus if the received light is uniformly depolarized, the parallel polarizer and perpendicular polarizer will each pass equal intensities, since the polarized intensity I_p would be zero. As such, the equations are:

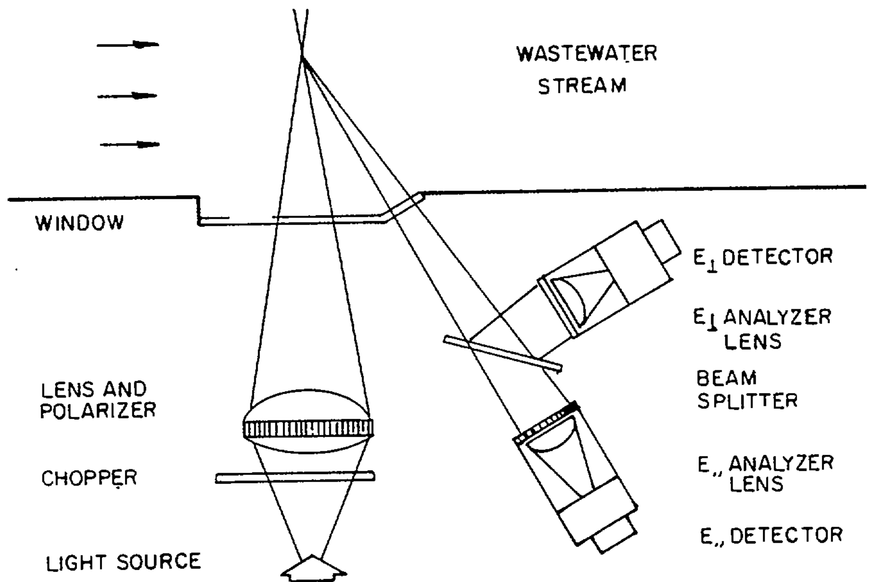


FIGURE 84. Schematic of Suspended Solids Monitor

$$I_\perp = \tfrac{1}{2}I_d$$

$$I_\parallel = I_p + \tfrac{1}{2}I_d$$

where $I_\perp$ = perpendicular polarized intensity

$I_\parallel$ = parallel polarized intensity

I_p = polarized intensity

I_d = depolarized intensity

The sensor notations generally include the subscripts '$_\perp$' or '$_\parallel$' as appropriate, and it must be understood that these are not synonomous with '$_p$' and '$_d$'.

If we assume an output voltage proportional to the received intensity at each sensor, the following equations are appropriate.

$$E_\perp = k_\perp\ (\tfrac{1}{2}I_d)$$

$$E_\parallel = k_\shortmid\ (I_p + \tfrac{1}{2}I_d)$$

where $E_\perp$ = perpendicular component

$E_\parallel$ = parallel component

$K_\perp$ and $K_\parallel$ = proportionately constants

In the American-Standard unit, $k_\perp$ and $k_\parallel$ were made equal, and the ratio of $E_\perp$ to $E_\parallel$ obtained as the depolarization ratio $D_\shortmid$.

$$D_1 = \frac{\frac{1}{2}I_d}{I_p + \frac{1}{2}I_d}$$

Note that at low densities ($I_d = 0$), $D_1 = 0$ whereas at high densities ($I_p = 0$), $D_1 = 1$.

Richardson (106) proposed using a slightly different ratio, $2E_\perp$ to $(E_\perp + E_\parallel)$. Note that if $k_\perp = k_\parallel$, then this ratio, call it D_2 is:

$$D_2 = \frac{I_d}{I_p + I_d}$$

The ratio D_2 also ranges from 0 to 1 but its denominator always relates to the total received light.

A third ratio, obtained by forming the ratio of $E_\perp$ to $(E_\parallel - E_\perp)$ assuming $k_\perp = k_\parallel$ yields R as

$$R = \frac{I_d}{I_p}$$

It is felt that R is the true depolarization ratio. Furthermore, it was found that the American Standard data, presented as D_1 versus P (concentration), when converted to R versus P was linear over most of its range.

Ambient Light Effects--

The previous discussion assumed that the only light present was initially emitted by the polarized source. Should there be any ambient light its effect must be considered. Furthermore, it is not possible to assume that the ambient light will be uniformly depolarized, since light incident to the air-water interface may be slightly polarized to favor the horizontal component under water. With low densities, this effect may not be completely erased by multiple scatter before being sensed. Consequently the intensity equations become:

$$I_{\perp a} = \tfrac{1}{2}I_{da} \qquad I_{\perp a+s} = \tfrac{1}{2}I_{da} + \tfrac{1}{2}I_{ds}$$

$$I_{\parallel a} = I_{pa} = \tfrac{1}{2}I_{da} \qquad I_{\parallel a+s} = I_{pa} + I_{ps} + \tfrac{1}{2}I_{ds} + \tfrac{1}{2}I_{ds}$$

where a = ambient light

s = polarized light source

Similarly, the equations for $E_\perp$ and $E_\parallel$ must be modified.

Analysis by Poisson Statistics--

Assuming that the depolarization of polarized light due to multiple scattering can be approximately described by Poisson statistics, the probability that an incident beam of polarized light would be received fully polarized might be expressed as:

$$p_{\parallel} = \exp(-\alpha Px),$$

where P is the concentration of scattering material, x is the distance and α is a constant. Similarly, the probability of receiving the light depolarized would be,

$$p_{\perp} = 1 - \exp(-\alpha Px).$$

The ratios involving $E_{\parallel}$ and E_T which have been proposed were examined, keeping in mind that $E_{\perp} = \frac{1}{2}I_d$ and $E_{\parallel} = I_p + \frac{1}{2}I_d$ and that the total received light is given by $I_t = I_p + I_d$.

$$R_1 = E_{\perp}/E_{\parallel}$$

This is the ratio used by American Standard which, in terms of the received intensities, can be expressed as

$$R_1 = \frac{1}{2}I_d(_I p + \frac{1}{2}I_d)$$

and in terms of the Poisson functions as

$$R_1 = \{1-\exp(-\alpha Px)\}/\{1+\exp(-\alpha Px)\}$$

or $$R_1 = \{\exp(\alpha Px)-1\}/\{\exp(\alpha Px)+1\}.$$

This ratio varies from 0 to 1 as P varies from 0 to infinity. Approximating $\exp(\alpha Px)$ by the first two terms in its series expansion, i.e., $1 + \alpha Px$ (an approximation which is valid if and only if the exponential term (αPx) is sufficiently small that the strong inequality $1 \gg (\alpha Px)^{\eta-1}$, is satisfied for all integer $(\eta \geq 2)$ yields

$$R_1 = \alpha PxX/(2 + \alpha Px).$$

In similar fashion, several additional ratios have been investigated. The results are summarized in Table 64. Note that the ratio R_4 is the most linear and has uniform sensitivity to the particle concentration. This ratio has a singularity $(E_{\parallel} - E_{\perp})$ and thus for high concentrations its accuracy depends on an accurate balance of the parallel and perpendicular detectors and their associated electronics. Should the approximations of the exponential require additional terms, they can be included more easily in the form R_4 than in any of the others. Tests of the existing unit would indicate that the assumption's validity is supported.

Prototype Development--

Emitter and Sensor-- The large size of the American Standard feasibility transducer (1.0 ft^3) (0.028 cu m) made it quite difficult to use. Since a large portion of the inside area was void, considerable weight was required to submerge it. The size and submergence problem dictated a complete

TABLE 64. RATIOS INVOLVING THE PERPENDICULAR AND PARALLEL COMPONENTS OF DEPOLARIZED LIGHT

	Expressed In Terms of			
Ratio	$E_{\parallel}$ & $E_{\perp}$	I_p & I_d	Poisson Statistics	Linearized Poisson
R_1	$E_{\perp}/E_{\parallel}$	$\frac{1}{2}I_d/(I_p+\frac{1}{2}I_d)$	$\{\exp(\alpha Px)-1\}/\{\exp(\alpha Px)+1\}$	$\alpha Px/(2+\alpha Px)$
R_2	$2E_{\perp}/(E_{\parallel}+E_{\perp})$	$I_d/(I_p+I_d)$	$\{\exp(\alpha Px)-1\}/\exp(\alpha Px)$	$\alpha Px/(1+\alpha Px)$
R_3	$(E_{\parallel}-E_{\perp})/(E_{\parallel}+E_{\perp})$	$I_p/(I_p+I_d)$	$1/\exp(\alpha Px)$	$1/(1+\alpha Px)$
R_4	$2E_{\perp}/(E_{\parallel}-E_{\perp})$	I_d/I_p	$\exp(\alpha Px)-1$	αPx

FEATURES

Ratio	Range $P=0$	Range $P=\infty$	Linearity	Sensitivity $S=\partial R/\partial P$	$P=0$	$P=\infty$	Remarks
R_1	0 →	1	linear only when $\alpha Px \ll 2$	$2\alpha x/(2+\alpha Px)^2$	$\alpha x/2$	0	Poor linearity, not very sensitive at high P.
R_2	0 →	1	linear only when $\alpha Px \ll 1$	$\alpha x/(1+\alpha Px)^2$	αx	0	More linear & sensitive than R_1 the true depolarization ratio.
R_3	1 →	0	non linear	$-\alpha x/(1+\alpha Px)^2$	$-\alpha x$	0	Really a 'polarization' ratio not useful here
R_4	0 →	∞	linear	αx	αx	αx	Linear, uniform sensitivity over the entire range. Requires careful balance of $E_{\parallel}$ and $E_{\perp}$ to achieve.

redesign of the entire transducer portion of the monitor. The design goal was for a transducer element which would be approximately 1 in. in diameter and 6 in. in length. The completed transducer unit contained both the emitter and sensor elements as well as electronics required immediately adjacent to the emitter-sensor elements. Such an arrangement allowed remote location of the major electronics package.

The feasibility unit emitter used a single tungsten bulb which emitted a broad spectrum white light. This light was columnated and polarized. Extensive tests with relatively inexpensive polarizing materials indicated that most polarizing plates do not uniformly polarize broad spectrum light. Such a non-uniform polarization would cause the unit's sensitivity to be non-uniform for different particle sizes. Ideally the unit should respond uniformly regardless of particle size. For this reason the spectral content of the transducer emitter was restricted.

The search for an emitter source of sufficiently narrow spectral content indicated that those which had adequate intensity generally were in the infra-red or near infra-red spectrum. This presented an additional problem in that not all polarizing materials worked well in the near infra-red or infra-red region. However a new material being investigated by the Polaroid Corporation proved adequate for near infra-red emitters. The emitter finally selected developed twelve milliwatts radiated power at 9400 Å $\pm$ 300 Å with only a 0.33 watt drive. The polarizers used were highly effective at this wave length and had the added benefit that they were nearly opaque to visible light, reducing ambient light effects significantly. Since it has been shown that depolarization in backscatter radiation tends to fall off dramatically when the particle size is less than 1/2 wave length in diameter, the use of 9400 Å light would dictate that the unit would be sensitive to all particle sizes greater than 0.5μ.

The development of the new sensor paralleled that of the new emitter system. Initial efforts tried to duplicate in miniature the beam splitter arrangement used in the American Standard feasibility unit. However, it was found that the beam splitters which were available at an acceptable price did not perform satisfactorily in the infra-red range, and added a great deal in the terms of complexity and loss of intensity. Additionally, it had to physically be displaced from the window, permitting light reflection internally which added to depolarization, causing false indications. For this reason two separate sensors were used, each mounted directly behind its polarizing plate as closely together as possible, and each located the same distance from the emitter by placing the emitter and two sensors at the points of an equilateral triangle.

The objective of small size for the transducer housing required that the emitter be placed in close proximity to the sensors. Due to the high intensity of the emitter it was found that ordinary plastics were not sufficiently opaque to totally prevent transmission from the emitter region to the sensor. The typical mechanism for such transmission was that as the emitter light would pass through its polarizing plate, internal reflections would cause sideways transmission through the plate into the receptor or sensor polarizers where the very sensitive sensors would see the light as returned energy. To avoid this problem the sensor emitter housing was fabricated from stainless steel with inset polarizing plates.

Electronics-- Due to the extreme sensitivity of the sensor units, it was necessary to mount the operational amplifiers and buffer amplifiers in a sensor emitter package as close as possible to the sensors. In the final prototype this entire electronics package was mounted on small circuit board approximately 0.75 in. by 3 in. and imbedded in the same plotting material used to surround the sensors and emitter.

In order to minimize or eliminate the ambient light effects, the emitter was alternately turned on and off at a rate of several hundred times per second. The received information from the sensors was then coherently detected and filtered, eliminating all effects of constant ambient light as well as pulsating effects due to fluorescent lighting fixtures. The final electronics system was found to be totally immune to sunlight as well as fluorescent and incandescent light. Figure 85 presents a photograph of the final electronic package and probe of the SS Meter.

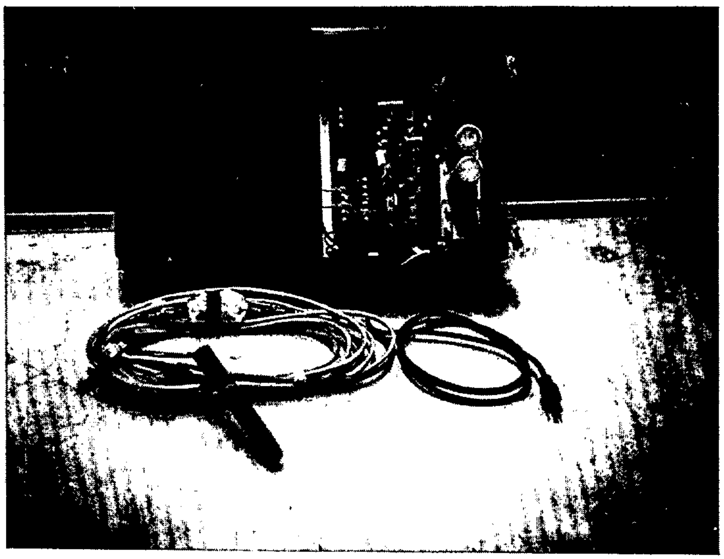


FIGURE 85. Electronics Package for Suspended Solids Meter

Prototype Testing--

The prototype testing program was accomplished in a series of extensive tests done in the laboratory followed by some limited testing in the field. The field testing consisted of two phases; tests performed at the Tulsa, Oklahoma, Mohawk Park Sewage Treatment Plant and tests performed at selected locations at the Syracuse Demonstration facility at Maltbie Street.

Laboratory Tests -- For the most part laboratory tests were performed using ungraded white kaolin clay. This clay was used in all color tests using both ordinary food coloring dyes as well as nigrosine black dye.

Some limited tests were performed using graded white kaolin clay in size ranges 5,10,15 and 30μ. The ungraded kaolin clay was generally sized greater than or equal to one micron. Additionally some limited tests were run on pumice, rottonstone, fine sand and biological material which was grown in the laboratory to assure purely biological nature.

The test apparatus consisted of a 5.3 gal (20 l) cylindrical container which was kept in constant agitation by means of an axially driven bottom disturbing vane. The solids meter probe was suspended in the circulating fluid approximately 1.0 in. (2.5 cm) from the wall at a depth of approximately 4.0 in. (10.2 cm) so that it was aimed directly across the cylindrical container.

The results of a typical test on ungraded kaolin clay are shown on Figure 86. Note how the depolarization ratio, R, of the low range of clay concentration from 10 to 1000 mg/l tends to increase at an increasing rate (roll up) while it increases at a decreasing rate (roll off) in the range from 10,000 to 100,000 mg/l. The latter occurrence is a result of a slight mismatch in the gains of the two channels (P for polarized component and D for the depolarized component). Figure 87 shows the same data after a correction has been made to the raw data to represent balanced gains. Notice how the gain adjustment has straightened the high end of the data but has little effect on the results at the low end. The low end effect is believed to be the result of a depolarization offset. Fortunately, in all the tests performed, this low end phenomenon appeared to be predictable enough so that it could be compensated for electronically. In the existing prototype unit compensation was not provided, however.

The high end roll off problem stemming from the imbalanced gains on the depolarized and polarized channels is a difficult one to resolve. To date the only method which has been successful in adjusting that gain has been to immerse the probe in a media which is at least ten times more dense than the densest media to be observed and adjusting the gains for as close to proper output as can be achieved.

In Figures 86 through 88 the solid straight line represents the linear response. As already indicated, Figure 87 shows the corrected response when unbalanced depolarization-polarization gains are accounted for. The equation

$$R_m = (1 + \varepsilon) \quad R_o + \varepsilon$$

where R_m is the measured ratio and
ε is an empirical correction factor for probe rescatter
R_o is the ideal linear relationship,

represents the effect of reradiation from the probe face. Empirically choosing a value for ε and correcting Figure 87, Figure 88 is obtained. The low end roll up has now been straightened with little if any effect on high end data, resulting in a nearly linear response for the unit from 10 to 10,000 mg/l of ungraded kaolin clay solution.

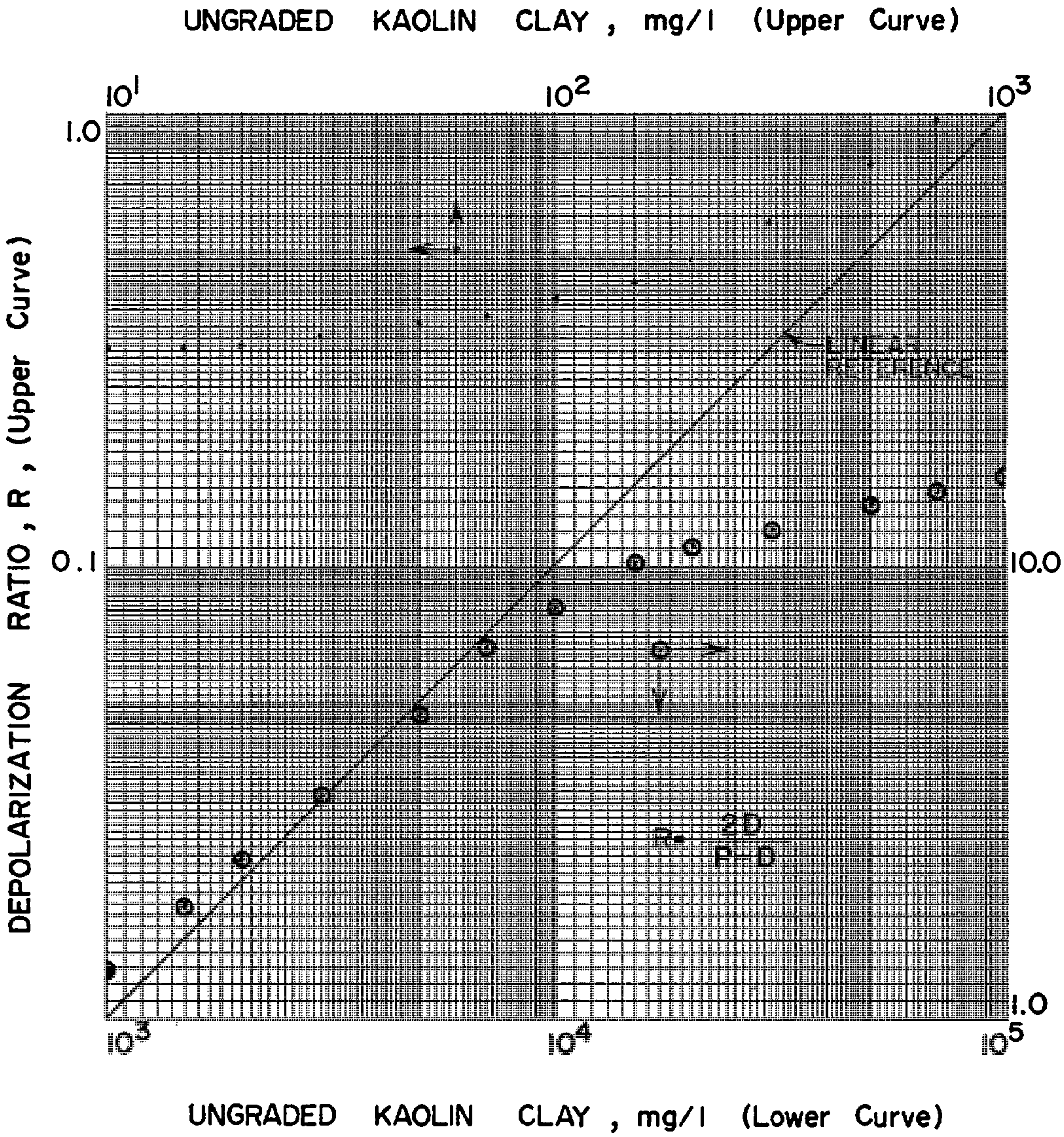


FIGURE 86. Depolarization Ratio vs Ungraded Kaolin Clay Concentration - No Depolarization-Polarization Gain Adjustment

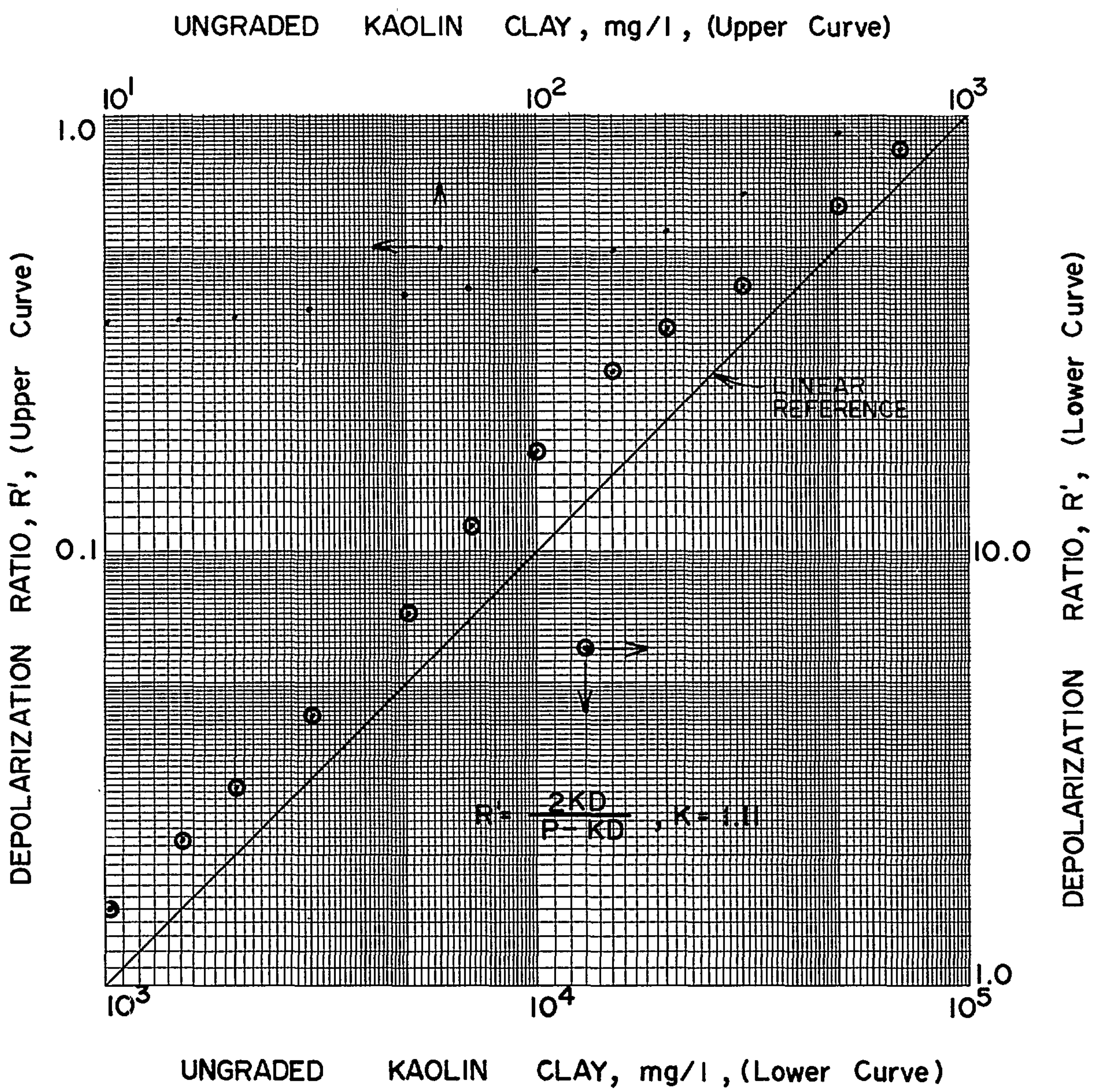


FIGURE 87. Depolarization Ratio vs Ungraded Kaolin Clay Concentration - Unbalanced Depolarization-Polarization Gain Adjusted

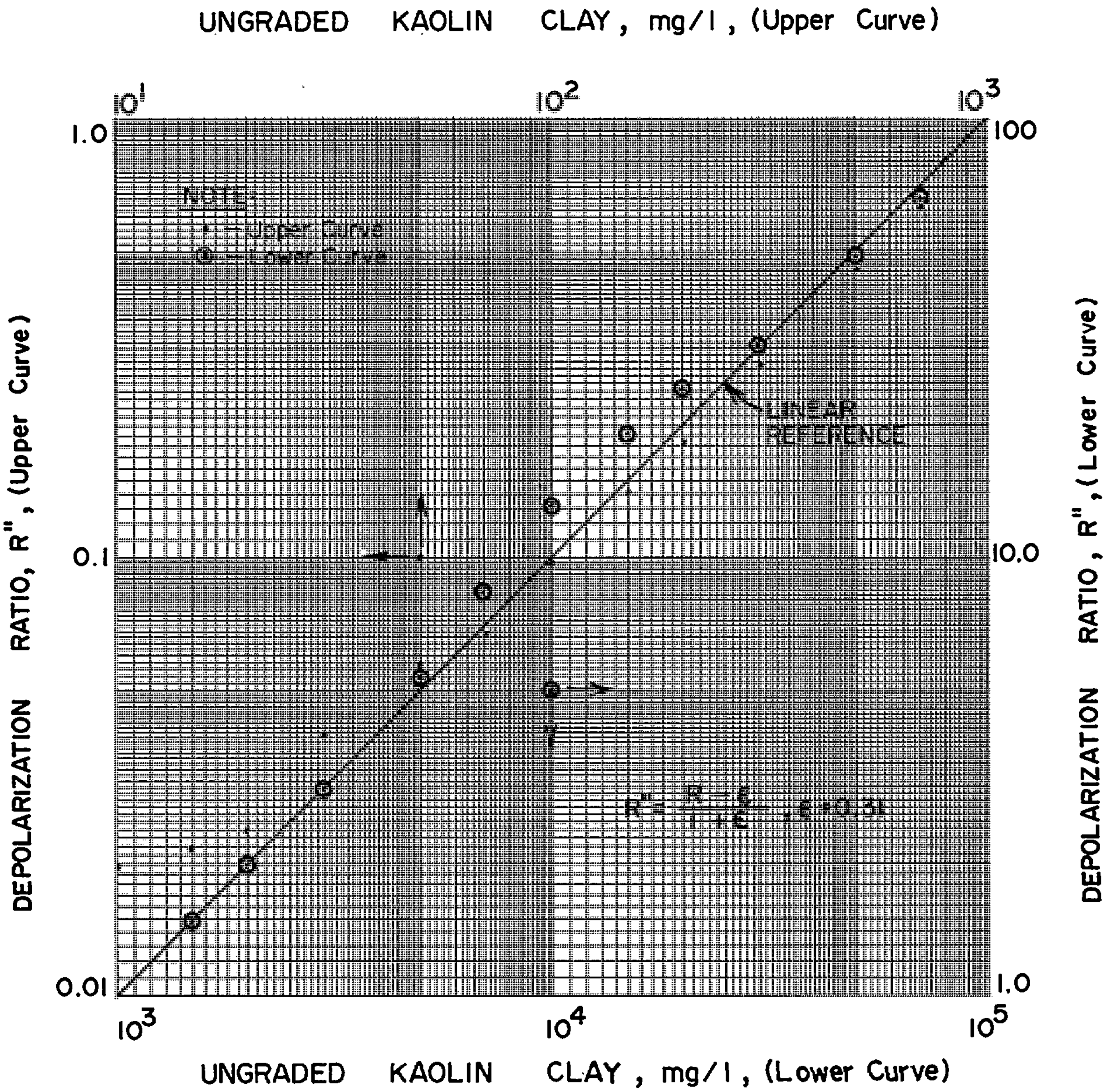


FIGURE 88. Depolarization Ratio vs Ungraded Kaolin Clay Concentration - Adjusted for Reradiation Effects

Limited tests using 5, 10, 15 and 30μ graded kaolin samples yielded results which were indiscernibly different from those of random kaolin samples whose sizes are greater than or equal to one micron (see Figure 89). This is in keeping with Liskowitz's observation that the backscatter method is relatively insensitive to particle size provided that particle size is greater than 1/2 wavelength.

In limited tests, considerable difference could be discerned between the kaolin, diatomaceous earth, pumice and rottonstone, indicating the sensitivity of the procedure to particles of different refractive indices. Refer to Figure 90.

Biological solids tests show that such particles had a slightly different response, most liekly due to their different refractive index as well as some variation in the amount of reflected illumination from the probe itself. The variations show slightly different behavior at the low end, becoming linear at the upper end but with an offset calibration variation. Additionally the tests were performed in a beaker which further influenced the lower concentration reading due to reflections from the beaker sides. See Figure 91.

Ordinary food coloring in varying concentrations and colors from dark blue to dark red had no noticeable effect on the output of the unit. The nigrisine black did have an effect when the concentration of the nigrisine black in mg/l became equal to approximately five times the concentration of the particulate matter. See Figures 92 and 93. It is felt that the concentration level of this intensely black dye would be unrealistic in most applications. At the levels which caused change in the readings the absorption is so high that return energy from multiple scatters is significantly reduced making the concentration appear artificially lower than it is.

Field Tests-- The procedure used in the field tests consisted of immersing the suspended solids monitor probe at each of several selected sites and obtaining grab samples concurrently with recording the monitor output. The grab samples were then returned to the lab where gravimetric tests were performed to obtain total SS. In the tests performed at the local sewage treatment plant these grab samples were analyzed in a laboratory within several hours, whereas the samples obtained at the remote sites in Syracuse were obtained with automatic sampling apparatus and these samples were not analyzed in the laboratory for up to 48 hr after they were obtained.

On two separate occasions the SS monitor was taken to Tulsa's Mohawk Park Sewage Treatment Plant where several samples were obtained at various places throughout the treatment process as indicated on Figure 94. Samples were obtained in the clarifier effluent, raw influent, and aeration basins. The data shown in Figure 94, for the ratio, R as displayed by the SS monitor, is the raw data and has not been corrected. The spread in the clarifier effluent and aeration basin data is probably caused by instability in the SS monitor. In general there appears to be good qualitative and

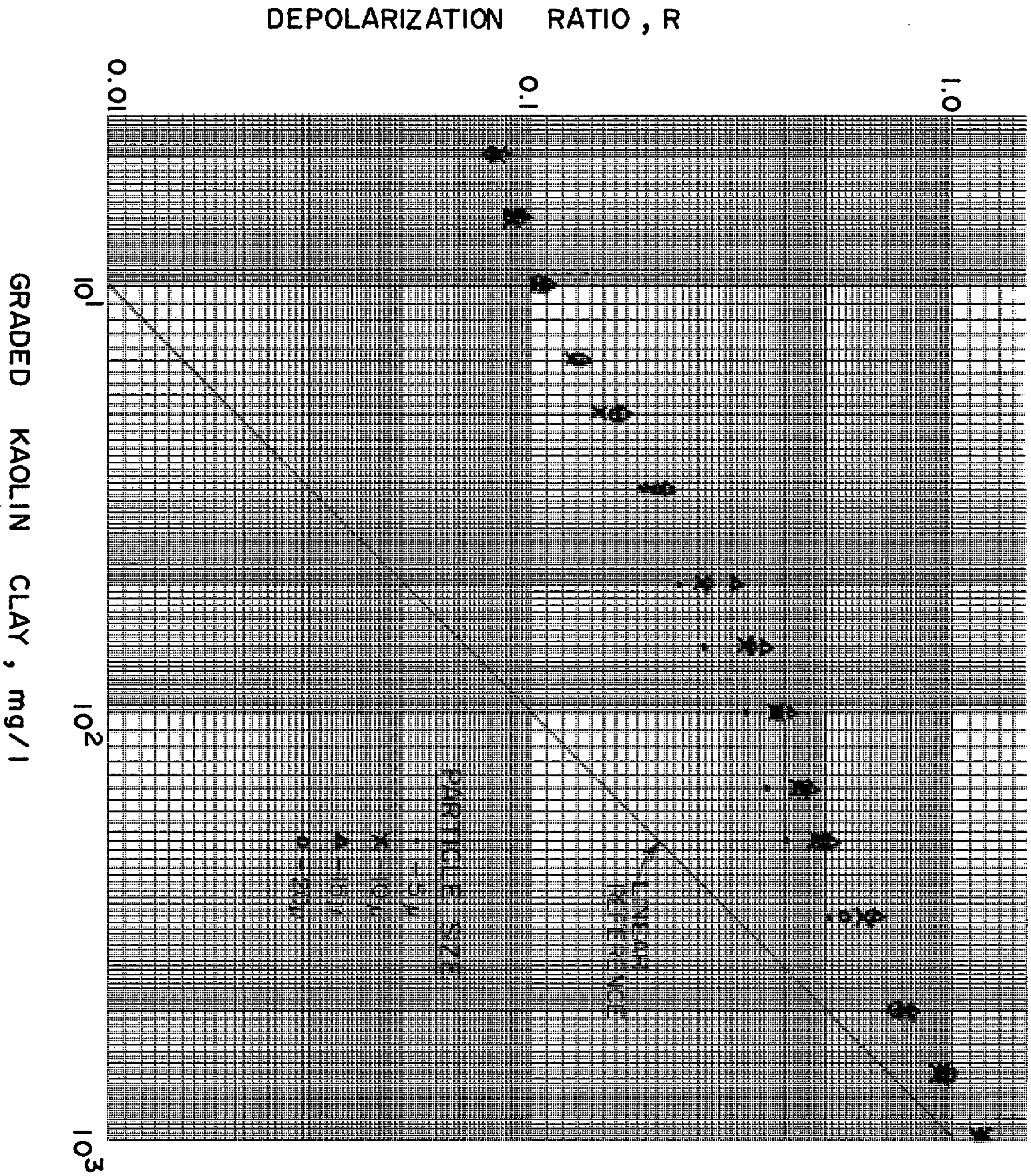


FIGURE 89. Depolarization Ratio vs Graded Kaolin Concentration

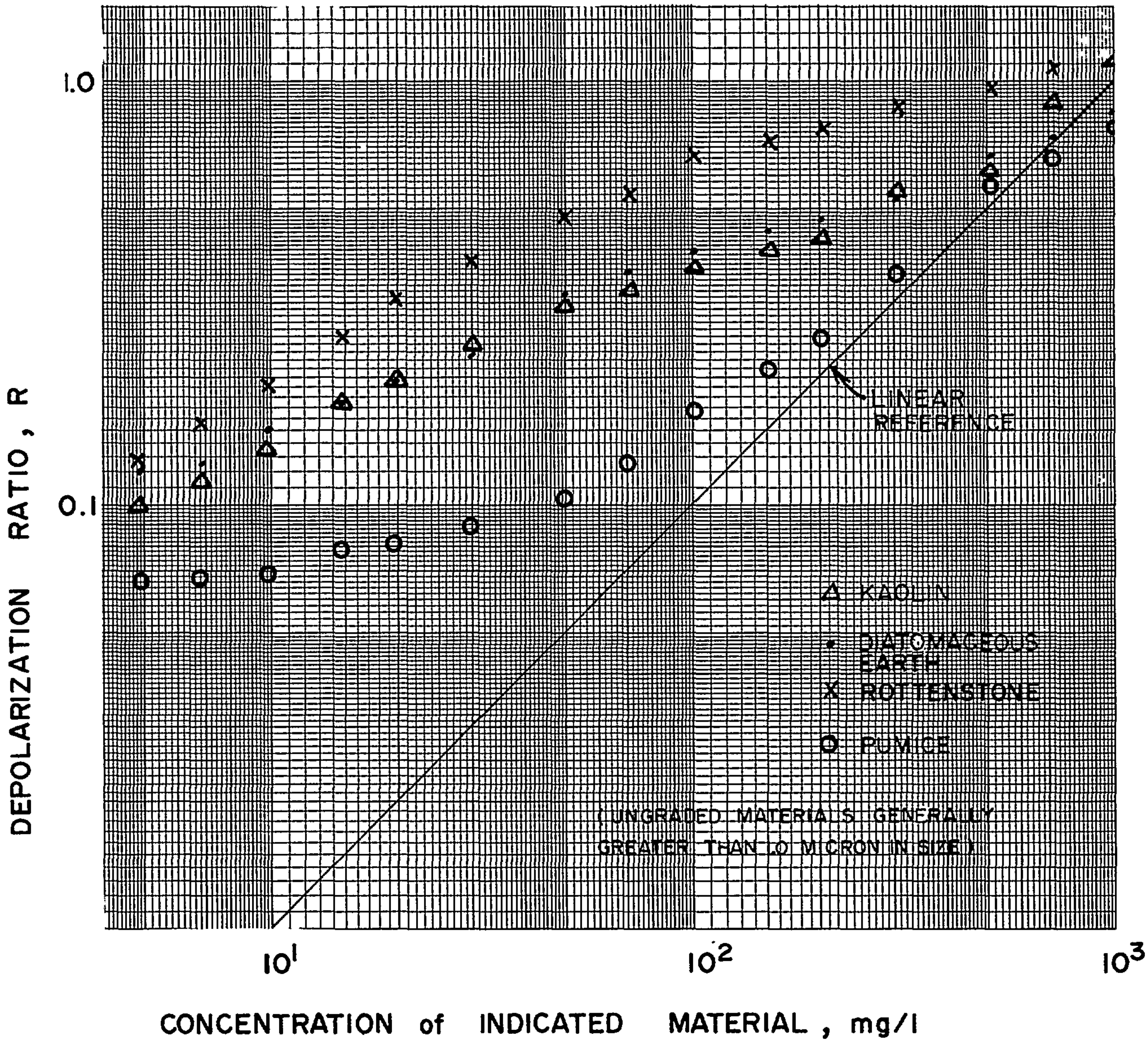


FIGURE 90. Depolarization Ratio vs Kaolin, Diatomaceous Earth, Rottenstone, and Pumice Concentrations

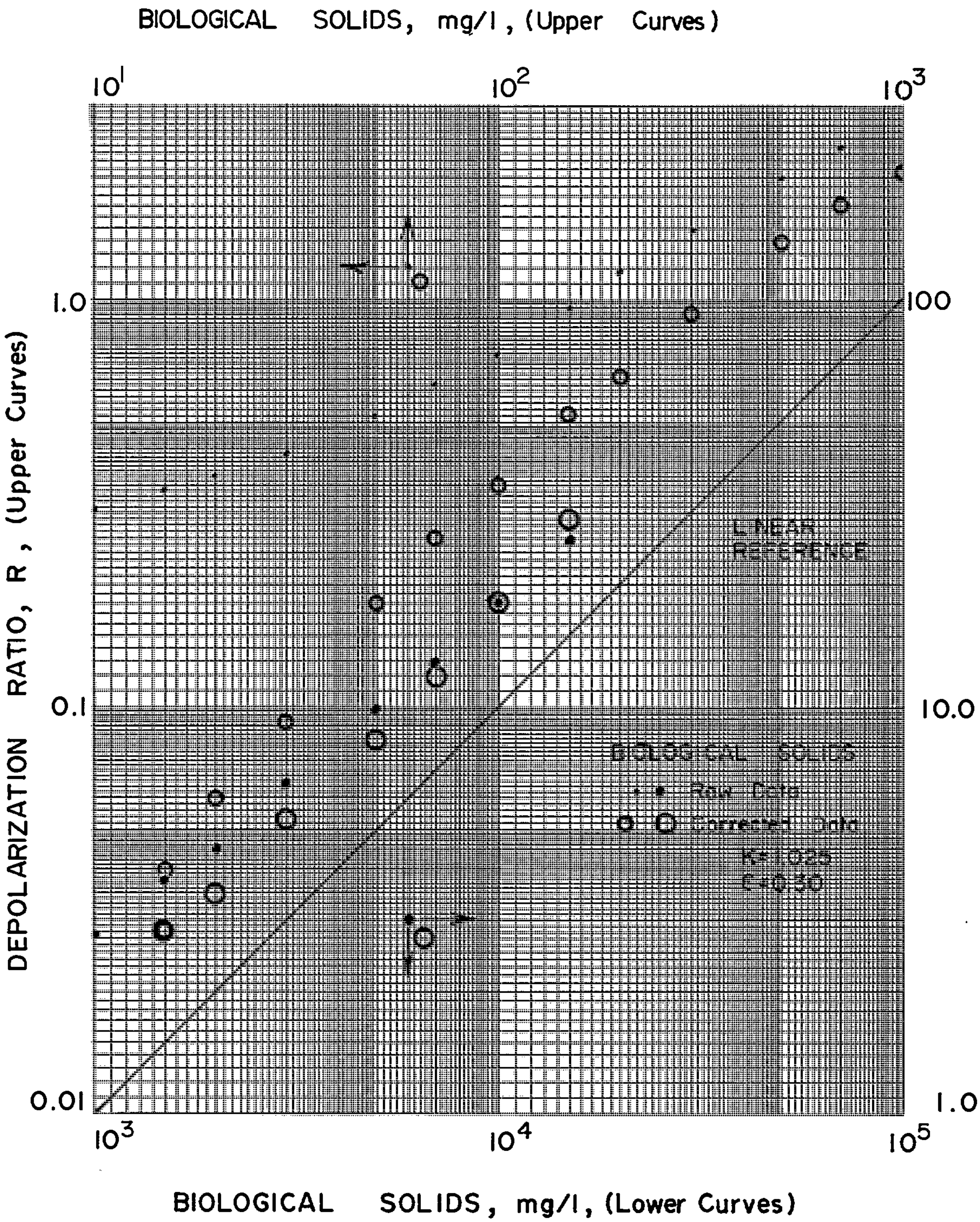


FIGURE 91. *Depolarization Ratio vs Biological Solids Concentration*

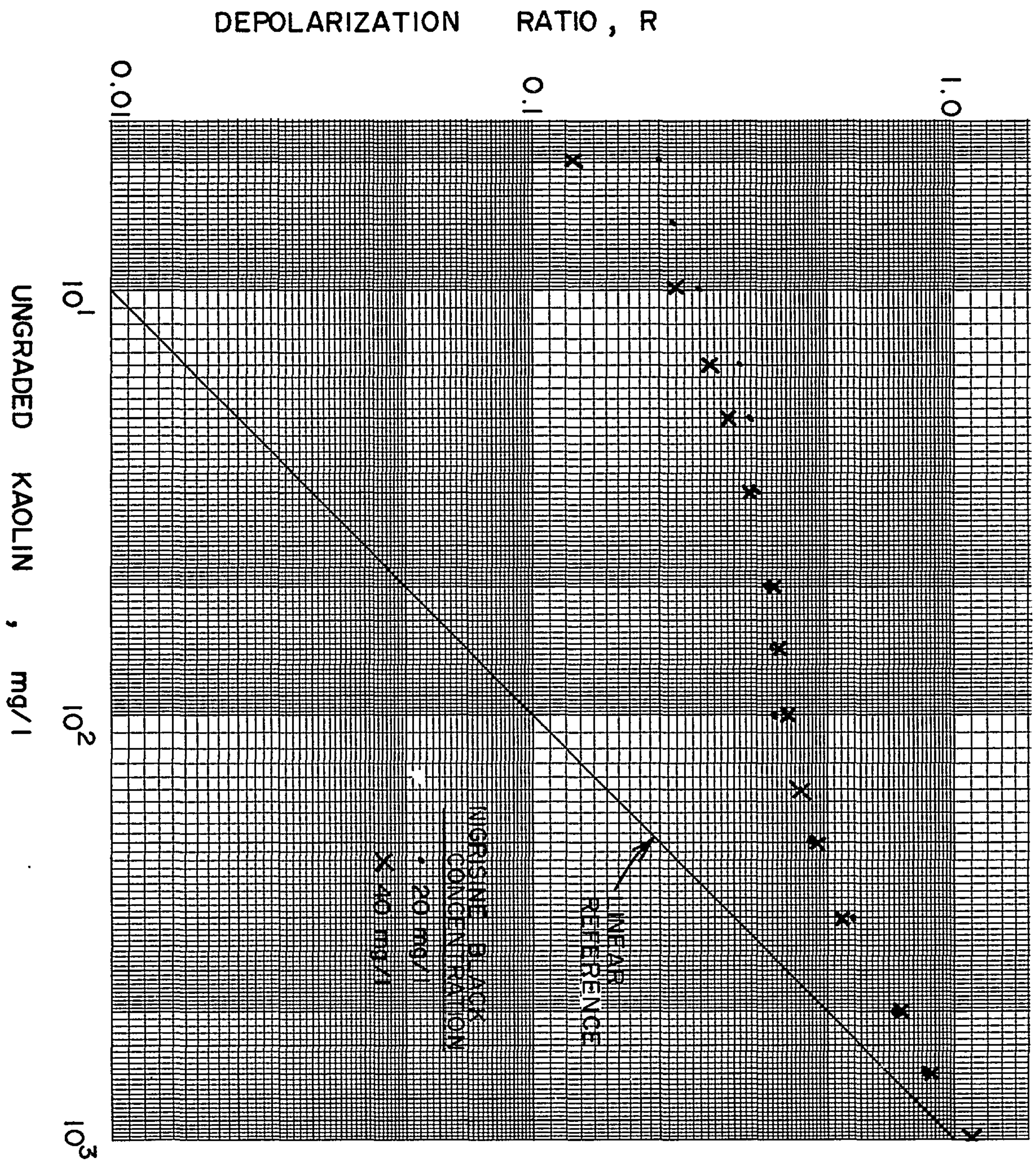


FIGURE 92. Depolarization Ratio vs Nigrisine Black Concentration

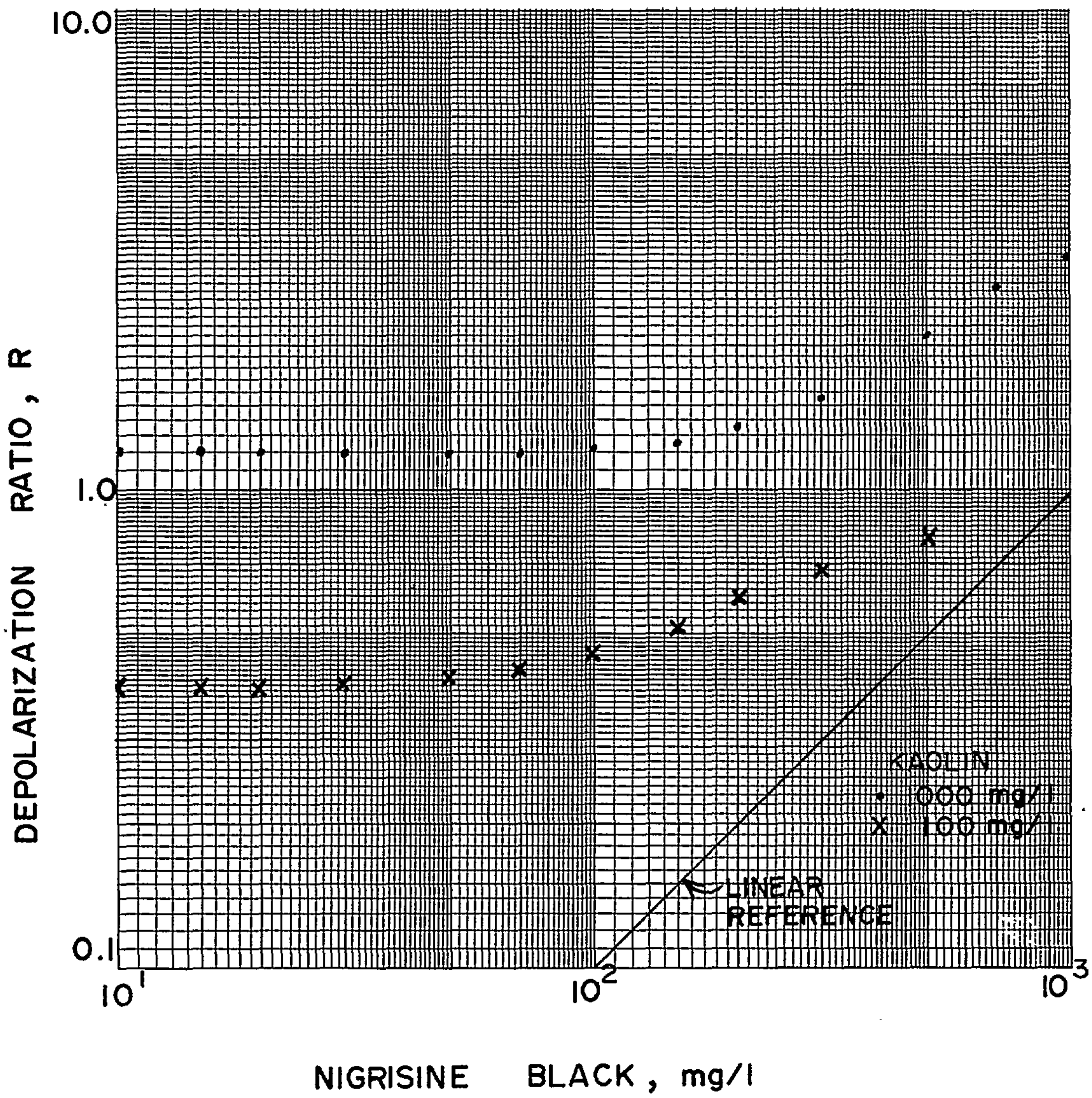


FIGURE 93. Depolarization Ratio vs Nigrisine Black and Kaolin Concentrations

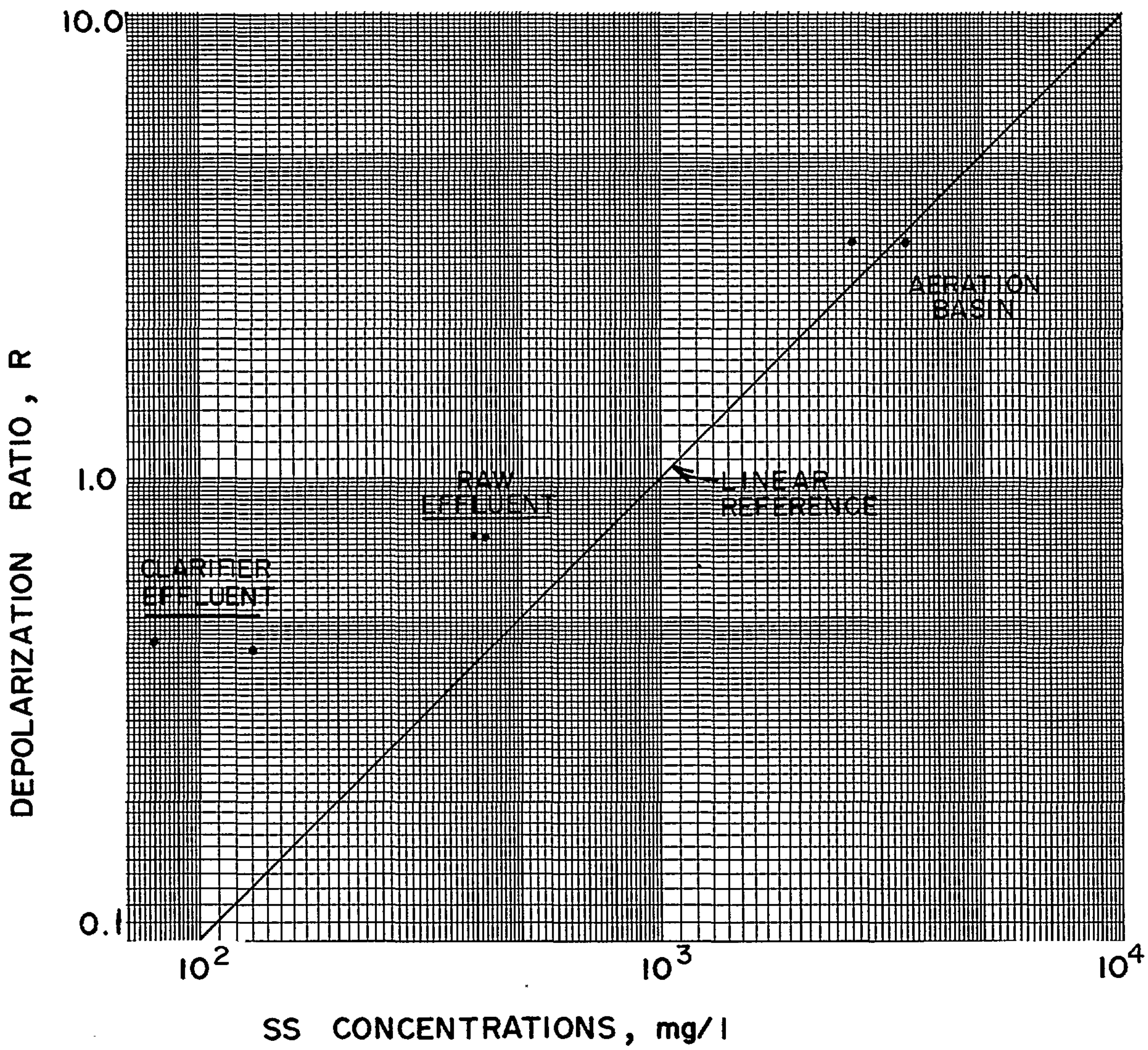


FIGURE 94. Depolarization Ratio vs Treatment Process SS Concentrations

quantitative correspondence in these tests with the earlier laboratory biological tests and the tests performed using kaolin.

In the Syracuse test program the probe was installed at several sites where CSO was treated. Consequently data obtained at these sites was limited to those times during which there was a storm event. Results of this testing were limited for a number of reasons. Initially the recorder used at the site was not sufficiently fast enough in the time scale to allow accurate interpolation of the transient events. Additionally, since the SS monitor did not have an auto-arranging capability the unit had to be left in a specified SS measuring range over which it was hoped that the majority of the SS values measured during the storm event would occur. As such the field calibration was not precisely known. An additional factor which limits direct comparison of SS monitor values with the gravimetric SS determinations is that the automatic samplers tend to attentuate peak SS values since samples are pumped into the containers over some specified time frame, however short it may be. Thus instantaneous SS values are not directly measurable for those samples collected automatically. However, there were several events for which adequate data was obtained. One such event is shown as Figure 95 in which the measured total SS for the SS monitor are shown versus the time of the event. Note the good correlation between the SS monitor output and the total SS over the period of the event. Unfortunately the precise calibration of the SS monitor is not known. The output plot was adjusted so that the peak events of the two overlie one another. It would be expected that the actual peak obtained on the SS monitor would be somewhat higher than that obtained by the automatic samplers; however, the significant factor in this figure is the good qualitative correlation between the two curves.

Summary of SS Monitor Results--

The objective of this project was to fabricate a prototype SS monitor based on the principle of depolarization of backscattered polarized light as investigated by Liskowitz and to test this prototype both in the laboratory and in the field. To a large extent this was accomplished with only the field testing in the storm overflow environment providing less data than expected due to the field sampling difficulties discussed earlier. The salient observations are itemized in the following.

1. Color Sensitivity

   The prototype was shown to be insensitive to the color of dissolved solids for practical concentrations. Nigrisine black dye concentration of less than 10 percent of the SS concentration had no effect. Similar results were obtained for red, green, and other color dyes.

2. Particle Shape and Size

   For particle sizes exceeding the radiation half wavelength, no size effects were noted. While direct shape testing was not

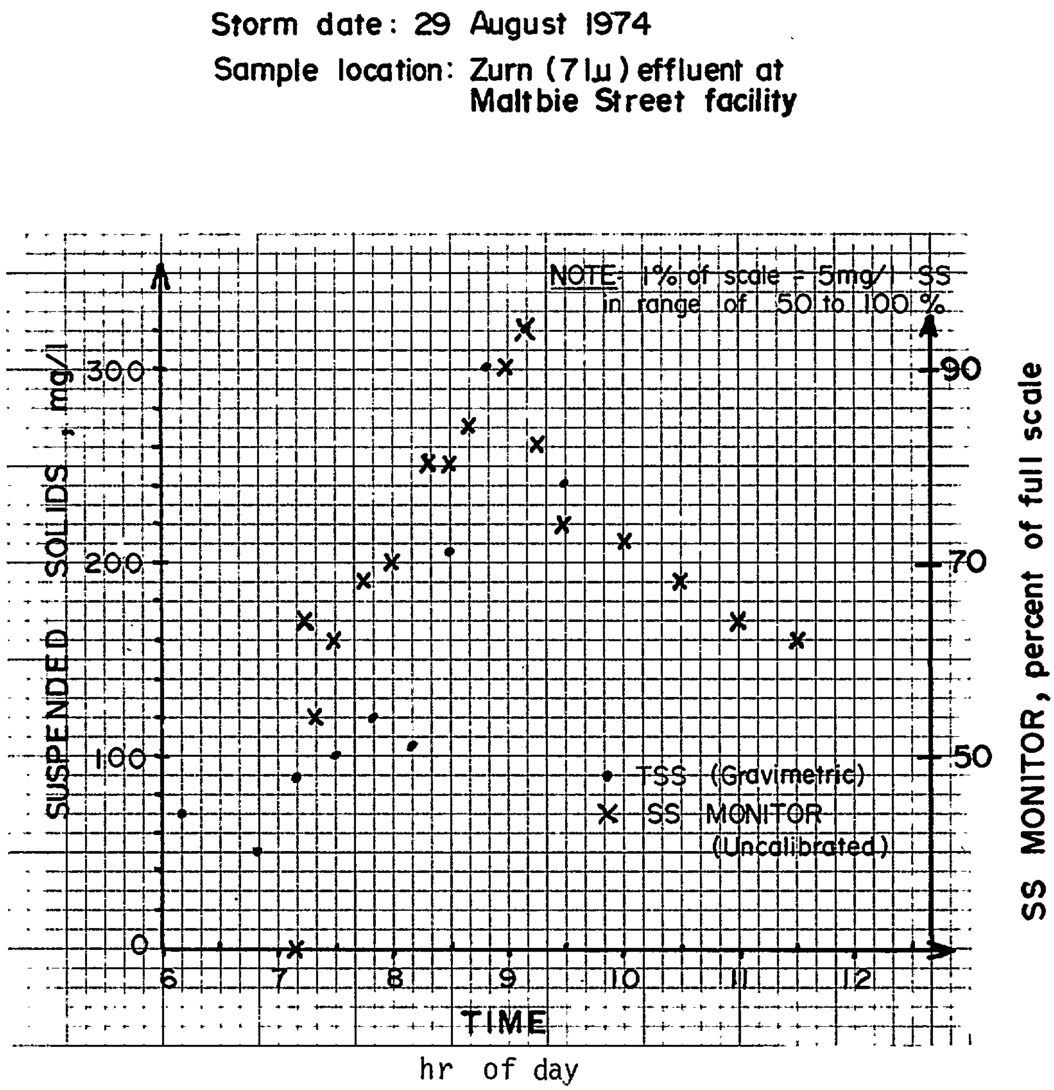


FIGURE 95. SS Monitor - Percent of Scale Readings and SS Concentrations in Syracuse Field Tests vs Time

accomplished, no effort was employed to control shape. There was no evidence of particle shape interference.

3. Ambient Light

Operation in ambient light conditions is limited only by sensor saturation. At low concentrations, where light shielding is negligible, operation in direct sunlight produced no interference. This characteristic is vital to portable survey usage.

4. Particle Concentration Range

Most tests were confined to measurements in the 10 mg/l to 100,000 mg/l SS range. The unit demonstrated measurement capability from less than 1 mg/l to over 1,000,000 mg/l.

5. Probe Physical Characteristic

The transducer probe size of 1 in. (2.5 cm) diameter and 6 in. (15.2 cm) length allows operation in confined spaces and permits portable operation.

6. Refractive Index

As with any optical device, sensitivity to particle refractive index is present. Calibration shifts on widely divergent refractive indices were noted. For refractively homogenous slurries, calibration stability was demonstrated. For instance, in sewage sludge the composition generally has less than 20 percent grit. The remainders are generally suspended and dissolved solids of a biological nature. Close examination of the monitor's response to biological solids as opposed to sand, pumice or stone dust shows that the overall interference by the suspended grit and the determination of suspended biological solids would not exceed one or two percent of error. Similarly in dredging operations where most of the suspended matter is largely grit, proper calibration would permit accurate display of suspended solids.

7. Probe Configuration

In low concentration measurements, nonlinearity of measurement was traced to two sources. First the shape of the probe face enhanced reradiation (by reflection) of transmitted energy which increased the depolarization ratio significantly. Secondly, the backscatter of light from the container sidewalls (such as a laboratory beaker) also enhanced the depolarization ratio. When measurements of low concentration slurries is attempted, the container volume should be increased over that used in the tests. Similarly the flat face probe design should be modified to reduce secondary reflections.

8. Comparative Results

While no direct comparative tests were carried out, the depolarization method has been shown to be insensitive to most of the parameters known to affect transmissive, nephelometric, and forward scatter measurement means.

REFERENCES

1. Lager, John A., and William G. Smith. Urban Stormwater Management and Technology: An Assessment. USEPA Report No. EPA-670/2-74-040. NTIS No. PB 240 687. September 1974. p.23.

2. Camp, Dresser and McKee Consulting Engineers. Comprehensive Sewerage Study, Onondaga County, New York. Project No. WPC-CS-169, Department of Public Works, Division of Drainage and Sanitation, County of Onondaga, New York, 1968. 62 pp.

3. O'Brien and Gere Engineers, Inc. Industrial Waste Monitoring Program. Project No. 115.246, Department of Public Works, Division of Drainage and Sanitation, County of Onondaga, New York, 1975.

4. O'Brien and Gere Engineers, Inc. Onondaga Lake Study. USEPA Project No. 11060 FAE, County of Onondaga, New York, April, 1971.

5. O'Brien and Gere Engineers, Inc. Onondaga Lake Monitoring Program, Jan-Dec 1973. Project No. 115.238, Department of Public Works, Division of Drainage and Sanitation, County of Onondaga, New York, 1975.

6. O'Brien and Gere Engineers, Inc. Unpublished Data for Onondaga County Department of Public Works, Division of Drainage and Sanitation, 1959-1969.

7. Moffa, P.E., et al. Bench-Scale High Rate Disinfection of Combined Sewer Overflows with Chlorine and Chlorine Dioxide. USEPA Report No. EPA-670/2-75-021. NTIS No. PB 242 296. April 1975.

8. Murphy, C.B., and Orest Hrycyk. High Rate Nutrient Removal for Combined Sewer Overflows. USEPA Report No. EPA-600/2-78-056. NTIS No. PB 285 473. June 1978.

9. Glover, George E., and George R. Herbert. Microstraining and Disinfection of Combined Sewer Overflows - Phase II. USEPA Report No. EPA-R2-73-124. NTIS No. PB 219 879. January 1973.

10. Maher, M.B. Microstraining and Disinfection of Combined Sewer Overflows-Phase III. USEPA Report No. EPA-670/2-74-049. NTIS No. PB 235 771. August 1974.

11. Keilbaugh, W.A., et al. Microstraining - with Ozonation and Chlorination-of Combined Sewer Overflows. Combined Sewer Overflow Seminar Papers, November 1969. USEPA Report No. 11020---03/70. NTIS No. PB 199 361. 1970.

12. American Public Works Association. Combined Sewer Overflow Regulation and Management-A Manual of Practice. USEPA Report No. 11022 DMU 08/70. NTIS No. PB 195 676. 1970.

13. Sullivan, R.H., et al. The Swirl Concentrator as a Combined Sewer Overflow Regulator Facility. USEPA Report No. EPA-R2-72-008. NTIS No. PB 214 687. September 1972.

14. Sullivan, R.H., et al. Relationship Between Diameter and Height for the Design of a Swirl Concentrator as a Combined Sewer Overflow Regulator. USEPA Report No. EPA-670/2-74-039. NTIS No. PB 234 646. July, 1974.

15. Field, Richard. The Dual Functioning Swirl Combined Sewer Overflow Regulator/Concentrator. USEPA Report No. EPA-670/2-73-059. NTIS No. PB 227 182/3. September, 1973.

16. Hsiung, G.D., and J.R. Henderson. Diagostic Virology. Yale University Press, New Haven, Connecticut, 1964.

17. Kalter, S.S. Procedures for Routine Laboratory Diagnosis of Virus and Rickettsial Diseases. Burgess Publishing Company, Minneapolis, Minnestoa, 1963.

18. Meynell, G.G., and E. Maynell. Theory and Practice in Experimental Bacteriology. Cambridge University Press, Cambridge, Massachusetts, 1965.

19. Rosenbaum, J.J., E.J. Sullivan, and E.A. Edwards. Techniques for Cell Cultivation in Microtitration Plates and Their Application in Biological Assays. In: Animal Tissue Culture, G.D. Wasley, ed. Williams and Wilkins Co., Baltimore, 1973.

20. Leibowitz, A. The Growth and Maintenance of Tissue-Cell Cultures in Free Gas Exchange with the Atmosphere. American Journal of Hygiene, 78:173-178, 1963.

21. Sobsey, M.D., C. Wallis, M. Henderson, and J.L. Melnick. Concentration of Enteroviruses from Large Volumes of Water. Applied Microbiology, 26: 529-534, 1973.

22. Homma, A., M.D. Sobsey, C. Wallis, and J.L. Melnick. Virus Concentration from Sewage. Water Research, 7:945-950, 1973.

23. Berg, G. Transmission of Viruses by the Water Route. John Wiley and Sons, New York, New York, 1967.

24. Hill, W.F., Jr., E.W. Akin, and W.H. Benton. Detection of Viruses in Water: A Review of Methods and Applications. Water Research, 5:967-995, 1971.

25. Wallis, C., A. Homma, and J.L. Melnick. Apparatus for Concentrating Viruses from Large Volumes of Water. Journal of the American Water Works Association, 64:189-196, 1972.

26. Wallis, C., A. Homma, and J.L. Melnick. A Portable Virus Concentrator for Water Testing in the Field. Water Research, 6:1249-1256, 1972.

27. Kruse, C.W., V. Olivieri, and K. Katvata. The Enhancement of Viral Inactivation by Halogens. Water and Sewage Works, 118:187, 1971.

28. Wallis, C., M. Henderson, and J.L. Melnick. Enterovirus Concentration on Cellulose Membranes. Applied Microbiology, 23: 476-480, 1972.

29. Wallis, C., and J. L. Melnick. A Portable Virus Concentrator for Use in the Field. In: Advances in Water Pollution Research, Jerusalem, 1972, S.H. Jenkins, ed. Pergamon, Oxford, England, 1973. pp 119-126.

30. Metcalf, T.G., C. Wallis, and J. L. Melnick. Concentration of Viruses from Seawater. In: Advances in Water Pollution Research, Jerusalem, 1972. S.H. Jenkins, ed. Pergamon, Oxford, England, 1973. pp. 109-115.

31. Metcalf, T.G., C. Wallis, and J.L. Melnick. Environmental Factors Influencing Isolation of Enteroviruses from Polluted Waters. Applied Microbiology, 27: 920-926, 1974.

32. Dahling, D.R., G. Berg, and D. Berman. BGM, A Continuous Cell Line More Sensitive Than Primary Rhesus and African Green Kidney Cells for the Recovery of Viruses from Water. Health and Laboratory Sciences, 11:275-282, 1974.

33. American Public Works Association. Combined Sewer Regulator Overflow Facilities. USEPA Report No. 11022 DMU 07/70, July 1970. p. 139.

34. Smisson, B. Design, Construction and Performances of Vortex Overflows. In: Proceedings, Symposium on Storm Sewage Overflows, Institute of Civil Engineers, Great Britain, May 4, 1967. p. 99.

35. Ackers, P., A.J.M. Harrison, and A.J. Brewer. Laboratory Studies of Storm Overflows with Unsteady Flow. In: Proceedings, Symposium on Storm Sewage Overflows, Institute of Civil Engineers, London, Great Britain, May 4, 1967. p. 37.

36. Prus-Chacinski, T.M., and J.W. Wielgorski. Secondary Motions Applied to Storm Sewage Overflows. In: Proceedings, Symposium on Storm Sewage Overflows, Institute of Civil Engineers, London, Great Britain, May 4, 1967.

37. Kromis, H. Full Scale Evaluation of High Rate Screening Devices for Treatment of Sanitary Sewage Bypass Flow, Interim Report. Ontario Ministry of the Environment, Ontario, Canada, Letter correspondence, June 20, 1975.

38. Krus, C.W. Improvement in Terminal Disinfection of Sewage Effluents. Water and Sewage Works, 120: 57-64, 1973.

39. Glover, G.E. High-Rate Disinfection of Combined Sewer Overflow. In: Combined Sewer Overflow Seminar Papers, USEPA Report No. 670/2-73-077. NTIS No. PB 231 836. November 1973.

40. National Oceanic and Atmospheric Administration, Environmental Data Service. Magnetic Tape Decks 345 and 488, 1948 to 1973. U.S. Department of Commerce, Ashville, North Carolina.

41. Rainfall Frequency Atlas of the United States, Technical Paper No. 40. U.S. Department of Commerce, Washington, D.C. 1961.

42. Lager, John A., Theodor Didriksson, and George B. Otte. Development and Application of a Simplified Stormwater Management Model. USEPA Report No. EPA-600/2-76-218. NTIS No. PB 258 074. August, 1976.

43. Fram Corporation. Strainer/Filter Treatment of Combined Sewer Overflows. USEPA Report No. 11010 EXV07/69. NTIS No. PB 185 949. July 1969.

44. Gupta, Mahendra K., et al. Screening/Flotation Treatment of Combined Sewer Overflows, Volume I - Bench Scale and Pilot Plant Investigations. USEPA Report No. EPA-600/2-77-069a. NTIS No. PB 272 834. 1977.

45. Rex Chainbelt, Inc. Screening/Flotation Treatment at Combined Sewer Overflows. USEPA Report No. 11020 FDC 01/72, 1972.

46. Nebolsine, Ross, et al. High Rate Filtration of Combined Sewer Overflows. USEPA Report No. 11023 EYI 04/72. NTIS No. PB 211 144. April 1972.

47. Neketin, T.H., and H.K. Dennis. Demonstration of Rotary Screening for Combined Sewer Overflows. USEPA Report No. 11023 FDD 07/71. NTIS No. PB 206 814. July 1971.

48. Lager, John A., et al. Urban Stormwater Management and Technology: Update and User's Guide. USEPA Report No. EPA-600/8-77-014. NTIS No. PB 275 654. September 1977.

49. Colston, N.V., Jr. Characterization and Treatment of Urban Land Runoff. USEPA Report No. EPA-670/2-74-096. NTIS No. PB240 978. December 1974.

50. Mytelka, Alan I., et al. Combined Sewer Overflow Study for the Hudson River Conference. USEPA Report No. EPA-R2-73-152. NTIS No. PB 227341. January 1973.

51. Dalrymple, Robert J., et al. Physical and Settling Characteristics of Particulates in Storm and Sanitary Wastewaters. USEPA Report No. EPA-670/2-75-011. NTIS No. PB 242 001. April 1975.

52. Gupta, M.K., et al. Handling and Disposal of Sludges Arising From Combined Sewer Overflow Treatment: Phase I Characterization. USEPA Report No. EPA-600/2-77-053a. May 1977.

53. Rich, L.G. Unit Processes in Sanitary Engineering. John Wiley & Sons, Inc., New York, New York, 1963. pp 147-164.

54. Drehwing, Frank J., et al. Combined Sewer Overflow Abatement Program, Rochester, New York - Pilot Plant Evaluations. USEPA Project No. Y005141, Final Report. 1978.

55. Fair, Gordon M., and John C. Geyer. Elements of Water Supply and Waste-Water Disposal. John Wiley and Sons, Inc., New York, New York, 1965. pp. 480-486.

56. Benjes, Henry H. Jr. Cost Estimating Manual - Combined Sewer Overflow Storage and Treatment. USEPA Report No. EPA-600/2-76-286. NTIS No. PB 266 359. December 1976.

57. 1975 Dodge Guide for Estimating Public Works Construction Costs. Dodge Building Costs Services, McGraw-Hill Information Systems Company, New York, New York, 1975. 201 pp.

58. Rao, V.C., V. Chandorkar, N.V. Rao, P. Kumeran, and S.B. Lahke. Unpublished Data In: Second International Congress for Virology, Budapest, Hungary, 1971.

59. Metcalf, T.G. Personal Communication. 1974.

60. Field, Richard, et al. Proceedings of Workshop on Microorganisms in Urban Stormwater. USEPA Report No. EPA-600/2-76-244. NTIS No. PB 263 030. November 1976.

61. Long, W.N., and F.A. Bell, Jr. Health Factors and Reused Water. Journal of the American Water Works Association, 64: 220, 1072.

62. Taylor, F.B. Viruses - What Is Their Significance in Water Supplies. Journal of the American Water Works Association, 66: 306-311, 1974.

63. Potable Waters. Hearings Before the Committee on Commerce, U.S. Senate, on Amendment 410 to S1478, Serial No. 92-57. U.S. Government Post Office, Washington, D.C., 1972.

64. Luria, S.E., et al. General Virology. John Wiley & Sons, New York, New York, 3rd Edition, 1978. 578 pp.

65. Rook, J.J. Formation of Halogens During Chlorination of Natural Waters. Water Treatment Examiner, Part 2, 23:234-243, 1974.

66. Harris, R.H. The Implications of Cancer Causing Substances in Mississippi River Waters. Environmental Defense Fund, Washington, D.C. 1974.

67. Morris, J.C., I.W. Weil and R.H. Culver. Kinetic Studies on the Breakpoint Reaction with Ammonia and Glycine. Presented at: International Congress of Pure and Applied Chemistry, New York, New York, 1952.

68. American Public Health Association, Inc. Standard Methods for the Analysis of Water and Wastewater, 14th Edition. New York

69. Federal Register, 38:22298, August 17, 1978.

70. Dowty, B., D. Carlisle, and J.L. Laseter. New Orleans Drinking Water Sources Tested by Gas Chromatography-Mass Spectrometry. Environmental Science & Technology, 9:762, 1975.

71. American Public Health Association, Inc. Standard Methods for the Examination of Water and Wastewater, 13th Edition. New York, 1971.

72. Chambers, C.W. Chlorination for Control of Bacteria and Viruses in Treatment Plant Effluents. Journal Water Pollution Control Federation, 43:228, 1971.

73. Robeck, G.G. Substitution of Residual Measurement for Distribution Bacteriological Sampling. In: Proceedings of the AWWA Water Quality Technology Conference, Dallas, Texas, December 2-3, 1974. American Water Works Association, Denver, Colorado, 1975.

74. Baker, R.J. Engineering Considerations in Disinfection. In: Proceedings of the National Specialty Conference on Disinfection, July 8-10, 1970. American Society of Civil Engineers, New York, 1970.

75. Reasoner, D.J., and E.E. Geldreich. Rapid Bacteriological Methods. In: Proceedings of the AWWA Water Quality Technology Conference, Dallas, Texas, December 2-3, 1974, American Water Works Association, Denver, Colorado, 1975.

76. McElroy, W.D. Energy Source for Bioluminescence in an Isolated System. Proceedings National Academy of Sciences (USA), 33: 342-345, 1947.

77. Levin, G.V., et al. Rapid Method for Detection of Microorganisms by ATP Assay: It's Possible Application in Virus and Cancer Studies. Bioscience, 14:37-38, 1964.

78. Holm-Hansen, O. Determination of Microbial Biomass in Ocean Profiles. Limnology & Oceangraphy, 14:740-747, 1969.

79. Heunnekens, F.M., and H.R. Whitely. Phosphoric Acid Anhydrides and Other Energy-rich Compounds. Comparative Biochemistry, 1:107-180.

80. Ponnamperuma, C., R. Mariner, and C. Sagan. Formation of Adenosine by Ultra-violet Irradiation of a Solution of Ademine and Ribose. Nature, 198:1199, 1963.

81. Ponnamperuma, C., C. Sagan, and R. Mariner. Synthesis of Adenosine Triphosphate Under Possible Primitive Earth Conditions. Nature, 199: 222, 1963.

82. Levin, G.L., C. Chen, and G. Davis. Development of the Firefly Bioluminescent Assay for the Rapid Quantitative Detection of Microbial Contamination of Water. Aerospace Medical Research Laboratories Report No. AMRL-TR-67-71, Wright-Patterson Air Force Base, Ohio, 1967.

83. McElroy, W.D., M. Deluca, and J. Travis. Molecular Uniformity in Biological Catalyses. Science, 157:150, 1967.

84. Plant, P.J., E.H. White, and W.D. McElroy. Decarboxylation of Luciferin in Firefly Bioluminescence. Biochemical and Biophysical Research Communications, 31:98, 1968.

85. McElroy, W.D., H.H. Seliger, and E.H. White. Mechanism of Bioluminescence, Chemiluminescence and Enzyme Function in the Oxidation of Firefly Luciferin. Photochemistry and Photobiology, 10:153, 1969.

86. Rhodes, W.C., and W.D. McElroy. Synthesis and Function of Luciferyl-adenylate and Oxyluciferyl-adenylate. Journal of Biological Chemistry, 233:1528, 1958.

87. McElroy, W.D., and H.H. Seliger. Chemistry of Light Emission. Advances in Enzymology, 25:119, 1963.

88. Rudd, J.W.M., and R.D. Hamilton. Measurement of Adenosine Triphosphate (ATP) in Two Precambrian Shield Lakes of Northwestern Ontario. Journal of the Fisheries Research Board of Canada, 30:1537, 1973.

89. Hamilton, R.D., O. Holm-Hansen, and J.D.H. Strickland. Notes on the Occurrence of Living Microscopic Organisms in Deep Water. Deep-Sea Research, 15:651, 1968.

90. Kao, I.C., et al. ATP Pools in Pure and Mixed Cultures. Journal Water Pollution Control Federation, 45:926, 1973.

91. Hobson, P.N., and R. Summers. ATP Pool and Growth Yield in Selenomonas ruminantum. Journal of General Microbiology, 70:351, 1972.

92. Levin, G.V., J.R. Schrot, and W.C. Hess. Methodology for Application of Adenosine Triphosphate Determination in Wastewater Treatment. Environmental Science & Technology, 9:961, 1975.

93. Smith, J.E., and J.L. McVea. Virus Inactivation by Chlorine Dioxide and Its Application to Storm Water Overflow. In: Proceedings of the 166th National Meeting, American Chemical Society, Chicago, Illinois, 13:2, 177-185, August 1973.

94. Instruction Manual Luminescence Biometer, Catalog No. 760018. Instruments Products Division, E.I. Dupont et Nemours, Wilmington, Delaware, 1970.

95. Chappelle, E.W., and G.V. Levin. Use of the Firefly Bioluminescent Reaction for Rapid Detection and Counting of Bacteria. Biochemical Medicine, 2:41, 1968.

96. Dufresne, L., and H.J. Gitelman. Semiautomated Procedure for Determination of Adenosine Triphosphate. Analytical Biochemistry 37:402, 1970.

97. Proceedings of the First Microbiology Seminar on Standardization of Methods. USEPA Report No. EPA-R4-73-022, Washington, D.C. 1973.

98. Johnson, R., J.H. Gentile, and S. Cheer. Automatic Sample Injector: Its Application in the Analysis of Adenosine Triphosphate. Analytical Biochemistry, 60:115, 1974.

99. VanDyke, K., R.Stitzel, T. McClellan, C. Szutkiewicz. Automated Analysis of ATP: Its Application to On-Line Continuous-Flow Incubations and Measurement of ATP-Coupled Enzyme Systems. Advances in Automated Chemistry, 1:47, 1969.

100. Technicon Corporation. Automating Manual Methods Using Technicon Auto Analyzer II System Techniques. Publication No. TNI-0170-01, Tarrytown, N.Y., 1972.

101. Raytheon Company, Environmental Systems Center. Stormwater Total Organic Carbon Analyzer. Portsmouth, Rhode Island, 1975. 32 pp.

102. Tulumello, A. Automatic Organic Monitoring System for Storm and Combined Sewers. USEPA Report No. EPA-670/2-75-067. NTIS No. PB 244 142. 1975.

103. Liskowitz, J.W. An Empirical Method for Determining the Concentration of Solids in Suspension. Environmental Science and Technology, 12:1206-1211, 1971.

104. Liskowitz, John W., et al. Suspended Solids Monitor. USEPA Report No. EPA-670/2-75-002. NTIS No. PB 241 581. 1975.

105. Liskowitz, J.W., and G.J. Franey. Measurement of Suspended Solids Concentration in Sewage by Use of a Depolarization Method. Environmental Science and Technology, 1:43-47, 1972.

106. Richardson, Allyn C. Special Assistant for Advanced Technology, USEPA Region I, Boston, Massachusetts. Personal Communication, 1973.

107. Areawide Assessment Procedures Manual - Volume I, Volume II and Volume III. USEPA Report No. EPA-600/9-76-014. Washington, D.C. July 1976.

APPENDIX A

Typical Preliminary Monitoring Data

APPENDIX A LEGEND

STRM NO	-	Storm Number
PRIM LOC	-	Primary CSO Location 1- Maltbie Street 2- Newell Street 3- Rowland Street
SEC LOC	-	Secondary Location 1- Untreated CSO
SEQ NO	-	Sequential Sample Number
TYPE	-	Type of Sample 1- Automatic Sequential Sample
DATE	-	Date Overflow Occurred
TIME	-	Time of Sample Collection
SAMPLE	-	Number Assigned to Sample for Analytical Purposes
FLOWRATE	-	Rate of Discharge, MGD
RAINACC	-	Total Accumulation of Precipitation, in.
RAININTS	-	Rain Intensity, in./hr
pH	-	Dimensionless
TCOLI	-	Total Coliform, cells/100 ml
TOC	-	Total Organic Carbon, mg/l
COD-M	-	Chemical Oxygen Demand (Wet Chemistry Analysis), mg/l
COD	-	Chemical Oxygen Demand (Automated Analysis), mg/l
TSS	-	Total Suspended Solids, mg/l
VSS	-	Volatile Suspended Solids, mg/l
TKN	-	Total Kjeldahl Nitrogen, mg/l
TIP	-	Total Inorganic Phosphorus, mg/l
Cl	-	Chloride, mg/l
FCOLI	-	Fecal Coliform, cells/100 ml
FSTREP	-	Fecal Streptococcus, cells/100 ml
TS	-	Total Solids, mg/l
VS	-	Volatile Solids, mg/l
TDS	-	Total Dissolved Solids, mg/l
VDS	-	Volatile Dissolved Solids, mg/l
BOD_5	-	Biochemical Oxygen Demand (5-Day), mg/l

Syracuse Combined Sewer Overflow Study
Maltbie Street Location
Preliminary Monitoring Data

STRM NO	PRIM LOC	SEC LOC	SEW NO	TYPE	DATE	TIME	SAMPLE	FLOWRATE	RAINACC	RAININTS	PH	TCOLI	TOC	COD-M	COD	TSS	VSS
4	1	1	1	1	06/30/72	345	226	14.9000	.35	.20	6.9	440000.	14.		27.	90.0	20.0
4	1	1	2	1	06/30/72	400	227	11.1500	.45	.40	7.2	200000.	9.		10.	60.0	24.0
4	1	1	3	1	06/30/72	415	228	8.9200	.50	.20	7.4	420000.	6.		10.	75.0	5.0
4	1	1	4	1	06/30/72	430	229	14.9000	.55	.20	7.5	340000.	7.		5.	110.0	.0
4	1	1	5	1	06/30/72	445	230	14.9000	.70	.60	7.6	480000.	6.		4.	70.0	.0
4	1	1	6	1	06/30/72	500	231	14.9000	.80	.40	7.5	390000.	4.		7.	80.0	.0
4	1	1	7	1	06/30/72	515	232	12.7200	.90	.40	7.7	330000.	11.		5.	75.0	.0
4	1	1	8	1	06/30/72	530	233	14.9000	1.00	.40	7.7	80000.	6.		5.	75.0	.0
4	1	1	9	1	06/30/72	545	234	8.9200	1.05	.20	7.6	290000.	12.		9.	40.0	15.0
4	1	1	10	1	06/30/72	600	235	5.6400	1.10	.20	7.7	630000.	15.		8.	375.0	50.0
4	1	1	11	1	06/30/72	615	236	3.8700	1.12	.08	7.7	160000.	14.		14.	170.0	28.0
4	1	1	12	1	06/30/72	630	237	4.0500	1.15	.12	7.7	20000.	20.		15.	160.0	20.0
4	1	1	13	1	06/30/72	645	238	2.4700	1.15	.00	6.5	330000.	16.		13.	640.0	70.0
4	1	1	14	1	06/30/72	700	239	1.7100	1.15	.00	7.1	70000.	14.		15.	350.0	40.0
4	1	1	15	1	06/30/72	715	240	1.3500	1.15	.00	7.4	53000.	12.		18.	540.0	40.0
4	1	1	16	1	06/30/72	730	241	1.2200	1.15	.00	7.5	700000.	15.		18.	310.0	30.0
4	1	1	17	1	06/30/72	745	242	.9800	1.15	.00	7.1	560000.	21.		36.	290.0	.0
4	1	1	18	1	06/30/72	800	243	.8700	1.15	.00	7.5	310000.	33.		32.	180.0	.0
4	1	1	19	1	06/30/72	815	244	.8100	1.15	.00	7.0	560000.	15.		23.	170.0	.0
4	1	1	20	1	06/30/72	830	245	.9200	1.15	.00	7.1	600000.	21.		22.	170.0	.0
4	1	1	21	1	06/30/72	845	246	.8800	1.15	.00	7.3	610000.	17.		25.	105.0	.0
4	1	1	22	1	06/30/72	900	247	.6900	1.15	.00	7.5	950000.	25.		24.	150.0	.0
4	1	1	23	1	06/30/72	915	248	.6300	1.15	.00	7.6	1050000.	23.		27.	290.0	.0
4	1	1	24	1	06/30/72	930	249	.5200	1.15	.00	7.6	430000.	21.		25.	290.0	.0
4	1	1	25	1	06/30/72	1030	280	.4800	1.30	.15	7.1	18000.	19.		24.	198.0	28.0
4	1	1	26	1	06/30/72	1130	281	.4500	1.30	.00	7.3	36000.	24.		30.	176.0	8.0
4	1	1	27	1	06/30/72	1230	282	.6400	1.30	.00	7.7	48000.	42.		50.	136.0	12.0
4	1	1	28	1	06/30/72	1330	283	.3700	1.30	.00	8.0	4000.	36.		44.	12.0	4.0
4	1	1	29	1	06/30/72	1430	284	.2500	1.30	.00	8.2	10000.	25.		35.	33.0	.0
4	1	1	30	1	06/30/72	1530	285		1.30	.00							
4	1	1	31	1	06/30/72	1630	295	.2000	1.30	.00	8.2	9000.	27.		35.		
4	1	1	32	1	06/30/72	1730	296	.1600	1.30	.00	7.6	48000.	31.		42.	304.0	20.0
4	1	1	33	1	06/30/72	1830	297	.0700	1.30	.00	7.6	23000.	34.		44.	108.0	28.0
4	1	1	34	1	06/30/72	1930	298	.0700	1.30	.00	7.9	13000.	136.		155.	84.0	32.0
4	1	1	35	1	06/30/72	2030	299	.3600	1.30	.00	8.1	12000.	16.		18.	160.0	16.0
4	1	1	36	1	06/30/72	2130	300	.6900	1.35	.05			67.		50.		
4	1	1	37	1	06/30/72	2230	301		1.35	.00							
4	1	1	38	1	06/30/72	2330	302	.1800	1.35	.00	8.1		21.		29.	40.0	.0
4	1	1	39	1	07/01/72	30	303	.0900	1.35	.00	8.5		24.		25.	40.0	.0
4	1	1	40	1	07/01/72	130	304	.0600	1.35	.00	8.3		25.		17.	44.0	.0
4	1	1	41	1	07/01/72	230	305	.0900	1.35	.00	8.2	130000.	18.		14.	48.0	8.0
4	1	1	42	1	07/01/72	330	306	.1300	1.35	.00	8.1	90000.	21.		12.	44.0	4.0
4	1	1	43	1	07/01/72	430	307	.1400	1.35	.00	8.2	50000.	19.		12.	40.0	8.0
4	1	1	44	1	07/01/72	530	308	.2300	1.35	.00	8.3	140000.	17.		13.	44.0	12.0
4	1	1	45	1	07/01/72	630	309	.4900	1.35	.00	8.3	70000.	14.		12.	32.0	4.0
4	1	1	46	1	07/01/72	730	310	1.4100	1.35	.00	8.3	200000.	18.		10.	36.0	.0
4	1	1	47	1	07/01/72	830	311	1.2100	1.35	.00	8.3	230000.	17.		11.	40.0	4.0
4	1	1	48	1	07/01/72	930	312	1.5300	1.35	.00	8.3	170000.	17.		12.	44.0	4.0

STRM NO	PRIM LOC	SEC LOC	SEQ NO	TYPE	DATE	TIME	SAMPLE	TKN	NH3N	ORGN	NO2NO3	NO3N	NO2N	TALK	TIP	CL	FCOLI	FSTREP	TS	VS	TDS	VDS	BODS
4	1	1	1	1	06/30/72	345	226	.9	.20	.7	.01	.00	.013	40.	.19	8.	48000.	19000.	172.0	34.0	82.0	14.0	
4	1	1	2	1	06/30/72	400	227	.9	.25	.6	.03	.00	.026	41.	.13	10.	32000.	24000.	100.0	24.0	40.0	.0	
4	1	1	3	1	06/30/72	415	228	1.9	.25	1.6	.08	.05	.026	41.	.12	10.	33000.	22000.	162.0	26.0	87.0	21.0	
4	1	1	4	1	06/30/72	430.	229	.9	.15	.7	.08	.06	.016	41.	.10	7.	35000.	23000.	224.0	56.0	114.0	56.0	
4	1	1	5	1	06/30/72	445	230	.4	.15	.2	.07	.06	.014	41.	.09	6.	32000.	25000.	230.0	72.0	160.0	72.0	
4	1	1	6	1	06/30/72	500	231	1.2	.15	1.0	.06	.04	.016	42.	.11	7.	26000.	26000.	216.0	84.0	136.0	84.0	
4	1	1	7	1	06/30/72	515	232	.4	.30	.1	.06	.04	.021	49.	.09	8.	63000.	17000.	214.0	90.0	139.0	90.0	
4	1	1	8	1	06/30/72	530	233	.9	.15	.7	.07	.05	.021	53.	.10	10.	19000.	15000.	206.0	84.0	131.0	84.0	
4	1	1	9	1	06/30/72	545	234	.7	.35	.3	.06	.04	.021	58.	.09	11.	34000.	19000.	200.0	128.0	160.0	113.0	
4	1	1	10	1	06/30/72	600	235	.7	.48	.2	.08	.04	.036	57.	.18	16.	67000.	16000.	636.0	146.0	261.0	96.0	
4	1		11	1	06/30/72	615	236	1.2	.35	.8	.10	.07	.031	83.	.11	20.	34000.	34000.	324.0	28.0	154.0	.0	
4	1	1	12	1	06/30/72	630	237	1.3	.35	.9	.21	.18	.031	86.	.32	23.	8000.	1000.	352.0	66.0	192.0	46.0	
4	1	1	13	1	06/30/72	645	238	1.4	.50	.9	.11	.07	.036	81.	.36	25.	34000.	79000.	802.0	104.0	162.0	34.0	
4	1	1	14	1	06/30/72	700	239	2.2	.60	1.6	.18	.14	.044	116.	.50	32.	10000.	3000.	470.0	64.0	120.0	23.0	
4	1	1	15	1	06/30/72	715	240	2.9	1.40	1.5	.26	.20	.060	135.	.49	36.	38000.	44000.	656.0	150.0	116.0	110.0	
4	1	1	16	1	06/30/72	730	241	3.2	1.68	1.5	.19	.15	.040	149.	.44	44.	162000.	150000.	472.0	108.0	162.0	78.0	
4	1	1	17	1	06/30/72	745	242	2.7	1.28	1.4	.25	.17	.075	163.	.25	46.	178000.	22000.	464.0	130.0	174.0	130.0	
4	1	1	18	1	06/30/72	800	243	2.7	2.30	.4	.25	.16	.094	194.	.60	52.	22000.	47000.	416.0	138.0	236.0	138.0	
4	1	1	19	1	06/30/72	815	244	4.2	1.90	2.3	.31	.25	.060	190.	2.49	54.	207000.	77000.	400.0	126.0	229.0	126.0	
4	1	1	20	1	06/30/72	830	245	4.0	1.55	2.4	.35	.29	.060	192.	.65	55.	150000.	119000.	406.0	112.0	236.0	112.0	
4	1	1	21	1	06/30/72	845	246	4.3	2.50	1.8	.31	.25	.057	192.	1.40	53.	164000.	490000.	376.0	112.0	271.0	112.0	
4	1	1	22	1	06/30/72	900	247	3.3	1.05	2.2	.33	.27	.063	157.	.72	45.	35000.	62000.	574.0	166.0	424.0	166.0	
4	1	1	23	1	06/30/72	915	248	3.9	.88	3.0	.32	.24	.081	168.	1.21	52.	42000.	99000.	816.0	204.0	526.0	204.0	
4	1	1	24	1	06/30/72	930	249	4.8	1.40	3.4	.33	.24	.087	180.	.70	57.	31000.	52000.	764.0	164.0	474.0	164.0	
4	1	1	25	1	06/30/72	1030	280	5.3	1.65	3.6	.35	.28	.068	182.	.47	58.	800.	50000.	574.0	148.0	386.0	120.0	
4	1	1	26	1	06/30/72	1130	281	6.8	1.65	5.1	1.47	1.22	.250	197.	1.24	64.	11000.	70000.	588.0	376.0	412.0	368.0	
4	1	1	27	1	06/30/72	1230	282	5.7	.30	5.4	.95	.68	.266	185.	.95	54.	18000.	95000.	520.0	336.0	384.0	324.0	
4	1	1	28	1	06/30/72	1330	283	7.7	.35	7.3	.51	.40	.106	193.	.76	66.	400.	13000.	417.0	120.0	405.0	116.0	
4	1	1	29	1	06/30/72	1430	284	7.2	.98	6.2	.64	.52	.117	212.	1.81	69.	700.	14000.	800.0	629.0	766.0	628.0	
4	1	1	30	1	06/30/72	1530	285																
4	1	1	31	1	06/30/72	1630	295	5.6	.75	4.8	.60	.50	.097	198.	.76	77.	3000.	26000.					
4	1	1	32	1	06/30/72	1730	296	5.6	.75	4.8	.90	.51	.386	203.	.79	79.	4000.	21000.	702.0	140.0	398.0	126.0	
4	1	1	33	1	06/30/72	1830	297	6.4	.75	5.6	2.71	1.90	.813	224.	1.06	94.	16000.	41000.	510.0	80.0	402.0	52.0	
4	1	1	34	1	06/30/72	1930	298	3.1	.35	2.7	.16	.13	.031	179.	.28	83.	5000.	52000.	508.0	210.0	424.0	178.0	
4	1	1	35	1	06/30/72	2030	299	2.1	.15	1.9	.23	.16	.073	213.	.24	67.	6000.	137000.	540.0	48.0	380.0	32.0	
4	1	1	36	1	06/30/72	2130	300	3.0	.15	2.8	2.07	1.84	.328	155.	.19	54.							
4	1	1	37	1	06/30/72	2230	301																
4	1	1	38	1	06/30/72	2330	302	4.1	.58	3.5	.81	.66	.148	175.	5.55	69.		88000.	350.0	44.0	310.0	44.0	
4	1	1	39	1	07/01/72	30	303	3.4	.25	3.1	.71	.62	.094	242.	4.37	91.	4000.	11000.	400.0	74.0	360.0	74.0	
4	1	1	40	1	07/01/72	130	304	3.6	.60	3.0	1.56	1.47	.094	231.	9.24	103.	9000.	21000.	536.0	102.0	492.0	102.0	
4	1	1	41	1	07/01/72	230	305	3.1	.55	2.5	1.54	1.45	.089	246.	4.88	97.	18000.	20000.	548.0	86.0	500.0	78.0	
4	1	1	42	1	07/01/72	330	306	2.9	.50	2.4	1.48	1.39	.092	257.	3.01	92.	58000.	15000.	544.0	114.0	500.0	110.0	
4	1	1	43	1	07/01/72	430	307	2.8	.55	2.2	1.56	1.46	.099	275.	2.71	93.	3000.	14000.	590.0	164.0	550.0	156.0	
4	1	1	44	1	07/01/72	530	308	2.5	.35	2.1	1.43	1.35	.078	271.	2.03	81.	6000.	7000.	546.0	118.0	502.0	106.0	
4	1	1	45	1	07/01/72	630	309	3.3	.32	3.0	1.15	1.10	.052	261.	1.15	71.	6000.	5000.	562.0	130.0	530.0	126.0	
4	1	1	46	1	07/01/72	730	310	3.1	.25	2.8	.30	.27	.031	253.	.56	66.	6000.	10000.	542.0	136.0	506.0	136.0	
4	1	1	47	1	07/01/72	830	311	3.1	.20	2.9	.29	.25	.036	257.	.60	66.	14000.	13000.	532.0	158.0	492.0	154.0	
4	1	1	48	1	07/01/72	930	312	2.2	.20	2.0	.24	.21	.031	257.	.52	68.	12000.	14000.	480.0	122.0	436.0	118.0	

Syracuse Combined Sewer Overflow Study
West Newell Street Location
Preliminary Monitoring Data

STRM NO	PRIM LOC	SEC LOC	SEQ NO	TYPE	DATE	TIME	SAMPLE	FLOWRATE	RAINACC	RAININTS	PH	TCOLI	TOC	COD-M	COD	TSS	VSS
22	2	1	1	1	09/22/72	600	879	.1180	.10	.02	8.7	24400000.	73.		110.	110.0	88.0
22	2	1	2	1	09/22/72	615	880	.0780	.10	.00	9.1	77000000.	76.		140.	90.0	68.0
22	2	1	3	1	09/22/72	630	881	.0780	.10	.00	9.3	9300000.	78.		90.	116.0	84.0
22	2	1	4	1	09/22/72	645	882	.0970	.10	.00	9.1	11100000.	69.		100.	96.0	72.0
22	2	1	5	1	09/22/72	700	883	.1120	.10	.00	9.4	34000000.	71.		130.	92.0	64.0
22	2	1	6	1	09/22/72	715	884	.0970	.10	.00	8.9	11000000.	77.		80.	106.0	84.0
22	2	1	7	1	09/22/72	730	885	.0650	.10	.00	9.5	91000.	75.		110.	88.0	68.0
22	2	1	8	1	09/22/72	745	886	.0310	.10	.00	9.1	132000.	76.		100.	136.0	100.0
22	2	1	9	1	09/22/72	800	887	.1250	.10	.00	9.7	830000.	87.		230.	260.0	144.0
22	2	1	10	1	09/22/72	815	888	.1250	.10	.00	9.8	90000.	72.		129.	240.0	128.0
22	2	1	11	1	09/22/72	830	889	.1250	.10	.00	9.6	4290000.	70.		120.	164.0	104.0
22	2	1	12	1	09/22/72	845	890	.1250	.10	.00	9.8	5300.	70.		130.	152.0	80.0
22	2	1	13	1	09/22/72	900	891	.0970	.10	.00	9.8	1700.	83.		240.	100.0	76.0
22.	2	1	14	1	09/22/72	915	892	.0640	.10	.00	8.8		46.		210.	68.0	44.0
22	2	1	15	1	09/22/72	930	893	.0460	.10	.00		30000000.	97.				
22	2	1	16	1	09/22/72	945	894	.0950	.10	.00	9.8	100.	84.		180.	104.0	64.0
22	2	1	17	1	09/22/72	1000	895	.0830	.10	.00	9.3		68.		100.	84.0	72.0
22	2	1	18	1	09/22/72	1015	896	.0570	.10	.00	10.1		115.		300.	327.0	240.0
22	2	1	19	1	09/22/72	1030	897	.0920	.10	.00	10.2	140.	80.		220.	100.0	68.0
22	2	1	20	1	09/22/72	1045	898	.0920	.10	.00	10.1	1160.	84.		220.	68.0	48.0
22	2	1	21	1	09/22/72	1100	899	.0610	.10	.00	9.5		96.		170.	108.0	80.0
22	2	1	22	1	09/22/72	1115	900	.0290	.10	.00	8.2		75.		170.	80.0	60.0
22	2	1	23	1	09/22/72	1130	901	.0100	.10	.00	8.9		73.		210.	88.0	68.0
22	2	1	24	1	09/22/72	1145	902	.1080	.10	.00	10.0		98.		200.	133.0	93.0

STRM NO	PRIM LOC	SEC LOC	SEQ NO	TYPE	DATE	TIME	SAMPLE	TKN	NH3N	ORGN	NO2NO3	NO3N	NO2N	TALK	TIP	CL	FCOLI	FSTREP	TS	VS	TDS	VDS	BOD5
22	2	1	1	1	09/22/72	600	879	11.0	6.02	5.0	.02	.01	.012	490.	4.26	51.	18000000.	43000.	900.0	244.0	790.0	156.0	60.0
22	2	1	2	1	09/22/72	615	880	13.6	2.32	11.3	.02	.01	.012	510.	2.10	55.	200000.	22000.	798.0	248.0	708.0	180.0	78.0
22	2	1	3	1	09/22/72	630	881	11.2	4.61	6.6	.02	.01	.010	480.	2.16	58.	4000000.	26000.	746.0	194.0	630.0	110.0	59.0
22	2	1	4	1	09/22/72	645	882	11.0	4.89	6.1	.02	.01	.010	520.	1.94	58.	900000.	2000.	776.0	162.0	680.0	90.0	54.0
22	2	1	5	1	09/22/72	700	883	17.3	4.48	12.8	.26	.02	.237	570.	2.63	68.	600000.	1000.	888.0	178.0	796.0	114.0	57.0
22	2	1	6	1	09/22/72	715	884	15.5	4.48	11.0	.09	.00	.089	489.	2.26	74.	880000.	100.	860.0	168.0	752.0	84.0	53.0
22	2	1	7	1	09/22/72	730	885	12.7	3.48	9.2	.43	.05	.382	600.	1.79	66.	50000.	4000.	836.0	186.0	748.0	118.0	59.0
22	2	1	8	1	09/22/72	745	886	14.1	8.16	5.9	.04	.03	.012	488.	1.82	113.	87000.	26600.	728.0	168.0	592.0	68.0	77.0
22	2	1	9	1	09/22/72	800	887	11.7	7.36	4.3	.39	.03	.358	570.	1.61	340.	15000.	86000.	1444.0	326.0	1184.0	182.0	101.0
22	2	1	10	1	09/22/72	815	888	11.5	4.69	6.8	.43	.27	.162	492.	1.23	334.	30000.	39000.	1378.0	328.0	1138.0	200.0	55.0
22	2	1	11	1	09/22/72	830	889	17.2	11.40	5.8	.38	.06	.321	610.	1.91	166.	150000.	64000.	1130.0	328.0	966.0	224.0	70.0
22	2	1	12	1	09/22/72	845	890	13.9	8.11	5.8	.63	.47	.159	740.	2.49	153.	600.	122000.	1140.0	230.0	988.0	150.0	75.0
22	2	1	13	1	09/22/72	900	891	16.7	8.47	8.2	.48	.39	.086	540.	3.03	155.	1000.	60.	914.0	182.0	814.0	106.0	91.0
22	2	1	14	1	09/22/72	915	892	20.1	9.50	10.6	.40	.35	.045	384.	2.86	117.			622.0	100.0	554.0	56.0	
22	2	1	15	1	09/22/72	930	893										4870000.	820.					
22	2	1	16	1	09/22/72	945	894	10.6	5.39	5.2	.48	.40	.084	610.	2.10	92.	100.	20.	968.0	192.0	864.0	128.0	78.0
22	2	1	17	1	09/22/72	1000	895	11.7	6.10	5.6	.44	.37	.066	381.	2.17	79.			704.0	194.0	620.0	122.0	67.0
22	2	1	18	1	09/22/72	1015	896	13.1	6.30	6.8	.48	.39	.091	770.	3.43	91.			1338.0	472.0	1011.0	232.0	201.0
22	2	1	19	1	09/22/72	1030	897	8.2	4.54	3.7	.46	.38	.081	710.	1.96	68.	0.	450.	980.0	264.0	880.0	196.0	93.0
22	2	1	20	1	09/22/72	1045	898	10.2	4.23	6.0	.49	.40	.091	610.	2.49	75.	0.	800.	960.0	320.0	892.0	272.0	101.0
22	2	1	21	1	09/22/72	1100	899	10.0	5.51	4.5	.40	.32	.076	367.	3.57	68.			724.0	272.0	616.0	192.0	120.0
22	2	1	22	1	09/22/72	1115	900	8.2	5.52	2.7	.39	.36	.025	291.	3.19	98.			690.0	202.0	610.0	142.0	75.0
22	2	1	23	1	09/22/72	1130	901	7.9	4.79	3.1	.31	.20	.111	362.	3.00	160.			868.0	250.0	780.0	182.0	70.0
22	2	1	24	1	09/22/72	1145	902	9.1	3.53	5.6	.52	.41	.111	700.	2.97	76.			1164.0	428.0	1031.0	335.0	120.0

Syracuse Combined Sewer Overflow Study
Rowland Street Location
Preliminary Monitoring Data

STRM NO	PRIM LOC	SEC LOC	SEQ NO	TYPE	DATE	TIME	SAMPLE	FLOWRATE	RAINACC	RAININTS	PH	TCOLI	TOC	COD-M	COD	TSS	VSS
4	3	1	1	1	06/30/72	415	253		.65	.40	7.1	240000.	17.		12.	100.0	.0
4	3	1	2	1	06/30/72	430	254	10.1000	.95	.40	7.4	780000.	13.		5.	125.0	5.0
4	3	1	3	1	06/30/72	445	255	11.5300	.95	.00	7.6	620000.	10.		5.	115.0	.0
4	3	1	4	1	06/30/72	500	256	10.1000	1.05	.40	7.6	500000.	8.		3.	135.0	5.0
4	3	1	5	1	06/30/72	515	257	10.1000	1.05	.00	7.7	1090000.	10.		3.	125.0	.0
4	3	1	6	1	06/30/72	530	258	8.0800	1.05	.00	7.9	810000.	10.		4.	120.0	.0
4	3	1	7	1	06/30/72	545	259	10.1000	1.10	.20	7.4	130000.	12.		5.	125.0	.0
4	3	1	8	1	06/30/72	600	260	10.1000	1.10	.00	7.7	180000.	12.		4.	182.0	120.0
4	3	1	9	1	06/30/72	615	261	8.9800	1.10	.00	7.9	140000.	11.		3.	110.0	.0
4	3	1	10	1	06/30/72	630	262	8.0800	1.10	.00	8.0	70000.	10.		7.	175.0	45.0
4	3	1	11	1	06/30/72	645	263	6.7300	1.10	.00	8.0	180000.	11.		4.	185.0	40.0
4	3	1	12	1	06/30/72	700	264	5.7800	1.10	.00	7.4	100000.	13.		8.	165.0	40.0
4	3	1	13	1	06/30/72	715	265	5.7800	1.10	.00	7.8	70000.	11.		8.	145.0	.0
4	3	1	14	1	06/30/72	730	266	5.0600	1.10	.00	7.9	150000.	13.		11.	140.0	35.0
4	3	1	15	1	06/30/72	745	267	4.0400	1.10	.00	7.9	40000.	14.		12.	135.0	25.0
4	3	1	16	1	06/30/72	800	268	3.3600	1.10	.00	8.0	220000.	13.		11.	115.0	30.0
4	3	1	17	1	06/30/72	815	269	3.3600	1.10	.00	7.6	240000.	12.		13.	110.0	35.0
4	3	1	18	1	06/30/72	830	270	2.7900	1.15	.20	7.5		16.				
4	3	1	19	1	06/30/72	845	271	3.1100	1.15	.00	7.6	18000.	15.		15.	105.0	35.0
4	3	1	20	1	06/30/72	900	272	3.1100	1.15	.00	7.2	12000.	14.		18.	125.0	40.0
4	3	1	21	1	06/30/72	915	273	3.1100	1.15	.00	7.7	19000.	12.		12.	120.0	35.0
4	3	1	22	1	06/30/72	930	274	3.1100	1.15	.00	7.8	9000.	13.		18.	100.0	25.0
4	3	1	23	1	06/30/72	945	275	3.1100	1.15	.00	7.9	25000.	12.		16.	140.0	85.0
4	3	1	24	1	06/30/72	1000	276	3.1100	1.15	.00	7.8	7000.	14.		17.	145.0	55.0
4	3	1	25	1	06/30/72	1100	316	2.0400	1.15	.00	7.8	120000.	17.		19.	33.0	13.0
4	3	1	26	1	06/30/72	1200	317	1.8000	1.15	.00	6.9	120000.	18.		17.	23.0	20.0
4	3	1	27	1	06/30/72	1300	318	1.8000	1.15	.00	7.5	90000.	18.		17.	20.0	18.0
4	3	1	28	1	06/30/72	1400	319	1.8000	1.15	.00	7.6	110000.	23.		15.	20.0	7.0
4	3	1	29	1	06/30/72	1500	320	1.7000	1.20	.05	7.6	130000.	22.		19.	15.0	10.0
4	3	1	30	1	06/30/72	1600	321	1.7000	1.20	.00	7.5	60000.	21.		16.	27.0	20.0
4	3	1	31	1	06/30/72	1700	322	1.8000	1.20	.00	7.5	290000.	24.		17.	19.0	10.0
4	3	1	32	1	06/30/72	1800	323	1.7000	1.20	.00	7.6	330000.	26.		20.	18.0	11.0
4	3	1	33	1	06/30/72	1900	324	1.8000	1.20	.00	7.8	260000.	23.		17.	27.0	13.0
4	3	1	34	1	06/30/72	2000	325	1.7500	1.20	.00	7.9	200000.	26.		19.	21.0	12.0
4	3	1	35	1	06/30/72	2100	326	1.9100	1.20	.00	7.7	220000.	26.		21.	32.0	11.0
4	3	1	36	1	06/30/72	2200	327	1.7500	1.20	.00	7.6	330000.	25.		18.	10.0	3.0
4	3	1	37	1	06/30/72	2300	328	1.7500	1.20	.00	7.7	260000.	19.		17.	10.0	5.0
4	3	1	38	1	06/30/72	2359	329	1.7000	1.20	.00	8.0	190000.	16.		14.	17.0	11.0
4	3	1	39	1	07/01/72	100	330	1.4500	1.20	.00							
4	3	1	40	1	07/01/72	200	331	1.7500	1.20	.00	8.0	400000.	12.		12.	14.0	4.0
4	3	1	41	1	07/01/72	300	332	1.5300	1.20	.00	7.8	230000.	20.		12.	12.0	5.0
4	3	1	42	1	07/01/72	400	333	1.4500	1.20	.00	7.8	290000.	13.		10.	15.0	7.0
4	3	1	43	1	07/01/72	500	334	1.4500	1.20	.00	7.8	440000.	16.		9.	17.0	10.0
4	3	1	44	1	07/01/72	600	335		1.20	.00	8.1	410000.	15.		10.	13.0	3.0
4	3	1	45	1	07/01/72	700	336		1.20	.00	8.0	190000.	17.		11.	14.0	6.0
4	3	1	46	1	07/01/72	800	337		1.20	.00	8.0	1020000.	20.		13.	10.0	3.0
4	3	1	47	1	07/01/72	900	338		1.20	.00	8.0	1560000.	19.		18.	18.0	5.0
4	3	1	48	1	07/01/72	1000	339		1.20	.00	7.9	920000.	23.		23.	20.0	10.0

STRM NO	PRIM LOC	SEC LOC	SEQ NO	TYPE	DATE	TIME	SAMPLE	TKN	NH3N	ORGN	NO2NO3	NO3N	NO2N	TALK	TIP	CL	FCOLI	FSTREP	TS	VS	TDS	VDS	BOD5
4	3	1	1	1	06/30/72	415	253	1.0	.15	.8	.17	.16	.010	137.	.13	40.	17000.	7000.	156.0	68.0	56.0	68.0	
4	3	1	2	1	06/30/72	430	254	.8	.20	.6	.08	.06	.016	120.	.14	25.	9000.	7000.	154.0	34.0	29.0	29.0	
4	3	1	3	1	06/30/72	445	255	1.2	.25	.9	.11	.09	.016	125.	.12	23.	11000.	3000.	1740.0	1440.0	1624.3	1440.4	
4	3	1	4	1	06/30/72	500	256	1.2	.25	.9	.11	.10	.014	126.	.11	27.	7000.	6000.	160.0	136.0	25.0	131.0	
4	3	1	5	1	06/30/72	515	257	1.1	.30	.8	.11	.09	.016	149.	.13	33.	260000.	11000.	264.0	126.0	139.0	126.0	
4	3	1	6	1	06/30/72	530	258	1.2	.32	.9	.14	.12	.016	159.	.11	37.	80000.	5000.	254.0	110.0	134.0	110.0	
4	3	1	7	1	06/30/72	545	259	1.0	.28	.7	.15	.13	.019	150.	.12	32.	23000.	1000.	164.0	164.0	39.0	164.0	
4	3	1	8	1	06/30/72	600	260	1.4	.25	1.1	.14	.12	.019		.12	36.	9000.	2500.	300.0	130.0	118.0	10.0	
4	3	1	9	1	06/30/72	615	261	1.0	.25	.7	.17	.15	.019		.13		12000.	3100.	328.0	152.0	218.0	152.0	
4	3	1	10	1	06/30/72	630	262	1.3	.40	.9	.18	.16	.021		.16		16000.	15000.	376.0	158.0	201.0	113.0	
4	3	1	11	1	06/30/72	645	263	1.9	.55	1.3	.21	.19	.021		.19		8000.	12000.	416.0	154.0	231.0	114.0	
4	3	1	12	1	06/30/72	700	264	2.0	.65	1.3	.22	.20	.021		.22		4000.	4000.	380.0	232.0	215.0	192.0	
4	3	1	13	1	06/30/72	715	265	2.5	.85	1.6	.23	.20	.026		.29		22000.	800.	432.0	158.0	287.0	158.0	
4	3	1	14	1	06/30/72	730	266	2.5	1.08	1.4	.26	.23	.029		.27		12000.	14000.	450.0	56.0	310.0	21.0	
4	3	1	15	1	06/30/72	745	267	3.5	2.02	1.5	.28	.25	.029	274.	.53	78.	1000.	2000.	482.0	80.0	347.0	55.0	
4	3	1	16	1	06/30/72	800	268	4.6	3.83	.8	.30	.27	.031	285.	.53	82.	13000.	1000.	534.0	70.0	419.0	40.0	
4	3	1	17	1	06/30/72	815	269	5.2	3.52	1.7	.33	.30	.031	232.	.57	81.	3000.	8000.	524.0	82.0	414.0	47.0	
4	3	1	18	1	06/30/72	830	270	4.5	2.30	2.2	.34	.31	.031	239.	.51	85.							
4	3	1	19	1	06/30/72	845	271	3.9	2.70	1.2	.34	.27	.065	234.	.81	85.	9000.	10000.	566.0	220.0	461.0	185.0	
4	3	1	20	1	06/30/72	900	272	4.2	2.70	1.5	.39	.34	.050	236.	.73	85.	3000.	3000.	554.0	200.0	429.0	160.0	
4	3	1	21	1	06/30/72	915	273	4.0	2.62	1.4	.38	.32	.060	238.	.54	86.	9000.	7000.	540.0	86.0	420.0	51.0	
4	3	1	22	1	06/30/72	930	274	4.8	3.70	1.1	.39	.30	.085	247.	1.08	88.	2000.	7000.	572.0	128.0	472.0	103.0	
4	3	1	23	1	06/30/72	945	275	4.4	3.12	1.3	.46	.34	.117	246.	.95	88.	10000.	8000.	564.0	90.0	424.0	5.0	
4	3	1	24	1	06/30/72	1000	276	4.4	3.02	1.4	.52	.41	.109	253.	1.26	88.	2000.	2000.	600.0	148.0	455.0	93.0	
4	3	1	25	1	06/30/72	1100	316	4.7	2.28	2.4	.31	.27	.040	255.	1.19	88.			502.0	102.0	469.0	69.0	
4	3	1	26	1	06/30/72	1200	317	4.1	1.93	2.2	.34	.30	.036	253.	1.22	89.			524.0	110.0	501.0	90.0	
4	3	1	27	1	06/30/72	1300	318	3.9	1.43	2.5	.34	.30	.036	255.	.86	92.			536.0	136.0	516.0	118.0	
4	3	1	28	1	06/30/72	1400	319	3.4	1.30	2.1	.34	.30	.036	259.	1.06	92.			532.0	262.0	512.0	255.0	
4	3	1	29	1	06/30/72	1500	320	3.4	1.26	2.1	.38	.33	.054	257.	1.77	92.			540.0	278.0	525.0	268.0	
4	3	1	30	1	06/30/72	1600	321	4.0	1.15	2.8	.38	.33	.054	259.	.94	93.			514.0	338.0	487.0	318.0	
4	3	1	31	1	06/30/72	1700	322	4.1	1.68	2.4	.35	.32	.031	259.	.53	92.			468.0	238.0	449.0	228.0	
4	3	1	32	1	06/30/72	1800	323	2.8	1.65	1.1	.50	.40	.104	259.	.48	93.			528.0	344.0	510.0	333.0	
4	3	1	33	1	06/30/72	1900	324	2.3	1.68	.6	.41	.36	.052	257.	.54	88.			538.0	368.0	511.0	355.0	
4	3	1	34	1	06/30/72	2000	325	2.7	2.20	.5	.35	.30	.047	257.	.50	93.			524.0	280.0	503.0	268.0	
4	3	1	35	1	06/30/72	2100	326	3.6	2.50	1.1	.36	.16	.200	249.	.37	89.			498.0	218.0	466.0	207.0	
4	3	1	36	1	06/30/72	2200	327	2.2	1.30	.9	.34	.30	.041	247.	.52	89.			480.0	102.0	470.0	99.0	
4	3	1	37	1	06/30/72	2300	328	2.8	1.78	1.0	.35	.32	.031	261.	.73	92.			516.0	134.0	506.0	129.0	
4	3	1	38	1	06/30/72	2359	329	3.4	2.45	.9	.34	.31	.031	269.	.39	93.			566.0	248.0	549.0	237.0	
4	3	1	39	1	07/01/72	100	330																
4	3	1	40	1	07/01/72	200	331	2.4	.65	1.7	.34	.32	.020	260.	.19	92.			528.0	160.0	514.0	156.0	
4	3	1	41	1	07/01/72	300	332	1.5	.70	.8	.34	.32	.020	253.	.29	91.			508.0	98.0	496.0	93.0	
4	3	1	42	1	07/01/72	400	333	1.4	.50	.9	.35	.33	.020	257.	.11	93.			488.0	106.0	473.0	99.0	
4	3	1	43	1	07/01/72	500	334	1.9	.50	1.4	.35	.33	.016	259.	.10	93.			564.0	118.0	547.0	108.0	
4	3	1	44	1	07/01/72	600	335	1.9	1.02	.9	.35	.33	.021	263.	.15	92.			562.0	230.0	549.0	227.0	
4	3	1	45	1	07/01/72	700	336	2.5	1.08	1.4	.35	.32	.026	261.	.12	92.			546.0	150.0	532.0	144.0	
4	3	1	46	1	07/01/72	800	337	4.2	3.65	.6	.36	.33	.029	269.	.46	91.			544.0	196.0	534.0	193.0	
4	3	1	47	1	07/01/72	900	338	6.7	4.25	2.5	.41	.36	.045	269.	.81	90.			578.0	218.0	560.0	213.0	
4	3	1	48	1	07/01/72	1000	339	5.2	4.30	.9	.47	.41	.063	273.	1.81	89.			594.0	232.0	574.0	222.0	

APPENDIX B

Chlorine Dioxide Analytical Data

APPENDIX B LEGEND

Actual Log Kill	-	Log (base 10) reduction of fecal coliform
ClO_2 Dose	-	Dosage of ClO_2 injected into CSO, mg/l
NH_3N	-	Ammonia nitrogen concentration at point of disinfectant injection, mg/l
Mixing Intensity GT	-	Mixing intensity expressed as a product of the velocity gradient (G, sec^{-1}) and detention time (T, sec), dimensionless
Influent SS	-	Suspended solids concentration at point of disinfectant injection, mg/l
pH	-	Log of the hydrogen ion concentration, dimensionless
Influent FC	-	Fecal coliform concentration at point of disinfectant injection, cells/100 ml
Predicted Log Kill	-	Log (base 10) of the number of fecal coliform killed by ClO_2 disinfection as predicted by the developed performance models
Residual	-	Difference between the predicted log kill and the actual log kill.

Analytical Data for ClO_2 Disinfection Tests

Sample No.	Actual Logkill FC	ClO_2 Dose, mg/l	NH_3N mg/l	Mixing Intensity, GT	Influent SS, mg/l	pH	Influent FC Count/100 ml	Predicted Logkill FC	Residual
SWEC02 1	.7	6.1	1.34	1000	840	7.0	105500	2.0	-1.3
SWEC02 3	.3	6.1	.17	1000	400	7.2	36000	2.0	-1.7
SWEC02 4	1.3	6.1	.89	3180	283	7.3	22000	2.2	-.9
SWEC02 5	1.5	6.1	1.01	3180	315	7.3	14500	2.2	-.7
SWEC02 6	1.6	6.1	1.01	3180	232	7.2	41000	2.3	-.7
SWEC02 7	2.3	6.6	1.20	2840	190	7.2	48000	2.5	-.2
SWEC02 8	3.1	6.1	1.11	3180	202	7.2	14500	2.2	.9
SWEC02 9	.4	1.6	1.73	1000	247	7.1	38000	.8	-.4
SWEC02 10	.7	1.6	1.68	1000	122	7.1	76000	.9	-.2
SWEC02 11	3.6	4.3	1.66	3885	110	7.1	164667	2.1	1.5
SWEC02 12	1.8	4.5	1.79	4167	122	7.2	122000	2.2	-.4
SWEC02 15	.3	3.2	1.44	2000	135	7.0	52000	1.5	-1.2
SWEC02 16	.9	3.3	1.56	2000	117	7.1	60000	1.6	-.7
SWEC03 3	.3	.3	1.53	2000	140	6.4	117500	.3	.0
SWEC03 4	.3	.4	1.30	2000	139	6.6	25000	.3	.0
SWEC03 8	.5	.4	1.24	2000	136	6.8	23000	.4	.1
SWEC03 9	.8	.3	1.21	2000	144	7.0	25000	.3	.5
SWEC03 10	1.1	6.5	1.13	2000	88	7.0	60000	2.5	-1.4
SWEC03 11	1.7	4.7	1.02	3375	160	6.8	151667	2.1	-.4
SWEC03 12	1.6	4.5	1.30	3620	236	6.5	131500	2.0	-.4
SWEC06 4	1.6	7.6	3.90	3933	143	5.8	1175000	3.2	-1.6
SWEC06 5	1.6	5.5	3.45	3307	172	5.6	1025000	2.5	-.9
SWEC06 6	1.8	5.9	3.45	3429	132	5.6	3400000	2.9	-1.1
SWEC06 7	2.1	6.2	2.95	3410	100	6.2	895000	2.9	-.8
SWEC06 8	1.5	6.5	3.45	3429	96	5.9	725000	2.9	-1.4
SWEC06 9	2.3	5.7	4.80	3410	300	5.1	3800000	2.6	-.3
SWEC06 10	2.9	6.0	3.55	3047	404	5.0	2045000	2.5	.4
SWEC06 11	2.8	6.3	2.20	3484	540	5.8	1790000	2.6	.2
SWEC06 12	2.0	6.7	1.85	3516	576	5.8	1700000	2.7	-.7
SWEC06 13	3.5	6.7	1.50	3516	555	6.2	2990000	2.9	.6
SWEC06 14	4.0	6.7	1.35	3516	452	6.2	5500000	3.0	1.0
SWEC06 15	4.1	6.7	.90	3516	412	6.5	3800000	3.0	1.1
SWEC06 16	4.2	6.7	.80	3516	305	6.9	5100000	3.2	1.0
SWEC06 17	3.9	6.7	.70	3516	180	6.6	2200000	3.1	.8
SWEC06 18	5.0	6.1	.55	3522	184	6.6	1425000	2.8	2.2
SWEC06 19	5.0	6.7	.60	3662	252	6.5	1280000	2.9	2.1
SWEC06 20	5.0	7.0	.55	3704	124	6.5	930000	3.1	1.9
SWEC06 21	3.5	5.6	1.30	3358	116	6.5	735000	2.7	.8
SWEC06 22	3.3	5.7	.70	3324	120	6.4	515000	2.6	.7
SWEC06 23	3.8	6.1	.75	3429	268	6.7	925000	2.7	1.1
SWEC06 24	3.5	6.7	.70	3541	468	6.5	1945000	2.9	.6
SWEC08 1	1.0	3.3	.51	2170	480	6.9	1210000	1.7	-.7
SWEC08 3	1.4	3.6	2.29	2157	641	6.8	4900000	1.9	-.5
SWEC08 4	1.1	3.7	.34	2157	634	6.6	3940000	1.9	-.8
SWEC08 5	1.5	3.3	.46	2157	534	6.6	8300000	1.8	-.3
SWEC08 6	2.5	3.3	.52	2157	540	6.6	7900000	1.8	.7
SWEC08 7	2.2	3.6	.51	2157	407	6.7	7900000	2.0	.2
SWEC08 8	2.4	3.7	.49	2157	527	6.6	7500000	2.0	.4
SWEC08 9	2.6	3.3	.54	2157	393	6.6	8900000	1.9	.7
SWEC08 10	2.4	3.4	.40	2177	334	6.5	6600000	1.9	.5
SWEC08 11	3.0	3.7	.97	2204	327	6.6	9900000	2.1	.9
SWEC09 3	2.2	5.2	.53	2344	499	6.2	535000	2.1	.1
SWEC09 4	2.9	4.5	.26	2005	507	6.0	1900000	2.0	.9
SWEC09 5	1.9	4.0	.34	1994	200	6.1	160000	1.7	.2
SWEC09 6	2.0	3.9	.35	1905	340	5.9	480000	1.7	.3
SWEC09 7	2.3	4.0	.26	1885	161	5.8	740000	1.9	.4
SWEC09 8	3.4	4.1	.30	1865	280	5.4	515000	1.8	1.6
SWEC09 9	3.1	3.6	.30	1837	220	5.9	245000	1.6	1.5
SWEC09 10	2.7	3.7	.30	1837	267	6.1	163667	1.6	1.1
SWEC09 11	2.3	3.9	.28	1851	280	6.2	415000	1.8	.5
SWEC09 14	3.1	5.8	.22	2496	227	6.6	130000	2.3	.8
SWEC09 15	3.0	6.3	.50	2581	86	6.4	99333	2.5	.5
SWEC09 16	3.5	5.7	.65	2620	187	5.9	510000	2.4	1.1
SWEC09 17	3.0	5.9	.51	2620	240	5.8	101500	2.2	.8
SWEC010 5	1.2	7.7	.08	1837	114	6.9	32000	2.7	-1.5
SWEC010 6	3.1	8.0	.12	3484	93	6.8	17000	2.8	.3
SWEC010 7	4.0	8.3	.06	3484	54	6.8	140000	3.4	.6
SWEC010 8	3.3	8.7	.04	3484	104	6.8	39500	3.1	.2
SWEC010 9	3.1	7.7	.06	3484	67	7.0	19500	2.9	.2
SWEC010 10	2.0	8.0	.06	3484	52	6.9	266667	3.5	-1.5
SWEC010 11	3.4	8.3	.10	3484	56	6.8	43500	3.2	.2
SWEC010 12	3.8	8.7	.09	3484	92	6.9	83000	3.3	.5
SWEC010 13	3.4	7.7	.01	3484	16	6.9	46000	3.3	.1
SWEC010 14	3.4	8.0	.01	3484	21	6.9	52000	3.4	.0
SWEC010 15	3.6	8.3	.01	3484	118	6.9	64500	3.1	.5
SWEC010 16	3.9	4.9	.01	2170	129	6.9	93667	2.1	1.8
SWEC010 17	4.0	4.4	.02	2170	157	7.1	106000	2.0	2.0
SWEC010 18	3.4	4.5	.09	2170	192	7.0	46000	1.9	1.5
SWEC010 19	4.0	4.7	.07	2170	216	7.0	147333	2.0	2.0
SWEC010 20	3.1	4.9	.04	2170	128	6.9	24000	1.9	1.2
SWEC010 21	3.7	4.4	.04	2170	106	7.0	74000	2.0	1.7
SWEC010 22	2.0	4.5	.07	2170	192	7.2	25000	1.8	.2
SWEC010 23	3.2	4.7	.01	2170	119	7.0	34000	1.9	1.3
SWEC010 24	3.3	4.9	.01	2170	162	7.2	40500	2.0	1.3
SWEC011 4	1.9	5.5	.01	3033	122	6.9	50000	2.3	-.4
SWEC011 5	2.9	5.5	.34	3033	128	7.0	149000	2.4	.5
SWEC011 6	2.8	5.5	.06	3033	106	7.0	114300	2.4	.4
SWEC011 7	3.8	5.7	.08	3133	98	7.0	100000	2.5	1.3

SWECOS	1	2.0	7.1	.66	5100	36	7.0	1000	2.5	-.5
SWECOS	2	2.0	7.4	1.22	5100	9	6.8	1000	2.8	-.8
SWECOS	3	3.4	7.8	1.15	5100	9	6.6	47000	3.6	-.2
SWECOS	4	3.4	8.1	.82	5100	10	6.6	52000	3.6	-.2
SWECOS	5	4.0	7.1	1.09	5100	92	6.6	137500	3.0	1.0
SWECOS	6	4.0	7.4	.91	5100	81	6.7	100333	3.1	.9
SWECOS	7	2.0	7.8	.91	5100	35	6.4	1000	2.6	-.6
SWECOS	8	3.1	8.1	.36	5100	64	6.6	23000	3.0	.1
SWECOS	9	3.4	7.1	.35	5100	38	7.0	47500	3.1	.3
SWECOS	10	4.1	7.4	.55	5100	80	6.8	143000	3.2	.9
SWECOS	11	4.1	7.8	.82	5100	21	6.7	164500	3.6	.5
SWECO	12	3.8	8.1	.60	5100	14	6.7	82000	3.7	.1
SWECOS	13	4.3	7.1	.52	5100	52	6.7	360333	3.3	1.0
SWECOS	14	4.2	7.4	.46	5100	116	6.9	332667	3.2	1.0
SWECO	15	3.8	7.8	.70	5100	56	6.7	79500	3.2	.6
SWECOS	16	4.0	8.1	.76	5100	66	6.7	133000	3.4	.6
SWECOS	17	4.1	7.1	.71	5100	32	6.7	146000	3.3	.8
SWECOS	18	4.0	7.4	.71	5100	80	6.7	95000	3.1	.9
SWECOS	19	4.5	7.8	.49	5100	64	6.7	640000	3.6	.9
SWECOS	20	4.4	8.1	.48	5100	94	6.8	495000	3.6	.8
SWECOS	21	4.1	7.1	.68	5100	104	6.7	153333	3.0	1.1
SWECOS	22	4.1	7.4	.55	5100	96	6.7	245000	3.2	.9
SWECOS	23	4.1	7.8	.55	5100	132	6.7	195333	3.2	.9
SWECOS	24	4.2	8.1	.71	5100	78	6.7	307000	3.5	.7
ZURN 3	10	1.5	4.4	1.22	4791	96	7.3	86500	2.2	-.7
ZURN 3	11	1.6	2.6	1.20	6009	128	7.6	24500	1.4	.2
ZURN 5	13	.9	3.7	.13	4791	84	7.0	925000	2.2	-1.3
ZURN 5	14	1.7	3.7	.53	8226	240	5.5	15400000	2.3	-.6
ZURN 5	15	2.0	3.7	.57	8226	200	5.8	5400000	2.3	-.3
ZURN 5	16	1.7	3.7	.46	8226	220	5.9	1475000	2.1	-.4
ZURN 5	17	1.0	3.7	.32	8226	176	6.2	1545000	2.2	-1.2
ZURN 5	18	1.3	3.7	.48	8226	220	6.0	4600000	2.2	-.9
ZURN 5	21	1.0	3.7	.55	8226	323	5.9	2400000	2.1	-1.1
ZURN 5	22	2.5	5.0	.74	8226	400	6.0	6000000	2.7	-.2
ZURN 5	23	1.3	5.0	1.01	8226	488	6.0	3900000	2.6	-1.3
ZURN 5	24	2.3	5.0	.80	8226	684	6.1	1290000	2.4	-.1
ZURN 6	1	2.5	5.4	4.25	5573	200	7.5	79000	2.4	.1
ZURN 6	2	1.9	5.4	3.90	5573	336	7.0	3750000	2.8	-.9
ZURN 6	3	1.5	5.4	6.20	5573	284	6.0	2600000	2.7	-1.2
ZURN 6	4	1.3	5.4	4.45	5573	244	6.2	1655000	2.7	-1.4
ZURN 6	5	1.4	5.5	2.95	5573	84	6.4	2700000	3.0	-1.6
ZURN 6	6	1.7	5.5	1.85	5573	240	6.5	1305000	2.7	-1.0
ZURN 6	7	2.9	5.5	4.55	6500	508	5.0	2455000	2.5	.4
ZURN 6	8	4.7	6.8	2.35	7909	836	6.0	11700000	3.2	1.5
ZURN 6	9	4.0	7.1	2.15	8236	944	6.7	3300000	3.2	.8
ZURN 6	10	4.9	7.1	2.00	8322	1233	6.9	7600000	3.3	1.1
ZURN 6	11	4.4	7.2	2.80	8410	1472	7.1	9100000	3.4	1.0
ZURN 6	12	4.5	7.0	1.35	8152	900	6.9	10200000	3.4	1.1
ZURN 6	13	4.1	7.5	.70	8686	640	7.3	1820000	3.4	.7
ZURN 6	14	4.1	7.5	.40	8686	432	7.2	2135000	3.5	.6
ZURN 6	15	3.9	7.5	.45	8686	328	7.3	3000000	3.7	.2
ZURN 6	16	4.0	7.5	.35	8686	304	7.4	1805000	3.6	.4
ZURN 6	17	4.1	7.5	.25	8686	224	6.7	280000	3.2	.9
ZURN 6	18	3.5	6.8	.35	7989	236	6.7	550000	3.1	.4
ZURN 6	19	3.7	7.1	.35	8322	223	6.6	485000	3.2	.5
ZURN 6	20	5.0	7.1	.30	8322	396	7.0	1315000	3.3	1.7
ZURN 6	21	4.9	6.0	.25	7071	404	7.0	1320000	2.9	2.0
ZURN 6	22	4.4	6.2	.25	7329	112	6.9	485000	3.0	1.4
ZURN 6	23	3.7	6.3	.10	7397	304	7.0	345000	2.8	.9
ZURN 6	24	4.7	6.4	.55	7466	728	7.0	760000	2.8	1.9
ZURN 8	1	3.3	5.5	.23	9153	496	6.8	4445000	2.9	.4
ZURN 8	2	1.2	5.9	.23	7466	456	6.7	2420000	2.9	-1.7
ZURN 8	3	1.8	6.5	.20	7466	488	6.5	8000000	3.3	-1.5
ZURN 8	4	3.7	7.3	.19	11670	440	6.5	1120000	3.3	.4
ZURN 8	5	4.0	10.0	.26	15163	448	6.6	955000	4.2	-.2
ZURN 8	6	4.0	8.1	.02	15163	420	6.5	1330000	3.7	.3
ZURN 8	7	2.5	8.1	1.64	12755	320	6.6	510000	3.5	-1.0
ZURN 8	8	2.4	7.6	1.74	12009	396	6.5	6300000	3.8	-1.4
ZURN 8	9	3.5	7.9	1.58	12370	392	6.6	6400000	3.9	-.4
ZURN 8	10	5.4	4.5	1.34	7712	356	6.4	6200000	2.6	2.8
ZURN 8	11	5.1	5.4	1.33	8965	312	6.6	2440000	2.8	2.3
ZURN 8	12	1.1	6.1	1.32	7466	288	6.5	2600000	3.0	-1.9
ZURN 8	13	2.5	3.1	1.36	5573	248	6.8	6800000	2.0	.5
ZURN 8	14	1.0	3.8	1.39	6689	144	8.1	480000	2.3	-1.3
ZURN 8	15	1.1	3.8	1.07	6689	148	7.0	1545000	2.3	-1.2

APPENDIX C

Chlorine Analytical Data

APPENDIX C LEGEND

Actual Log kill	-	Log (base 10) reduction of fecal coliform
Cl_2 Dose	-	Dosage of Cl_2 injected into CSO, mg/l
NH_3N	-	Ammonia nitrogen concentration at point of disinfectant injection, mg/l
Influent SS	-	Suspended solids concentration at point of disinfectant injection, mg/l
pH	-	Log of the hydrogen ion concentration, dimensionless
Influent FC	-	Fecal coliform concentration at point of disinfectant injection, cells/100 ml
GT Mixing Intensity	-	Mixing intensity expressed as a product of the velocity gradient (G, sec^{-1}) and detention time (T, sec), dimensionless
Predicted Log kill	-	Log (base 10) of the number of fecal coliform killed by ClO_2 disinfection as predicted by the developed performance model
Residual	-	Difference between the predicted log kill and the actual log kill.

Analytial Data for Cl_2 Disinfection Tests

Sample No.	Actual Logkill FC	Cl_2 Dose, mg/l	NH_3N mg/l	Influent SS,mg/l	pH	Influent FC Counts/100ml	Mixing Intensity GT	Predicted Logkill FC	Residual
SAMPLE 1	.3	.0	1.28	80	6.0	765000	5022	.3	.0
SAMPLE 2	.3	.0	1.13	180	7.0	2100000	5022	.3	.0
SAMPLE 3	.3	.0	1.98	160	6.0	1800000	5022	.3	.0
SAMPLE 4	5.1	23.3	1.83	392	6.0	2190000	5022	5.1	.0
SAMPLE 5	6.0	23.3	1.52	436	5.0	11900000	5022	5.6	.4
SAMPLE 6	6.0	23.3	1.58	360	5.0	9700000	5022	6.0	.0
SAMPLE 7	6.0	23.3	.96	275	5.0	14600000	5022	6.1	-.1
SAMPLE 8	6.0	23.3	.73	230	5.0	10900000	5022	6.0	.0
SAMPLE9	6.0	23.3	.68	220	5.0	10900000	5022	5.8	.2
SAMPLE10	6.0	23.3	.62	167	6.0	11300000	5022	5.8	.2
SAMPL 11	6.0	23.3	.62	144	6.0	11000000	5022	5.8	.2
SAMPLE12	6.0	23.3	.62	116	6.0	13500000	5022	5.9	.1
SAMPLE13	6.0	23.3	.62	92	6.0	10100000	5022	6.0	.0
SAMPLE14	6.0	23.3	.73	96	6.0	13200000	5022	6.0	.0
SAMPLE15	6.0	23.3	.73	176	6.0	11200000	4098	5.1	.9
SAMPLE16	6.0	18.6	1.30	236	6.0	910000	4098	4.4	1.6
SAMPLE17	2.2	18.6	1.30	228	6.0	385000	4098	4.3	-2.1
SAMPLE18	2.6	18.6	1.13	232	7.0	665000	4098	4.2	-1.6
SAMPL 19	4.1	24.0	.76	180	6.0	3800000	2336	4.0	.1
SAMPLE20	3.2	24.0	.45	247	7.0	8600000	2336	3.9	-.7
SAMPLE21	3.7	24.0	.56	211	7.0	2000000	2336	3.8	-.1
SAMPLE22	3.0	24.0	.59	167	7.0	1445000	2336	3.8	-.8
SAMPLE23	2.8	24.0	.52	410	7.0	560000	2336	3.5	-.7
SAMPLE24	2.4	24.0	.30	250	7.0	650000	2336	3.7	-1.3
SAMPLE25	4.0	18.0	.34	87	7.0	210000	2336	3.5	.5
SAMPLE26	3.3	18.0	.40	64	6.0	330000	2336	3.6	-.3
SAMPLE27	2.3	18.0	.40	40	7.0	540000	2336	3.7	-1.4
SAMPLE28	3.5	9.0	.28	68	7.0	540000	2336	2.7	.8
SAMPLE29	3.0	9.0	.28	56	7.0	112000	2336	2.7	.3
SAMPLE30	3.0	9.0	.28	64	7.0	135500	2336	2.7	.3
SAMPLE32	3.6	9.0	.25	112	6.0	116333	2336	2.7	.9
SAMPLE33	3.6	9.0	.26	832	6.0	340000	2336	2.4	1.2
SAMPLE34	3.7	9.0	.40	44	7.0	525000	2336	2.9	.8
SAMPLE35	4.0	9.0	.45	44	6.0	235000	2336	2.8	1.2
SAMPLE36	1.5	9.0	.53	70	6.0	191667	2336	2.8	-1.3
SAMPLE37	4.5	12.0	.32	588	7.0	4800000	2828	2.9	1.6
SAMPLE39	2.6	12.0	.11	480	7.0	2200000	2828	2.9	-.3
SAMPLE40	2.5	12.0	.11	250	7.0	2115000	2828	3.0	-.5
SAMPLE41	2.6	12.0	.09	240	7.0	3100000	2828	3.1	-.5
SAMPLE42	2.6	12.0	.21	256	7.0	1090000	2828	2.9	-.3
SAMPLE43	2.2	12.0	.15	220	7.0	1340000	2828	3.0	-.8
SAMPLE44	2.9	12.0	.17	204	7.0	610000	2828	3.0	-.1
SAMPLE45	2.4	12.0	.08	193	7.0	810000	2828	3.0	-.6
SAMPLE46	2.9	12.0	.17	148	8.0	125667	2828	2.7	.2
SAMPLE47	2.4	12.0	.21	156	9.0	221000	2828	2.7	-.3
SAMPLE48	2.8	12.0	.17	132	8.0	209000	2828	2.9	-.1
SAMPLE49	3.1	12.0	.28	140	7.0	645000	2828	3.0	.1
SAMPLE50	2.2	12.0	.51	148	7.0	290000	2828	3.0	-.8
SAMPLE51	4.4	12.0	.43	168	7.0	450000	2828	3.0	1.4
SAMPLE52	3.4	12.0	.55	156	7.0	465000	2828	3.0	.4
SAMPLE53	4.2	12.0	.59	92	7.0	325000	2828	3.1	1.1
SAMPLE54	2.4	12.0	.45	81	7.0	290000	2828	3.1	-.7
SAMPLE55	3.5	12.0	.23	76	7.0	280000	2828	3.1	.4
SAMPLE56	4.2	12.0	.20	60	7.0	290000	2828	3.2	1.0
SAMPLE57	3.2	12.0	.25	64	7.0	315000	2828	3.2	.0
SAMPLE58	4.3	12.0	.35	66	7.0	365000	2828	3.2	1.1
SAMPLE59	4.1	12.0	.40	92	7.0	164000	2828	3.0	1.1
SAMPLE 5	2.6	12.0	.65	260	7.0	630000	4098	3.4	-.8
SAMPLE 6	3.4	12.0	.67	250	8.0	230000	4098	3.3	.1
SAMPLE 7	3.4	12.0	.65	202	8.0	430000	4098	3.3	.1
SAMPLE 8	4.0	12.0	.71	186	7.0	250000	4098	3.4	.6
SAMPLE 9	4.5	12.0	.59	186	7.0	530000	4098	3.5	1.0
SAMPLE 10	4.2	12.0	.60	175	7.0	800000	4098	3.5	.7
SAMPLE 11	4.2	12.0	.61	164	7.0	1300000	4098	3.6	.6
SAMPLE 12	4.0	12.0	.44	130	7.0	100000	4098	3.5	.5
SAMPLE 13	5.7	12.0	1.77	118	7.0	900000	4098	3.6	2.1
SAMPLE77	3.0	12.0	.20	88	7.0	7400000	4098	3.9	-.9
SAMPLE78	3.8	12.0	1.12	1240	6.0	370000	4098	3.3	.5
SAMPLE79	3.2	12.0	1.97	1660	6.0	675000	4098	3.3	-.1
SAMPLE80	3.2	12.0	1.97	3130	6.0	970000	4098	3.1	.1
SAMPLE81	2.1	12.0	.26	1770	7.0	19500	4098	2.8	-.7
SAMPLE82	3.1	12.0	.28	750	7.0	380000	4098	3.2	-.1
SAMPLE83	4.0	12.0	.36	500	7.0	440000	4098	3.3	.7
SAMPLE84	3.1	12.0	.11	520	7.0	21500	4098	3.1	.0
SAMPLE85	4.3	12.0	.16	530	7.0	710000	4098	3.4	.9
SAMPLE86	3.7	12.0	.12	500	7.0	440000	4098	3.4	.3
SAMPLE87	3.2	12.0	.17	360	7.0	530000	4098	3.4	-.2
SAMPLE88	3.2	12.0	.17	280	7.0	460000	4098	3.5	-.3
SAMPLE92	4.1	12.0	.39	230	7.0	185000	4098	3.4	.7
SAMPLE93	1.7	12.0	.90	60	8.0	110333	4098	3.6	-1.9
SAMPLE94	4.1	12.0	.62	20	7.0	180000	4098	3.9	.2

APPENDIX D

Analytical Data for Sequential Addition of Disinfectants

APPENDIX D LEGEND

Actual Log kill	-	Log (base 10) reduction of fecal coliform
Influent FC	-	Fecal coliform concentration at point of disinfectant injection, counts/100 ml
Influent SS	-	Suspended solids concentration at point of disinfectant injection, mg/l
pH	-	Log of the hydrogen ion concentration, dimensionless
Mixing Intensity GT	-	Mixing intensity expressed as a product of the velocity gradient (G, sec^{-1}) and detention time (T, sec), dimensionless
Cl_2 Dose	-	Dosage of Cl_2 injected into CSO, mg/l
ClO_2 Dose	-	Dosage of ClO_2 injected into CSO, mg/l
Predicted Log kill FC	-	Log (base 10) of the number of fecal coliform killed by sequential addition of disinfectants as predicted by the developed performance models
Residual	-	Difference between the predicted logkill and actual logkill

Analytical Data for Sequential Addition of ClO_2 and Cl_2

Sample No.	Actual Logkill FC	Influent FC Counts/100 ml	Influent SS,mg/l	pH	Mixing Intensity GT	Cl_2 Dose mg/l	ClO_2 Dose mg/l	Predicted Logkill FC	Residual
CRANE 2	3.5	538000	140	6.6	4220	8.0	2.0	3.3	.2
CRANE 3	3.1	165000	100	7.4	4220	8.0	2.0	3.0	.1
CRANE 4	3.1	183000	60	7.0	4220	8.0	2.0	3.0	.1
CRANE 5	3.1	209000	240	7.1	4220	8.0	2.0	3.0	.1
CRANE 6	3.3	385000	150	6.6	4220	8.0	2.0	3.2	.1
CRANE 7	3.1	181000	200	6.7	4220	8.0	2.0	3.0	.1
CRANE 8	3.1	210000	130	6.8	4220	8.0	2.0	3.0	.1
CRANE 10	3.6	660000	120	6.1	4220	8.0	2.0	3.4	.2
CRANE 12	3.3	580000	120	6.5	4220	8.0	2.0	3.4	-.1
CRANE 13	3.3	370000	100	6.7	4220	8.0	2.0	3.2	.1
CRANE 14	2.5	400000	90	6.5	4220	8.0	2.0	3.2	-.7
CRANE 15	3.2	380000	70	6.9	4220	8.0	2.0	3.2	.0
ZURN 2	4.3	4100000	170	6.6	7600	8.0	2.0	4.1	.2
ZURN 3	2.8	81000	220	7.4	7600	8.0	2.0	2.8	.0
ZURN 4	3.1	262000	90	7.0	7600	8.0	2.0	3.1	.0
ZURN 5	3.1	180000	430	7.1	7600	8.0	2.0	3.0	.1
ZURN 6	3.0	126000	750	6.6	7600	8.0	2.0	2.9	.1
ZURN 7	3.2	310000	1230	6.7	7600	8.0	2.0	3.2	.0
ZURN 8	3.6	670000	1490	6.8	7600	8.0	2.0	3.4	.2
ZURN 9	3.6	760000	1060	6.6	7600	8.0	2.0	3.5	.1
ZURN 10	3.4	440000	1490	6.1	7600	8.0	2.0	3.3	.1
ZURN 11	3.4	430000	730	6.0	7600	8.0	2.0	3.3	.1
ZURN 12	3.4	430000	790	6.5	7600	8.0	2.0	3.3	.1
ZURN 13	2.6	360000	640	6.7	7600	8.0	2.0	3.2	-.6
ZURN 14	3.0	490000	260	6.5	7600	8.0	2.0	3.3	-.3
ZURN 15	3.3	360000	390	6.9	7600	8.0	2.0	3.2	.1

APPENDIX E

Analytical Data for Swirl Prototype SS Removal

APPENDIX E LEGEND

Actual % SS Removed	-	Percent of suspended solids removed
Flow	-	Flowrate, MGD (1 MGD x 3785 = 1 cu m/day)
Influent SS	-	Influent suspended solids concentration, mg/l
Foul Fraction	-	Percent of influent flow which is removed via the foul sewer outlet
Predicted % SS Removal	-	Suspended solids removed (in terms of concentration) as predicted by the developed performance model
Residual	-	Difference between the predicted SS concentration removal and the actual SS concentration removal, percent

Swirl Prototype SS Removals

Sample No.		Actual % SS Removed	Flow, MGD	Influent SS, mg/l	Foul Frction, %	Predicted % SS Removal	Residual
SWIRL 2	1	53	1.60	380	38	47.0	6.0
	2	48	2.10	780	29	48.0	.0
	5	37	2.47	540	24	45.9	-8.9
	6	44	1.97	410	30	46.2	-2.2
	7	34	2.10	410	29	45.8	-11.8
	8	29	2.05	240	29	44.2	-15.2
SWIRL 3	2	60	.71	88	86	46.5	13.5
	4	36	.82	59	73	44.4	-8.4
	5	31	.90	320	67	49.6	-18.6
SWIRL 7	4	37	2.25	104	27	41.2	-4.2
	5	48	2.50	96	24	40.5	7.5
	7	37	1.10	108	54	44.9	-7.9
SWIRL10	1	64	4.80	436	13	41.9	22.1
	2	32	4.10	208	15	40.4	-8.4
	3	44	2.35	222	26	43.3	.7
	4	29	1.60	186	38	44.7	-15.7
	7	48	2.60	260	23	43.3	4.7
	8	39	2.45	210	24	42.9	-3.9
SWIRL14	2	65	2.20	284	27	44.4	20.6
	3	29	2.30	204	26	43.1	-14.1
	6	50	2.58	128	23	41.2	8.8
SWIRL 1	13	52	.51	460	39	54.3	-2.3
	14	53	.51	430	39	54.1	-1.1
	15	52	.51	460	39	54.3	-2.3
	16	50	.67	420	63	52.3	-2.3
	17	51	.67	410	63	52.2	-1.2
	18	42	.67	330	63	51.4	-9.4
	19	53	.35	340	46	55.5	-2.5
	20	61	.35	360	46	55.7	5.3
	21	61	.35	330	46	55.4	5.6
	22	59	.45	320	80	53.7	5.3
	23	71	.45	310	80	53.6	17.4
	24	66	.45	320	80	53.7	12.3
SWIRL 2	2	49	1.97	760	32	48.2	.8
	3	55	1.94	710	34	48.1	6.9
	4	47	1.76	600	28	48.1	-1.1
	5	41	1.56	540	38	48.4	-7.4
	7	60	1.32	400	45	48.2	11.8
	11	55	1.16	212	35	46.8	8.2
	12	32	.98	110	27	45.5	-13.5
	15	55	1.18	200	23	46.5	8.5
	17	38	1.69	280	40	45.7	-7.7
SWIRL 6	1	51	2.70	156	6	41.5	9.5
	4	61	1.92	1200	21	50.0	11.0
	11	33	1.38	408	34	48.1	-15.1
	12	39	1.30	476	33	48.9	-9.9
	13	50	.76	516	58	52.3	-2.3
SWIRL12	5	65	3.15	788	19	45.8	19.2
	6	60	2.30	475	26	45.8	14.2
	9	77	1.20	672	39	50.6	26.4
	12	51	5.45	390	11	40.9	10.1
	24	41	4.20	242	14	40.8	.2
	27	38	2.40	326	25	44.4	-6.4
	32	66	1.00	166	60	46.8	19.2
SWIRL13	17	43	4.73	160	11	39.0	4.0
	21	55	1.65	124	13	43.2	11.8
	22	30	1.17	236	15	47.1	-17.1
	23	45	1.13	258	16	47.6	-2.6
	2	31	3.75	232	28	41.2	-10.2
	5	27	5.20	364	29	40.9	-13.9
	12	58	3.04	106	2	39.9	18.1
	13	57	2.83	110	1	40.3	16.7
	14	58	2.97	84	4	39.3	18.7
	15	45	2.93	66	2	38.7	6.3
	16	43	2.94	40	1	37.3	5.7
	19	41	1.59	37	2	39.8	1.2
	20	36	1.34	39	2	40.8	-4.8
	21	49	1.08	43	3	42.1	6.9
	22	32	.81	41	4	43.3	-11.3
	23	28	.73	57	4	44.9	-16.9
SWIRL15	3	60	2.55	60	61	39.1	20.9
	6	49	2.95	158	53	41.2	7.8
	7	42	3.04	136	51	40.6	1.4
	8	38	3.06	104	51	39.8	-1.8
	10	41	3.49	74	44	38.2	2.8
	11	38	3.35	90	57	39.0	-1.0
	13	53	3.03	104	64	39.8	13.2
	14	32	3.03	74	64	38.9	-6.9
	16	65	3.03	210	64	41.9	23.1
	17	57	3.37	150	58	40.4	16.6
	18	64	3.71	144	53	39.8	24.2
	22	32	3.56	50	56	37.1	-5.1
	23	35	3.33	40	60	36.8	-1.8
SWIRL16	1	30	7.50	100	12	35.8	-5.8
	2	24	7.30	310	10	38.9	-14.9
	3	20	7.10	220	8	38.1	-18.1
	4	26	6.50	150	15	37.5	-11.5

APPENDIX F

Swirl Prototype Storm-Averaged Data

APPENDIX F LEGEND

Actual % SS Removal	-	Suspended solids removed (in terms of concentration), percent
Average Flow	-	Average flow to the swirl unit, MGD (1 MGD x 3785 = 3 cu m/day)
Average Influent SS	-	Influent SS concentration, mg/l
Foul Fraction	-	The flow removed via the foul sewer outlet as expressed as a percentage of influent flow to the unit
Predicted % SS Removal	-	Suspended solids removed (in terms of concentration) as predicted by the developed performance model, percent
Residual	-	Difference between the predicted SS concentration removal and the actual SS concentration removal, percent

Swirl Prototype Storm-Averaged Data

Storm No.	Actual % SS Removed	Average Flow, MGD	Average Influent SS, mg/l	Foul Fraction, %	Predicted % SS Removed	Residual
SWIRL 2	36	2.15	535	28	28.2	7.8
SWIRL 3	23	1.00	182	59	39.7	-16.7
SWIRL 7	18	2.50	110	24	26.3	-8.3
SWIRL 10	29	2.91	230	21	24.8	4.2
SWIRL 14	22	2.22	159	27	27.8	-5.8
SWIRL 1	55	.50	374	58	39.4	15.6
SWIRL 2	41	1.43	342	36	31.7	9.3
SWIRL 6	24	1.42	342	27	27.8	-3.8
SWIRL 12	20	2.79	291	22	25.3	-5.3
SWIRL 14	27	2.73	156	14	20.6	6.4
SWIRL 15	52	3.25	115	58	39.4	12.6

APPENDIX G

ADENOSINE TRIPHOSPHATE (ATP) DATA MEASUREMENTS

TABLE G-1. BACTERIAL REDUCTIONS AND ATP MEASUREMENTS FOR SINGLE STAGE DISINFECTION USING Cl_2 or ClO_2

Disinfectant	Dose, mg/l	Contact Time sec	Bacteria, Cells/100 ml TC	Bacteria, Cells/100 ml FS	ATP µg/l
Cl_2	0	30	4,700,000	328,000	6.320
Cl_2	5	30	4,900,000	326,000	1.250
Cl_2	10	30	4,000,000	154,000	0.750
Cl_2	15	30	250,000	13,300	0.730
Cl_2	25	30	1,300	260	0.068
Cl_2	100	30	0	0	0.011
Cl_2	0	300	4,700,000	328,000	6.320
Cl_2	5	300	3,400,000	59,000	0.730
Cl_2	10	300	44,000	9,700	0.220
Cl_2	15	300	1,010	590	0.070
Cl_2	25	300	5	5	0.039
Cl_2	100	300	0	0	0.009
ClO_2	0	30	33,000,000	322,000	5.120
ClO_2	5	30	1,150,000	39,000	1.100
ClO_2	10	30	300,000	21,800	0.740
ClO_2	15	30	130,000	8,000	0.210
ClO_2	25	30	6,900	730	0.047
ClO_2	100	30	300	41	0.039
ClO_2	0	300	33,000,000	322,000	5.120
ClO_2	5	300	760,000	17,600	0.460
ClO_2	10	300	40,000	5,300	0.280
ClO_2	15	300	34,800	576	0.057
ClO_2	25	300	700	36	0.019
ClO_2	100	300	0	0	0.006

TABLE G-2. BACTERIAL REDUCTIONS AND ATP MEASUREMENTS FOR TWO-STAGE DISINFECTION WITH ClO_2 (TRIAL A)

Disinfectant Dosage, mg/l ClO_2	Disinfectant Dosage, mg/l ClO_2	Contact Time sec	Bacteria, Counts/100 ml TC	FC	FS	ATP µg/l
2	0	0	6,380,000	615,000	99,000	5.08
2	2	15	407,000	63,500	92,400	2.24
2	2	30	165,000	11,000	77,000	1.09
2	2	45	22,000	4,620	39,600	0.79
2	2	60	49,500	3,300	33,000	0.60
2	2	90	22,000	2,200	16,500	0.46
2	2	120	5,000	2,300	24,900	0.30
2	0	0	4,840,000	594,000	110,000	5.03
2	0	15	1,430,000	231,000	86,900	2.53
2	2	30	1,060,000	117,000	77,000	3.02
2	2	45	16,500	7,240	40,700	0.78
2	2	60	14,300	3,520	18,700	0.52
2	2	90	11,000	2,200	8,580	0.32
2	2	120	5,500	1,540	2,970	0.16
2	0	0	2,860,000	308,000	77,000	1.920
2	4	15	715,000	41,800	67,100	0.840
2	4	30	29,500	1,850	36,300	0.150
2	4	45	7,700	460	1,100	0.065
2	4	60	3,300	2,200	900	0.054
2	4	90	1,100	660	770	0.040
2	4	120	770	405	220	0.044
2	0	0	4,180,000	322,000	114,000	1.850
2	0	15	1,050,000	97,000	91,300	1.080
2	4	30	836,000	52,000	83,600	0.990
2	4	45	16,500	1,100	11,000	0.078
2	4	60	2,640	600	3,310	0.048
2	4	90	1,430	250	540	0.033
2	4	120	480	60	385	0.027

TABLE G-3. BACTERIAL REDUCTIONS AND ATP MEASUREMENTS FOR TWO-STAGE DISINFECTION WITH ClO_2 (TRIAL B)

Disinfectant Dosage, mg/l		Contact Time	Bacteria, Counts/100 ml			ATP
ClO_2	ClO_2	Sec	TC	FC	FS	μg/l
4	0	0	1,870,000	39,600	121,000	3.160
4	2	15	340,000	1,100	30,800	0.500
4	2	30	10,000	999	10,000	0.110
4	2	45	8,800	110	2,530	0.083
4	2	60	9,900	110	2,640	0.056
4	2	90	2,200	330	1,430	0.046
4	2	120	999	77	1,100	0.046
4	0	0	1,430,000	29,700	130,000	1.950
4	0	15	80,300	1,760	57,200	0.430
4	2	30	18,700	360	14,900	0.190
4	2	45	4,070	66	2,860	0.052
4	2	60	2,970	77	1,590	0.052
4	2	90	2,900	77	1,100	0.045
4	2	120	1,900	55	350	0.036
4	0	0	803,000	24,800	131,000	2.500
4	4	15	28,600	550	14,300	0.320
4	4	30	880	20	1,180	0.120
4	4	45	880	10	820	0.075
4	4	60	580	10	600	0.049
4	4	90	410	10	300	----
4	4	120	275	0	280	----
4	0	0	1,120,000	880	124,000	2.020
4	0	15	126,000	440	28,300	0.520
4	4	30	19,300	440	7,000	0.057
4	4	45	1,100	0	1,200	----
4	4	60	460	20	500	----
4	4	90	250	0	385	----
4	4	120	120	0	120	----

TABLE G-4. BACTERIAL REDUCTIONS AND ATP MEASUREMENTS FOR TWO-STAGE DISINFECTION WITH Cl_2 and ClO_2 (TRIAL A)

Disinfectant Dosage, mg/l Cl_2	ClO_2	Contact Time sec	Bacteria, Counts/100 ml TC	FC	ATP µg/l
4	0	0	3,430,000	308,000	4.52
4	2	15	572,000	84,700	3.53
4	2	30	113,000	59,400	3.20
4	2	45	473,000	82,500	3.13
4	2	60	123,000	40,700	2.68
4	2	90	1,100	38,500	2.44
4	2	120	52,800	12,100	2.67
4	0	0	2,220,000	110,000	3.87
4	0	15	924,000	86,900	2.94
4	2	30	506,000	74,800	2.77
4	2	45	187,000	38,500	2.73
4	2	60	62,700	39,600	2.33
4	2	90	132,000	40,700	2.00
4	2	120	75,900	13,200	1.98
4	0	0	1,870,000	113,000	2.60
4	4	15	1,870,000	84,700	1.82
4	4	30	407,000	18,700	1.03
4	4	45	176,000	8,800	0.68
4	4	60	187,000	5,500	0.68
4	4	90	80,300	8,910	0.61
4	4	120	59,400	6,160	0.53
4	0	0	2,640,000	110,000	1.33
4	0	15	1,430,000	47,300	1.68
4	4	30	1,050,000	56,100	1.02
4	4	45	143,000	102,000	0.43
4	4	60	96,800	8,800	0.20
4	4	90	60,500	4,180	0.29
4	4	120	13,200	3,410	0.23

TABLE G-5. BACTERIAL REDUCTIONS AND ATP MEASUREMENTS FOR TWO-STAGE DISINFECTION WITH Cl_2 and ClO_2 (TRIAL B)

Disinfectant Dosage, mg/l Cl_2	ClO_2	Contact Time sec	Bacteria, Counts/100 ml TC	FC	FS	ATP µg/l
8	0	0	187,000	12,100	82,500	0.720
8	2	15	121,000	1,430	38,500	1.030
8	2	30	90,000	99	2,200	0.280
8	2	45	1,100	99	3,300	0.290
8	2	60	1,210	99	3,850	0.120
8	2	90	660	99	1,980	0.070
8	2	120	110	9	1,870	0.055
8	0	0	220,000	9,900	72,900	0.960
8	0	15	61,600	2,420	17,600	0.610
8	2	30	4,400	2,250	3,300	0.140
8	2	45	1,100	320	990	0.150
8	2	60	330	165	2,750	0.074
8	2	90	99	9	1,980	0.073
8	2	120	99	11	1,320	0.065
8	0	0	660,000	30,200	11,000	1.010
8	4	15	-------	------	-----	0.710
8	4	30	-------	------	-----	0.089
8	4	45	999	220	999	0.049
8	4	60	-------	------	-----	0.027
8	4	90	-------	------	-----	0.039
8	4	120	99	9	99	0.027
8	0	0	100,000	22,500	74,800	0.820
8	0	15	-------	------	-----	1.020
8	4	30	18,700	3,630	8,800	0.520
8	4	45	-------	------	-----	0.052
8	4	60	110	90	110	0.035
8	4	90	-------	------	-----	0.025
8	4	120	99	9	44	0.019

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA-600/2-79-134	2.	3. RECIPIENT'S ACCESSION NO.
4. TITLE AND SUBTITLE DISINFECTION/TREATMENT OF COMBINED SEWER OVERFLOWS Syracuse, New York		5. REPORT DATE August 1979 (Issuing Date) 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Frank J. Drehwing, Arthur J. Oliver, Dwight A. MacArthur, Peter E. Moffa		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS O'Brien & Gere Engineers, Inc. 1304 Buckley Road Syracuse, New York 13221		10. PROGRAM ELEMENT NO. 1BC822, SOS #1, Task 18 11. CONTRACT/GRANT NO. S802400 (11020HFR)
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Environmental Research Laboratory--Cin.,OH Office of Research & Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268		13. TYPE OF REPORT AND PERIOD COVERED Final 1971-1978 14. SPONSORING AGENCY CODE EPA/600/14

15. SUPPLEMENTARY NOTES Supplement to EPA-670/2-75-021, "Bench-Scale High-Rate Disinfection of Combined Sewer Overflows" - Project Officer: Richard Field (201) 321-6674
FTS 340-6674

16. ABSTRACT

The Syracuse demonstration program was designed to evaluate high-rate disinfection/treatment of CSO. The study covered field evaluations of high-rate treatment and disinfection by the following unit processes: three separate microscreening devices, swirl regulator/concentrator, and disinfection utilizing chlorine and chlorine dioxide.

The three microscreening units were evaluated employing various hydraulic loading rates. Using multiple regression analysis techniques, mathematical performance models were developed for each unit relating suspended solids removal efficiencies to hydraulic and solid loading rates, and the results are presented in the reports. Similarly, performance models were developed for the treatment efficiency of the swirl regulator/concentrator and are reported. Multiple regression modeling of the disinfection data yielded statistically significant performance equations for the high-rate disinfection systems. The models provided an analysis of sensitivity to mixing intensity and detention time for the two disinfectants.

Capital and operating cost estimates indicated that solids removal via swirl regulation/concentration followed by disinfection by chlorine was the least expensive CSO abatement strategy for those abatement options evaluated in this study.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Combined sewers, Disinfection, Chlorine, Water pollution, Waste treatment, Sewage treatment, Wastewater, Sewage, Overflows, Sewers, Collection	High-Rate treatment/disinfection, CSO characterization, Microscreening, Suspended solids monitor, Swirl regulator/concentrator, Adenosine triphosphate, Total organic analyzer	13B

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS *(This Report)* UNCLASSIFIED	21. NO. OF PAGES 262
	20. SECURITY CLASS *(This page)* UNCLASSIFIED	22. PRICE

EPA Form 2220-1 (Rev. 4-77)

☆ U.S. GOVERNMENT PRINTING OFFICE: 1979 -657-060/5453