United States
Environmental Protection
Agency

Office of Research and
Development
Washington, DC 20460

EPA/600/R-00/020
January 2000

# Retrofitting Control Facilities For Wet-Weather Flow Treatment

Research Report

EPA#/600/R-00/020
January 2000

# Retrofitting Control Facilities For Wet-Weather Flow Treatment

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## Notice

The information in this document has been funded by the United States Environmental Protection Agency under Contract No. 8C-R056-NTSX. Although it has been subjected to the Agency's peer and administrative review and has been approved for publication as an EPA document, it does not necessarily reflect the views of the Agency and no official endorsement should be inferred. Also, the mention of trade names or commercial products does not imply endorsement by the United States government.

## Foreword

The U.S. Environmental Protection Agency is charged by Congress with protecting the Nation's land, air, and water resources. Under a mandate of national environmental laws, the Agency strives to formulate and implement actions leading to a compatible balance between human activities and the ability of natural systems to support and nurture life. To meet this mandate, EPA's research program is providing data and technical support for solving environmental problems today and building a science knowledge base necessary to manage our ecological resources wisely, understand how pollutants affect our health, and prevent or reduce environmental risks in the future.

The National Risk Management Research Laboratory is the Agency's center for investigation of technological and management approaches for reducing risk from threats to human health and the environment. The focus of the Laboratory's research program is on methods for the prevention and control of pollution to air, land, water and subsurface resources; protection of water quality in public water systems; remediation of contaminated sites and ground water; and prevention and control of indoor air pollution. The goal of this research effort is to catalyze development and implementation of innovative, cost-effective, environmental technologies; develop scientific and engineering information needed by EPA to support regulatory and policy decisions; and provide technical support and information transfer to ensure effective implementation of environmental regulations and strategies.

This publication has been produced as part of the Laboratory's strategic long-term research plan. It is published and made available by EPA's Office of Research and Development to assist the user community and to link researchers with their clients.

E. Timothy Oppelt, Director
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## Abstract

Available technologies were evaluated to demonstrate the technical feasibility and cost effectiveness of retrofitting existing facilities to handle wet-weather flow. Cost/benefit relationships were also compared to construction of new conventional control and treatment facilities.

Desktop analyses of 13 separate retrofit examples were performed for 1) converting or retrofitting primary settling tanks with dissolved air flotation and lamellae and/or microsand-enhanced plate or tube settling units, 2) retrofitting existing wet-weather flow storage tanks to provide enhanced settling/ treatment and post-storm solids removal, 3) converting dry ponds to wet ponds for enhanced treatment, 4) retrofitting wet-weather flow storage tanks for dry-weather flow augmentation, 5) using storage for sanitary sewer overflow control , 6) retrofitting for industrial wastewater control in a combined sewer system, and 7) bringing outdated/abandoned treatment plants back online as wet-weather flow treatment facilities.

This analysis demonstrated that retrofitting existing wet-weather flow facilities can be technically feasible in most cases and may be more cost effective than construction of new conventional control and treatment facilities. The feasibility and cost effectiveness of retrofitting was found to be a function of site-specific conditions and treatment requirements. Retrofitting processes will better enable communities to meet EPA's National CSO Policy and stormwater permitting program requirements.

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## Acronyms and Abbreviations

| | |
|---|---|
| ac-ft | Acre-feet |
| ASD | Augusta Sanitary District |
| AWPCP | Auxiliary Water Pollution Control Plant |
| BATS | Burnet Avenue Trunk Sewer |
| BMP | Best Management Practice |
| BOD | Biochemical Oxygen Demand |
| CCS | Combined Collection System |
| CDM | Camp Dresser & McKee |
| CEAM | Center for Exposure Assessment Modeling |
| cfs | Cubic Feet per Second |
| cm | Centimeter |
| CN | Curve Number |
| COD | Chemical Oxygen Demand |
| COV | Coefficient of Variation |
| CPE | Comprehensive Performance Evaluation |
| CSO | Combined Sewer Overflow |
| CSS | Combined Sewer System |
| cyd | Cubic Yards |
| CWA | Clean Water Act |
| D | Day |
| DAF | Dissolved Air Flotation |
| DEP | Department of Environmental Protection |
| DWF | Dry Weather Flow |
| EA | Environmental Assessment |
| EBSS | Erie Boulevard Storage System |
| EPA | U.S. Environmental Protection Agency |
| EMC | Event Mean Concentration |
| FDS | Flow Distribution Structure |
| $FeCl_3$ | Ferric Chloride |
| fps | Feet per Second |
| ft | Feet |
| gal | Gallon |
| gpd | Gallons per day |
| gpm | Gallons per minute |
| ha | Hectare |
| HP | Horse power |
| h | Hour |
| IDF | Intensity Distribution Frequency |
| In. | Inch |
| I/I | Infiltration and/or Inflow |
| JSRS | James Street Relief Sewer |
| kg | Kilogram |

## Acronyms and Abbreviations, Continued

| | |
|---|---|
| km | Kilometer |
| kW | Kilowatt |
| l | Liter |
| lbs | Pounds |
| m | Meter |
| mA | Milliampere |
| mm | Millimeter |
| µm | Micrometer |
| mi | Mile |
| Metro | Municipality of Toronto Metropolitan |
| METRO | Syracuse Metropolitan Wastewater Treatment Facility |
| MG | Million gallons |
| MGD | Million gallons per day |
| mg | Milligram |
| mg/L | Milligrams per Liter |
| mg | Microgram |
| min | Minute |
| MIS | Main Interceptor Sewer |
| ml | Milliliter |
| msl | Mean Sea Level |
| mt | Metric ton |
| MTP | Main Treatment Plant |
| NASA | National Aeronautics and Space Administration |
| NOAA | National Oceanic and Atmospheric Administration |
| NPDES | National Pollution Discharge Elimination System |
| NTIS | National Technical Information Services |
| NURP | Nationwide Urban Runoff Program |
| NWS | National Weather Service |
| NYCDEP | New York City Department of Environmental Protection |
| NYSDEC | New York State Department of Environmental Conservation |
| O&M | Operation and Maintenance |
| ORF | Overflow Retention Facility |
| POTWs | Publicly Owned Treatment Plants |
| ppm | Parts per Million |
| PPS | Particulate Pollutant Strength |
| PRF | Peak Reduction Factors |
| psi | Pounds Per Square Inch |
| RCP | Reinforced Concrete Pipe |
| rpm | Revolutions per Minute |
| s | Seconds |
| SCADA | Supervisory Control and Data Acquisition |
| scfm | Standard Cubic Feet Per Minute |

## Acronyms and Abbreviations, Continued

| | |
|---|---|
| SCS | Soil Conservation Service |
| SESD | South Essex Sewerage District |
| SOR | Surface Overflow Rate |
| STLC | Soluble Threshold Limit Concentrations |
| SS | Suspended Solids |
| SSES | Sewer System Evaluation Survey |
| SSO | Sanitary Sewer Overflow |
| SW | Storm water |
| SWMM | Stormwater Management Model |
| SYNOP | Synoptic Rainfall Analysis Program |
| TB | Tipping Buckets |
| TKN | Total Kjeldahl Nitrogen |
| TOC | Total Organic Carbon |
| TTLC | Total Threshold Limit Concentration |
| USGS | U.S. Geological Survey |
| WET | Whole Effluent Toxicity |
| WHO | World Heath Organization |
| WPCP | Water Pollution Control Plant |
| WQO | Water Quality Objective |
| WWF | Wet Weather Flow |
| WWTF | Wastewater Treatment Facility |

## Acknowledgements

Moffa & Associates gratefully acknowledges the support of the project by the National Risk Management Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, under the direction of Mr. Richard Field, Wet-Weather Flow Research Program leader.

The cooperation of Thomas P. O'Connor, Project Officer, Urban Watershed Management Branch, Water Supply and Water Resources Division, National Risk Management Research Laboratory (NRMRL), Office of Research and Development (ORD), U.S. Environmental Protection Agency (EPA), is acknowledged as are Dr. David Fischer and Dr. Ronald Rovanesk, ORISE post graduates with EPA's UWMB, in Edison, New Jersey and Dan Murray, Technology Transfer Division, NRMRL, ORD, EPA, Cincinnati, Ohio for providing reviews for the report.

In addition, special thanks are given to the following consultants for their contributions to this report:

Steve D. Freedman, P.E., Earth Tech Inc.

Sorin L. Goldstein, P.E., Camp Dresser & McKee, Inc., Woodbury, NY 11797

Dwight A. MacArthur, P.E., O'Brien & Gere Engineers, Inc., Syracuse, NY 13221

Robert Pitt, Environmental Engineer, Consultant

Robert B. Stallings, P.E., Earth Tech Inc.

The assistance of Gregory J. Hotaling with preparation of graphical materials presented is hereby recognized, as is the assistance of Renee Davis for assistance with word processing.

This report was prepared by Moffa & Associates Consulting Engineers, by Howard M. Goebel, Daniel P. Davis, and John J. LaGorga under the direction of Peter E. Moffa, Principal.

# Section 1
# Introduction

## *Background*

Wet-weather flow (WWF) is currently one of the leading causes of water-quality impairment in the United States and improvement of controls is one of the two priority water focus-areas cited by the U.S. Environmental Protection Agency's (EPA) Office of Water in its National Agenda for the Future. Wet-weather pollution problems are primarily manifested in three different ways: separate sewer overflows (SSO), combined sewer overflows (CSO), and stormwater (SW) discharges. SSO are the result of the unplanned relief of a sewer system intended only for sanitary sewage but also carrying infiltration and inflow (I/I), illegal stormwater connections, and stormwater. CSO are the result of the "designed" relief of a sewer system that intentionally carries stormwater and sanitary sewage. Stormwater runoff discharges include flows that have been collected in a separate storm sewer system or a surface drainage system.

WWF pollution is extensive throughout the country. There are approximately 15,000 CSO discharge points within 1,100 municipalities that have combined sewers. SSO occur in more than 1,000 municipalities. SW discharges occur in over 6,500 municipalities with populations greater than 50,000 and at 1.2 million industrial, commercial, institutional, and retail sources.

It was not until data from several comprehensive municipal CSO and SSO studies were collected that the types of pollutants discharged by overflow systems and their relative impacts on receiving waters were initially understood. The primary parameters of importance are visible floatables and gross solids, bacteria and other pathogens, suspended solids (SS), biochemical oxygen demand (BOD), nutrients, and toxic organics and metals.

The projected costs for national CSO, SSO, and SW abatement are in the tens of billions of dollars. Consequently, inexpensive abatement alternatives are required to alleviate the high costs associated with conventional treatment. High-rate treatment technologies can increase sewage treatment system capacity to allow treatment of a significant portion of the WWF in sanitary and combined sewers. The primary benefits of high-rate treatment technologies are their ability to handle higher flow rates and compactness or small footprint compared to conventional treatment processes. High-rate treatment technologies require less tankage and space and in urbanized areas are often more cost effective than conventional treatment processes since storm flows are significantly greater than dry-weather flows.

Wet-weather discharges are reflections of the watersheds or sewersheds from which they are generated and the climatic patterns those areas experience. The impacts of wet-weather discharges are determined by a variety of complex relationships between the temporal and spatial distributions of discharges. These include the types and magnitudes of pollutant loads being transported, the ecological and hydraulic nature of the receiving water, as well as the designated beneficial uses, and the desires and

expectations of stakeholders. The result is a wide-scale problem that requires very site-specific solutions.

Solutions to WWF problems require investigation of a wide variety of management options and the relationships between management costs and water quality benefits. WWF must be controlled by storage-treatment systems since WWF are intermittent and highly variable in both pollutant concentrations and flow rates. An optimized system maximizes the use of the existing infrastructure prior to construction of new treatment/storage devices.

Retrofitting existing sewerage systems to handle increased WWF that result in SSO, CSO, and SW discharges may be a cost-effective alternative to the construction of new treatment systems. Retrofitting existing sewerage systems and treatment plants to convey and treat additional WWF can be accomplished by:

1. Increasing the hydraulic loadings at the control facilities (CSO and SSO);
2. Increasing the amount of storage throughout the conveyance system (All WWF); and
3. Providing the additional function of pollution removal by settling at flood-control-detention basins (SW).

***Summary of Analyses***

This report demonstrates real and hypothetical case studies. Retrofits of available technologies for conventional sewer systems are presented in the form of desktop analyses. The desktop analyses were conducted to determine the technical feasibility and cost effectiveness of retrofitting existing sewage and WWF systems in lieu of constructing new sewerage and conveyance systems. Site-specific analyses were completed for WWF storage-treatment systems and their associated cost due to the variability of wet-weather volume and treatability (quantity and quality).

The desktop analyses including conceptual layouts and designs of systems to increase treatment and storage of WWF through retrofits were performed for the following scenarios:

1. Converting or retrofitting primary settling tanks at the publicly owned treatment works (POTW) to alternative sedimentation or sedimentation enhancement methods or methods of treatment that provide equivalent or higher levels of treatment (SSO and CSO);
2. Retrofitting existing CSO storage tanks to provide enhanced settling/treatment and post-storm solids removal;
3. Converting existing “dry-ponds” and/or other flood control devices to SW detention ponds for separate SW systems to allow treatment through sedimentation;
4. Retrofitting existing WWF storage tanks and ponds to serve a second function as DWF equalization tanks;

5. Retrofitting for industrial wastewater control in a combined sewer system;
6. SSO control using storage; and
7. Bringing outdated and/or abandoned POTWs back online as satellite WWF treatment facilities (SSO and CSO).

The analyses were performed over a variety of orders of magnitude of flow and/or volume for each scenario. The desktop analyses of the retrofitted systems addressed the following design considerations, where appropriate:

1. Operations, maintenance and level of automation
2. Sludge handling requirements and disposal
3. Continuous operation versus storm flow event operations
4. Chemical storage and feed rate
5. Bypass requirements

Treatment values, pollutant and hydraulic loadings, and seasonal and diurnal variation are also provided, where appropriate.

Simplified simulations were completed using historical rainfall data for an one-year period using the EPA Stormwater Management Model (SWMM) with hourly time steps or an appropriate alternative. The yearly rainfall data is provided including seasonal variation, average antecedent duration, and storm intensity.

When feasible, the cost effectiveness of retrofitting versus construction of additional conventional treatment works and/or storage basins was completed as part of each desktop analysis. The effectiveness of each system, guidance on design and operation, cost breakdowns (new construction and modification of existing structure, system installation, and operation and maintenance [O&M]), and case studies are provided.

### *Regulatory Background*

Since the 1970s, the U.S. Environmental Protection Agency's Construction Grant Program financed the development and construction of hundreds of wastewater treatment facilities (WWTF) around the nation. During this time the main focus was regional WWTF designed to treat dry-weather sewage flows. Little or no money was invested to abate wet-weather pollution sources. Since then, many of the POTWs have exceeded their design flows, discharge standards have become more stringent, and federal regulations and policies have begun to address WWF pollution. All of these factors contribute to a need for more and better WWF treatment facilities. Today, federal funding for new environmental facilities is scarce and taxpayers carry the burden. Based on this climate, it becomes much more critical that any compliance needs should first consider optimizing existing infrastructure through retrofitting.

This report is intended to help municipalities who may be impacted by current policies and regulations (i.e., National Pollutant Discharge Elimination System (NPDES) Phase I permitting and the CSO Control Policy) and the newer regulations and policies that address WWF (i.e., the upcoming NPDES (Phase II) permitting regulations and SSO regulations). Also, this report can assist municipalities and others (e.g., industry and commercial property owners) who are subject to local regulations.

Stormwater Regulations and Permits

Congress added Section 402(p) to the Clean Water Act in 1987 to require the implementation of a comprehensive approach for addressing stormwater discharges. Section 402(p)(4) required EPA to develop permit application regulations under the NPDES, submission of NPDES permit applications, issuance of NPDES permits, and compliance with NPDES permit conditions. Section 402(p)(6) requires EPA to designate stormwater discharges to be regulated (within the statutory definitions provided in section 402(p)(2)) and establish a comprehensive regulatory program, which may include performance standards, guidelines, guidance, and management practices and treatment requirements (EPA, 1999).

The first phase of NPDES stormwater permit application regulations ("Phase I") was promulgated on November 16, 1990 (EPA, 1990). The provisions addressing municipal separate-storm-sewer systems (MS4s) cover systems serving a population of 100,000 or more. This includes 173 cities, 47 counties and additional systems designated by EPA or states. A total of 260 permits, covering approximately 880 operators (local governments, state highway departments, etc.) were identified as subject to Phase I permit application requirements. The CWA requires that MS4 permits effectively prohibit non-stormwater discharges into the storm sewers as well as reduce the discharge of pollutants to the maximum extent practicable (including management practices, control techniques and system, design and engineering methods, and other provisions appropriate for the control of such pollutants).

Phase I MS4 permittees were required to submit an application that included source-identification information, precipitation data, existing data on the volume and quality of stormwater discharges, a list of receiving water bodies and existing information on impacts on receiving waters, a field screening analysis for illicit connections and illegal dumping, and other information. The MS4 permittees were to gather and provide additional information including:

• discharge characterization data based on quantitative data from 5 to 10 representative locations in approved sampling plans; estimates of the annual pollutant load and event-mean concentration of system discharges for selected conventional pollutants and heavy metals; a proposed schedule to provide estimates of seasonal pollutant loads; and the mean concentration for certain detected constituents in a representative storm event; and

• a proposed management program including descriptions of: structural and source control measures that are to be implemented to reduce pollutants in runoff from commercial and residential areas; a program to detect and remove illicit discharges; and a program to control pollutants in construction site runoff.

The NPDES stormwater regulations for the second phase of stormwater discharge control ("Phase II") was proposed on January 9, 1998 (US EPA, 1998c). EPA was required to promulgate the Phase II rule in 1999 under a separate consent decree. The proposal designated two classes of facilities to be automatically covered on a nationwide basis under the NPDES program:

(1) small municipal separate storm sewer systems located in urbanized areas (about 3,500 municipalities would be included in the program); and
(2) construction activities (pollutants include sediments and erosion from these sites) that disturb equal to or greater than one and less than five acres of land (about 110,000 sites per year will be included in the program).

Those facilities designated above would need to apply for NPDES stormwater permits by 2002. EPA is anticipating that most permittees would be covered under general permits.

Municipalities and others may be able to use retrofitting to comply with the new Phase II rules by converting existing dry ponds (or a portion thereof), used typically for flood and drainage control, to wet ponds. This is consistent the Phase I management program to use structural and source control measures to reduce pollutants in runoff from commercial and residential areas. Wet ponds, because of the permanent pool and longer detention times are thought to remove more pollutants than dry ponds (e.g., SS and BOD and COD removals) and reduce or eliminate discharges.

Some facilities that EPA is proposing to cover under the new Phase II rule are already subject to state and/or local stormwater management requirements. The Rockland, Maine case study demonstrates how retrofitting was incorporated into the Maine's CSO Abatement Program permitting requirements. Among other planned retrofits a new force main was commissioned to separate an industrial loading from the municipal sewer and the wastewater treatment facility is being upgraded to handle additional WWF.

CSO Control Policy

The need for high-rate treatment systems comes from the National CSO Policy that requires treatment of additional WWF in existing POTW during high flow periods caused by rainstorms. The recent EPA National CSO Control Policy (59 Federal Register 18688) resulted in the publication of several guidance documents. These two guidance documents are "*Combined Sewer Overflow – Guidance for Nine Minimum Controls*" (EPA 832-B-95-003) and "*Combined Sewer Overflow – Guidance for Long Term Control Plan*" (EPA 832-B-95-002). Required CSO abatement technologies that are addressed in this report are:

- Maximization of flow to the POTW for treatment;
- Evaluation of alternatives that enable the permittee, in consultation with the NPDES permitting authority, Water Quality Standard (WQS) authority, and the public, to select CSO controls that will meet CWA requirements;
- Cost/performance considerations to demonstrate the relationships among a comprehensive set of reasonable control alternatives; and
- Maximization of WWF treatment at existing POTW treatment plants

Consensus Recommendation of SSO Federal Advisory Subcommittee

The EPA/State SSO work group distributed a set of draft NPDES permit regulations and related guidance for municipal sanitary sewer collection systems and SSO to a small municipal outreach group, the SSO Federal Advisory Subcommittee and other members of the public. The SSO Federal Advisory Subcommittee made consensus recommendations to EPA on:

- capacity, management, operation and maintenance (CMOM) program requirements;
- a permit prohibition for SSO;
- permit record keeping, reporting and public notification requirements; and
- peak excess flow treatment facilities.

The SSO subcommittee recommended to EPA the substance of the CMOM, Prohibition, Record Keeping, Reporting and Public Notification, Remote Treatment Facilities documents, and Satellite Collection Systems and watershed management agreed principals on October 20, 1999. In addition, the Subcommittee recommended principles for satellite collection systems and watershed management.

The protocols for the Federal Advisory Subcommittee provide that EPA is committed to reflect the consensus reached by the SSO Subcommittee in an upcoming notice of proposed rulemaking. While the Advisory Subcommittee made recommendation addressing core issues, some outstanding issues remain. The consensus recommendation of the SSO Federal Advisory Subcommittee clears the way for the next step in the development of a proposed rule.

Retrofitting an existing treatment plant is often a more cost-effective way to control additional flow than building new facilities. With the upcoming SSO regulations, investigating the economics of retrofitting existing systems to treat more flow, may become the first step in a compliance exercise.

## Section 2
## Conclusions

Desktop analyses were performed for 13 separate retrofit examples. The analyses demonstrated that in certain circumstances retrofitting existing WWF facilities is technically feasible and may be more cost effective than construction of new conventional control and treatment facilities. A summary of each retrofit is provided in Table 2-1 and is discussed below.

Table 2-1. Summary of Retrofitting Desktop Analysis Projects

| ***Project*** | ***Retrofit*** | ***Treatment Benefit*** | ***Cost Benefit*** |
|---|---|---|---|
| *Augusta WWTF* | *Headworks Upgrade - Enhanced Primary Treatment* | *Yes* | *Yes* |
| *Hypothetical WWTF* | *ACTIFLO System -Enhanced Primary Treatment* | *Yes* | *No* |
| *Spring Creek AWPCP* | *Plate Settlers for Enhanced Treatment in WWF Storage Facilities* | *No* | *No* |
| *Spring Creek AWPCP* | *Chemical Addition for Enhanced Treatment in WWF Storage Facilities* | *Marginal* | *Marginal* |
| *Spring Creek AWPCP* | *Post-Storm Solids Removal for WWF Storage Facilities* | *Yes* | *Not Determined* |
| *Sunnyvale Detention Basin* | *Retrofit for Enhanced Treatment* | *Yes* | *Not Determined* |
| *Monroe St. Detention Pond* | *Retrofit for Enhanced Treatment* | *Yes* | *Not Determined* |
| *Birmingham Dry-Ponds* | *Retrofit for Enhanced Treatment* | *Yes* | *Not Determined* |
| *Erie Boulevard Storage System* | *DWF Augmentation* | *Yes* | *Marginal* |
| *Spring Creek AWPCP* | *DWF Augmentation* | *Yes* | *Marginal* |
| *Hypothetical Sanitary Sewer System* | *SSO Control Using Storage* | *Yes* | *Yes* |
| *Rockland WWTF* | *Retrofitting for Industrial Wastewater Control in a CSS* | *Yes* | *Yes* |

The headworks of the Augusta, ME WWTF were retrofitted to maximize flow to the existing primary and secondary tanks and to minimize wet-weather bypass. This was a very cost-effective alternative by maximizing existing plant capacity. This retrofit allowed for minimizing the construction of new high-rate treatment facilities. However, there are only limited experiences of such retrofitting documented in the literature.

Enhanced sedimentation methods employ flocculation with lamellar settling to speed up the clarification process. One such system, the ACTIFLO, was hypothetically retrofitted to a primary treatment facility. This technology provides enhanced SS removal with short detention times and quick start-up as compared to conventional primary clarification. One of the benefits of the ACTIFLO system is the extremely small footprint

of the process. The downside is the high cost of chemicals and increased O&M requirements.

Retrofitting existing WWF storage tanks for flow equalization and enhanced treatment was not cost effective. The Spring Creek Auxiliary Wastewater Pollution Control Plant (AWPCP) in New York, NY was used to evaluate this retrofit option. This is largely due to the high variability of WWF and limitations of the technologies evaluated. Retrofitting existing WWF storage tanks with cross-flow plate settlers was found to be technically limiting due to the hydraulic loading rates of cross-flow plate settlers. This technology may be more appropriate for primary clarifiers where hydraulic load rates are more consistent than in WWF storage tanks.

Retrofitting existing WWF storage tanks with chemical addition to provide chemically enhanced sedimentation of WWF is a feasible option. This technology could result in 60—90% SS removal, 40—70% BOD5 removal, and 30—60% chemical oxygen demand (COD) removal. This technology does not affect the operation of the storage tanks to serve their primary role of equalization.

The AWPCP was also used to evaluate several retrofitting options for post-storm solids removal from WWF storage tanks including tipping buckets, flushing gates, and orifice headers. The results of pilot testing and design considerations including capital and O&M costs are provided for each technology.

Retrofitting existing dry-ponds to provide treatment through sedimentation was found to be a relatively simple and viable treatment option. Three dry ponds were evaluated from different regions of the country: 1) Santa Clara, CA, 2) Madison, WI, and 3) Birmingham, AL. The retrofits significantly reduced effluent flows and downstream channel erosion and also provided a water quality benefit. This was done largely by developing features to better control influent and effluent flows and to reduce short-circuiting within the ponds.

The Erie Boulevard Storage System (EBSS) in Syracuse, NY and the AWPCP were evaluated for the purpose of serving a dual role as wet-weather and dry-weather equalization facilities. This analysis showed that this could be done without drastically reducing the ability of the storage tanks to capture WWF. The primary benefit of equalization provided by the DWF storage is the dampening of peak flow rates that would otherwise result at the treatment plant, allowing the primary and secondary treatment facilities to operate at a more uniform rate thereby provide more consistent treatment efficiency throughout a typical day. However, this retrofit is only economically viable for a treatment plant that is operating at or near design capacity where increased O&M costs can be balanced against cost to increase capacity at the plant.

The retrofit of an unused aeration tank at the Rockland WWTF in Rockland, ME and the construction of a separate force main from an industrial facility to the WWTF provided a cost-effective alternative for the elimination of high-strength wastes from CSO discharges. This retrofit conveyed high-strength waste from the industrial facility to the

dry-weather headworks thereby eliminating the potential for CSO discharges of industrial wastewater and consequent impact on the receiving water.

Retrofitting SSOs to use storage from overflow retention facilities provided a cost-effective alternative to expansion of the sanitary collection system to convey the volume of overflow to the treatment plant for primary and secondary treatment. However, this alternative requires the availability of a sufficient amount of land to construct the storage facility.

Retrofitting abandoned POTWs for use as WWF treatment facilities was shown to be an effective treatment alternative. The Auburn WWTF in Auburn, NY was retrofitted using the abandoned primary treatment tanks as WWF storage tanks to reduce the impact associated with WWF in excess of the capacity of the existing treatment plant. This retrofit was an extremely cost-effective alternative to other abatement options due to the availability and condition of the existing facilities. The abandoned primary tanks were in good condition and required only minor modifications. This retrofit is a cost-effective alternative in cases where abandoned primary tanks are available in good condition for retrofitting. This type of retrofit may be a less cost-effective solution for cases where the existing tanks are in poor condition requiring a substantial cost to upgrade the tanks to serve a functional role.

The feasibility and cost effectiveness of retrofitting is in large part controlled by site-specific conditions and treatment requirements. The physical condition of existing facilities, age, and level of maintenance, size, layout, hydraulic factors, water quality and other site-specific factors all affect the suitability for retrofitting. As such, the opportunity for retrofitting existing WWF facilities must be evaluated on a case by case basis.

## Section 3
## Recommendations

The findings of this study indicate that retrofitting existing control facilities can be a cost-effective alternative to the construction of conventional treatment facilities. The appropriateness of retrofitting is however, a function of site-specific needs and conditions. Therefore, it is recommended that retrofitting existing control facilities be identified as an alternative for wet-weather abatement. Retrofitting should be evaluated early on in the decision making process for wet-weather abatement.

The advanced sedimentation methods such as the ACTIFLO merit further study given the results for high-rate treatment applications in Europe and due to the limited experience in North America. This could include controlled, side-by-side pilot demonstrations to assess the benefits and cost effectiveness of alternative advanced sedimentation methods.

# Section 4
# Methods and Materials

## 4.1 Literature Search

### *Introduction*

Typically, urban infrastructure collects, conveys, and treats DWF. In the case of a CSO community, a component of WWF is collected and conveyed in the same infrastructure as the DWF, but points of relief are purposely provided before treatment because of hydraulic constraints of the conveyance system and the capacity of the dry-weather treatment facilities. In the case of an SSO community, the DWF (sanitary sewage) and the stormwater are collected and conveyed in different systems. However, these separated sanitary systems inadvertently receive I/I which result in overflows and consequent pollutant concerns. Regardless of how WWF are manifested, the essential infrastructure is made up of the collection, conveyance, and treatment systems. In each of these systems, retrofitting specific structures can provide improved control and treatment for WWF. Often surprisingly simple and inexpensive retrofits can provide expanded capacity, enhanced treatment, extended facility lifespan, and reduced operating costs.

This literature review focuses on retrofits made to collection, conveyance, and treatment systems in order to better control and treat wet-weather pollution. A majority of the cited literature was found through queries in such databases as,

- Environmental Sciences & Pollution Management Database
- Compendex*Plus
- Wilson Applied Science & Technology Abstracts
- ProCite Database,
- National Center for Environmental Publication and Information

Other literature was found through requests for information made to consulting engineering firms. An emphasis was placed on national conference proceedings. The referenced databases are described in Appendix A.

### *General Retrofitting Information*

Martin (1988) developed a method to help government watershed planners and managers to: (1) assess the pollutant potential of runoff from existing urban development, (2) screen for feasible control measures, and (3) develop and implement retrofit management strategies. This method strongly prompts the implementation of watershed retrofits as remedial actions for stormwater pollution.

Walesh (1992) discusses the potential for retrofitting stormwater facilities. The paper summarizes the history of urban stormwater detention facilities and suggests that many of these existing facilities can be retrofitted to improve quantity control, include quality

control, improve O&M, reduce safety hazards, enhance aesthetic attributes and have recreational features. Beale (1992) and England (1995) also discuss retrofitting stormwater facilities. Both authors address design considerations for stormwater retrofitting techniques. Such techniques include detention ponds, retention ponds, underground vaults, exfiltration pipes, and baffle boxes. Palhegyi, Driscoll, and Mangarella (1991) evaluated the feasibility of retrofitting stormwater control facilities to improve pollutant-removal performance. This paper presents the results of an analysis that was performed to predict the removal efficiency of suspended sediments in storm runoff under existing conditions and under several practical retrofit alternatives. A flood risk analysis was performed for each condition. It was concluded that stormwater quality could be improved by modifying operating strategies of existing facilities, but a flood risk analysis must be considered.

Field and O'Connor (1997) discuss a strategy to optimize CSO control systems. The optimization strategy maximizes the use of the existing system and sizes the storage facility in accordance with the WWTF capacity to obtain the lowest-cost storage and treatment system. Roesner and Burgess (1992) discuss CSO rehabilitation strategies for urban areas. Roesner and Burgess address and cite examples for eliminating dry-weather overflows, rehabilitating regulators, and optimizing existing interceptor capacity.

### *Collection System Retrofits*

The practice of utilizing inlet control concepts to reduce the impacts of flooding from urban runoff and pollution through catchbasin adjustments is common. Most installations involve restriction of catchbasin outflow to cause surface inflow to be re-directed by overland routes to more suitable discharge points or to underground storage. The majority of installations involve retrofitting existing systems to mitigate collection-system surcharging, reduce exfiltration and infiltration exchange within adjacent sewer systems, and to cause reduction of downstream CSOs (Pisano, 1990).

The area known as the "Triangle" in Parma, Ohio, is a 12.1 ha (30 acre) low-relief area in a 117.4 ha (290 acre) drainage system. In the "Triangle" storm drains are generally inadequate and severe street flooding and basement flooding are common. Inlet controls such as overland flow berms, vortex throttles, and shallow drains to de-water street areas were implemented throughout the entire 117.4 ha (290 acre) drainage area. The project mitigated surface-water ponding and provided basement-flooding protection throughout the entire 117.4 ha (290 acre) area. Construction costs in 1984 were $875,000 (Pisano, 1982).

Lake Quinsigamond, Worchester, Massachusetts, was becoming eutrophic from stormwater discharges. Studies indicated that the dominant nutrient source was contaminated DWF from the storm sewer. It was believed that this contamination resulted from the mixing of the sanitary sewer with the storm sewer due to the "over/under" sewer configuration. In this configuration the sanitary sewer was placed over the stormwater sewer in the same trench. A system wide pipe replacement or rehabilitation was rejected based on cost. Therefore, a number of system controls were

installed; these were found to immediately eliminate much of the nutrient loading from the storm sewer base flow. A vortex flow throttle was installed in the storm sewer to divert the polluted base flow and the first flush to a nearby combined sewer. Inlet controls were then installed in the upper reaches of the combined system to increase its capacity to convey the polluted base flow and the first flush. Approximately 0.34 $m^3/s$ (12 cfs) of stormwater (base flow and first-flush) were diverted to the combined system. Total construction cost were estimated at $160,000 (Pisano, 1989).

### *Conveyance System Retrofits*

The conveyance system is the infrastructure component designed to move water from the site of origin to an acceptable receiving water body. In the case of dry-weather sewage flow, the conveyance system carries wastewater to a treatment plant. In the case of WWF, this water may be discharged into a water body without treatment. In many stormwater instances, detention ponds are an integral part of the conveyance system. In the past, these detention ponds were used primarily for the purpose of urban flood control. In some instances, ponds are retrofitted for the capture and treatment of stormwater, thereby improving the quality of the receiving water body.

#### Flow Equalization

Fairport Harbor, Connecticut, developed a unique solution to eliminate CSOs by converting an abandoned industrial fuel oil tank into a cost-effective wet-weather retention tank. The fuel oil tank was utilized for flow equalization after the original plan to construct a new tank and pump station near the outfall met local resistance. The conversion of the fuel oil tank has removed an environmental liability, saved the district money, and reduced CSOs. An initial examination of the 36.6 m (120 ft) diameter fuel oil tank indicated it had sufficient capacity 12,112 $m^3$ (3.2 MG) for the intended capture of the five-year intensity storm. However, of primary concern were structural integrity and the presence of industrial sludge and asbestos. Rehabilitation of the tank began with the removal of the crude oil sludge, interior piping, asbestos-covered exterior piping, and lead-based paint. Welds that did not meet American Welding Society standards were replaced. Utilization of the fuel oil tank also required building a pump station and force main designed for a peak flow rate of 19,870 $m^3/d$ (5.25 million gallons per day or MGD). The project total cost was $2.6 million (Shrout, 1994).

The Rohnert Park Demonstration Project included the design, construction, operation, testing and evaluation of a surge facility designed to provide flow equalization and some degree of treatment to all WWF and to provide rate control of wet-weather and dry-weather wastewater flows to interceptor sewers. The principal feature of the surge facility was a 2840 $m^3$ (0.75 million gallon or MG) sedimentation-equalization basin. It was tested under field conditions with influent flows varying from 2,270—20,800 $m^3/d$ (0.6—5.5 MGD). The ability of the surge facility to provide adequate hydraulic control to diurnal flow variation was documented. Under WWF conditions, the surge facility was able to remove approximately 45% of the influent SS. BOD removal under WWF

conditions was not significant. The total construction cost for the demonstration facility was $384,000 (Welbon, 1974).

Detention Basins

Santa Clara Valley, California is protected from flooding by a series of pump stations and levees. Based on a preliminary evaluation of the feasibility of retrofitting detention basins, a pilot study was implemented to retrofit a pump station and to conduct testing to measure water quality benefits and costs. In the pilot study, structural and operational retrofits were implemented at a pump station consisting of four primary pumps rated at 1.1 $m^3$/s (39 cfs) and one auxiliary pump rated at 0.25 $m^3$/s (9 cfs). The detention basin area is 1.8 ha (4.4 ac.) with a 37,005 $m^3$ (30 ac-ft) capacity and receives water from a 187.4 ha (463 acre) urbanized watershed. The structural retrofits included installing a weir at the outlet of the detention pond, filling a channel that connected the inlet and the outlet of the detention pond, and blocking off a drainage pipe that ran under the channel. The purpose of these structural retrofits was to reduce short-circuiting of the pond and provide better distribution of the flow into the outlet. The operational retrofit consisted of modifying the pump schedule to create a two-foot permanent pool in the pond. These retrofits provided greater retention time in the detention pond and therefore enhanced gravitation settling of SS. Post-retrofit water quality surveys showed removals of 29% for chromium, 42% for total copper, 53% for lead, 51% for total nickel, 44% for total zinc, and 50% for SS. The metals removals correlated well with the SS removal. The amortized cost over 20 years for retrofitting, operation, and maintenance was $8,200/year (Woodward-Clyde, 1996).

Dye-tracing experiments performed in a stormwater pond in Kingston Township, Ontario revealed short retention times, mainly due to the small size of the pond. Mathematical modeling of this pond indicated irregular circulation patterns, resulting in short-circuiting and dead zones. A study was conducted to retrofit the pond for the purpose of improving pollutant-removal characteristics. A series of three baffles were installed along the length of the pond, roughly perpendicular to the direction of flow, and extending an average of two-thirds of the width across the pond. The installation of the baffles increased the length-to-width ratio of the flow path in the pond from 1.5:1 to 4.5:1 and corrected the flow regime problems. In addition, an increase in the hydraulic efficiency of the pond (defined as the ratio of measured to volumetric retention times) from 0.65—0.86 was observed. An increase in pollutant removal through sedimentation processes was inferred from a comparison of retention time measured before and after baffle installation (Matthews et al., 1997).

Florida's Indian River Lagoon suffers from drainage problems such as increased volume of freshwater runoff to the estuarine receiving water and deposition of organic sediments, reduced water clarity because of increased discharge of SS, and eutrophication caused by nutrient loadings. A project was initiated to create a watershed control system for the Indian River Lagoon, develop management strategies to relieve stresses resulting from runoff to the lagoon, and address the feasibility of watershed retrofitting to reduce pollutant loads. The partially developed, 768.9 ha

(1,900 acre), industrial area of NASA's Kennedy Space Center was selected as a representative drainage catchment. After screening the retrofitting alternatives, it was determined that constructing a weir in the main drainage channel, with discharge of diverted flow to a wetland area, would provide the best system performance. The cause and effect relationships between the catchment hydrology, channel hydraulics, and pollutant loads were documented using a calibrated SWMM model. Model results concluded that the retrofit could satisfactorily achieve flood control, increase SS removal, and maximize water depth, but could have difficulty meeting the groundwater discharge and water level-fluctuation criteria. Without periodic drawdown from the wetland, water levels in the system would be nearly static due to the nearly flat relief. Fixed cost of the proposed retrofit would be $28,000. The annual O&M costs were estimated to be $17,100 (Bennett and Heaney, 1989).

Chemical Enhanced Treatment

As evidenced by the above-referenced literature, many structural retrofits have been implemented to improve the treatment characteristics of wet-weather detention basins. In addition to structural retrofits, some municipalities have experimented with chemically enhanced wet-weather detention basins to improve the treatment characteristics of these basins during wet-weather events. This low-structural retrofit requires small amounts of chemicals and polymer to promote the coagulation and flocculation of smaller, lighter particles into larger, heavier particles.

The Washington State Department of Transportation designed and constructed a scale model detention basin to investigate contaminant-removal capabilities. Using typical contaminants and concentrations, a simulated highway stormwater runoff was formulated and applied to the model detention basin over a range of flow rates. Four coagulants were evaluated for their ability to enhance removal of sediment and metals. Coagulant addition resulted in significant increases in metals removal over the range of stormwater flow rates studied. The greatest improvement was observed at the higher flow rates. Further improvements in contaminant removal were observed following the addition of an influent baffle. This baffle increased the hydraulic detention time by reducing short-circuiting with an associated improvement in contaminant removal (Price and Yonge, 1995).

A restoration project began in 1987 in an effort to improve water quality in Lake Ella, Tallahassee, Florida. This effort constituted the first use of alum for treatment of stormwater inputs into a receiving water body. Lake Ella is a shallow, 5.3 ha (13 acre) lake that received stormwater runoff through 18 separate storm sewers from 64.8 ha (160 acre) of mostly impervious urban watershed. With a volume of 113,550 $m^3$ (30 MG), Lake Ella receives 518,550 $m^3$ (137 MG) of runoff each year. The lake was highly eutrophic and fish kills were common before installation of the alum stormwater treatment system. Initially, common stormwater treatment technologies such as retention basins and exfiltration trenches were considered. Alum treatment was considered when it was determined that there was no available land surrounding the lake that could be used for retention and the cost of purchasing homes to acquire land

for construction of retention basins was cost prohibitive. The stormwater treatment system was implemented using flow meters and variable speed injection pumps to automatically inject liquid alum into the storm sewers. Mixing of the alum and stormwater occurs as a result of turbulence in the storm sewer line, and the floc produced settles on the lake bottom providing an added benefit of nutrient inactivation in the sediments. In addition to the non-structural alum retrofit, bar type trash traps were installed in underground structures upstream of the lake; the primary outfall lines were extended into the lake about 30.5 m (100 ft). The total cost of the alum injection system, excluding the extension of the storm sewer lines into the lake, was about $200,400. The capital cost of this system was $3,140/ha ($1,270/acre) of watershed area treated. The capital costs involved in constructing an alum treatment system are relatively independent of watershed size and dependent primarily upon the number of outfall locations treated (Harper and Herr, 1992).

Polymer injection has also been used for the control of sewer overflows in the city of Dallas, Texas. An EPA-sponsored research program studied a non-structural retrofit system wherein the capacity of a sewer was increased by the injection of certain water-soluble chemicals to reduce turbulent friction. A polymer injection rate of 0.54 kg/min (1.2 lbs/min) into a 38 cm (15 in.), surcharged pipe conveying 18.9 $m^3$/min (5,000 gallons per minute [gpm]) reduced head from 2.5 m (8 ft) above the pipe to 1.8 m (6 ft) above the pipe. The capacity of a sewage lift station was also increased by injection of a slurry of friction-reducing polymer into the pump intake. Polymer doses of 120 and 350 ppm were sufficient at increasing flow in a 15.2 cm (6 in.) force main by 35% and 64%, respectively (Chandler and Lewis, 1977).

### Real-Time Control

Real-time control systems are gaining a renewed interest for the control of wet-weather overflows. In 1967, the U.S. federal government funded demonstration projects to investigate available technologies for control of CSOs. Demonstration grants were awarded to sewer agencies in Cleveland, Ohio; Seattle, Washington; Minneapolis, Minnesota; and Detroit, Michigan (Chantrill, 1990). These projects used computers for the coordination of storage at different sites in a sewer collection system to optimize use of storage on a system wide basis. All of the projects experienced problems getting the central computers operational largely due to the limitation of computer technology at the time. Today, municipalities have many more options for implementing computer-based real-time control of wet-weather overflows.

Delattre (1990) outlined real-time control of combined sewer systems from the user's perspective. Delattre's discussion included design objectives and operating functions, implementation of supervision and control schemes, and evaluation of operation performance thorough case studies. Chantrill (1990) discussed the history of real-time control of combined sewers in the United States. Chantrill's discussion included the control software developed in Seattle, Washington and Lima, Ohio, and the development efforts by Cello Vitasovic and Wolfgang Schilling.

### *Treatment Facility Retrofits*

Increasing Capacity/Efficiency of Existing WWTF

Retrofitting dry-weather treatment plants to handle increased flows from wet-weather events has proven to have merit in some communities. Typically it involves increasing the capacity of the existing treatment plant by installing newer, more efficient equipment.

The town of Dexter, New York, solved an infiltration problem by upgrading a treatment plant with new aeration units rather than rehabilitating the collection system. Peak flows of more than 2,271 $m^3$/d (0.6 MGD) from wet-weather events and groundwater infiltration were washing bacteria out of the 379 $m^3$/d (0.1 MGD) activated sludge WWTF. The town of Dexter decided to treat the excess infiltration, but did not have enough space at the plant to build a larger mixed-liquor system to handle excess flows. Installing prefabricated fixed-film treatment units that offered improved treatment efficiency and quick bacterial growth (Myers and Wickham, 1994) solved this problem.

The city of Clatskanie, Oregon, conducted an infiltration/inflow (I/I) study and found that the collection system was subjected to severe I/I. Additionally, deficiencies existed in the trickling filter plant. A cost-effectiveness analysis indicated that the development of a joint dry/wet-weather facility without major collection system improvements was the best solution. The principal features of this alternative included a single primary clarifier followed by an activated sludge secondary treatment process with a DWF capacity of 1,893 $m^3$/d (0.5 MGD). The secondary process can be operated in a contact-stabilization mode, and the primary clarifier can be operated in a dissolved air flotation (DAF) mode. With these process modifications, flows up to 4,731 $m^3$/d (1.25 MGD) could be effectively treated. SS and BOD removals during DWF were both 94%. WWF removal efficiencies for flows ranging from 1,893—8,706 $m^3$/d (0.5—2.3 MGD) were 71% and 73% for SS and BOD respectively. The capital cost of the DAF-contact stabilization capability was estimated to be 14% more than the cost of a standard dry-weather plant (Whitney-Jacobsen and Associates, 1981).

Modifications to the clarification system at the main WWTF in New Orleans, Louisiana have resolved a recurring hydraulic overloading problem caused by inflow and infiltration. The modifications also have increased average waste activated sludge concentrations by 50%. Sluice gates were installed to direct flows equally to the clarifiers during normal flows. These gates are adjusted during high flows to allow for optimal flows through the clarifiers, thereby eliminating sludge-blanket scouring that occurred before the gates were installed. A polymer injection system was also added to the thickener-clarifier to further enhance settling (Berry, 1994).

The Acheres WWTF in Paris, France was designed to treat the carbonaceous portion of dry-weather sewage. The city is currently undergoing a major rehabilitation and retrofitting plan to upgrade the plant to include nutrient removal and wet-weather treatment. By the year 2001, the plant will be retrofitted with tertiary treatment for

nutrient removal and physical-chemical treatment for wet-weather treatment. The wet-weather treatment facility was designed to treat 0.76 $m^3$/min (200 gpm), almost twice the 0.42 $m^3$/min (110 gpm) DWF (Gousailles, 1995).

Physical-Chemical Treatment

The physical-chemical treatment system at the Acheres wastewater plant will use the ACTIFLO process. This process incorporates lamellar settling and weighted flocculation by microsand. Guibelin, Delsalle and Binot (1994) presented operational and performance results from the ACTIFLO process. They established that the process allows an upflow velocity of 131 m/h (0.12 fps) over the lamella. Under this flow regime, 80% SS and 60% BOD removal is achieved using 60g/$m^3$ of ferric chloride and 0.8g/$m^3$ of polyelectrolyte. The process is also noted for its quick start up time, an important criteria for treating WWF.

Another physical-chemical treatment system being studied in France is the Densadeg unit (Le Poder and Binot, circa 1995; Rovel and Mitchell circa 1995). This unit combines three principles: lamellar settling, coagulation-flocculation, and built-in thickening. The Densadeg pilot unit has been studied at WWTF and at representative sites along a combined sewer system. The researchers documented SS removals of 80% at a velocity of 50 m/h (0.5 fps) and noted that coagulant dose ($FeCl_3$) affected treatment results more than velocity. Densadeg units are to be installed at the new Colombes sewage treatment plant (240,000 $m^3$/d [63.4 MGD]) in Paris, France. A key feature of the Colombes is how it will be operated during wet-weather events. Rather than sequential operation of the biological stage as during dry-weather, the stages will switch to operate in parallel during periods of high flow (Hayward, 1996).

Nenov (1995) compared the effectiveness and costs of physical-chemical treatment versus the activated sludge process. Laboratory experiments confirmed the high efficiency of physical-chemical treatment for SS and BOD/COD removal. The data obtained show that chemical treatment over a large range of surface flow rate provides a reduction of SS and BOD/COD greater than 60% and 50%, respectively. Based on capital and O&M costs from the Ravad WWTF it was shown that physical-chemical treatment retrofitting could be done at a much lower cost than biological treatment expansion.

Chemical Enhanced Treatment

As an alternative to physical-chemical treatment, other municipalities have experimented with chemically enhanced treatment to increase plant capacity during wet-weather events.

Sarnia, Ontario, studied the effectiveness of chemically enhanced primary treatment to increase and sustain removal efficiencies during high plant flows for more than ten years at an existing primary plant. Very little decrease in treatment efficiency was observed at overflow rates as high as 114.1 $m^3/d/m^2$ (2,800 gpd/$ft^2$). The study

concluded that, with the addition of 14 ppm of ferric chloride and 0.3 ppm polymer, the SS and BOD removals averaged 84 and 60%, respectively (Heinke, Tay, and Qazi, 1980).

In March 1990, a 22 day test of chemically enhanced treatment was conducted at the South Essex Sewerage District (SESD) primary treatment plant in Salem, Massachusetts. The testing conditions were unique in that the flow was split and the plant was operated as two distinct and parallel treatment lines: half of the plant received chemical additives while the other half operated as a conventional primary plant. BOD removals on the conventional primary side averaged 17% but were 51% on the chemically treated side (a 200% increase). SS, fats, oils and grease removals increased by 60%. The test also demonstrated chemical addition can enable settling tanks to handle maximum stormwater conditions without deterioration in removal efficiency. Ferric chloride and polymer concentrations totaled approximately 30 ppm. Average overflow rates of 32.6 $m^3/d/m^2$ (800 $gpd/ft^2$) reached a peak of 77.4 $m^3/d/m^2$ (1,900 $gpd/ft^2$) during wet-weather events. The SESD plant's average DWF of 94,625 $m^3/d$ (25 MGD) increased to over 340,650 $m^3/d$ (90 MGD) during a storm on the ninth day of the test. SS and BOD removal rates did not decrease during or after this event (Harleman, Morrissey, Murcott, 1991).

Converting Dry-Weather WWTF to a Wet-Weather Facility

Conversion of existing or abandoned dry-weather treatment plants into wet-weather holding or treatment facilities is an option when communities build new dry-weather treatment plants.

The Municipality of Toronto Metropolitan (Metro) has recently completed the Main Treatment Plant (MTP) Environmental Assessment (EA). One of the goals of the EA was to establish a preferred alternative for meeting CSO treatment needs. Metro placed a high priority on source control as a means of reducing CSO's and associated pollutant loadings. Sewer separation was not considered as the only solution to the CSO problem because of concerns relating to increased stormwater discharges, the legality of separation on private properties, and high costs. The preferred strategy included, among other solutions, conversion of a WWTF to a CSO treatment plant (CG&S, 1997).

## 4.2 Use of SWMM

SWMM is a large, complex model capable of simulating the movement of precipitation and pollutants from the ground surface through pipe and channel networks, storage/treatment networks, and finally to receiving waters. The model is capable of simulating a single event or on a continuous basis for extended periods of time.

SWMM has been released under several different official versions (Metcalf and Eddy, Inc., et al. 1971; Huber et al. 1975, 1984; Roesner et al. 1984; Huber and Dickinson

1988; Roesner et al. 1988). The official versions were primarily designed for mainframe computer use; however, later versions have been modified for use on personal computers. The current "official" version, version 4, is available on disks or through the Internet from the EPA at the following addressees:

Center for Exposure Assessment Modeling (CEAM)
U.S. Environmental Protection Agency
960 College Station Road
Athens, Georgia 30613
Phone: 706/355-8400
E-mail: ceam@epamail.epa.gov
WWW: ftp://ftp.epa.gov/epa_ceam/wwwhtml/ceamhome.htm

The complete SWMM model documentation (Huber and Dickinson 1988; Roesner et al. 1988) can be obtained from the National Technical Information Services (NTIS) or Oregon State University:

National Technical Information Service
5285 Port Royal Road
Springfield, Virginia 22161
Phone: 703/487-4650
NTIS no. PB88 236 641 – Huber and Dickinson (1988)
NTIS no. PB88 236 658 – Roesner et al. (1988)

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Phone: 503/737-4934
E-mail: huberw@ccmail.orst.edu

The SWMM consists of five basic blocks (or components) that can be used together or separately. The blocks are RUNOFF, TRANSPORT, EXTENDED TRANSPORT (EXTRAN), STORAGE TREATMENT (S/T) and RECEIVING WATER (RECEIVE). The RUNOFF Block simulates rainfall, the resulting surface runoff quantities, and hydrographs for each drainage basin of the combined sewer collection system. The EXTRAN Block routes the surface runoff hydrographs developed in the RUNOFF Block through the collection system to the point of interception and overflow.

SWMM was used to perform simplified simulations using historical rainfall data for a period of one year or the average of multiple years of hourly rainfall data with hourly time steps or an appropriate equivalent. This was done using the RUNOFF and TRANSPORT Blocks of SWMM. The RUNOFF Block generates surface runoff hydrographs from a drainage area in response to precipitation based on watershed characteristics including size, shape, imperviousness, and slope. The RUNOFF block

uses the continuity equation that tracks the volume and depth of water on the ground surface and Manning's equation that governs the rate of surface runoff.

These hydrographs were then used as input into the TRANSPORT Block that routes flow through a sewer system that is depicted as a series of manholes, conduits, pump stations, storage tanks, and overflow structures. TRANSPORT uses a simplified version of the momentum equation where flow is a function of depth to route flow through the collection system.

Unless otherwise noted, simplified simulations conducted for these analyses were done using existing calibrated and validated RUNOFF and TRANSPORT models, modified to estimate the pre- and post-retrofit conditions to assess the treatment/capture effectiveness of the retrofit.

### 4.3 Case Studies and Hypothetical Retrofit Examples

The desktop analyses provided in this document are a mixture of case studies of actual retrofit examples and hypothetical retrofit examples where actual retrofit examples were not readily available. The case studies include site specific details of each retrofit including description of facilities, operational parameters, flow rates, and costs. The hypothetical examples are used to provide a conceptual overview of retrofitting opportunities and allow comparison to other retrofitting examples. In some instances, actual case studies were labeled as "hypothetical" to allow the use of information without revealing the source to protect confidential information. In these instances, actual information was used to the greatest extent possible to increase the accuracy of the desktop analysis.

## Section 5
## Desktop Evaluation

### 5.1 Conversion of Primary Treatment Tanks

Conversion or retrofitting primary settling tanks at POTWs to provide equivalent or higher levels of treatment may be a technically feasible and economically attractive alternative to the construction of new conventional primary settling tanks. The principal downside of conventional primary treatment facilities is the large footprint required due to the low surface overflow rate (SOR). Alternative sedimentation methods including micro carrier enhancement, chemical precipitation, dissolved air floatation, and plate/tube settling may provide an alternative to conventional primary treatment.

Two desktop analyses are presented to demonstrate the retrofit of primary treatment tanks to provide equivalent or higher levels of treatment:

1. Retrofit of the Augusta Wastewater Treatment Facility, Augusta, ME
2. Hypothetical retrofit of the POTW using the ACTIFLO process

#### *5.1.a. Augusta Wastewater Treatment Facility, Augusta, Maine*

The Augusta, Maine Sanitary District (ASD) owns and operates a secondary wastewater treatment facility (WWTF) that discharges to the Kennebec River, which is a Class C waterbody for the state of Maine. The WWTF was originally constructed in 1962 with upgrades in 1966, 1982 and 1997. The ASD WWTF provides secondary treatment using pure-oxygen activated sludge. The WWTF was designed to handle a monthly-average flow of 30,280 $m^3$/d (8 MGD) and an instantaneous-peak flow of 60,560 $m^3$/d (16 MGD). The WWTF serves a partially combined collection system (CCS). ASD's existing collection system services approximately 90% of the residential population, nearly all of the commercial properties, and a portion of the industrial properties within the limits of the city of Augusta. The system also serves four neighboring communities, the city of Hallowell and the towns of Manchester, Winthrop and Monmouth. The system contains three types of sewers:

- combined sanitary and storms
- separate sanitary
- storm

The majority of the collection system within the limits of Augusta flows by gravity to the WWTF. Ten pumping stations exist serving the lowlands and outlying areas. Currently, there are over 167 km (104 miles) of gravity sewer ranging from 15—107 cm (6—42 in.) in diameter. Pipe materials include stone and brick (dating back to the Civil War era) as well as more recent installations of vitrified clay, asbestos-cement, concrete, cast iron, and PVC pipe. According to the 1993 CSO Facilities Plan there are presently 38 diversion

manholes and 29 permitted CSO located within the system. In an average year 219,530 $m^3$ (58 MG) are discharged.

A majority of the wastewater currently entering the ASD sewer system is domestic sewage from residential and commercial sources. The original WWTF design included an allowance for a significant organic loading from local industry; however, many of the local industries closed or relocated soon after Secondary Treatment System upgrade became operational in the early 1980's. Consequently, the WWTF is operating with less than one-half of its original design flow and organic loading. Currently, annual average wastewater flows are 15,140 $m^3$/d (4 MGD).

Prior to implementation of the CSO Abatement Program, the WWTF used the following unit processes to provide preliminary, primary and secondary treatment:

- One mechanically-cleaned bar screen with 3.8 cm (1½in.) clear spacing located in a 1.5 m (57 in.) wide channel
- One detritor-type grit chamber 4.3 m (14 ft) in diameter
- Two 91.5 cm (36 in.) comminutors with a manual bypass screen
- Three primary clarifiers, of which two are 16.8 m (55 ft) in diameter and one is 24.4 m (80 ft) in diameter, for a combined surface area of 910 $m^2$ (9,800 $ft^2$)
- Four main primary effluent lift pumps
- Two pure-oxygen activated sludge aeration tanks with a total volume of 2,725 $m^3$ (0.72 MG)
- Three secondary clarifiers, each at 24.4 m (80 ft) diameter with a total surface area of approximately 1400 $m^2$ (15,100 $ft^2$.)
- One chlorine contact tank with a volume of 613 $m^3$ (162,000 gal)

Figure 5.1.a-1 shows the existing site layout prior to implementing the CSO Abatement Program. Figure 5.1.a-2 shows a schematic of the existing WWTF prior to retrofitting. When influent peak flow approaches 34,065 $m^3$/d (9 MGD), excess flow is bypassed to the chlorine contact tank following degritting in the detritor. The bypass was automatically activated by a side overflow weir located along the grit chamber outlet channel. The water level was regulated by a 30.5 cm (12 in.) Parshall Flume located downstream of the weir.

The CSO Abatement Program approved by the Maine Department of Environmental Protection recommended a four-phase plan to address CSO discharges. Phase I of the CSO Abatement Program included improvements to provide treatment for WWTF bypass that is the third largest CSO in the CCS based on projected peak flows. The following is a summary of Phase I Improvements, which is a retrofit to the existing facilities:

Figure 5.1.a-1. Existing Site Layout Prior to Retrofit

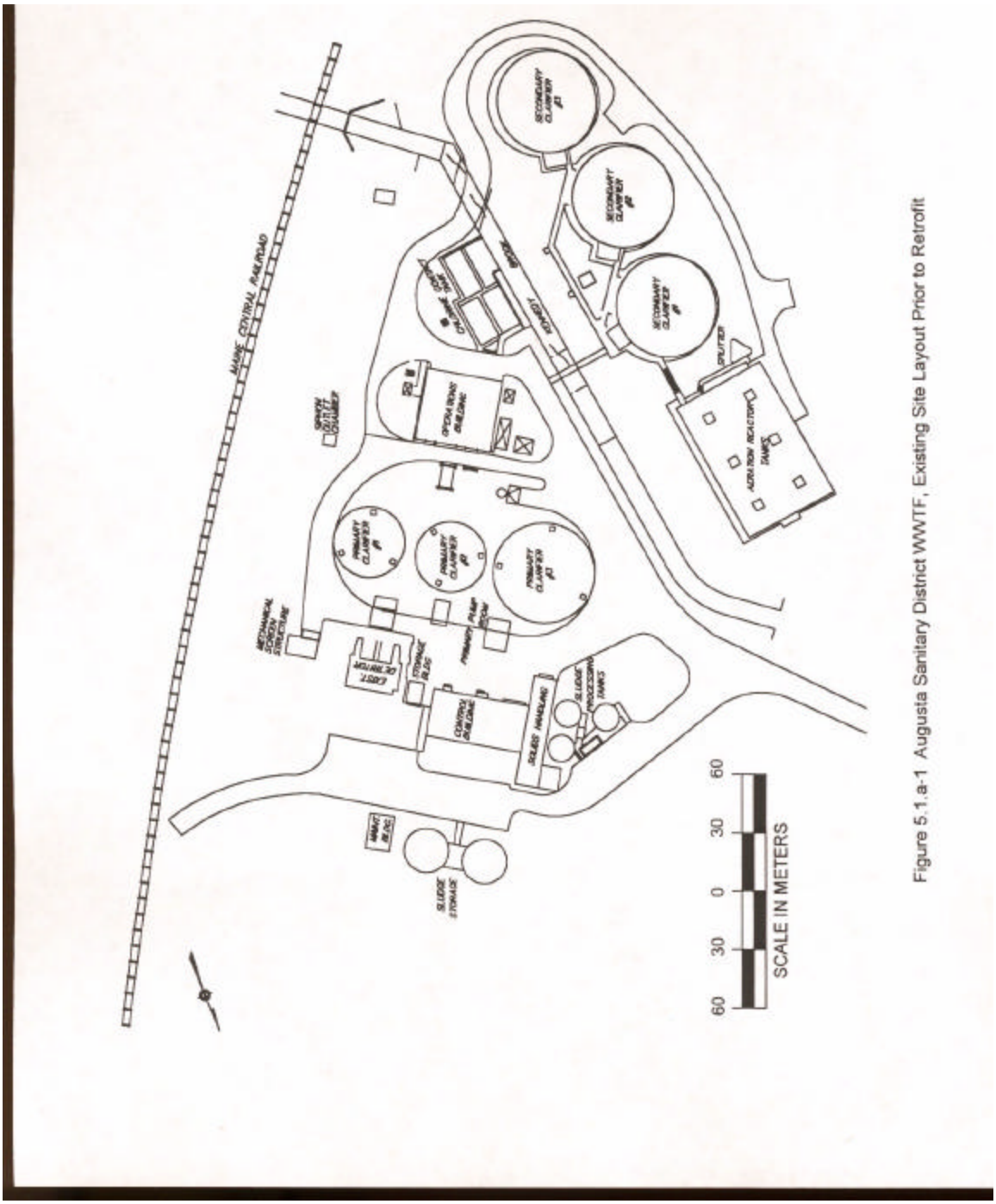


Figure 5.1.a-1 Augusta Sanitary District WWTF, Existing Site Layout Prior to Retrofit

Figure 5.1.a-2. Process Flow Schematic Prior to Retrofit

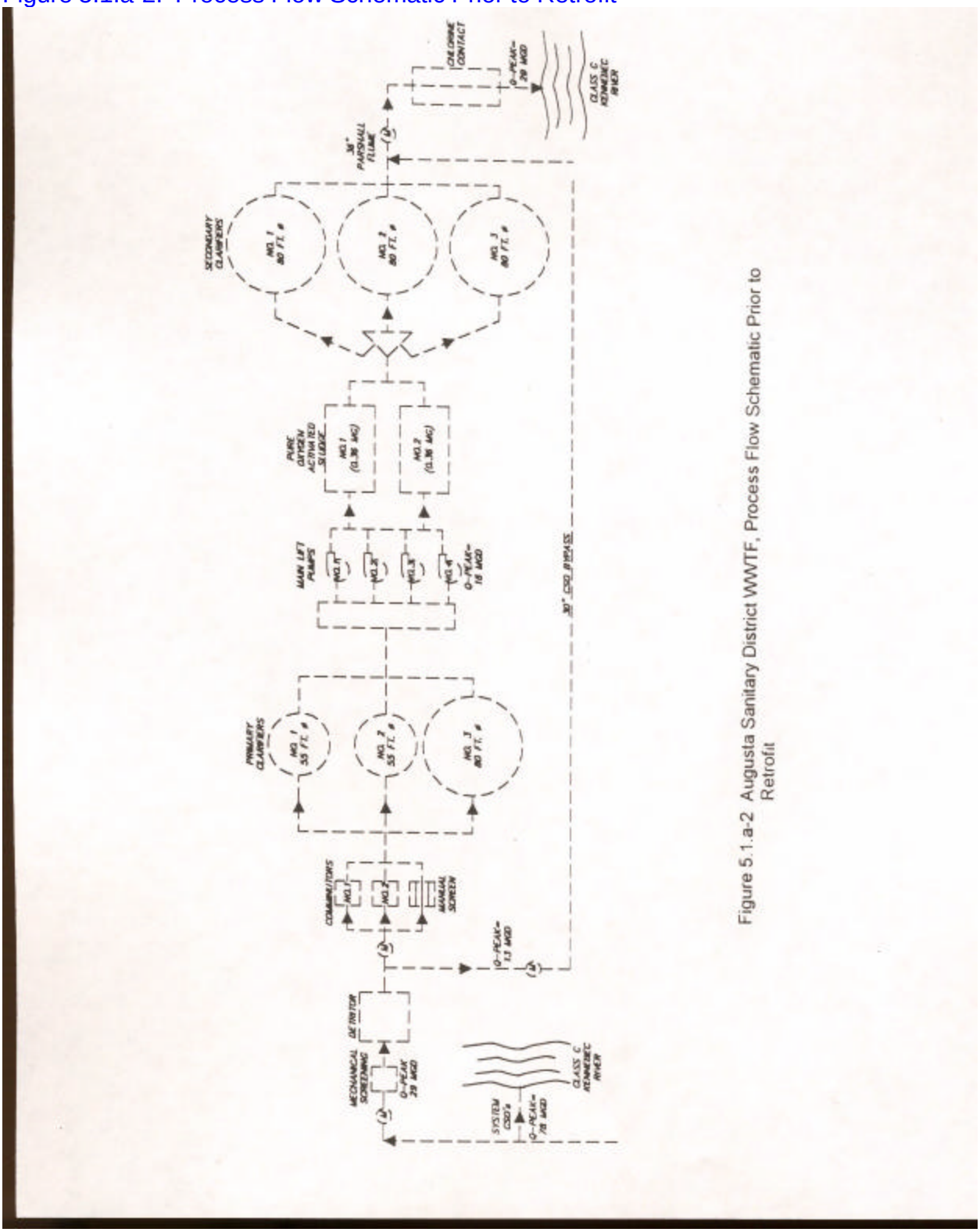


Figure 5.1.a-2 Augusta Sanitary District WWTF, Process Flow Schematic Prior to Retrofit

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- A new headworks containing two mechanical screens with 2.5 cm (1 in.) clear spacing located in a 1.5 m (60 in.) wide channels, two 167 $m^3$ (44,000 gal) aerated grit chambers, and one 0.9 m (36 in.) parshall flume. The mechanically cleaned bar screens have screenings compactors to remove excess water. The grit is removed from the aerated grit chambers using recessed impeller type pumps which discharge to cyclone classifiers for washing and dewatering.

- A new flow distribution structure (FDS No. 1) for the primary clarifier influent. The distribution structure utilizes motorized weir gates that will allow off-line tanks to be activated during a wet weather event.

- A new primary effluent distribution structure (FDS No. 2) with motorized weir gate to regulate flow receiving secondary treatment and flow to the CSO bypass.

- A new 250 $m^3$ (66,000 gal) chlorine contact tank with mechanical mixers using sodium hypochlorite for high-rate disinfection. A 24 $m^3$ (6,400 gal) bulk storage tank is provided for sodium hypochlorite storage. An 11 $m^3$ (3,000 gal) dechlorination contact chamber with mechanical flash mixer is provided. A 16 $m^3$ (4,150 gal) bulk storage tank is provided for sodium bisulfite. Chemical metering pumps are provided with flow pacing from an ultrasonic flow meter.

These improvements will allow the full 109,800 $m^3$/d (29 MGD) of connected interceptor capacity to receive preliminary and primary treatment. Daily-maximum flows of 45,400 $m^3$/d (12 MGD) would receive secondary treatment while 64,300 $m^3$/d (17 MGD) would be diverted to the high-rate disinfection system after receiving primary treatment.

Later phases of the CSO abatement program (Phases II, III and IV) will allow additional WWF from the Augusta CCS to be treated at the WWTF. This will be accomplished through the construction of:

- Consolidation conduits to intercept CSO discharges throughout the system and convey the flow to the WWTF;

- Three new vortex separators, each 10.7 m (35 ft) diameter, and 2.4 m (8 ft) side water depth to provide the equivalent of primary treatment;

- Additional high-rate disinfection and dechlorination tank capacity to increase the tank volume to 1,250 $m^3$ (330,000 gal) for disinfection, and 62.5 $m^3$ (16,500 gal) for dechlorination;

- A new wet-weather outfall from the WWTF to the Kennebec River.

After all phases are completed, the total influent peak flow capacity to the WWTF would be increased from 109,800 $m^3$/d (29 MGD) to 405,000 $m^3$/d (107 MGD). Up to 295,200 $m^3$/d (78 MGD) will be treated in the vortex separators. The remaining 109,800 $m^3$/d (29 MGD) would be conveyed to the upgraded headworks. Approximately 268,700 $m^3$/d (71 MGD) of vortex separator effluent will be disinfected and dechlorinated in the expanded high-rate

disinfection tank. Approximately 26,495 $m^3$/d (7 MGD) of vortex separator underflow would be conveyed to the upgraded headworks. A total flow of 136,260 $m^3$/d (36 MGD) would be treated within the headworks and primary clarifiers (109,765 $m^3$/d or 29 MGD influent plus 26,495 $m^3$/d or 7 MGD vortex underflow). Primary clarifier effluent flows of approximately 45,420 $m^3$/d (12 MGD) would receive secondary treatment. The remaining 90,840 $m^3$/d (24 MGD) of primary clarifier effluent will receive high-rate disinfection and dechlorination. The basis of design of these facilities is based upon SWMM results using a one-year, two-hour storm event.

Figure 5.1.a-3 shows a site layout following completion of the CSO Abatement Program. Figure 5.1.a-4 shows a schematic of the ASD WWTF following completion of the CSO Abatement Program.

It should be noted that the 1993 CSO Facilities Plan is currently being updated. The updated plan will re-assess the earlier recommendations for the later phases of the program.

Stormwater Management Model Results

The EPA SWMM Version 4 was used to assess the net benefit of the retrofit by comparing annual WWF volume treated at the WWTF under pre- and post-retrofit conditions. A simplified model was developed to simulate hourly rainfall, based upon the previously calibrated model developed for individual rainfall events.

The data used for these projections consists of 21 years of hourly rainfall data from the National Weather Service (NWS) station in Portland, Maine for the years from 1971—1991. These data were used as input to the simplified RUNOFF model that projected flows that were used to simulate long-term overflow using the TRANSPORT block of SWMM. The TRANSPORT model was constructed as a simplified network consisting of overflows and regulator pipes. The simplified model was used to project the annual volume of treated WWF at the WWTF.

Based on this analysis, the average annual WWF conveyed to the treatment plant was found to be 582,890 $m^3$ (154 MG). The layout of the pre-retrofit headworks allowed 484,480 $m^3$ (128 MG) to receive treatment consisting of primary and secondary treatment followed by disinfection while 98,410 $m^3$ (26 MG) of WWF annually bypassed the treatment plant and only received disinfection prior to discharge. The retrofitted headworks allowed the entire 582,890 $m^3$ (154 MG) to receive primary treatment and 484,480 $m^3$ (128 MG) received secondary treatment and disinfection. The remaining 98,410 $m^3$ (26 MG) that did not receive secondary treatment was bypassed to the high-rate disinfection system prior to discharge to the Kennebec River.

Figure 5.1.a-3. Site Layout Following Retrofit

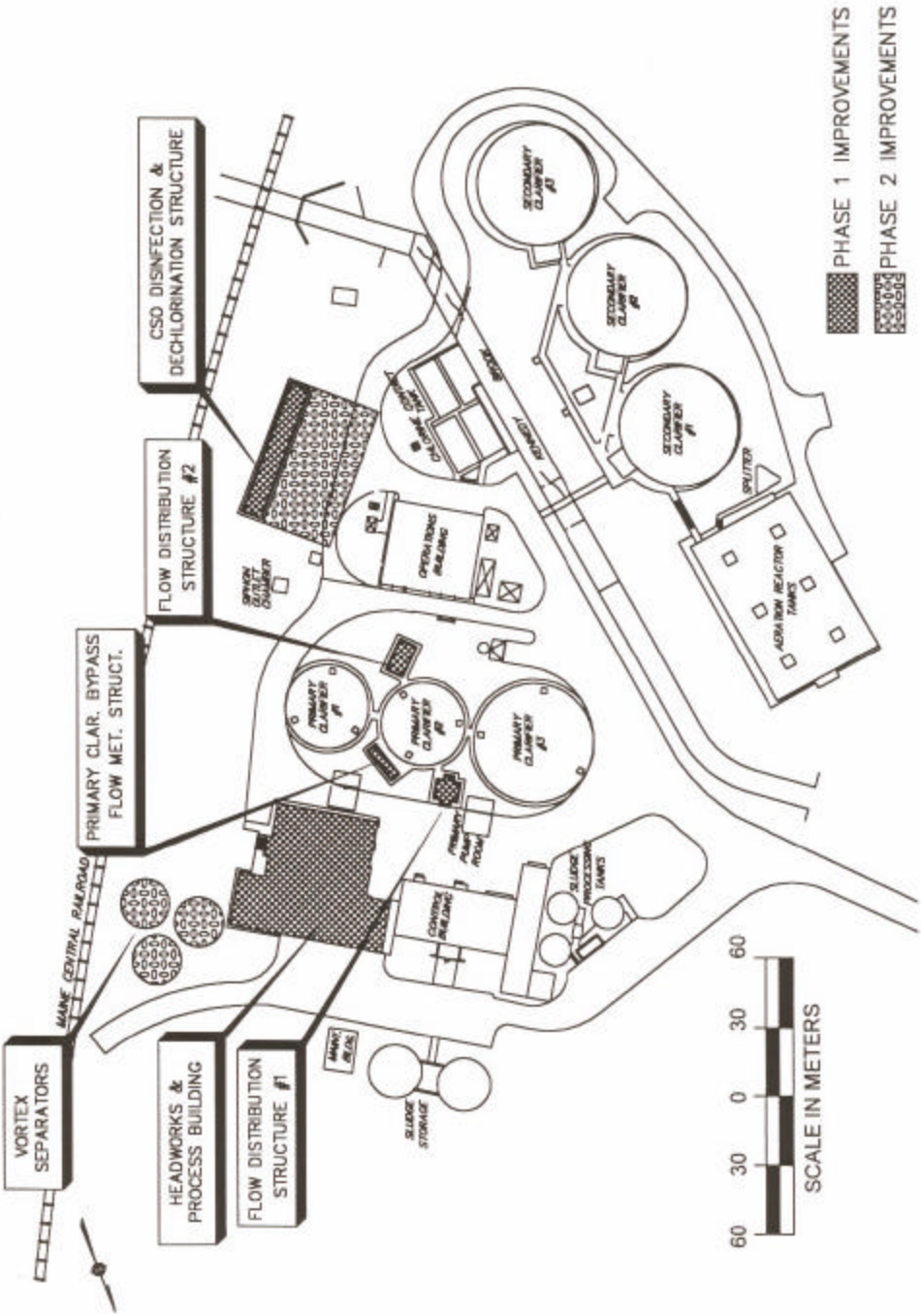
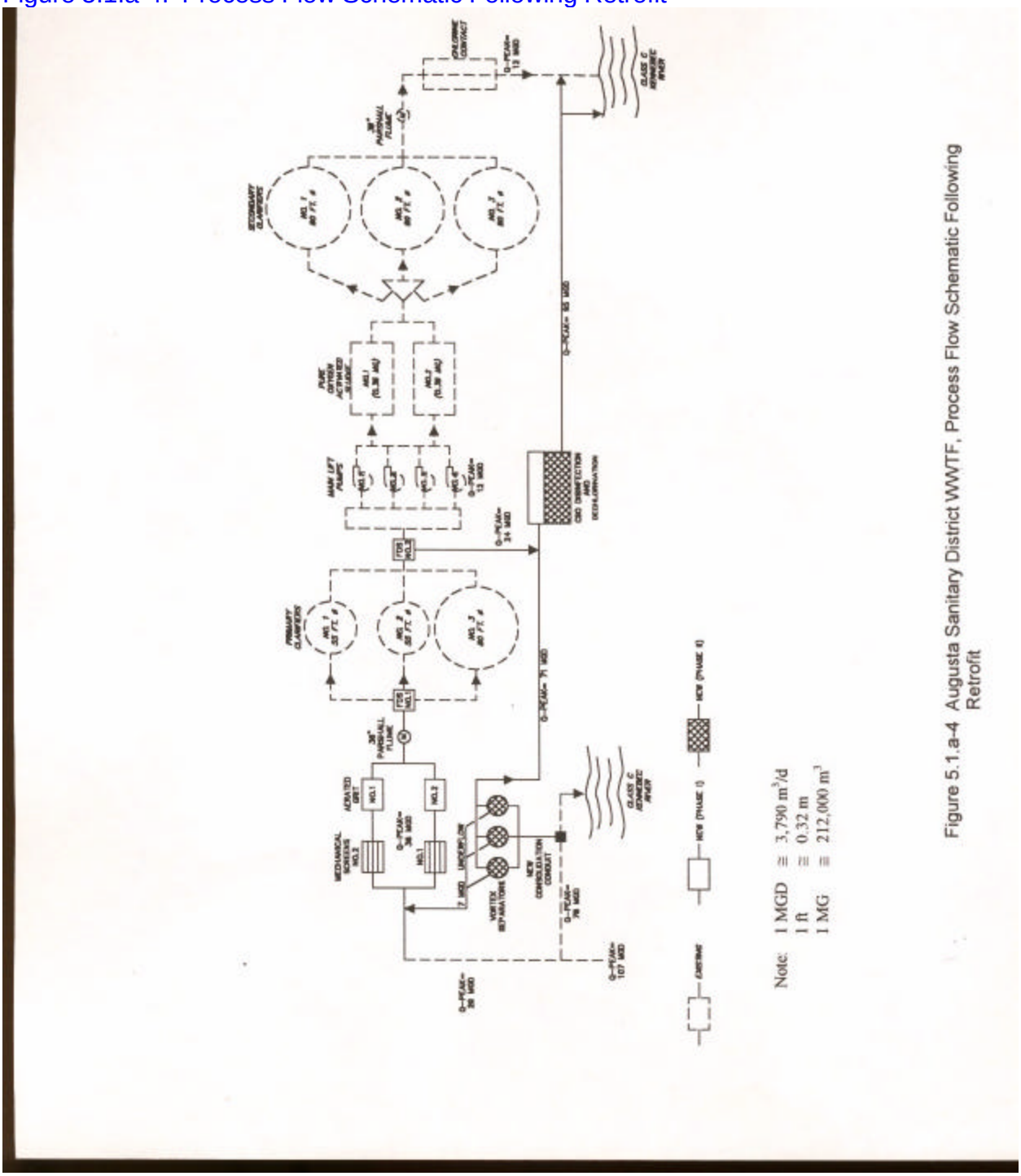



Figure 5.1.a-3 Augusta Sanitary District WWTF, Site Layout Following Retrofit

Figure 5.1.a-4. Process Flow Schematic Following Retrofit



Design Considerations

The retrofitting of the ASD WWTF to handle increased WWF required consideration of several factors:

- Pollutant and Hydraulic Loadings
- Treatment Objectives
- Operational Requirements
- Chemical Storage and Feed Requirements
- Sludge Handling Requirements
- Operations and Maintenance Requirements
- Construction Sequencing and Site Constraints

Each of these considerations is described in this section.

*Pollutant And Hydraulic Loadings* - Pollutant and hydraulic loadings were developed for the wet-weather treatment units at the ASD WWTF during the Facilities Planning stage. These values were refined and updated as the project progressed through the preliminary engineering and detailed design stages. The primary objective of the wet-weather treatment units is to remove floatable and settleable solids to promote effective disinfection.

Table 5.1.a-1 shows the design criteria for the wet-weather treatment processes at the ASD WWTF. The primary focus of the Phase I improvements are to remove hydraulic restrictions at the WWTF preventing flows of 109,800 $m^3$/d (29 MGD) from reaching the existing primary clarifiers. As noted previously, the completion of Phases II, III and IV will provide equivalent primary treatment and disinfection for additional peak WWF of approximately 295,200 $m^3$/d (78 MGD).

Table 5.1.a-1. Phase 1 CSO Abatement Program Wet-Weather Treatment Design Criteria

| **Description and Criteria** | **Phase I** | | **Phases II, III & IV** | |
|---|---|---|---|---|
| Vortex Treatment | | | | |
| Number of Units (each) | 0 | | 3 | |
| Design Flow | -- | | 0.30 $m^3$/d | 78.0 MGD |
| Diameter Each | -- | | 10.67m | 35.0 ft |
| Loading Rate | -- | | 1098 $m^3$/d /$m^2$ | 27,000 gpd/$ft^2$ |
| Overflow to High-Rate Disinfection | -- | | 268,735 $m^3$/d | 71.0 MGD |
| Underflow to Headworks | -- | | 26,495 $m^3$/d | 7.0 MGD |
| Mechanical Screening | | | | |
| No. of Bar Screens (ea.) | 2 | | 2 | |
| Type | Reciprocating Rake | | Reciprocating Rake | |
| Bar Size | 0.95x5.1cm | 3/8x2 in | 0.95x5.1 cm | 3/8x2 in |

| | | | | |
|---|---|---|---|---|
| Bar Spacing | 2.54 cm | 1 in | 2.54 cm | 1 in |

Table 5.1.a-1. Continued

| **Description and Criteria** | **Phase I** | | **Phases II, III & IV** | |
|---|---|---|---|---|
| Peak Influent Flow | 109,765 $m^3$/d | 29.0 MGD | 109,765 $m^3$/d | 29.0 MGD |
| Vortex Overflow | 0.0 | | 26,495 $m^3$/d | 7.0 MGD |
| Total Peak Flow | 109,765 $m^3$/d | 29.0 MGD | 136,260 $m^3$/d | 36.0 MGD |
| Approach Velocity Half Clogged | 0.76 m/s | 2.5 fps | 0.84 m/s | 2.75 fps |
| Screen Velocity Half Clogged | 2.07 m/s | 6.8 fps | 2.29 m/s | 7.5 fps |
| Differential Head | 27.43 cm | 0.9 ft | 33.22 cm | 1.09 ft |
| Estimated Wet Screening Quantity | $3.7x10^{-5}$ $m^3/m^3$ | (5 $ft^3$ /MG) | $3.7x10^{-5}$ $m^3/m^3$ | (5 $ft^3$ /MG) |
| Average De-watered Screenings Volume | 0.28 $m^3$/d | 0.37 cyd/d | 0.57 $m^3$/d | 0.74 cyd/d |
| Screenings Container Size | .57 $m^3$/d | 0.75 cyd | 0.57 $m^3$/d | 0.75 cyd |
| Screening Storage Time (days) | 2 | | 1 | |
| <u>Grit Removal</u> | | | | |
| No. of Tanks | 2 | | 2 | |
| Type | Aerated | | Aerated | |
| Average Flow | 15,140 $m^3$/d | 4.0 MGD | 30,280 $m^3$/d | 8.0 MGD |
| Peak Flow | 109,765 $m^3$/d | 29.0 MGD | 136,260 $m^3$/d | 36.0 MGD |
| Volume Each Tank | 167 $m^3$ | 44,000 gal | 167 $m^3$ | 44,000 gal |
| Detention Time - Avg. Flow (min) | 31.7 | | 16.8 | |
| Detention Time - Peak Flow (min) | 4.4 | | 3.5 | |
| Air Supply | 0.27 to 0.72 $m^3$/min/m | 3 to 8 cfm/ft | 0.27 to 0.72 $m^3$/min/m | 3 to 8 cfm/ft |
| No. of Air Blowers | 2+1 standby | | 2+1 standby | |
| Air Flow Each | 2.4 to 6.3 $m^3$/min | 84 to 224 cfm | 2.4 to 6.3 $m^3$/min | 84 to 224 cfm |
| Pressure | 3867 kg/ $m^2$ | 5.5 psig | 3867 kg/ $m^2$ | 5.5 psig |
| Blower Motor Hp | 5/10, two speed | | 5/10, two speed | |
| <u>Grit De-watering & Storage</u> | | | | |
| Type | Cyclone | | Cyclone | |
| No. of Classifiers | 2 | | 2 | |
| Flow | 1 $m^3$/min | 260 gpm | 1 $m^3$/min | 260 gpm |
| No. of Grit Pumps | 2+1 standby | | 2+1 standby | |
| Pump Head | 8.5 m | 28 ft | 8.5 m | 28 ft |
| Pump Speed (rpm) | 900 | | 900 | |
| Pump power | 7.5 kW (10 HP) | | 7.5 kW (10 HP) | |
| Grit Washer Diameter | 30.5 cm | 12 in | 30.5 cm | 12 in |
| Grit Washer Speed (rpm) | 18 | | 18 | |
| Estimated Grit Quantity | 45 $cm^3/m^3$ | 6.0 $ft^3$/MG | 45 $cm^3/m^3$ | 6.0 $ft^3$/MG |
| Grit Container Size | 4.6 $m^3$ | 6 cyd | 4.6 $m^3$ | 6 cyd |
| Grit Storage Time (days) | 7 | | 3 | |
| Description and Criteria | Phase I | | Phases II, III & IV | |
| <u>Primary Clarifiers</u> | | | | |
| Average Flow | 15,140 $m^3$/d | 4.0 MGD | 30,280 $m^3$/d | 8.0 MGD |

| | | | | |
|---|---|---|---|---|
| Peak Flow | 109,765 $m^3$/d | 29.0 MGD | 136,260 $m^3$/d | 36.0 MGD |
| BOD Loading [1] | 3220 kg/d | 7,100 lbs/d | 7258 kg/d | 16,000 lbs/d |
| SS Loading [1] | 5443 kg/d | 12,000 lbs/d | 7348 kg/d | 16,200 lbs/d |
| No. Of Tanks (ea.) | 3 | | 3 | |

Table 5.1.a-1 Continued

| **Description and Criteria** | **Phase I** | | **Phases II, III & IV** | |
|---|---|---|---|---|
| Diameter Each (ft) | 2 @ 16.7 m | 2 @ 55 ft | 2 @ 16.7 m | 2 @ 55 ft |
| | 1 @ 24.4 m | 1 @ 80 ft | 1 @ 24.4 m | 1 @ 80 ft |
| Sidewater Depth | 2.1 m | 7.0 ft | 2.1 m | 7.0 ft |
| Total Volume (MG) | 1938 $m^3$ | 0.512 | 1938 $m^3$ | 0.512 |
| Average Hydraulic Loading | 16.7 $m^3$/d/ $m^2$ | 409 gpd/$ft^2$ | 33.3 $m^3$/d/ $m^2$ | 818 gpd/$ft^2$ |
| Peak Hydraulic Loading | 121 $m^3$/d/$m^2$ | 2,966 gpd/$ft^2$ | 150 $m^3$/d/ $m^2$ | 3,682 gpd/$ft^2$ |
| Average Weir Loading | 83.2 $m^3$/d/m | 6,701 gpd/ft | 166.5 $m^3$/d/ m | 13,403 gpd/ft |
| Peak Weir Loading | 603.4 $m^3$/d/m | 48,584 gpd/ft | 749.0 $m^3$/d/ m | 60,311 gpd/ft |
| Detention Time – Average Flow (h) | 3.1 | | 1.5 | |
| Detention Time – Peak Flow (h) | 0.4 | | 0.3 | |
| Estimated $BOD_5$ Removal[1] | 1352 kg/d | 2,980 lbs/d | 2205 kg/d | 4,860 lbs/d |
| Estimated SS Removal[1] | 2994 kg/d | 6,600 lbs/d | 3674 kg/d | 8,100 lbs/d |
| Estimated Sludge Concentration (mg/L) | 40,000 | | 40,000 | |
| Estimated Sludge Quantity @4.0%[1] | 75 $m^3$/d | 0.0198 MGD | 92 $m^3$/d | 0.0243 MGD |
| Primary Sludge/Scum Pumps | 5 | | 5 | |
| Pump Type | Disc | | Disc | |
| Pump Flow Each | 0.3 $m^3$/min | 85 gpm | 0.3 $m^3$/min | 85 gpm |
| Total Head | 4.6 m | 15 ft | 4.6 m | 15 ft |
| Pump Speed (RPM) | 300 | | 300 | |
| Pump power | 3.7 kW (5 HP) | | 3.7 kW (5 HP) | |
| No. of Grinders | 5 | | 5 | |
| <u>CSO Disinfection</u> | | | | |
| Chemical Type | Sodium Hypochlorite | | Sodium Hypochlorite | |
| Peak Flow | 64,345 $m^3$/d | 17.0 MGD | 359,575 $m^3$/d | 95.0 MGD |
| Dose Rate at Peak Flow (mg/L) | 15.0 | | 15.0 | |
| Chemical Concentration (%) | 15.0 | | 15.0 | |
| $Cl_2$ Feed Rate | 149.8 kg/$m^3$ | 1.25 lbs/gal | 149.8 kg/$m^3$ | 1.25 lbs/gal |
| No. of Pumps | 1 + 1 standby | | 2 + 1 standby | |
| Pump Capacity each | 0.27 $m^3$/h | 70.0 gph | 0.76 $m^3$/h | 200.0 gph |
| Contact Time (min) | 5.5 | | 5.0 | |
| Contact Chamber Volume | 250 $m^3$ | 66,000 gal | 1250 $m^3$ | 330,000 gal |
| Length to Width Ratio | 7:1 | | 40:1 | |
| Description and Criteria | Phase I | | Phases II, III & IV | |
| <u>CSO Dechlorination</u> | | | | |
| Chemical Type | Sodium Bisulfite | | Sodium Bisulfite | |
| Peak Flow | 64,345 $m^3$/d | 17.0 MGD | 359,575 $m^3$/d | 95.0 MGD |
| Chlorine Residual (mg/L) | 5.0 | | 5.0 | |
| $NaHSO_3$ Dose (mg/L) | 8.06 | | 8.0 | |

| | | | | |
|---|---|---|---|---|
| $NaHSO_3$, Concentration (%) | 38.0 | | 38.0 | |
| Feed Rate $NaHSO_3$ | 504.4 kg/m$^3$ | 4.21 lbs/gal | 504.4 kg/m$^3$ | 4.21 lbs/gal |
| No. of Pumps | 1 + 1 standby | | 2+1 standby | |
| Pump Capacity each | 0.05 m$^3$/h | 12.0 gph | 0.12 m$^3$/h | 32.0 gph |
| Contact Time (s.) | 15.0 | | 15.0 | |
| Contact Chamber Volume | 11.4 m$^3$ | 3,000 gal | 62.5 m$^3$ | 16,500 gal |

*Treatment Objectives.* The state of Maine requires an Escherichia coliform (E-coli) concentration of 949 colonies per 100 ml to comply with the maximum daily E-coli limit in a Class C river. Sampling and monitoring performed during the development of the CSO Abatement Program indicated that the Kennebec River does not currently meet water quality criteria for bacteria during wet weather. The CSO Abatement Program recommended a plan to treat CSO discharges to meet the current maximum-daily-water-quality limit. Primary treatment and high-rate disinfection were selected as the most economical methods to meet the treatment objectives for this project.

*Operational Requirements.* The Phase I Improvements to the Augusta WWTF designed to operate on a continuous basis include:

- Mechanical Screening
- Grit Removal
- Primary Treatment

The high-rate disinfection and dechlorination systems are intended to operate in an event-based mode during periods of excessive WWF. Under current DWF conditions, the WWTF operates with only one primary clarifier in service. The other two primary clarifiers can be kept out of service and used as off-line storage during wet-weather events. If one 24.4 m diameter (80 ft) primary clarifier is in service, the remaining two 16.8 m (55 ft) diameter primary clarifiers provide approximately 943 m$^3$ (249,000 gal) of off-line storage. The WWTF staff can activate the off-line tanks using the new Supervisory Control and Data Acquisition (SCADA) System Controls. Motor-operated weir gates regulate the flow to the primary clarifiers.

After the WWF subsides the primary clarifiers can be drained using a new pump with variable speed controls. The primary clarifier tank drain pump discharge is returned to the new Primary Clarifier Flow Distribution Structure (FDS No. 1).

The future improvements of Phases II, III and IV will use an event-based mode of operation during wet weather periods. The existing overflow pipes located within the CSS will be intercepted by new consolidation conduits that will convey the combined sewerage to the WWTF. A series of vortex separators will be constructed to provide equivalent primary treatment (removing floatable and settleable solids) to facilitate effective disinfection. The high-rate disinfection and dechlorination tank will be expanded with a new parallel outfall pipe provided to the Kennebec River from the WWTF. At the end of the wet-weather event the vortex separators and high-rate disinfection tank will be drained using a pump with variable speed control to the WWTF headworks.

*Chemical Storage and Feed Requirements.* The CSO Abatement facilities will utilize two chemicals:

- Liquid Sodium Hypochlorite for disinfection
- Liquid Sodium Bisulfite for dechlorination

The liquid sodium hypochlorite system design will use on commercial-grade liquid sodium hypochlorite (15% solution) which contains the equivalent of 150 kg of chlorine/$m^3$ (1.25 lbs of chlorine/gal of solution). The metering pumps are designed to deliver a dosage of 15 milligrams per liter (mg/L) at the peak design flow with one unit out of service. A bulk storage tank with an effective working volume of 240.2 $m^3$ (6,400 gal) is provided. The tank is used for both dry-weather, secondary effluent disinfection, and high-rate wet-weather disinfection to minimize problems associated with product shelf life. The available chlorine in sodium hypochlorite solution declines with age. The anticipated sodium hypochlorite storage time will be 51 days initially, decreasing to 25 days at the future design flow.

The storage system design is based on commercial grade liquid sodium bisulfite (38% solution) which contains approximately 500 kg of sodium bisulfite per cubic meter of solution (4.2 lbs of sodium bisulfite/gal of solution). The metering pumps are designed to deliver sodium bisulfite dosages necessary to dechlorinate effluent with a chlorine residual of 5.0 mg/L. Approximately 1.6 kg of sodium bisulfite will be used per kg of chlorine residual. A bulk storage tank with an effective working volume of 15.7 $m^3$ (4,150 gal) is provided.

The sodium hypochlorite and sodium bisulfite metering pumps are flow paced. An ultrasonic level sensor is used for effluent flow measurement. A broad-crested rectangular weir 3.4 m (11 ft) in length is the primary measuring device.

*Sludge Handling Requirements.* Sludge handling requirements will increase during periods of WWF. The most pronounced increase is expected to occur during the initial period of WWF (first flush). The configuration of FDS No. 1 will capture a significant portion of the first flush solids in the off-line primary clarifiers.

A solids balance was performed to estimate the quantity of sludge produced during a storm with a one-year recurrence interval. The daily sludge quantities increase as follows:

- Current Estimated Sludge Quantity (kg/d) 3,290 (7,250 lbs/d)
- One-Year Storm Estimated Sludge Quantity (kg/d) 7,550 (16,650 lbs/d)

The sludge quantity is estimated to increase 230% during a one-year storm event. However, over the course of a week, the overall increase can be assimilated if sludge pumping and processing schedules are adjusted. The following is a summary of weekly sludge quantities during a period with a one-year storm:

- Initial Estimated Sludge Quantity (kg/wk) 23,010 (50,730 lbs/wk)

- One-Year Storm Projected Sludge Quantity (kg/wk) 27,270 (60,140 lbs/wk)

The estimated sludge quantity would only increase by 19% during a week with a one-year storm event.

*Operation and Maintenance Requirements.* The implementation of the CSO Abatement Program will result in increased O&M costs. The following list summarizes the areas of expected O&M cost increases at the WWTF:

- Labor to perform sampling, laboratory analyses, reporting, wet-weather process adjustments, and clean-up following an event.
- Power for in-plant pumping, sludge processing, and chemical feed systems.
- Chemicals for disinfection, dechlorination, and polymer for additional sludge quantities.
- Sludge disposal tipping and transport fees.

Table 5.1.a-2 summarizes the expected increase in annual O&M costs at the WWTF resulting from the initial Phase I and future Phases II, III and IV of the CSO Abatement Program.

Table 5.1.a-2. Estimated O&M Costs, CSO Abatement Program - All Phases

| Description | Estimated Annual Cost |
|---|---|
| Sampling | $800 |
| Laboratory Analyses | $3,200 |
| Report Preparation | $1,500 |
| Process Control Adjustments | $1,200 |
| Clean-Up of Wet-Weather Treatment Structures | $6,000 |
| Power | $1,000 |
| Sodium Hypochlorite | $4,100 |
| Sodium Bisulfite | $2,000 |
| Polymer for Sludge Dewatering | $1,000 |
| Sludge Disposal Transport and Tipping Fees | $11,500 |
| Total Estimated Cost | $32,300 |

*Construction Sequencing and Site Constraints.* The existing ASD WWTF site is crowded, with little room for expansion. Therefore, it was necessary to construct some of the CSO Abatement Facilities in locations previously used for other purposes. The following is a listing of special sequencing and site considerations related to the CSO Abatement Program at the WWTF:

- Portions of the existing screen structure and detritor grit chamber were demolished to facilitate the construction of the new headworks.
- A temporary junction structure and CSO bypass was needed.

- The existing primary clarifiers were taken out of service in sequence, to facilitate new interconnecting piping and mechanical refurbishment.
- The existing comminutor structure required temporary piping connections to keep it in operation during the new headworks construction.
- After the new headworks are built and placed into service, the existing comminutor structure will require demolition and removal to facilitate installation of new underground piping.

A completion time of 540 calendar days was specified for the Phase I CSO Abatement Program.

Project Costs

The bids for the Phase I CSO Abatement Program were opened on January 23, 1997. Costs for the CSO Abatement components are listed in Table 5.1.a-3. Costs included construction requirements and project services. Project services included design engineering, construction engineering, legal, and fiscal expenses but did not include contingencies, right-of-way expenses, or other miscellaneous items.

Table 5.1.a-3. Phase 1 CSO Abatement Program Project Costs

| Description | Cost[1] |
|---|---|
| General Conditions | $200,000 |
| Sitework and Yard Piping | $344,000 |
| Headworks Building and Chemical Feed Systems | $1,688,000 |
| CSO Disinfection and Dechlorination Structure | $536,000 |
| Primary Pump Room Renovations | $80,000 |
| Primary Clarifier Renovations | $40,000 |
| Flow Distribution Structure No. 1 | $117,000 |
| Flow Distribution Structure No. 2 | $122,000 |
| Operations Building Pump Room Renovations | $60,000 |
| Yard Electrical Ductbanks and Lighting | $88,000 |
| Instrumentation and SCADA System | $167,000 |
| Total Construction Cost | $3,422,000 |
| Project Services[2] | $761,000 |
| Total Project Cost | $4,183,000 |

[1] Based on January 23, 1997 Bid Prices
[2] Includes Design Engineering, Construction Engineering, Legal, and Fiscal Expenses; does not include contingencies and right-of-way costs.

Conclusions

This retrofit will utilize the entire capacity of the existing primary clarifiers to provide primary treatment for the maximum rate of influent flow, thereby increasing the portion of the WWF that will receive primary treatment from 60,560 $m^3$/d (16 MGD) to 109,765 $m^3$/d (29 MGD). This will result in net 20% increase: 484,480—582,890 $m^3$ (128—154 MG) in the annual

WWF that receives primary treatment. Only a few examples of retrofitting WWTF have been published. Often these examples are strongly influenced by site-specific constraints and therefore do not lend themselves to comparison with other retrofit situations. The retrofit solution described here has been selected based on the cost effectiveness. For comparison purposes, the alternative treatment scheme of off-line storage at the WWTF for the 13 MGD of WWF was estimated to be $8,336,000 versus the $4,183,000 for the selected retrofit.

### *5.1.b. Hypothetical Retrofit of Primary Treatment Facilities Using the ACTIFLO System*

## ACTIFLO System

This desktop analysis involves the retrofit of hypothetical primary treatment tanks for enhanced treatment of WWF in excess of the 908,400 $m^3$/d (240 MGD) peak wet-weather capacity of the existing WWTF. The treatment method selected for this retrofit scenario is the ACTIFLO process that combines microsand-enhanced flocculation with lamellar plate settling. The process has been shown to be effective in treating CSO/SSO and primary wastewater flows in installations in Paris, France (Le Poder and Binot 1994). Typical performance results are 85-95% reduction in SS, 50—80% reduction in Total BOD, 85—95% reduction of total phosphorus and 10—20% reduction in total kjeldahl nitrogen (TKN).

The hypothetical WWTF has eight primary clarifiers that are 41.2 m (135 ft) in diameter with a 3 m (10 ft) side water depth. The surface-settling rate of the existing units at the design peak flow of 908,400 $m^3$/d (240 MGD) is 85.6 $m^3/d/m^2$ (2,100 gpd/$ft^2$) with a hydraulic detention time of 48 min. Figure 5.1.b-1 shows the existing layout of the hypothetical WWTF prior to construction of the Phase I.

Due to the limited side water depth, retrofit of the existing primary clarifiers is not practical. A more economical approach is the demolition of one of the existing clarifiers and replacing it with a 908,400 $m^3$/d (240 MGD) ACTIFLO system which has a depth of 6.7 m (22 ft). The seven remaining clarifiers would be retained for storage of WWF during storm events or for future plant modifications.

The proposed 908,400 $m^3$/d (240 MGD) ACTIFLO system design consists of six 151,400 $m^3$/d (40 MGD) process trains. Each of the process trains is designed to operate at a nominal overflow rate of 1.2 $m^3/min/m^2$ (30 gpm/$ft^2$) with a total hydraulic retention time of 5.6 min.

Table 5.1.b-1. ACTIFLO Design Summary

| Design Parameter | Quantity | |
|---|---|---|
| Number of Process Trains | 6 | |
| Nominal Train Capacity | 151,400 $m^3$/d | 40 MGD |
| Total System Capacity | 908,400 $m^3$/d | 240 MGD |

Hydraulic Retention Time 5.6 min

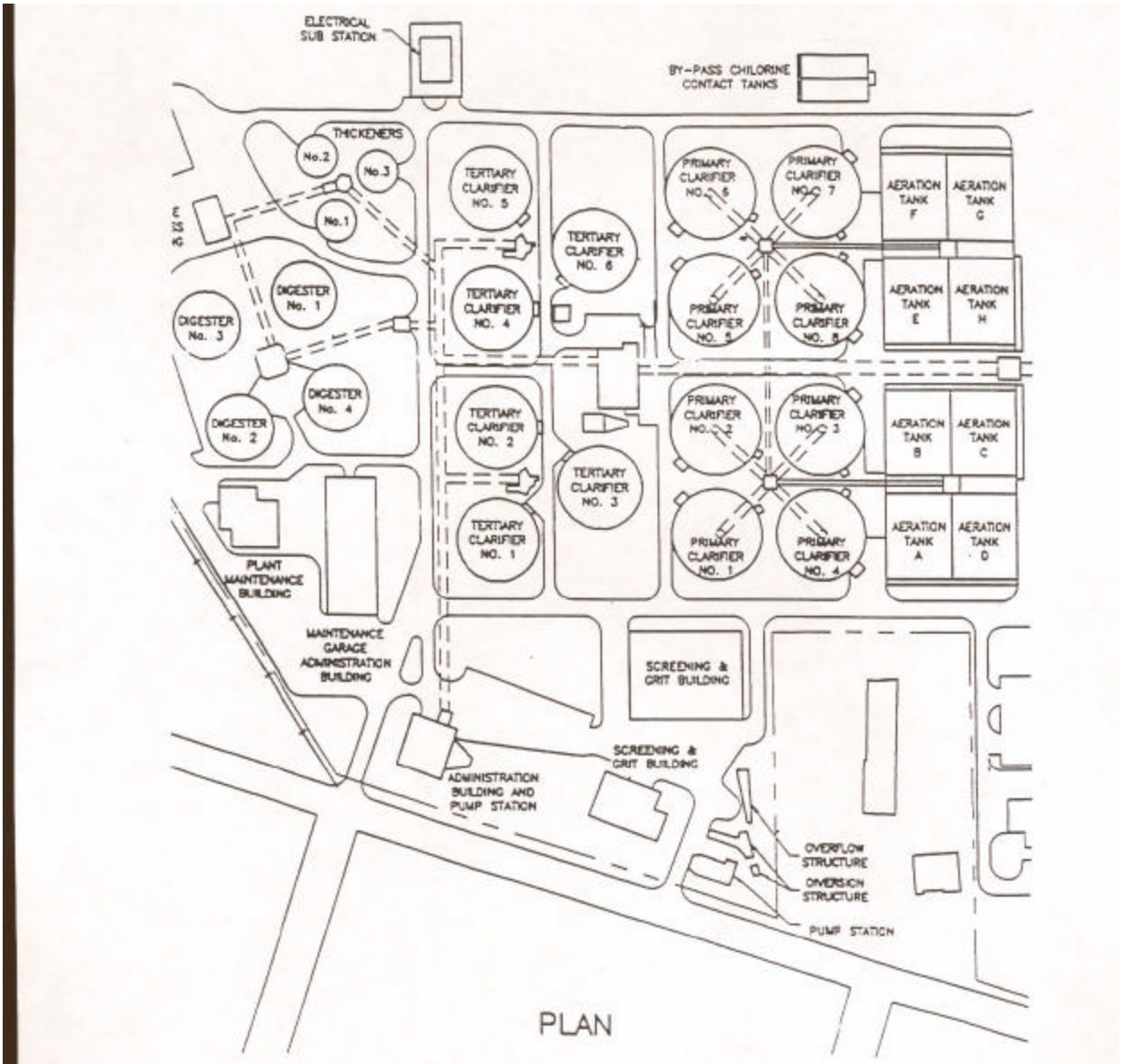



Figure 5.1.b-1. Layout of Hypothetical WWTF Prior to Construction of ACTIFLO System

Figure 5.1.b-1. Layout of Hypothetical WWTF Prior to Construction of ACTIFLO System

The high overflow rate and short retention time offered by the ACTIFLO process result in an extremely compact system. Overall, the proposed ACTILFO design will meet the treatment objectives of the project in approximately 5% of the space currently occupied by the existing primary clarifiers. A brief comparison of the existing facilities and the ACTIFLO design are shown in Table 5.1.b-2.

Table 5.1.b-2. Comparison of ACTIFLO and Existing Primary Treatment Systems

| Parameter | Existing System | ACTIFLO | Net Savings with ACTIFLO |
|---|---|---|---|
| Total Capacity | 908,400 $m^3$/d | 908,400 $m^3$/d | - |
| Number of Units | 8 | 6 | - |
| Unit Capacity | 113,550 $m^3$/d | 151,400 $m^3$/d | - |
| Area Required | 1,329.1 $m^2$ (per unit)<br>10,632.7 $m^2$ (total) | 87.8 $m^2$ (per train) 526.7 $m^2$ (total) | 94% per unit<br>95% total |
| Process Volume | 3,987 $m^3$ (per unit)<br>31,898 $m^3$ (total) | 3,533 $m^3$ (per unit)<br>3,588.3 $m^3$ (total) | 85% per unit<br>89% total |
| Hydraulic Retention Time | 51 min | 5.6 min | 89% |

A layout drawing of the hypothetical WWTF showing the location of the proposed ACTIFLO system is provided in Figure 5.1.b-2.

ACTIFLO Process Theory

Fundamentally, the ACTIFLO process for wastewater treatment is very similar to conventional water treatment technology of coagulation, flocculation and sedimentation. Although influent characteristics vary for different wastewater applications and influent streams, the removal of suspended materials is accomplished by the same mechanisms in each application. Therefore, for the purposes of this discussion, the term wastewater (WW) collectively refers to any domestic wastewater stream including primary sewage, CSO, SSO and/or inflow and infiltration (I/I).

As in conventional water treatment technology, a coagulant is used for the destabilization of suspended materials entering the process and a flocculent aid polymer is added to aggregate the destabilized solids into larger masses. The resulting flocs are then subsequently removed by settling for disposal. The primary advance made in the ACTIFLO process is the addition of very fine sand as a “seed” for the development of high-density floc. The resulting flocs, ballasted by the microsand, are more easily removed by settling. It is in this step that ACTIFLO differs significantly from conventional treatment processes. A brief overview of the physiochemical principals involved in conventional coagulation and flocculation is presented in "Section 5.2.b. Chemical Addition to AWPCP" to help understand the advantages of ACTIFLO over chemical coagulation alone.

Figure 5.1.b-2. Layout of Proposed ACTIFLO System in Hypothetical Setting

Figure 5.1.b-2. Layout of Proposed ACTIFLO System in Hypothetical Setting

The primary advance made in the ACTIFLO process is the addition of a very fine (60—140 $\mu$m mean diameter) clean silica sand or microsand in the flocculation process step. With its addition, the microsand serves several key roles in the ACTIFLO process:

- The high specific surface area to volume ratio of the microsand particles serves as a seed for the development of floc.

- Together, the microsand and polymer seed promote inter-particle contact and enmeshment of suspended materials resulting in development of large stable floc.

- The high microsand concentration within the ACTIFLO process effectively dampens the effects of fluctuating influent quality and provides for stable operation and treatment throughout changes in the raw water quality.

- The relatively high specific gravity of the microsand (~2.65) serves as a ballasting agent creating heavier floc that settle very quickly.

- The chemically inert microsand does not react with the process chemistry and allows the microsand to be efficiently separated from the sludge via a hydrocyclone and reused in the process.

The advantages of microsand ballasted flocculation allow ACTIFLO to offer surface loading or overflow rates many times higher than competing processes. Overflow rates as high as 3.3 $m^3/min/m^2$ (80 $gpm/ft^2$) are claimed in typical wastewater applications of ACTIFLO. The high loading rates result in a process that is extremely compact compared to conventional or competing processes of similar capacity. The high overflow rates and size savings provided by ACTIFLO translate directly into significant savings in project cost.

The use of microsand for ballasted flocculation provides ACTIFLO with several operational advantages in wastewater applications. The high solids content within the ACTIFLO process provides for effective treatment of influent with varying SS concentrations. In such applications, the high microsand concentration within the process allows for SS removal by particle enmeshment in the microsand-polymer complex. This performance is also extremely beneficial in chemical precipitation reactions such as tertiary phosphorus removal/effluent polishing or the treatment of dilute waste streams.

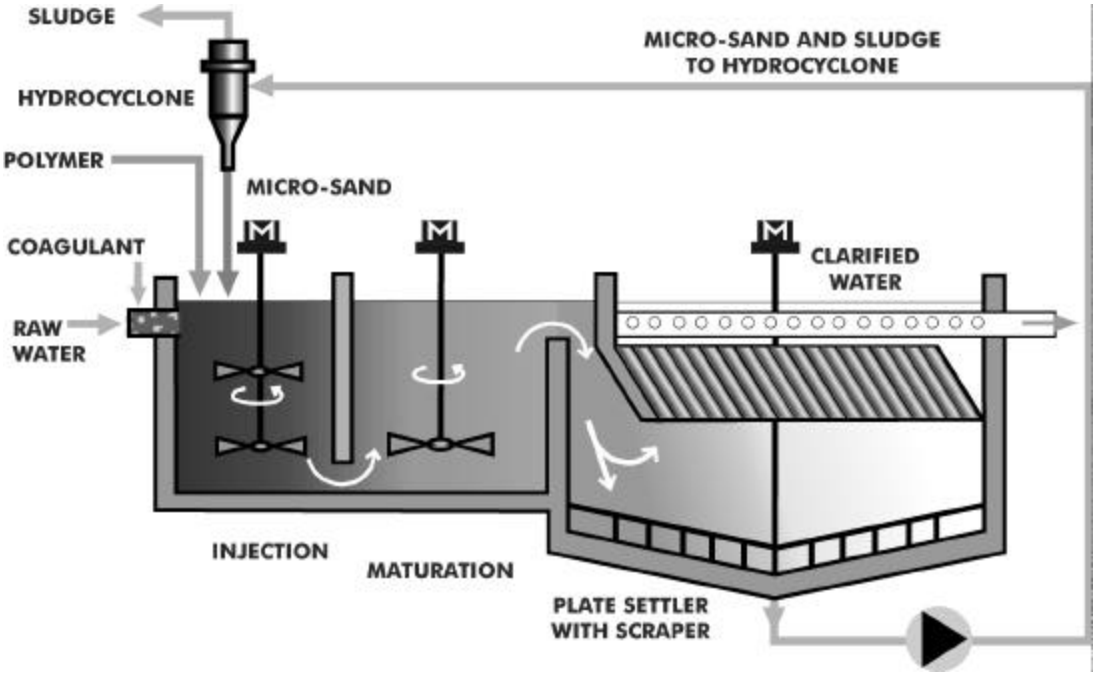
Microsand ballasted flocculation is also effective in the treatment of extremely dirty wastewater (high turbidity and SS). Here, the high microsand concentration within the process allows it to provide efficient removal of high-suspended solids concentrations without difficulty. For similar reasons, the process is virtually unaffected by sudden fluctuations in influent quality or volume. Minor adjustments to chemical dosage are all that are needed to provide effective treatment under such conditions.

Overall, the use of microsand results in the development of chemical floc that is significantly denser and more durable than floc from conventional clarification processes. The

ACTIFLO flocs have considerably higher settling velocities than conventional floc allowing much higher clarifier loading rates. Ballasted flocculation also provides for a process that is rapidly started-up and optimized, as well as stable in treating with variations in raw water quality. These characteristics make the ACITLFO process ideal for treating the difficult conditions posed by wastewater/CSO treatment.

ACTIFLO Process Description

The ACTIFLO® process is a compact high performance water clarification system that combines the advantages of microsand enhanced flocculation with lamellar settling. Wastewater treatment is accomplished through a series of consecutive process steps that consist of influent screening, coagulant addition, microsand and polymer injection (injection), floc maturation (maturation), settling and sand recirculation. Each step in the process is discussed in greater detail in the paragraphs that follow. A flow diagram/process schematic of the ACTIFLO system is provided in Figure 5.1.b-3.




Prior to entering the ACTIFLO process, raw wastewater is directed through fine screens to remove large debris and prevent equipment fouling in the process. Debris removed by the fine screening process is typically sent directly to a sanitary landfill for disposal. Chemical coagulant is then added to the screened raw water in the influent line via an in-line mechanical or jet mixing system to provide for thorough and instantaneous dispersion of the coagulant into the influent stream. The coagulant, typically ferric

chloride ($FeCl_3$) in wastewater applications, serves to destabilize suspended materials as they enter the process.

The coagulated water enters the ACTIFLO system in the injection tank. Here, flocculent aid polymer (polymer) and microsand are added to initiate floc formation. Microsand and polymer are incorporated into the coagulated water via mixing over 1 min of retention time. Together, the microsand and polymer form ballasted pin-floc that is allowed to develop further in the next process step.

Treatment continues as water passes through the underflow passage from the injection tank into the maturation tank. Although chemical floc formation actually begins with the addition of polymer and microsand in the injection tank, the majority of ballasted floc formation occurs during the maturation process step. Gentle mixing and increased hydraulic retention time of approximately 3 min provide ideal conditions for the formation of polymer bridges between the microsand and the destabilized SS. This process is further augmented by the large specific surface area of the microsand that provides enhanced opportunity for polymer bridging and enmeshment of suspended materials.

Fully formed ballasted floc leave the maturation tank and enter the settling tank where they rapidly settle and are removed from the treated water via lamellar settling. Laminar up flow through the lamellar settling zone provides effective removal of smaller floc and suspended materials in the treated water. Clarified water exits the ACTIFLO system by a series of collection troughs for subsequent treatment.

*Sludge Handling.* The ballasted floc sand-sludge mixture is collected at the bottom of the settling tank and withdrawn using a rubber-lined centrifugal slurry pump. The sand-sludge mixture is then pumped to the hydrocyclone for separation. Energy from pumping is effectively converted to centrifugal forces within the body of the hydrocyclone causing chemical sludge to be separated from the higher density microsand. Once separated, the microsand is concentrated and discharged from the bottom of the hydrocyclone and re-injected into the process for re-use. The lighter density sludge is discharged from the top of the hydrocyclone and sent for thickening and/or final disposal.

*Coagulant Process Data.* As previously discussed, chemical coagulant is added to the raw water to destabilize suspended materials as they enter the ACTIFLO system. To provide effective treatment, coagulant should be injected in the influent line upstream of the system. In-line mechanical or jet injection mixing, depending on flow rate should be provided to ensure nearly instantaneous incorporation of the coagulant into the influent stream**.**

Although any number of different coagulants could be used, $FeCl_3$ is typically used in wastewater and CSO treatment applications since it is less expensive, and thus more economical in high capacity treatment applications than other available coagulants.

$FeCl_3$ also offers the advantages of being less sensitive to pH and more aggressive in the removal of organics. The only disadvantage to $FeCl_3$ use is the need for corrosion

resistant coagulant dosing and mixing equipment in the influent line. Once incorporated into the raw water, $FeCl_3$ does not pose a serious corrosion threat to process equipment within the ACTIFLO system.

Stormwater Management Model (SWMM) Results

The EPA SWMM Version 4 was used to assess the benefit of the retrofit by comparing annual WWF volume treated at the WWTF under pre- and post-retrofit conditions for the hypothetical retrofit. A simplified model was developed to simulate hourly rainfall for the hypothetical collection system.

The data used for these projections consists of 30 years of hourly rainfall data for the years from 1962—1991. These data were used as input to the simplified RUNOFF model that projected flows that were used to simulate long-term overflow using the TRANSPORT block of SWMM. The TRANSPORT model was constructed as a simplified network consisting of hypothetical overflows and regulator pipes. The simplified model was used to project the annual volume of treated WWF at the WWTF.

Table 5.1.b-3 demonstrates that the hypothetical ACTIFLO system would provide considerable annual capture of WWF that is presently discharged without treatment. This is due to the use of the remaining seven primary treatment tanks as wet-weather storage devices with the ACTIFLO system. These tanks have a combined storage capacity of 28,388 $m^3$ (7.5 MG).

Table 5.1.b-3. Average Annual Overflow Volume from the Pre- and Post-Retrofit Facilities (1962 – 1991)

| | Average Annual Overflow Volume | |
|---|---|---|
| Pre-Retrofit WWTF Overflow | 439,060 $m^3$/d | 116 MG |
| Post-Retrofit | 34,065 $m^3$/d | 9 MG |
| Percent Capture | 92% | |

The proposed ACTIFLO system in combination with the remaining 7 primary clarifiers modified for storage of WWF would provide a system to treat 92% of the WWF that is presently discharged at the WWTF.

Project Costs

*Operational Costs.* The primary costs associated with the operation of the ACTIFLO system are electrical power and process chemicals. The major power components in the system are the mixers, scraper drive and microsand recirculation pumps.

Additionally, a small amount of electrical power is required for the PLC control panels, polymer mix and feed system, and coagulant feed system. Wherever possible, energy efficient equipment is selected for use in the ACTIFLO system to help reduce overall operating costs. In wastewater/CSO treatment applications, the ACTIFLO system is typically operated at less than full capacity, further reducing electrical costs.

Chemical costs are the other major expense associated with operation of the ACTIFLO process. The chemical costs consist of primarily of coagulant and flocculent aid polymer. Depending on the raw water characteristics, pH adjustment chemical costs might also be required in certain applications. As with any water treatment process, chemical dosages are difficult to predict without pilot or full-scale operating data from the site. Furthermore, chemical dosages are subject to vary with changes in the influent conditions. Therefore, a range of typical or anticipated chemical dosages is used to calculate operating costs.

Although chemicals are a major component in the overall operating costs of the ACTIFLO process, the chemicals are used very efficiently due to the mixing provided within the system. Overall, ACTIFLO is typically capable of producing similar or better treatment results with 20—50% reductions in chemical usage as compared to conventional physical-chemical treatment systems of similar capacity. The estimated annual O&M costs are provided in Table 5.1.b-4.

*Capital Costs.* Capital costs estimates for retrofitting the proposed ACTIFLO system into the existing WWTF that were provided by the ACTIFLO manufacturer (Kruger Inc.) are listed in Table 5.1.b-5. The costs include construction requirements and project services including design engineering, construction management, legal and fiscal expenses but do not include contingencies or other miscellaneous items.

The conventional abatement alternative for the hypothetical retrofit would be to construct wet-weather storage facilities adjacent to the existing primary treatment facilities to equalize WWF for ultimate treatment at the WWTF. This would require the construction of a 28,388 $m^3$ (7.5 MG) overflow retention facility (ORF). The estimated cost to construct this facility based upon EPA cost guidelines for CSO control technologies (EPA 1992) is $18.4 million derived from the following equation:

$$\text{Cost} = 3.577\ V^{0.812} \quad (1)$$

where,

Cost = Average construction cost, millions of dollars

V = Storage volume, MG

Table 5. 1. b-4. Estimated O&M Costs

| DESCRIPTION | Estimated Annual Cost |
|---|---|
| **SAND COMSUMPTION** | |
| Est. Sand Loss (g/m $^3$ water produced) | 1 |
| Design Flow Rate (m $^3$/d) | 908,400 (240 MGD) |
| Sand Loss (kg/day) | 793 (1,748 lbs/d) |
| Sand Loss (mt/y) | 263 (290 tons/y) |
| Sand Cost ($/ton) | $88 ($80/ton) |
| **Typical Operation Cost ($/y)** [(1)] | **$9,253** |
| **POLYMER COMSUMPTION** | |
| Est. Polymer Dosage (ppm) | 0.8 |
| Polymer Consumption (kg/d) | 205 (452 lbs/d) |
| Polymer Consumption (mt/y) | 68 (75 tons/y) |
| Polymer Cost ($/mt) | $3,858 ($3,500/ton) |
| **Typical Operation Cost ($/y)** | **$104,694** |
| **COAGULANT COMSUMPTION** | |
| Coagulant Type | Ferric Chloride |
| Est. Coagulant Dosage (ppm) | 65 |
| Coagulant Consumption (kg/d) | 26,008 (57,501 lbs/d) |
| Coagulant Consumption (mt/y) | 8,650 (8,540 tons/y) |
| Coagulant Cost ($/mt) | $220 ($200/ton) |
| **Typical Operation Cost ($/y)** | **$760,978** |
| **ENERGY COMSUMPTION** | |
| Injection Mixer kW per Train | 11 (15 HP) |
| Maturation Mixer kW per Train | 15 (20 HP) |
| Scraper kW per Train | 3.7 (5.0 HP) |
| Sand Pump kW per Train | 37 (50 HP) |
| kW per Train | 67 (90 HP) |
| Total kW | 403 (540 HP) |
| kW/1,000 m$^3$ | 0.44 (2.25 HP/MGD) |
| KW/d | 7,725 (9,688 HP/d) |
| **Typical Operation Cost ($/y)** | **$107,090** |
| **Total Typical Operation Cost ($/y)** | **$982,015** |

(1) Typical annual O&M costs are based on operation at 33% capacity 302,800 m$^3$/d (80 MGD) for 365 days per year with the provision for 75 wet-weather events lasting 12 h each requiring operation at 100% capacity. 75 wet-weather events per year are based upon long-term continuous simulation of 30 years of data through a hypothetical collection system using the TRANSPORT block of SWMM.

Table 5.1.b-5. Estimated ACTIFLO Retrofit Capital Costs

| **Description** | **Estimated Cost[1]** |
|---|---|
| Equipment Cost | $12,000,000 |
| Installation Cost | $960,000 |
| Concrete Cost | $700,000 |
| **Total Project Cost** | **$13,660,000** |

[1] Includes Design Engineering, Construction Engineering, Legal, and Fiscal Expenses based on information provided by ACTIFLO manufacturer, Kruger, Inc. Does not include contingencies and right-of-way costs

In comparison, the capital cost for the ACTIFLO process is less costly than the conventional storage facility, $14.5 million versus $18.4 million; however, the projected O&M cost of the ACTIFLO facility at almost $1 million per year make it a less cost-effective alternative to storage. This is a result of high chemical costs associated the ACTIFLO process.

Typical ACTIFLO Performance in WW/CSO Treatment

*ACTIFLO Pilot Testing Results.* A pilot plant with a nominal capacity of 150 $m^3$/h and a maximum capacity of 200 $m^3$/h was used by Kruger to conduct the pilot testing of the ACTIFLO system. The pilot plant consists of three mixing tanks with a total volume of 12.5 $m^3$ and two parallel hopper settling tanks each having a surface area of 1.2 $m^2$. Each mixing tank is equipped with an axial flow mixer designed to allow for optimal coagulation and flocculation. A rubber lined centrifugal pump extracts the sand/sludge slurry from the bottom of the hopper settling tanks. The major operating parameters of the pilot plant at the nominal flow rate of 150 $m^3$/h are summarized in Table 5.1.b-6.

Table 5.1.b-6. Major-operating parameters of ACTIFLO pilot plant at nominal flow rate of 150 $m^3$/h

| Parameters | Values | Units |
|---|---|---|
| Coagulation Tank Detention Time | 1 | min |
| Injection Tank Detention Time | 1 | min |
| Maturation Tank Detention Time | 1 | min |
| Settler Overflow Rate | 150 | m/h |
| Microsand Recirculation Rate | 8 | % |
| Microsand Concentration | 3—6 | kg/$m^3$ |
| Velocity Gradient (G) in Coagulation Tank | 200—300 | 1/s |
| Velocity Gradient (G) in Injection Tank | 200—300 | 1/s |
| Velocity Gradient (G) in Maturation Tank | 150—200 | 1/s |

Case Studies

*Primary Wastewater Treatment Application - Wastewater Reuse for Agricultural Irrigation in Mexico City, Mexico.* Wastewater reuse for agricultural irrigation must be treated for the removal of Helminth eggs and protozoa cysts, and elimination of pathogenic bacteria and inactivation of enteric viruses. Of great importance is the removal of Helminth eggs. They are extremely persistent, surviving in harsh environmental conditions, and due to their latency period, are transmitted primarily through uncooked food. The Helminth disease transmission has been identified as the top-priority heath problem of the wastewater reuse in developing country. The World Heath Organization (WHO) has recommended that the Helminth egg concentration be under 1 Helminth egg per liter for all kinds of agriculture (WHO, 1989).

After several studies, the Mexican authorities had determined that the best and most economical way to reuse the Mexico City wastewater for agriculture irrigation was to use advanced primary treatment. Due to the existing situation and the extremely large flow of 74.5 $m^3/s$ (1,700 MGD), the Mexican authorities decided to evaluate new treatment technologies and organized pilot studies to see their performances. A six-month pilot test was conducted to investigate the performance of the ACTIFLO process in 1997 (Le Poder 1997).

The results are summarized in Table 5.1.b-7 and in Figure 5.1.b-4 and 5.1.b-5 and show that it was possible to obtain very low SS (25 mg/l in average) and Helminth eggs residual concentration (1 egg per test in average) even at very high overflow (up to 200 m/h) and low chemical dosage (<60 ppm of alum). It confirms that the ACTIFLO process, even without the downstream filtration process, is able to produce an effluent water with less than 1 egg/liter in daily average samples the majority of the time, and less than 5 eggs/liter in all cases, which is the maximum amount permitted for certain kinds of agriculture.

Table 5.1.b-7. Removal efficiency from 20 daily samples in primary treatment in Mexico City, Mexico

| | **SS** | **Helminth Eggs** | **Fecal Coliform** | **COD** | **Total N** | **Total P** | **Sulfurs** |
|---|---|---|---|---|---|---|---|
| | mg/l | Eggs/l | /100 ml | mg/l | mg/l | mg/l | mg/l |
| Influent | 302 | 24.3 | $8x10^8$ | 442 | 18.4 | 10.3 | 7.1 |
| Effluent | 24.7 | 0.99 | $1.4x10^8$ | 172 | 15.5 | 1.9 | 1.3 |
| Removal (%) | 91.7 | 96 | 82.8 | 61.1 | 15.8 | 81.6 | 81.7 |

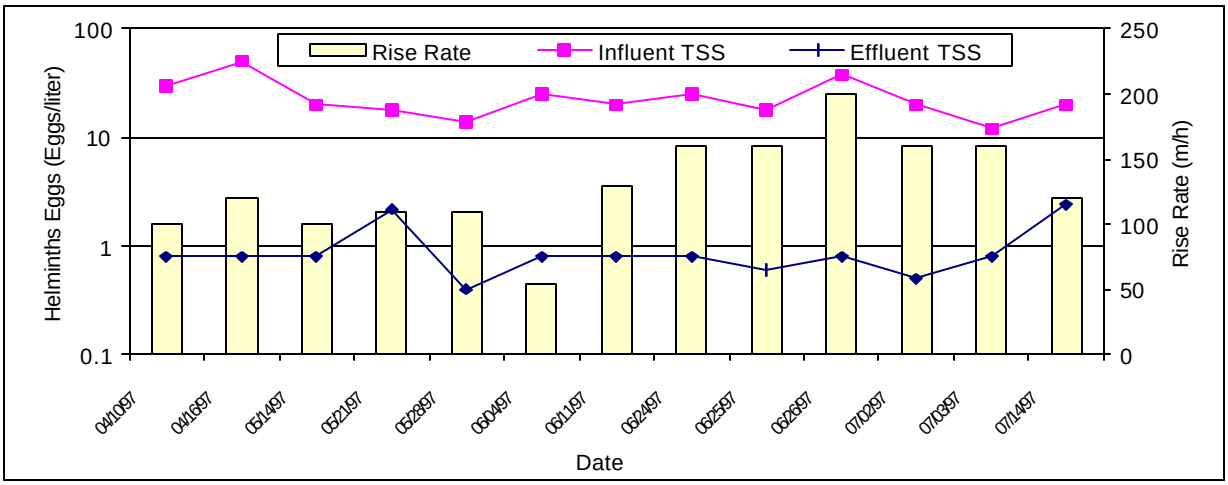



Figure 5.1.b-4. Helminth eggs removal in primary wastewater treatment in Mexico City, Mexico

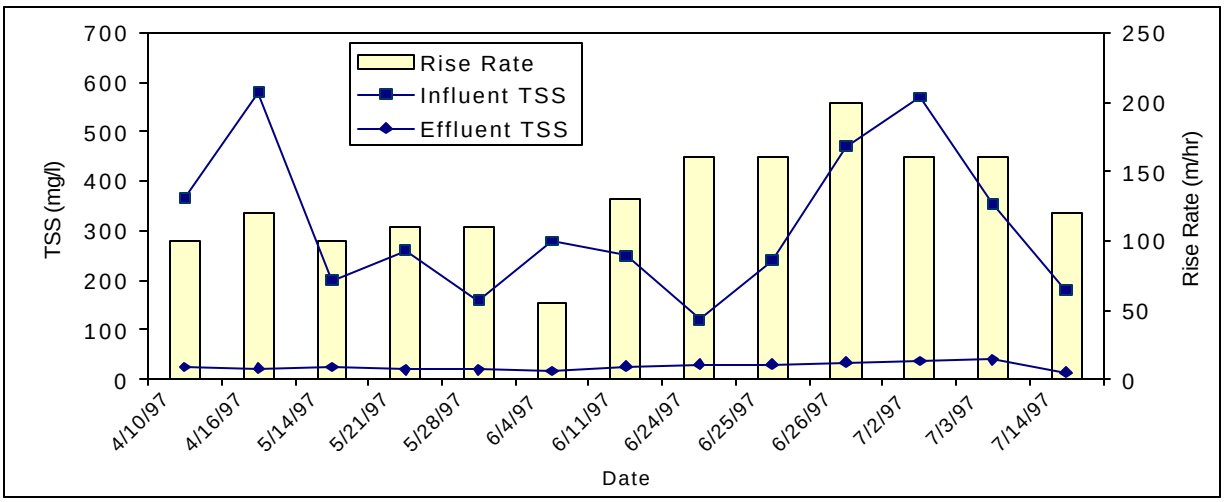



Figure 5.1.b-5. SS removal in primary wastewater treatment in Mexico City, Mexico

*CSO Treatment Application in Galveston, Texas, Pilot Study.* A pilot study was conducted by CDM (1998) in Galveston, TX to evaluate the effectiveness of the ACTIFLO process under simulated CSO conditions. In order to simulate CSO conditions raw wastewater was pumped through a screen into a holding basin where it was subsequently blended with the secondary effluent from the existing WWTF prior to entering the pilot unit.

The effect of coagulant dosage on SS, COD and $BOD_5$ removal efficiencies is shown in Table 5.1.b-8. The pilot unit was operated at an overflow of 75 m/h, a polymer dosage of 1.0 mg/l and a pH of coagulation of 6.0 while the coagulant dosage ($FeCl_3$) was varied from 75—125 mg/l. As shown in the figure, the SS and COD removal efficiencies improved as the $FeCl_3$ was increased from 75 mg/l to 100 mg/l. The removal efficiencies of SS and $BOD_5$ remained constant as the $FeCl_3$ was increased from 100 mg/l to 125 mg/l. Therefore, the optimum coagulant dosage was approximately 100 mg/l.

Table 5.1.b-8. Effect of Coagulant Dosage on Removal Efficiencies in Galveston, TX

| $FeCl_3$ | SS | | | COD | | | BOD5 | | |
|---|---|---|---|---|---|---|---|---|---|
| Dose Mg/l | Raw Mg/l | Settled Mg/l | % Rem. | Raw Mg/l | Settled Mg/l | % Rem. | Raw Mg/l | Settled Mg/l | % Rem. |
| 75 | 123 | 16.9 | 86% | 500 | 112 | 78% | 105 | 39 | 63% |
| 100 | 147 | 13.4 | 91% | 600 | 105 | 92% | 123 | 35 | 72% |
| 125 | 135 | 10.9 | 92% | 300 | 105 | 65% | 195 | 52 | 73% |

In the event of CSO conditions a process that is primarily used during overflow periods will have to start up and achieve steady state very quickly. Figure 5.1.b-6 illustrates the rapid start up capabilities of ACTIFLO process. The first SS and COD samples were taken after two hydraulic retention times (18 min), the time needed to achieve steady state. After the process reached steady state the SS and COD removal efficiencies were 88% and 75% respectively, and the average SS and COD removal efficiencies over the testing period were 91% and 73%, respectively. The short overall retention time of the process allows the rapid start up capabilities of ACTIFLO process. During this experiment the pilot unit was operated at an overflow rate of 75 m/h, a $FeCl_3$ dosage of 100 mg/l and a polymer dosage of 1.0 mg/l.

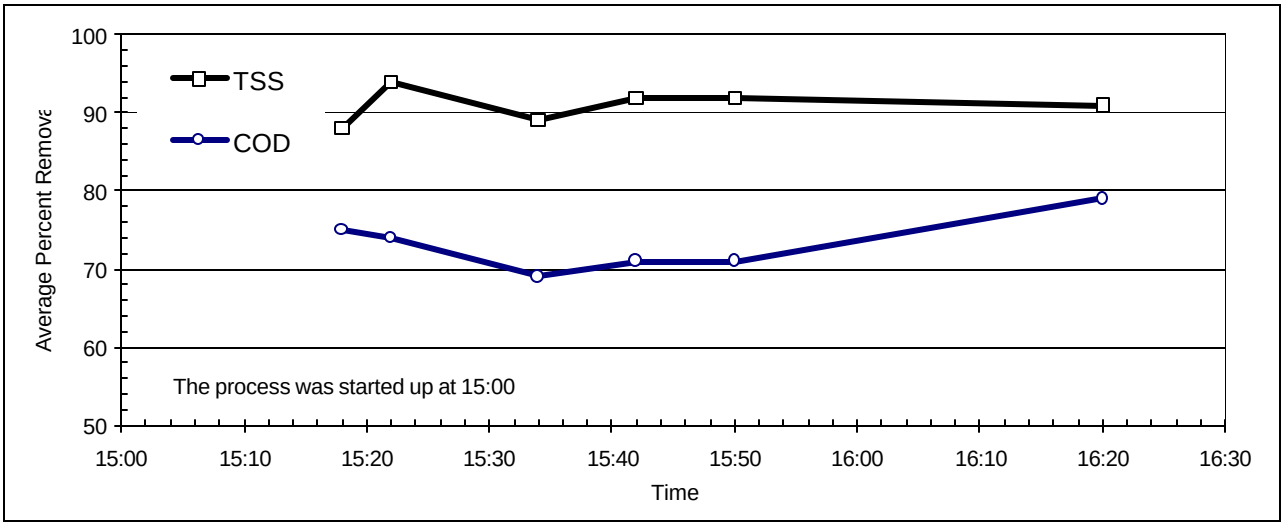



Figure 5.1.b-6. Start-Up Efficiency of ACTIFLO Process

*CSO Treatment Application in Cincinnati, Ohio Pilot Study.* Kruger Inc conducted a pilot study using the ACITFLO® process for the Metropolitan Sewer District at the Mill Creek WWTF in Cincinnati, OH (Kruger 1998). The testing period was from April 14, 1998 through April 29, 1998. This pilot study was conducted to demonstrate the effect of overflow rate on the percent removal of turbidity, $BOD_5$, and total phosphorous during CSO conditions. On April 17, 1998 the pilot unit was operated at a polymer dosage of

1.0 mg/l, a $FeCl_3$ dosage of 35 mg/l while the overflow was varied from 75 m/h to 150 m/h. Table 5.1.b-9 depicts the effect of overflow rate on the removal efficiencies.

Table 5.1.b-9. Effect of overflow rate on removal efficiencies, Cincinnati, OH

| Rise | Turbidity | | | $BOD_5$ | | | Total Phosphorous | | |
|---|---|---|---|---|---|---|---|---|---|
| Rate | Raw | Settled | % | Raw | Settled | % | Raw | Settled | % |
| m/h | NTU | NTU | Rem. | mg/l | mg/l | Rem. | mg/l | mg/l | Rem. |
| 75 | 64 | 2.06 | 96.6% | 125 | 23 | 81.6% | 0.60 | ND[1] | >99% |
| 100 | 72 | 2.35 | 96.8% | 139 | 20 | 85.6% | 1.03 | ND | >99% |
| 150 | 80 | 4.65 | 94.2% | 71 | 20 | 71.8% | 2.99 | ND | >99% |

[1] ND = None Detected

ACTIFLO process achieved greater than 94% removal of turbidity and > 70% removal of $BOD_5$ and >99% removal of total phosphorous. The process performance was not compromised at the higher overflow rates in regards to total phosphorous removal. By comparing the settled water results, higher overflow rates do not adversely effect the process performance.

An overnight run was performed to demonstrate the stability and the performance of the process during CSO conditions. The results of this overnight run are shown in Figure 5.1.b-7. During the night of April 22 and the morning of April 23, the process was operated at an overflow rate of 100 m/h, a polymer dosage of 1.0 mg/l to 1.3 mg/l and a $FeCl_3$ dosage of 45 mg/l to 100 mg/l. The SS removal varied from 60% to 97.2% with an average percent removal of 78.8% and a standard deviation of 7%. The $BOD_5$ percent removal varied from 17.1% to 71.6% with an average percent removal of 45.4% and a standard deviation of 18.2%.

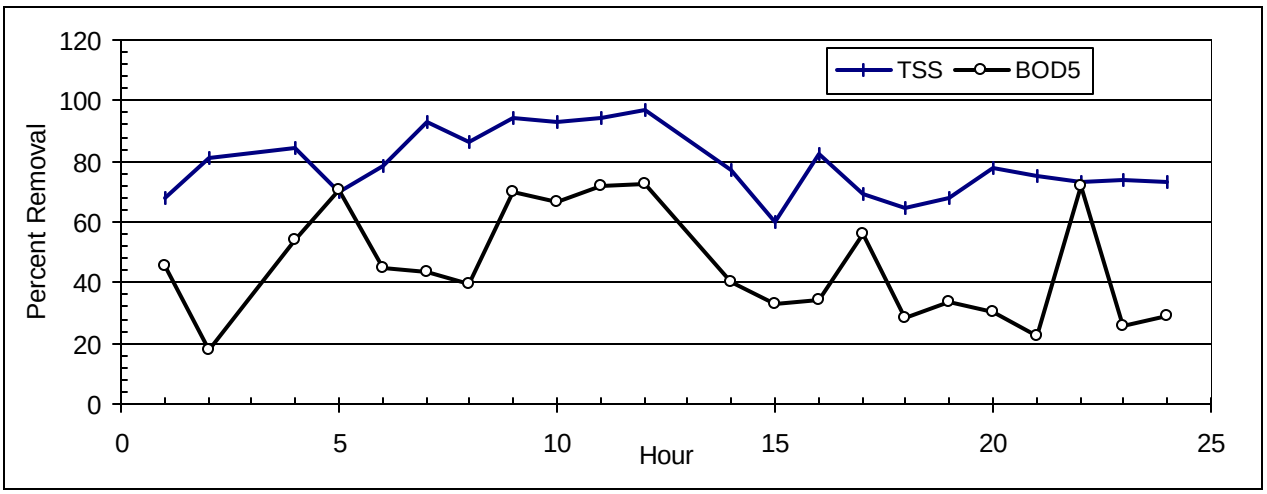



Figure 5.1.b-7. Twenty-four Hour Continuous Demonstration Run, Cincinnati, Ohio

Conclusions

As a primary clarification process, the ACTIFLO process can achieve high removal efficiencies of pollutants including SS and Total Phosphorous. Additionally, the quick start-up and short detention time of ACTIFLO process make it a viable alternative for CSO treatment.

This desktop analysis demonstrates that while the ACTIFLO process may be a viable treatment process, the capital and O&M costs are high versus conventional alternatives. One of the primary benefits of the ACTIFLO system is the extremely small footprint of the process. This can make the ACTIFLO system a more cost-effective alternative for sites where space for large storage facilities is limited.

This process can essentially be adapted and retrofitted into existing primary treatment facilities and for CSO control.

## 5.2 Retrofitting Existing WWF Storage Tanks to Provide Enhanced Settling/ Treatment and Post-Storm Solids Removal. Spring Creek AWPCP, New York, NY

Background

The Spring Creek Auxiliary Water Pollution Control Plant (AWPCP) is a wet-weather combined sewage detention facility with an existing maximum storage volume of approximately 49,200 $m^3$ (13 MG): 37,900 $m^3$ (10 MG) in basin storage and 11,400 $m^3$ (3 MG) in influent barrel storage. The AWPCP facility is located on Spring Creek in Queens, New York, approximately 1 mile east of the 26 Ward Water Pollution Control Plant (WPCP), near the Brooklyn-Queens border. The function of the AWPCP is to capture combined sewage, above elevation -7.00, from tributary drainage areas in Brooklyn and Queens. Flow is conveyed to the plant by the four overflow barrels from the Autumn Avenue regulator located in Brooklyn, and by two overflow barrels from the 157$^{th}$ Avenue regulator located in Queens. The plant has six basins that fill with combined sewage to a depth of 4.6 m (15 ft) and up to a maximum elevation of +1.00, depending on Jamaica Bay tide elevations. Once the basins fill to elevations of -0.50 or greater depending on tides, CSO is released into the bay through the flap gates at the south end of the basins. As flow recedes in the sewers, combined sewage retained within the basins and influent barrels above elevation -7.00 flow by gravity back into the sewer system for treatment at the 26 Ward WPCP. Retained combined sewage below elevation -7.00 is screened and pumped back into the sewer system through the 0.6 m (24 in.) diameter plant effluent sewer. The layout of AWPCP is shown in Figure 5.2-1.

Overview

This desktop analysis evaluates the concept of retrofitting the AWPCP to provide enhanced treatment of stored WWF and removal of solids accumulated in the basin following storm events. WWF is presently retained in the AWPCP to minimize wet-weather overflow through equalization, however, no provision for treatment is provided.

The residual solids and debris deposited within the storage basins are currently cleaned by a system of traveling bridges equipped with a horizontal spray header and a water cannon. Each bridge, one per basin, is operated in multiple passes over the basin length and uses spray water (brackish bay water) to clean the basin walls and floor.

Two alternatives, cross-flow plate settlers and chemical addition, were evaluated to determine the dual effectiveness of retrofitting AWPCP to serve as an equalization system and to enhance the removal of solids prior to discharge to Jamaica Bay.

The criteria for retrofitting the AWPCP for enhanced treatment were selected to allow conceptual design of each retrofit. The primary design factor of these alternatives is the peak flow rate that must be conveyed and treated. The one-year frequency, two-hour duration design storm was selected for this analysis. The peak flow rate associated with the one-year design storm is 66 $m^3$/d (2,330 cfs) based on SWMM modeling of the combined sewer system tributary to Spring Creek and the 26 Ward WPCP.

## 5.2-1 Spring Creek AWPCP Layout

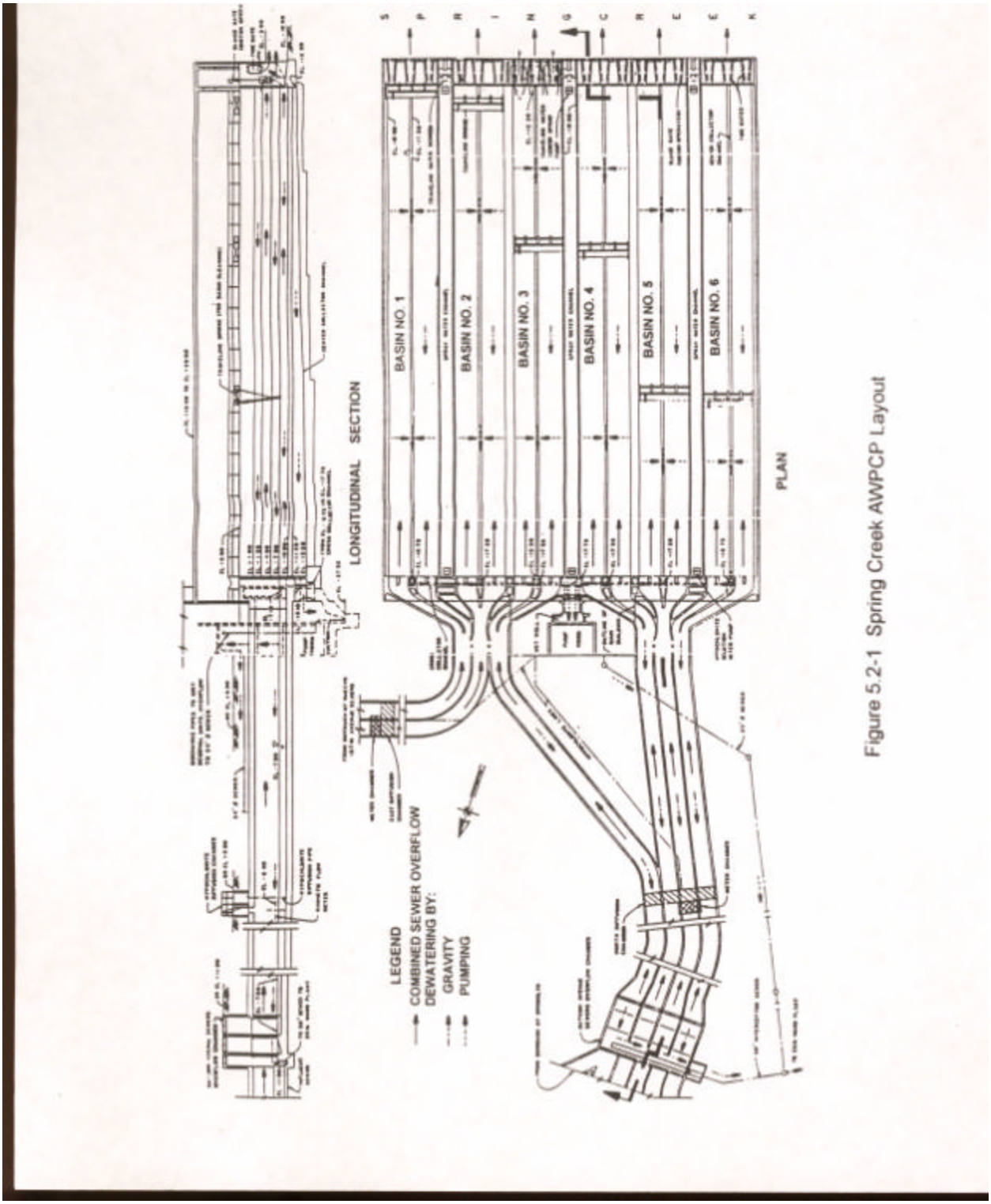



Figure 5.2-1 Spring Creek AWPCP Layout

The existing dimensions of the AWPCP and the design flow rate for the proposed treatment facilities are shown in Table 5.2-1.

Table 5.2-1. Existing Dimensions of the AWPCP and Design Flow Rate

| **Design Data** | **Quantity** |
|---|---|
| Design Storm Flow (one-year storm) | 63.2 $m^3$/s (2,230 cfs) |
| Number of Retention Basins | 6 |
| Flow / Basin | 11.0 m/s (388 cfs) |
| Length of Basin | 152.4 m (500 ft) |
| Width of Basin | 15.2 m (50 ft) |
| Depth of Basin | 3.7 m (12.2 ft) |
| Basin Volume | 8,670 m3 (306,145 $ft^3$) |
| Total Volume | 52,000 $m^3$ (1,836,158 $ft^3$) |
| Basin Surface Area | 2,322 $m^2$ (25,000 $ft^2$) |
| Total Surface Area | 13,935 $m^2$ (150,000 $ft^2$) |
| Cross Sectional Area / Basin | 56.7 $m^2$ (610 $ft^2$) |
| Surface Loading Rate | 410 m3/d/m2 (10,067 gpd/$ft^2$) |
| Hydraulic Detention Time | 15 min |

### *5.2.a. Cross-Flow Plate Settlers*

Cross-flow plate settlers use lamellar clarification to remove solids from wastewater. This process takes advantage of the decantation principle for SS that are denser than water for clarification. Cross-flow plate settlers have performed as well as conventional clarifiers, with a smaller footprint to achieve a similar level of settling.

Cross-flow lamellar clarification are principally used for the removal of oils and grease present in residual liquids emanating from industrial activities in the petrochemical, chemical, mechanical, metallurgical, paper-processing and food-processing sectors. The cross-flow plate settler technology has also been successfully utilized in conventional primary clarifiers and for gravity settling of surface water runoff.

Case History: Shell Montreal Canada

The Shell Company in Montreal, Canada retrofitted their existing storage basin with cross-flow plate settlers to improve the water quality of the effluent prior to discharge. This concrete storage basin collects surface runoff from the refinery area. The

parameters that were used for design of the cross-flow plate settlers are shown in Table 5.2.a-1.

Table 5.2.a-1. Design Parameters for the Cross-Flow Plate Separator

| Parameter | Quantity |
|---|---|
| Treatment Capacity (peak flow) | 7,570 L/min (2,000 gpm) |
| Influent TSS Concentration | 200 mg/L |
| Average TSS Density | 1,159 mg/L |
| Temperature | 20 deg. C |
| pH | 8 to 9 |
| Overflow Rate | 404 $m^3/d/m^2$ (9,930 $gpd/ft^2$) |

The concrete tanks were modified to include cross-flow plate settlers that provided a 27 $m^2$ (290 $ft^2$) surface area for a total projected surface of 223 $m^2$ (2,400 $ft^2$). The installation included:

- Inlet deflectors
- Inlet screening
- Two cross-flow plate cells
- A V-notch weir
- An adjustable skimmer
- An effluent weir

The layout of the retrofitted settling tanks is shown on Figure 5.2.a-1.

Limited evaluation of this retrofit has shown that SS concentrations have been reduced to between 100—125 mg/L throughout a range of influent flow rates and water qualities.

This retrofit would improve the treatment capacity of the existing settling tank without the requirement for costly expansion or the construction of new settling tanks to achieve comparable levels of SS removal. Cost information for this retrofit is unavailable.

Proposed Retrofit of AWPCP with Cross Flow Settlers

This retrofit consists of converting the existing Spring Creek basins to include cross-flow lamellar plate settlers to provide treatment through gravity settling of dense particles. The use of inclined plates significantly increases the ratio of settling surface to footprint that would be required for conventional primary clarifiers. The capacity of primary clarifiers is primarily a function of the surface area rather than depth. The settling area is a function of the surface area. Theoretically, the settling area of a given spatial area could be increased by stacking horizontal plates to achieve a settling area equal to the surface area times the number of plates. However, this arrangement does not allow for removal of the accumulated sludge.

## 5.2.a-1 Layout of Shell Co. Cross-Flow Plate Settler, Montreal Canada

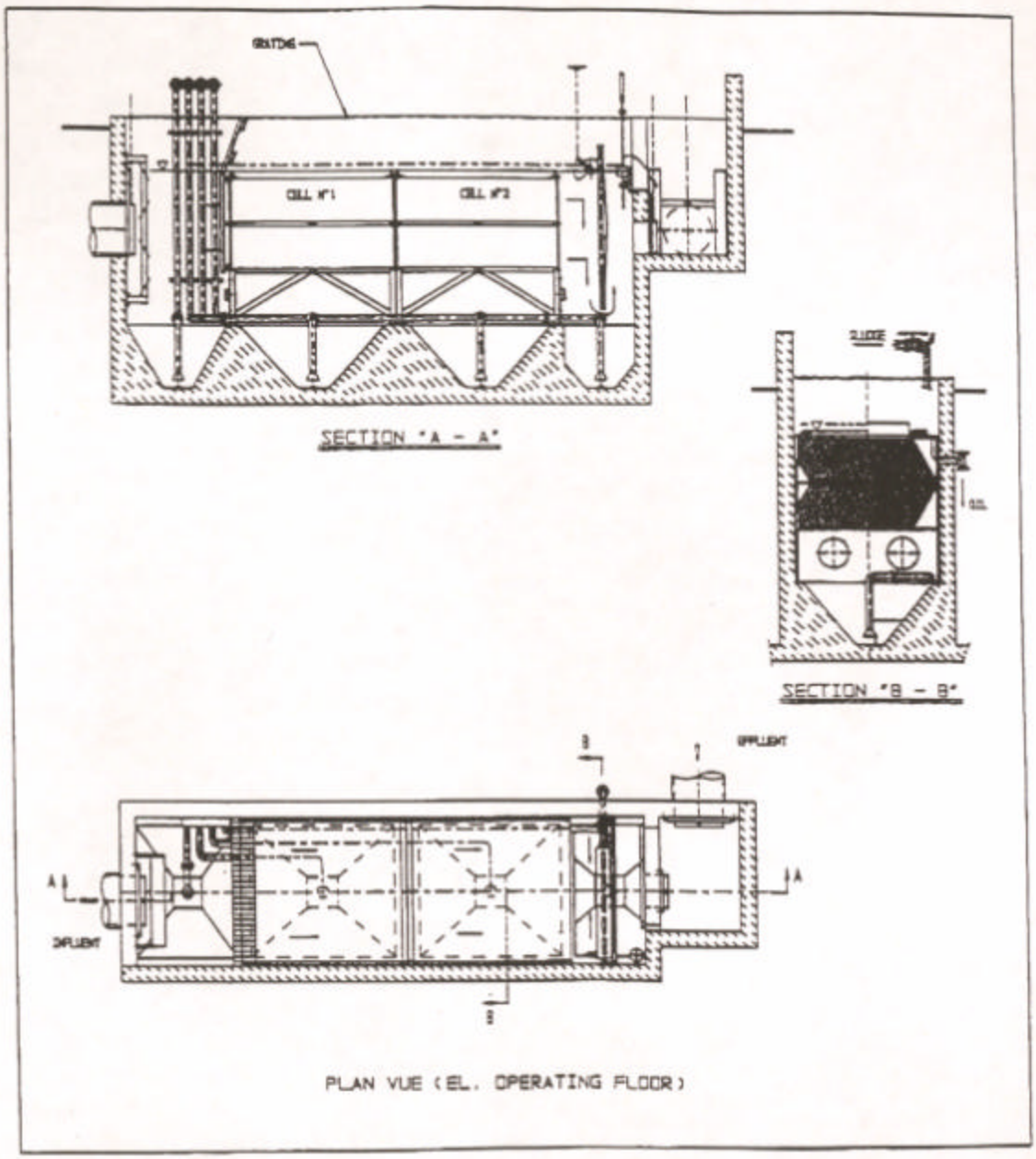



Figure 5.2.a-1 Layout of Shell Co. Cross-Flow Plate Settler, Montreal, Canada

Inclining the plates would make sludge removal possible. Each plate would have an effective settling area equal to its horizontal projection. The projections of inclined plates packed close together (e.g. 4 in. apart) overlap. The total settling area is determined by adding the horizontal projections. This can increase the theoretical settling area by 10 times the settling area compared to a conventional clarifier (Huebner 1979).

There are two major force vectors acting on SS contained within wastewater in the cross-flow separator. The force of gravity acts to pull the SS down while the velocity vector pushes the SS up along the length of the plates. The resultant of these forces directs the SS to the plate where particles impinges on the plate and come in contact with other particles sliding downward along the plate. The flow between the plates is maintained in the laminar range to promote settling.

The treatment process begins with the screened influent entering the lamellar zone that consists of cross-flow cells. Particle settling occurs by gravity according to Stoke's Law that describes the velocity of a particle falling through a fluid. Stoke's Law states that the velocity of a particle is equal to the square of the particle radius.

In the design of sedimentation basins, the usual procedure is to select a particle with a terminal velocity $V_c$ and to design the basin so particles that have a terminal velocity equal to or greater than $V_c$ will be removed.

The lamellar cell design is a practical application of Hazen's theory on settling that provides a large equivalent surface of separation (as noted by the projected horizontal area for each plate). Hazen's theory is stated by Huebner (1979) where the velocity is:

V = average linear velocity $= \dfrac{q}{d \cdot w}$ (2)

$q$ = volumetric flow rate

$d$ = distance between plates

$w$ = width of plates

$\Theta$ = angle of elevation to the horizontal

$V_s$ = particle settling velocity

$d_s$ = maximum settling distance

L = length of plates

Particles are removed from the flow stream when its settling time is less than its flow through time.

$$t_s \leq t_q \quad (3)$$

$$t_s = \frac{d_s}{V_s} \quad (4)$$

$$t_q = \frac{L}{V} = \frac{L}{q/d \cdot w} = \frac{L \cdot d \cdot w}{q} \qquad (5)$$

The maximum settling distance, $d_S$ can be calculated as a function of the distance between the plates, d and the angle of inclination of the plate, Θ.

$$d_s = \frac{d}{Cos\,\Theta} \qquad (6)$$

Which equates to:

$$\frac{d/Cos\,\Theta}{V_s} \leq \frac{L \cdot d \cdot w}{q} \qquad (7)$$

and further simplifies to:

$$\frac{q}{L \cdot w} \leq V_s\, Cos\,\Theta \qquad (8)$$

Hazen's theory provides the surface loading rate (Y) for a conventional clarifier, as:

$$Y = \frac{q}{L \cdot w} \qquad (9)$$

which is equivalent to the simplified inclined plate formula derived above in that q is the total flow divided by the number of plates. The conventional surface loading rate occurs when the angle Θ goes to zero, resulting in a $Cos\,\Theta$ of 1.

The principal design parameters of cross-flow plate settlers are:

*Wastewater flow rate* - Settling systems are designed to accommodate the peak flow rate, but the maximum removal efficiency is normally achieved at a lower design flow rate.

*Settling rate of solids* – The settling rate of solids is measured in laboratory tests. A design velocity is then selected according to the required removal efficiency. The required settling area is calculated by dividing the design flow rate by the design velocity.

*Angle/spacing of the plate* – The angle of the plates typically varies from 55—65 degrees and the spacing varies from 2.5—10.2 cm (1—4 in.) depending upon the nature and the concentration of solids and space limitations. The 10.2 cm (4 in.) spacing is commonly used in wastewater applications.

A depiction of the cross-flow plate is shown on Figure 5.2.a-2.

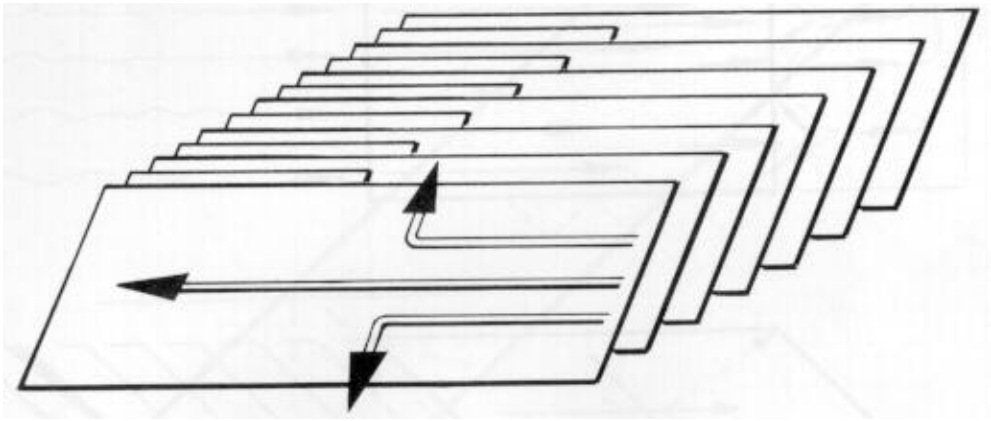


Figure 5.2.a-2. Horizontal Projections of Inclined Cross-Flow Lamellar Plates

Cross Flow Plate Settler Equipment

The cross-flow plate settlers selected for this retrofit are from Aquarius Services & Technologies Inc., Quebec. The major equipment of the cross-flow lamellar plate settlers is shown in Figures 5.2.a-3 and 5.2.a-4, and include the following:

*1. Flow Deflectors* – Flow deflectors at the inlet of each basin dissipate the energy generated by the influent flow. This encourages flocculation and distributes the flow evenly along the width of the basin and wet perimeter of the lamellar cell.

*2. Inlet Bar Screen* - Floating material that may clog the lamellar cells is removed by a inlet bar screen with 5.1 cm (2 in.) square openings

*3. and 4. Cross Flow Cells* - Cross-flow type lamellar cells consist of inclined plates at a 60° angle and are constructed of either aluminum, stainless steel or synthetic materials. Spacing between plates may vary according to the design requirements. For the purpose of this retrofit, stainless steel plates with 10.2 cm (4 in.) spacing were selected based on the manufacturer's suggestions for this installation.

*5. Skimming Device* - Oils and hydrocarbons are skimmed from the surface through an adjustable skimmer.

*6. Scum Baffle* - A scum baffle is installed in back of the skimmer to retain the floatables for collection and disposal through the skimmer.

*7. Weir* - A V-notch weir is installed at the discharge end of the basin to regulate the liquid level in the basin.

*8. Sludge Vacuum Collector* **-** Each sludge silo is equipped with a sludge vacuum collector to transfer and dispose of collected sludge.

Figures 5.2.a-3. Plan View of Aquarius Cross-Flow Plate Settler

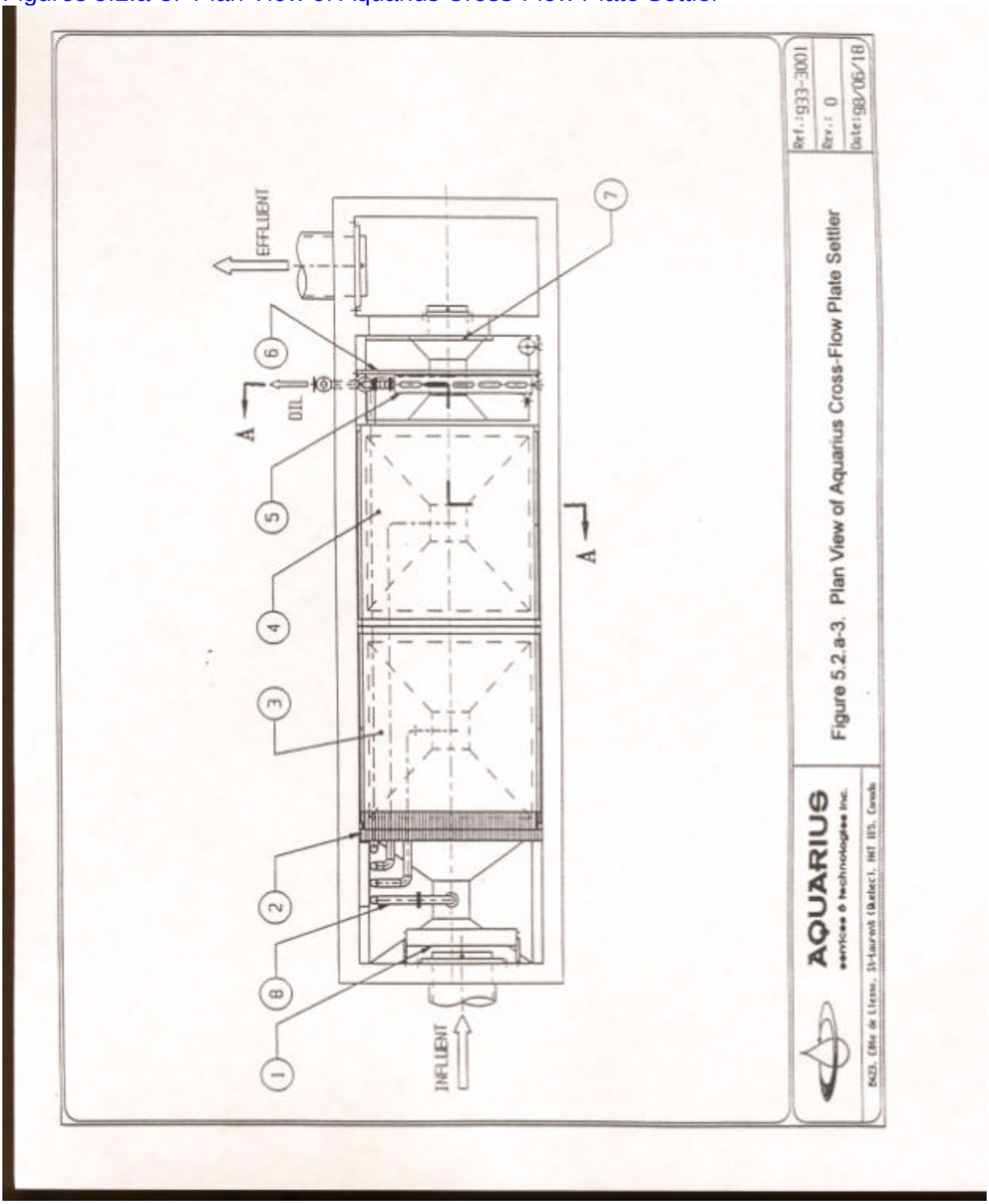
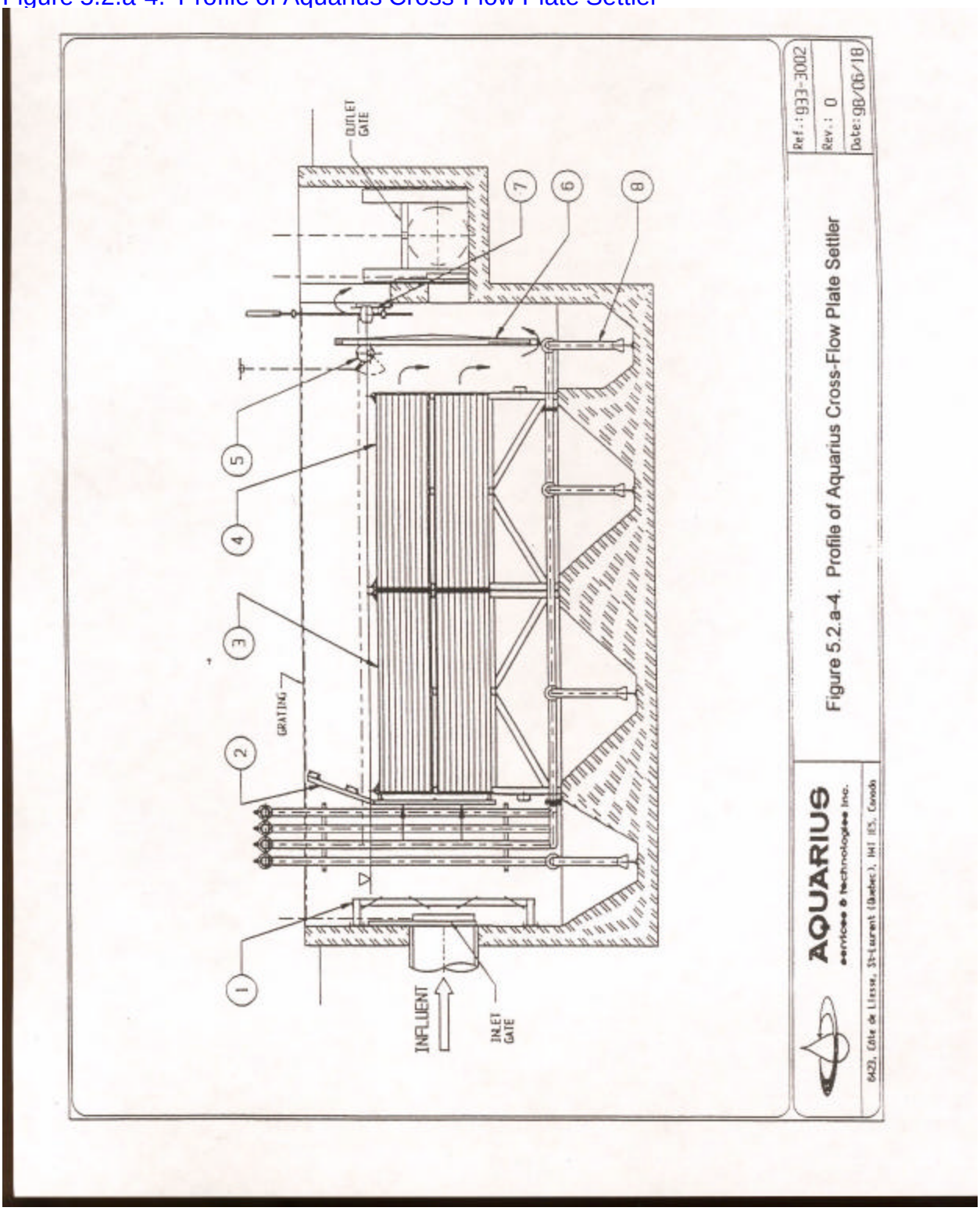


Figure 5.2.a-4. Profile of Aquarius Cross-Flow Plate Settler



Following the completion of the storm, the basins would be automatically washed and the sludge will be transferred for disposal.

The hydraulic design criteria for the cross-flow plate settler technology is shown on Table 5.2.a-2.

Table 5.2.a-2. Cross-Flow Plate Settler Design Data

| Design Data | Quantity |
|---|---|
| Hydraulic Loading Rate* | 0.15 $m^3$/s/100 $m^2$ (0.5 cfs/100 sq. ft) |
| Overflow Rate | 132 $m^3$/day/$m^2$ (3,250 gpd/sq. ft.) |

* Loading Rate furnished by Aquarius Services & Technologies Inc. Quebec

Under the above loading rates and available basin surface area, the one-year design flow rate of 63 $m^3$/s (2,230 cfs) would not be accommodated by a cross flow plate settler system. The proposed system is capable of treating a maximum flow rate of 1,835,725 $m^3$/d (485 MGD) or 32% of the design flow rate based on the physical size of the Spring Creek basins and the limited hydraulic loading rates of the cross-flow plate settlers. The proposed facilities would provide treatment under a small percentage of wet-weather events observed at Spring Creek and would create a hydraulic limitation to WWF entering the facility. Therefore, retrofit with cross-flow plate settlers is not considered viable for this site.

### *5.2.b. Chemical Addition to AWPCP*

This desktop analysis involves retrofitting the AWPCP by chemical addition to provide enhanced settling/treatment. Chemical addition describes a method of treatment where chemicals are added to enhance settling of particles present in the wastewater. Treated water is then decanted and conveyed for further treatment or discharged. The resultant sludge is collected for disposal and can be de-watered to reduce the volume of solids. Chemical precipitation can be used to remove metals, fats, oils and greases (FOG), SS and some organics. It can also to be used to remove phosphorus, fluoride, ferrocyanide and other inorganics.

Conventional wastewater clarification processes primarily involve the destabilization and subsequent removal of colloidal SS materials that are not readily removed by gravitation alone. These suspended materials can be natural or synthetic organic or inorganic compounds, microorganisms, and/or viruses that typically range in size from $10^{-4}$ $\mu$m to 1,000 $\mu$m. In most natural systems, the stability of colloidal suspended materials is attributed to a net negative surface charge that causes individual particles to repel each other and remain in suspension. To counteract these repulsive forces a highly charged ionic chemical, such as alum ($Al_2$ $(SO_4)_3$), $FeCl_3$, ferric sulfate ($Fe_2(SO_4)_3$), poly-aluminum chloride (PACl), is added to bring about a net reduction in the repulsive force between the suspended materials. This process, termed coagulation, results in the

destabilization and/or attraction of the SS to form chemical floc. Although destabilized, the chemical floc may remain in solution due to its extremely small size and low mass. These floc, often called “pin floc,” are difficult to remove from the treated water without first aggregating the smaller particles together into larger, heavier, more settleable floc by flocculation.

Flocculation is the term used to describe the aggregation of pin floc into larger, heavier, more settleable floc. This is most readily accomplished with the addition of a chemical (flocculant aid) polymer. Such polymers are available from many sources in both liquid and dry form with varying molecular weights and net surface charges. In most cases selection of the best polymer for a specific application and set of conditions is largely a matter of trial and error. Larger, more settleable flocs form by inter-particle polyelectrolyte bridges between floc and destabilized solids. Although the formation of inter-particle bridges can take many forms, the end result is the enmeshment of large volumes of particles to form large dense floc that are more readily removed by gravitational settling.

The use of chemical addition would enhance the settling of discrete particles and suspended matter thereby increasing the overall removal of $BOD_5$ and SS. This would be accomplished by retrofitting the existing basins with chemical feed, mixing and contact facilities. The retrofit would also include dedicated sludge collection and removal facilities. Chemical coagulation of raw wastewater before sedimentation promotes flocculation of finely divided solids into more readily settleable flocs, thereby increasing SS, $BOD_5$, and phosphorous removal efficiencies. Sedimentation with coagulation may remove 60—90% of the SS, 40—70% of the $BOD_5$, 70—90% of the phosphorous, and 80—90% of the bacterial loadings. In comparison, sedimentation without coagulation may remove only 40—70% of the SS, 25—40% of the $BOD_5$, 5—10% of the phosphorous loadings and 50—60% of the bacteria loading.

Chemical Coagulants

Advantages of coagulation include greater removal efficiencies, the ability to treat higher flow rates, and more consistent performance. Disadvantages of coagulation include an increased mass of primary sludge that is often difficult to thicken and de-water, and increased in operational cost and operator attention (WEF 1992). Iron salts, particularly $FeCl_3$, are the most common coagulant used for primary treatment. WWTF using lime for primary treatment have found that it produces more sludge than do metal salts and is more difficult to store, handle and introduce into the waste stream. Coagulants should be selected for performance, reliability and cost and evaluations made to determine dosages and effectiveness.

Case Histories

Table 5.2.b-1 lists coagulant dosages and performance data for six (6) treatment plants that employ chemical coagulation for advanced primary performance in sedimentation tanks. The Joint Water Pollution Control Plant (JWPCP) at Carson, California evaluated 30 different polymers and found anionic polymers to be the most effective type for enhancing primary sedimentation. The Hyperion plant in Los Angeles, California uses to great success $FeCl_3$ and an anionic polymer at the headworks before primary sedimentation. This process increased primary sludge production 45% through improved SS and colloidal matter removal. This increase was partially offset by a decrease in waste activated sludge production.

The Los Angeles and San Diego sedimentation tanks with chemical coagulation operate with overflow rates between 69.3—81.5 $m^3/d/m^2$ (l700—2000 $gpd/ft^2$) as compared to conventional overflow rates of 32.6 $m^3/d/m^2$ (800 $gpd/ft^2$) for primary settling. Research in Sarnia and Windsor, Ontario indicated that overflow rates up to 97.8 $m^3/d/m^2$ (2,400 $gpd/ft^2$) did not significantly affect effluent quality.

Design Considerations

The design flow rate for this retrofit is 66 $m^3/s$ (2,330 cfs) as shown in Table 5.2-1 with a corresponding overflow rate of 410.1 $m^3/d/m^2$ (10,067 $gpd/ft^2$) based on the dimensions of AWPCP. This high surface SOR is greater than what would typically be applied for a chemical coagulation system; however, for the purposes of this retrofit it is assumed that during the higher SOR's, a reduced efficiency will result. It is important to note that the frequency at which these high SOR's are likely to occur are expected to be relatively infrequent (e.g., once per year).

The proposed retrofit consists of equipping the existing influent barrels with a chemical feed and mixing system. These facilities would provide the necessary mixing and contact for the rapid mix and flocculation steps, which include the addition of $FeCl_3$ and an anionic polymer. An important element in this analysis is an evaluation of the hydraulic and physical parameters dictated by the existing basins as depicted on Table 5.2-1.

This retrofit would include the specific elements outlined below. Based on a ratio of 452 kW / $m^3/d$ (0.16 HP/MGD) at a peak flow of 953,820 $m^3/d$ (252 MGD) per barrel, a total of 30 kW (40 HP) is necessary to achieve the mixing required for the $FeCl_3$ per barrel. There will also be 2 mixers per basin for mixing the polymer. Therefore, 24—15 kW (20 HP) mixers (4 /barrel) are necessary for the 6 basins. Chemical storage requirements are based on the need to treat approximately 2,500,000 $m^3$ (660 MG) of CSO per year. Storage of $FeCl_3$ (37% solution) and polymer (0.25 mg/l) to treat a month (on average) of CSO would require approximately 16 $m^3$ (4,200 gal) of chemical storage.

Table 5.2.b-1. Coagulant Dosages and Performance Data for Six Treatment Plants That Employ Chemical Coagulation for Advanced Primary Performance in Sedimentation Tanks

| | | | Advanced Primary Performance | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | BOD | | | TSS | | | Chemical Addition | | |
| | Flow | | Influent | Effluent | Rem. | Influent | Effluent | Rem. | Type | Conc. | Duration |
| **Location** | ($m^3$/day) | (MGD) | mg/L | mg/L | % | mg/L | mg/L | % | | mg/L | |
| Point Loma<br>City of San Diego | 722,935 | 191 | 276 | 119 | 57 | 305 | 60 | 80.3 | $FeCl_3$<br>Anionic Poly | 35<br>0.26 | Continuous |
| Orange County<br>Plant No. 1 | 227,100 | 60 | 263 | 162 | 38 | 229 | 81 | 64.6 | $FeCl_3$<br>Anionic Poly | 20<br>0.25 | 8 hours<br>Peak Flow |
| Orange County<br>Plant No. 2 | 696,440 | 184 | 248 | 134 | 46 | 232 | 71 | 69.4 | $FeCl_3$<br>Anionic Poly | 30<br>0.14 | 12 hours<br>Peak Flow |
| JWPCP<br>Los Angeles County | 1,438,300 | 380 | 365 | 210 | 42 | 475 | 105 | 77.9 | Anionic Poly | 0.15 | Continuous |
| Hyperion<br>Los Angeles City | 1,400,450 | 370 | 300 | 145 | 52 | 270 | 45 | 83.3 | $FeCl_3$<br>Anionic Poly | 20<br>0.25 | Continuous |
| Sarnia<br>Ontario, Canada | 37,850 | 10 | 98 | 49 | 50 | 124 | 25 | 79.8 | $FeCl_3$<br>Anionic Poly | 17<br>0.30 | Continuous |

Chemical feed pumps will be necessary to deliver the correct dosing of chemical during wet-weather events. The $FeCl_3$ solution will require a feed rate of approximately 0.05 $m^3$/min (13 gpm) per barrel at peak flow; resulting in 2 metering pumps per barrel for a total of 24. The pumps would be hose (or peristaltic type) pumps. Half as many feed pumps would be required to deliver the polymer to the influent barrels.

Operators could control the chemical feed system using a new control panel and SCADA system housed in the existing control room. The capital costs for this retrofit are shown on Table 5.2.b-2.

Table 5.2.b-2. Estimated Capital Costs Chemical Addition Retrofit

| Description | Cost |
|---|---|
| General Conditions | $25,000 |
| Sitework | $350,000 |
| Chemical Storage | $25,000 |
| Chemical Feed System | $75,000 |
| Chemical Mixers (4 in each of 6 basins) | $720,000 |
| Instrumentation and SCADA System | $25,000 |
| Total Construction Cost | $1,220,000 |
| Project Services[1] | $240,000 |
| **Total Project Cost** | **$1,460,000** |

[1] Includes Design & Construction Engineering, Legal, and Fiscal Expenses
Does not include contingencies and right-of-way costs

Operational Requirements

The existing AWPCP operates intermittently based on wet-weather events. Following CSO events, the facility is de-watered by gravity and pumping. These operations would not change under the proposed retrofit. The following list summarizes the areas of expected O&M cost increases at the facility:

- Increased labor to perform wet-weather process adjustments and clean-up following a wet-weather event.
- Power for chemical feed and mixing systems , and
- Chemicals

The costs are based on the facility treating approximately 2,498,100 $m^3$ (660 MG) of CSO per year and maintaining a chemical storage of approximately one month. The annual volume of CSO is based on modeling performed for a typical year using the Storm Water Management Model (SWMM). Hourly rainfall data were used from a local NWS Weather Station as input to the model. It is estimated that approximately 100 wet-weather events occur per year and that each event is approximately 6 h in duration. The O&M costs are shown in Table 5.2.b-3.

Table 5.2.b-3. Estimated Chemical Addition O&M Costs

| Description | Estimated Annual Cost |
|---|---|
| Coagulant Consumption | |
| Coagulant Type | Ferric Chloride |
| Coagulant Dosage (ppm) | 20 |
| Design Flow Rate (MGD) | 1,510 |
| Coagulant Consumption (lbs./day) at Peak Flow | 252,000 |
| Coagulant Consumption (lbs/month) | 9,200 |
| Coagulant Consumption (tons/year) | 44 |
| Coagulant Cost ($/ton) | 200 |
| Coagulant Cost ($/year) | **$8,800** |
| Polymer Consumption | |
| Est. Polymer Dosage (ppm) | 0.25 |
| Design Flow Rate (MGD) at peak flow | 1,510 |
| Polymer Consumption (lbs./day) at Peak Flow | 3,100 |
| Polymer Consumption (lbs./month) | 110 |
| Polymer Consumption (tons/year) | 0.53 |
| Polymer Cost ($/ton) | 3,500 |
| Polymer Cost @ 100% /24/365 ($/year) | **$1,848** |
| Labor | |
| Cleanup/Operations | **$21,000** |
| Energy Consumption | |
| Rapid Mixer HP per Basin | 30 |
| Chemical Feed HP per Basin | 2 |
| Total HP (for 6 basin compartments) | 192 |
| kWh/year | 115,200 |
| Total Annual Energy Cost ($/year) | $6,912 |
| **Total Annual Operation Cost ($/year)** | **$38,600** |

6 hours per event

## Conclusions

This retrofit utilizes the existing storage facilities to provide chemically enhanced sedimentation of WWF. This could result in the removal of 60—90% of the SS, 40—70% of the $BOD_5$, and 30—60% of the COD from the effluent to the receiving water. This is an expected 58% increase over current performance without chemical addition. At the one-year design flow, the facility will exceed typical SORs by a factor of 4—5, however, these would occur relatively infrequently.

### *5.2.c. Retrofitting Existing WWF Storage Tanks For Post-Storm Solids Removal, Spring Creek AWPCP, New York, NY*

In 1997, CDM conducted a basin-cleaning prototype testing program at the Spring Creek AWPCP in New York City (CDM 1997). The prototype testing was performed in response to the New York City Department of Environmental Protection's (NYCDEP) desire to investigate alternatives to the existing traveling bridge-cleaning system. This analysis was performed to evaluate the results of the prototype testing and provide recommendations for basin cleaning system design.

Background

The residual solids and debris deposited within the storage basins are actively cleaned. Currently, for cleaning of solids and debris within the basins, the facility uses a system of traveling bridges equipped with a horizontal spray header and a water cannon. Each bridge, one per basin, is operated in multiple passes over the basin length and uses spray water (brackish bay water) to clean the basin walls and floor.

The basin cleaning alternatives evaluated through prototype installation and testing at the facility were:

- Tipping buckets
- Fixed spray nozzles
- Orifice headers
- Flushing gate (Hydroself Fluid Flap)
- Re-suspension/mixing (used to enhance any of the above)

Objectives and Scope

The effectiveness of the cleaning systems at the Spring Creek AWPCP were evaluated to establish design criteria for feasible alternatives for full implementation and to develop operational requirements and operational and capital costs associated with each of the systems. The data collected from the prototype study was used to either determine or indicate the following:

- Effectiveness of floor cleaning
- Effectiveness of wall cleaning
- Water usage
- Power usage
- Overall cleaning time required
- Special operating requirements
- Safety concerns (if any) associated with each system
- Cost

Each of the systems listed above was installed in test areas within three of the six basins at Spring Creek. As discussed below, each system was arranged in different

configurations and operated during actual basin cleaning events for a total of 16 cleaning test events, over the period from July 1 to December 16, 1997. The cleaning systems were tested under conditions where solids accumulation varied widely. Each system was operated under a variety of cleaning water volumes, flow rates, and pressures. As the testing proceeded, it became apparent which technologies were not providing effective cleaning. As a result, the original test protocol was modified to focus on, and optimize the performance of the most effective systems.

The actual test conditions and the equipment used in the study are summarized below.

*Basin Cleaning Prototype Equipment and Operation.* The prototype testing consisted of 16 test runs, from July 1, 1997 through December 17, 1997. The prototype fieldwork was performed in conjunction with NYCDEP's routine basin cleaning operations to benchmark the existing cleaning operations at the facility.

The following parameters were monitored and recorded during the prototype testing:

- Solids depth within basins
- Equipment test parameters (flow rate, total wash-water volume, pressure, etc.)
- System cleaning effectiveness
- Cleaning duration

The overall cleaning effectiveness was based on a qualitative assessment of the degree of cleaning and the approximate percentage of the floor area cleaned. The test area was judged to be clean when approximately 99% of the test area was without solids. Water consumption and operating pressures were measured using flow meters and pressure gauges set up throughout the system. Cleaning time was measured for the prototype systems and then estimated regarding full-scale implementation of each system.

An evaluation of the existing basin cleaning operation was performed and provided a benchmark for the prototype cleaning systems. This benchmarking was performed by observing the cleaning operations, documenting the cleaning time and using the rated pump capacity to calculate the flow rate and water used by the existing traveling bridges.

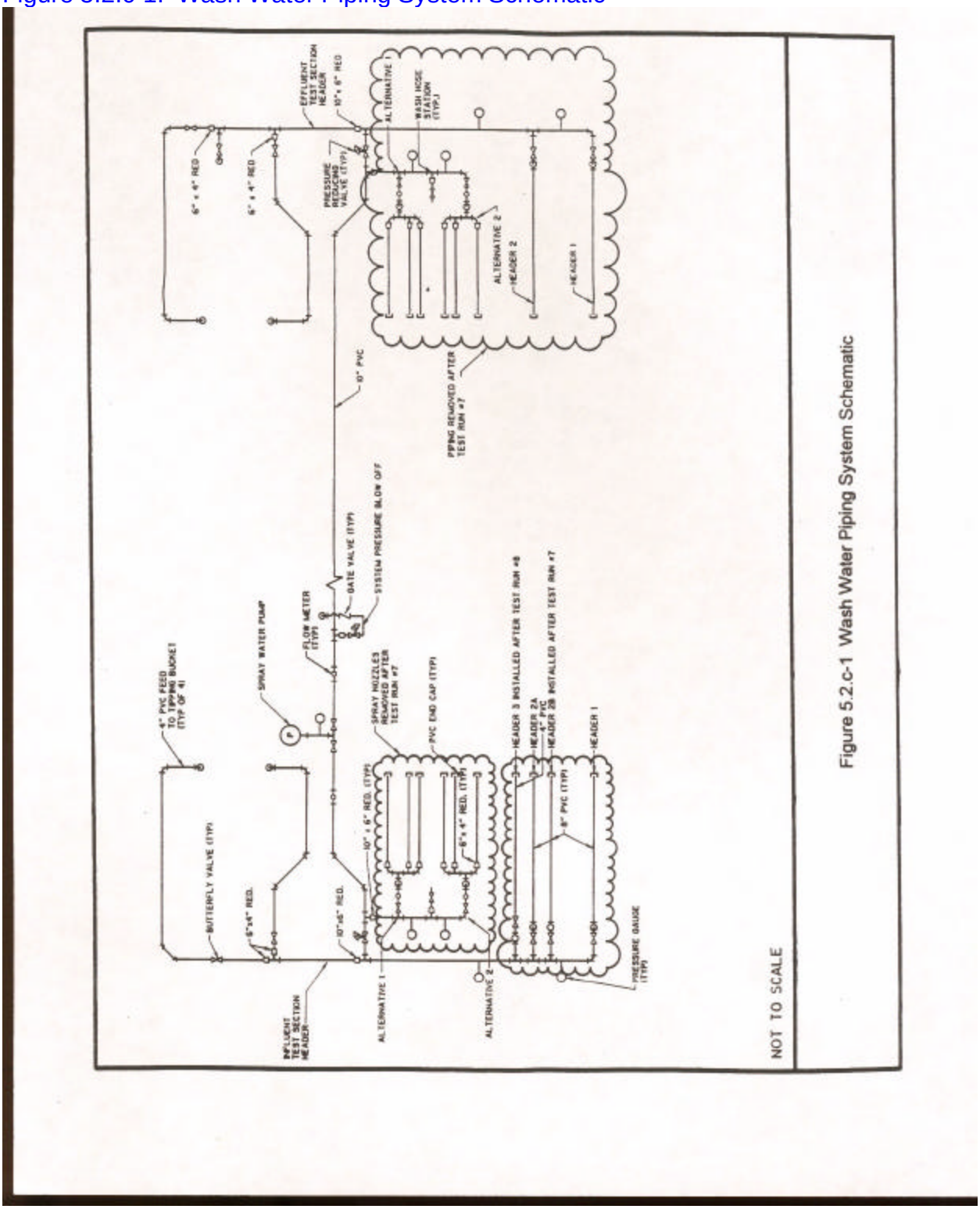
Using the traveling bridges, the basins were generally cleaned one side at a time. Typically, the traveling bridge requires a minimum of two passes per side. The first pass loosens the solids and provides some cleaning, while the latter passes provide for a more thorough cleaning. A manually controlled water cannon breaks up large heavy solids. In addition to cleaning the basin floor, the water cannon cleans the sidewalls of the basin and the walkways of the basin area.

*General Prototype Systems Installation and Operation.* Five prototype cleaning systems were tested: tipping buckets, flushing gate, fixed spray headers with nozzles, fixed spray headers with orifices, and a submersible mixer. The prototype systems were located in basins 1, 2 and 3.

Bay water was used as the source of cleaning water for the prototype systems. Cleaning water was supplied from a single, 37.3 kW (50 HP), three stage vertical turbine wash-water pump, installed in the existing spray water channel between basins 1 and 2. The pump was sized for a flow rate of 3 $m^3$/min (800 gpm) against a head of 50.3 m (165 ft). A PVC piping system distributed wash-water to the various cleaning systems. A flow schematic of the wash-water equipment and piping is provided in Figure 5.2.c-1. Flow meters, pressure gages, and butterfly valves were installed throughout the cleaning-water piping-system to provide a positive means of regulating the system flow and pressure. Pressure control valves were installed within the piping system to regulate the delivery pressure of the cleaning water to the required level. A pressure sustaining valve was installed on the main piping header to relieve excess flow and to maintain system pressure during times the full 3 $m^3$/min (800 gpm) flowed but was not required.

Prototype testing occurred in conjunction with NYCDEP's normal basin cleaning operations. The basins were pumped and drained by plant staff prior to the normal cleaning operations. Plant staff cleaned the length of the basins, except for the 6.1 m (20 ft) test sections located in basins 1, 2, and 3 using the existing traveling bridge system. On occasion, some test sections were not available for testing either due to cleaning of the test section by plant staff or by inflow of the tide through leaking tide gates. Generally after the plant staff completed cleaning, the prototype system testing began. Sediment depth in the prototype system test sections was measured and recorded by entering the basins. The distribution of solids within each test section was documented. The tipping buckets, spray nozzles and orifice headers at the influent end of the basins were generally tested first. After the testing at the influent end of the basins was finished, the tipping buckets, flushing gate, spray nozzles and orifice headers at the effluent end of the basins were then tested. During several test runs, the prototype testing was halted early due to overloading of solids in the AWPCP’s cross collector channel and the chamber screen channels. As discussed later, sudden slugs of cleaning water, carrying large quantities of solids, overloaded the channels causing the testing to be ended prematurely.

Figure 5.2.c-1. Wash Water Piping System Schematic



As the results of the prototype testing were analyzed, the operating conditions were adjusted and the actual prototype system was modified or the entire prototype system was removed from testing when deemed ineffective.

*Tipping Buckets.* Grande, Novac & Associates, Inc., of Montreal, Canada supplied the units tipping buckets. Four tipping buckets were installed in basin 1. Two buckets, one each on opposite sidewalls, were installed at elevation +1.5 m (+ 5.00 ft) at both the influent (TB-1A and TB-2A) and overflow (TB-1B and TB-2B) test sections of the basin, as shown in Figures 5.2.c-2 and 5.2.c-3. The buckets were installed at either end of the basin to test the cleaning effectiveness of the highest and lowest solids loading in the basin. The overflow end buckets were used to determine the ability of the buckets to convey the solids down the full length of the center velocity channel.

Each bucket was 6.1 m (20 ft) long and had a liquid capacity of 0.15 $m^3$ per linear meter (11 gal per linear foot [gal/ft]) for a total volume of 0.8 $m^3$ (220 gal) per bucket. During testing, a constant supply of cleaning water was fed into the buckets. When the maximum water level was reached in the buckets, the buckets would automatically tip due to the change in the bucket's center of gravity, a 0.8 $m^3$ (220 gal) wave of water down the basin wall and along the floor, to the velocity channel.

A training wall, stretching the width of the basin, from the velocity channel to the basin sidewall, was originally installed along the edge of the test area. The purpose of the training wall was to prevent the wave of cleaning water from dispersing laterally, thereby maintaining the full force of the wave within the test section. In addition, curved wall fillets were installed at the bottom of the sidewall to allow for a smooth transition of water flow from the wall to the floor. Figure 5.2.c-3 shows the approximate size and location of the training walls and wall fillets.

Once the NYCDEP's cleaning operations were completed and the sediment in the basins measured, filling of the tipping buckets commenced. The two buckets at the influent end of the basins were filled simultaneously and then discharged automatically. The process was repeated as necessary until the test section was considered clean. Table 5.2.c-1 summarizes the test conditions for the tipping buckets along with the other prototype systems tested.

After the installation of the equipment it was found that bucket TB-2B did not tip automatically. A counter weight was added to correct bucket mounting/misalignment and to permit the bucket to tip. Periodically, bucket TB-1B also experienced hesitation in tipping.

Following the initial test runs, modifications were made to the training walls and fillets to improve their effectiveness at eliminating dead zones within the basins. The original training walls, made of steel angle approximately 15—30 cm (0.5—1 ft) in height, were bolted to the basin floor. The training walls extended from the walls at the column line to the velocity channel. During testing it was observed that the solids along the training

Figure 5.2.c-2. Tipping Buckets – Enlarged Plan

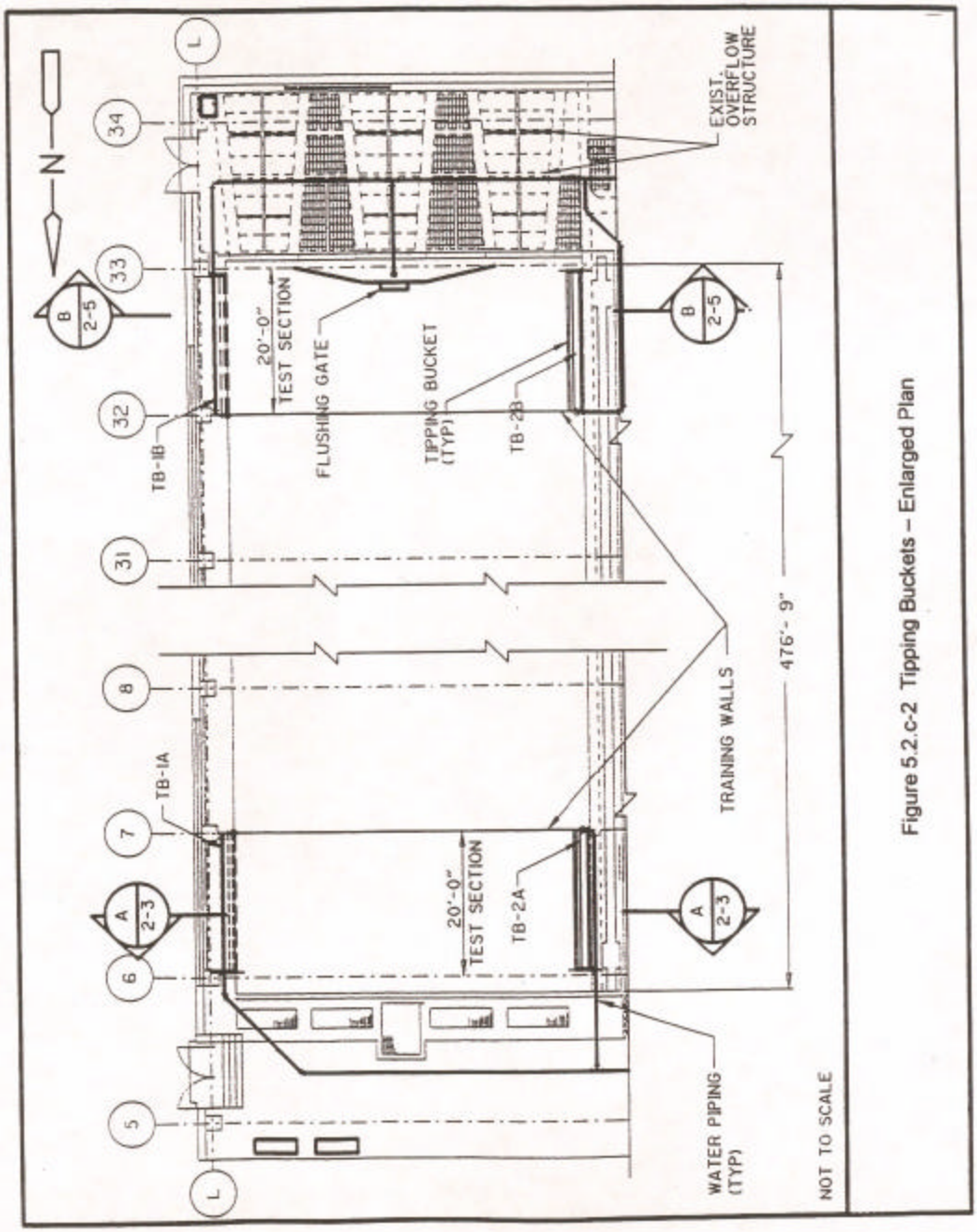


Figure 5.2.c-3.  Tipping Buckets – Section A

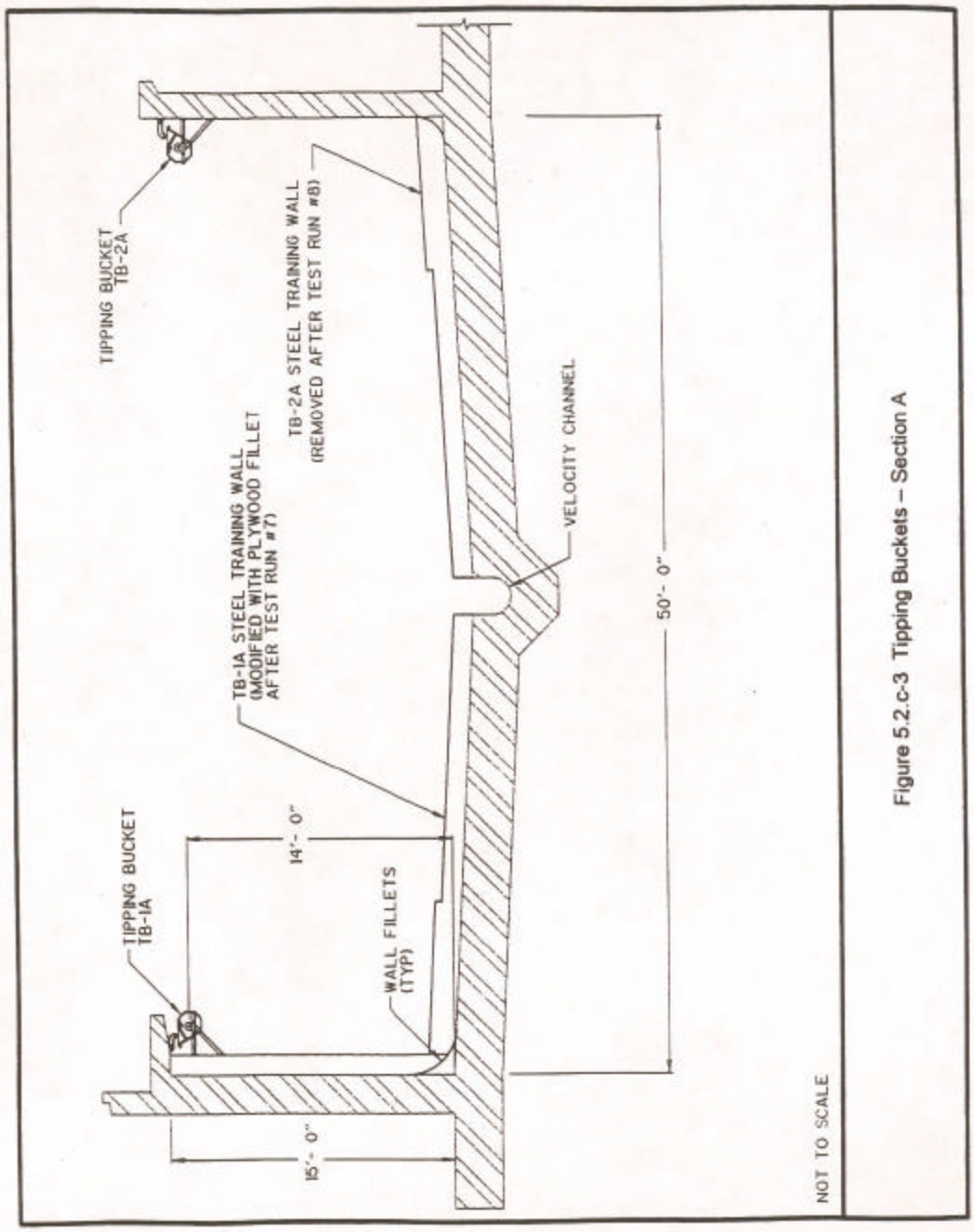


Table 5.2.c-1. Work Plan Prototype Test Matrix

**Table 5.2.c-1 Work Plan Prototype Test Matrix**

| *System* | *Test Parameter* | *Run Number* | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | *1* | *2* | *3* | *4* | *5* | *6* | *7* | *8* | *9* | *10* | *11* | *12* |
| Tipping Buckets | Volume of Water ($m^3$/bucket LM) * | 0.137 | 0.137 | 0.137 | 0.087 | 0.087 | 0.087 | 0.062 | 0.062 | 0.062 | 0.137 (2) | 0.087 (2) | 0.062 (2) |
| Flushing Gate | Volume of Water * | Full | 1/2 | 1/2 | Full | 1/2 | 1/2 | Full | 1/2 (2) | 1/2 (2) | Full (2) | 1/2 (2) | 1/2 (2) |
| Fixed Spray Header with Nozzles (Alt no 1&2) | Wash Water Flow Rate ($m^3$/min/bank): - Alternative 1 | 0.598 | 0.598 | 0.598 | 0.333 | 0.333 | 0.333 | TBD | TBD | TBD | TBD | TBD | TBD |
| | - Alternative 2 | 0.908 | 0.908 | 0.908 | 0.492 | 0.492 | 0.492 | TBD | TBD | TBD | TBD | TBD | TBD |
| | Wash Water Pressure ($kg/m^2$) * | 42,186 | 42,186 | 42,186 | 10,547 | 10,547 | 10,547 | TBD | TBD | TBD | TBD | TBD | TBD |
| Fixed Spray Header | Wash Water Flow Rate ($m^3$/m/bank): | 3.028 | 3.028 | 3.028 | 2.142 | 2.142 | 2.142 | TBD | TBD | TBD | TBD | TBD | TBD |
| | Wash Water Pressure ($kg/m^2$) * | 7,031 | 7,031 | 7,031 | 3,516 | 3,516 | 3,516 | TBD | TBD | TBD | TBD | TBD | TBD |
| Submersible Mixer | Mixer Starting Point (3) | DRE | DRE | DRE | SD | SD | SD | 1-DAY | 1-DAY | 1-DAY | TBD | TBD | TBD |

Notes:

English Equivalents are provided in Appendix B.

1. "*" - denotes test parameter to be varied during the test runs. LM denotes per linear meter
2. Staging denotes sand / sand bags / concrere blocks, etc.
3. Mixer starting points are denoted as follows:
   "DRE" - denotes during rain event,
   "SD" - denotes at the start of basin dewatering for cleaning,
   "1-DAY" - denotes 1 day prior to the start of basin dewatering for cleaning.
4. Run numbers may be rearranged based on field conditions (e.g. perform run No. 4 immediately after run No. 1, etc.)
5. Test parameter may be varied to suite initail test results.
6. "TBD" - denotes to be determined.

wall were the most difficult to clean. It was believed that this was due to a dead zone between the edge of the bucket and the basin wall pilaster. A modified plywood training wall was installed along the width of the pilaster to eliminate this dead zone as shown in Figure 5.2.c-4.

In separate subsequent tests, the training wall from bucket TB-2A and the wall fillets from buckets TB-1B and TB-2B were removed. The training wall was removed to determine what effect its removal would have on the solids located in the dead zones. The wall fillets were removed for use in the modified orifice header configuration.

*Flushing Gate.* The flushing gate system was tested for possible installation in the facility's influent barrels, and/or cross collector channels. Currently, accumulated solids in the influent barrels must be removed manually. The wave produced by a flushing gate can produce a high velocity flow of water over a relatively long distance. The basin velocity channel offered an opportunity to test the effectiveness of the flushing gate without actually entering the influent barrels. Grande, Novac & Associates, Inc. of Montreal, Canada supplied the flushing gate. A single flushing gate, 1.22 m (4 ft) wide with two 2.74 m (9 ft) long steel subwalls was installed at the overflow end of basin 1. The flushing gate was centered at the head of the existing velocity channel, while the two subwalls were located on either side of the flushing gate. The purpose of the subwalls was to accommodate the installation of the gate and to provide a storage volume of approximately 15.1 $m^3$ (4,000 gal) of flushing water. This system utilized both CSO and bay water as its source of flushing water. Cross sectional views of the flushing gate are shown in Figure 5.2.c-5.

When the desired volume of water behind the flushing gate was obtained, a remote hydraulic actuator opened the flushing gate and sent a wave of cleaning water down the length of the velocity channel. The time of travel for the leading edge of the wave produced by the gate was measured at two points in the center velocity channel, 30.5 m (100 ft) and 76.2 m (250 ft) downstream of the gate. The velocity versus time of the flow following the wave was determined by measuring the time of travel of objects floating in the channel.

There was no active means of closure for the flushing gate. After the stored wash-water volume had discharged, the flap portion of the gate would close under its own weight. The flap was required to sit flush against the gate frame in order for the hydraulic unit to lock the flap in the closed position. As a result, any debris or leaking tide water flow would interfere with the gate closure. When the gate closed, debris caught between the flap and frame would often cause leakage of the stored volume. Several times during the testing, a person was required to enter the basin in order to clear debris from the gate and allow it to be closed. On one occasion, an automobile tire became wedged in the gate opening.

Figure 5.2.c-4. Modified Plywood Training Wall Schematic

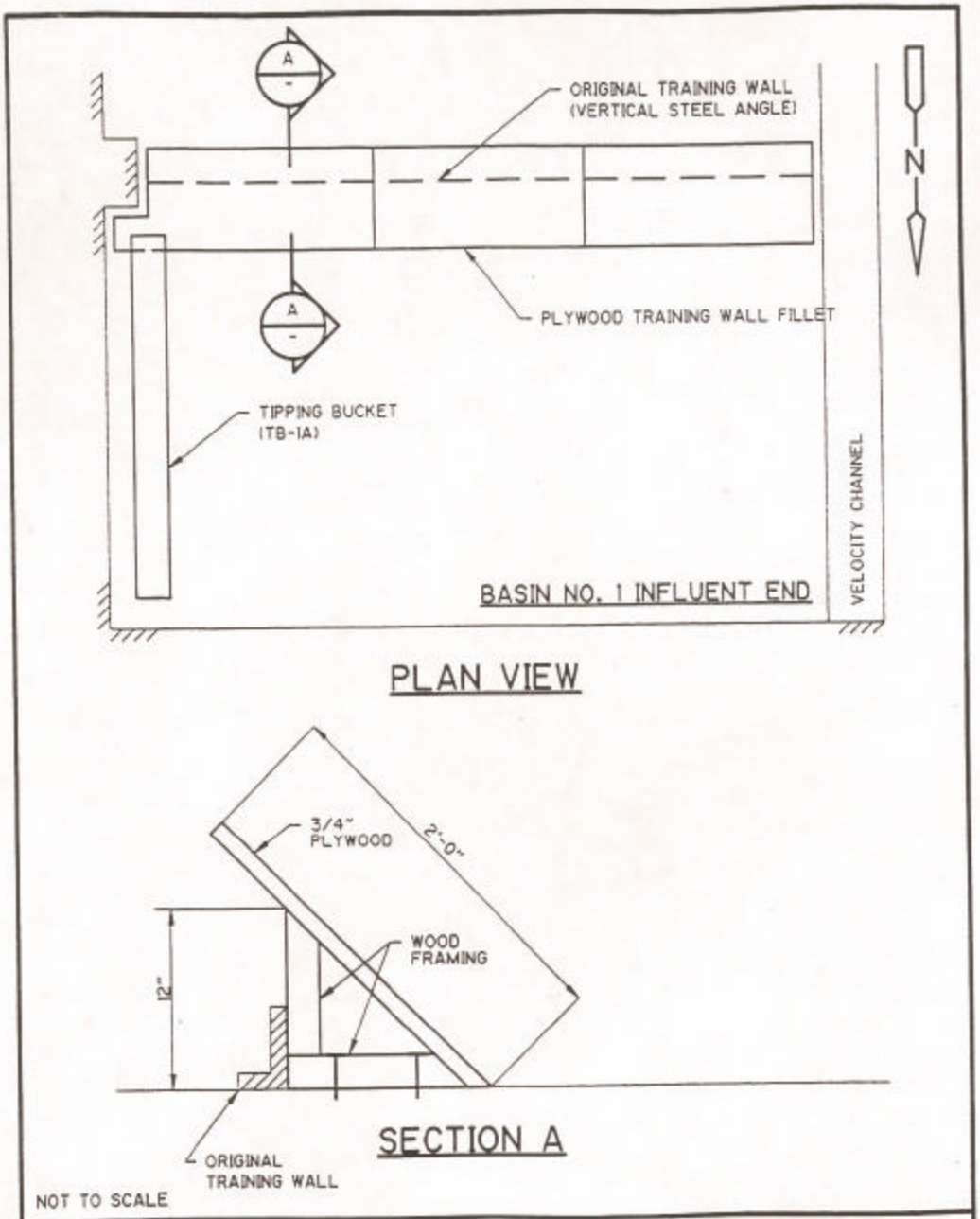
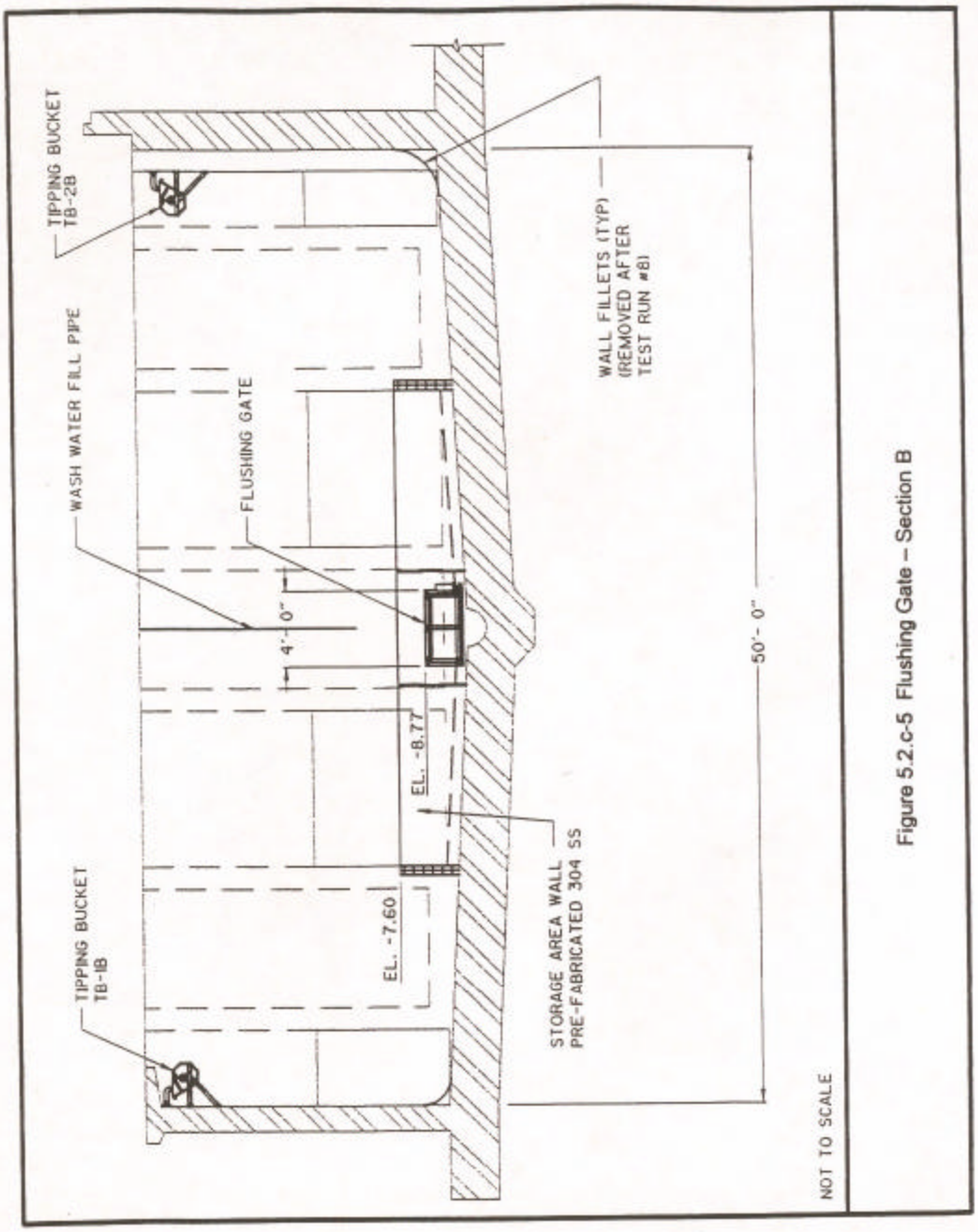



Figure 5.2.c-4 Modified Plywood Training Wall Schematic

Figure 5.2.c-5. Flushing Gate – Section B



*Fixed Header with Spray Nozzles.* Two separate spray header arrangements (Alternative-1 and Alternative-2) were installed in basin 2. Spraying Systems Inc. of Wheaton, Illinois provided spray nozzles installed on these headers. Alternative-1 used the model HH30100 nozzle that delivered a full cone spray pattern of 0.05 $m^3$/min/nozzle (12.3 gpm/nozzle) at approximately 42,190 kg/$m^2$ (60 psi). Alternative-2 used the model PSS35200 nozzle that delivered a flat spray pattern of 0.1 $m^3$/min/nozzle (24 gpm/nozzle) at approximately 42,190 kg/$m^2$ (60 psi). Both arrangements also utilized a flat-spray-pattern-nozzle-model 50200 [delivering approximately 0.1 $m^3$/min/nozzle (24 gpm/nozzle) at 42,190 kg/$m^2$ (60 psi) to clean the sidewalls. Each spray manifold was installed with 3 separate 10.2 cm (4 in.) header pipes.

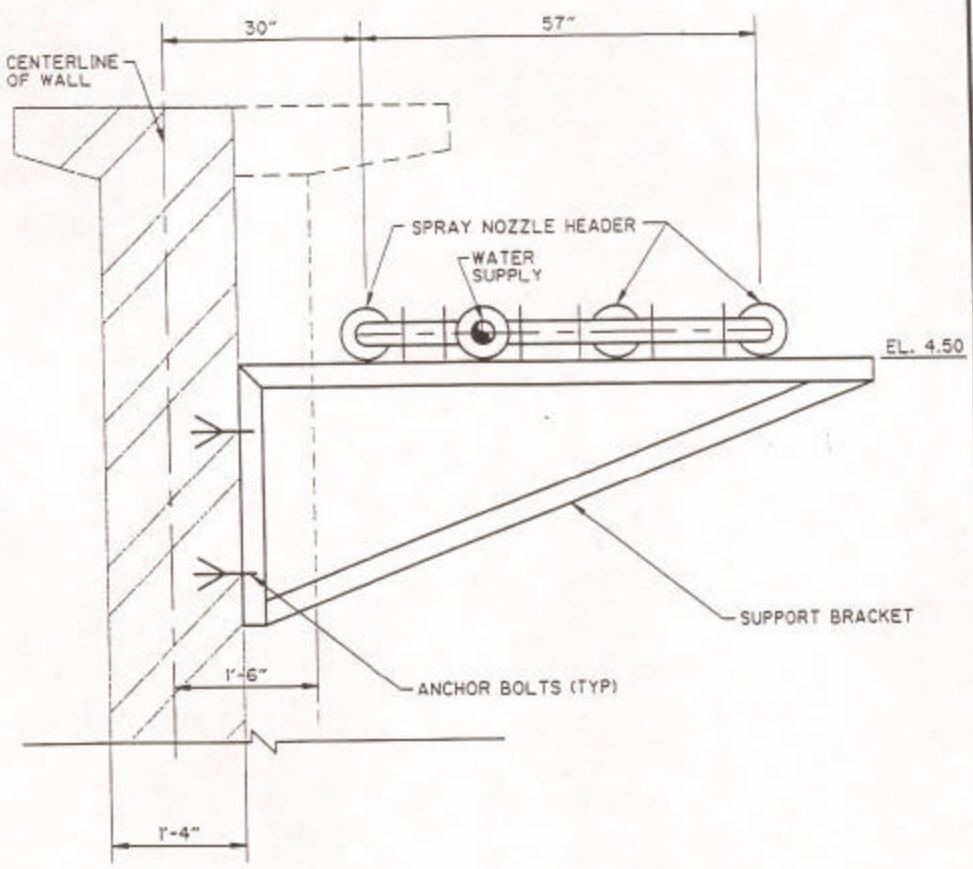
Two pipes were dedicated to clean the floor, and one pipe was dedicated for wall cleaning as shown in Figure 5.2.c-6. The spray water manifolds were located on opposite walls at both the influent and overflow ends of the basins at elevation +1.5 m (+5.00 ft). At this elevation the nozzles were approximately 4.3 m (14 ft) from the basin floor. This maintained the nozzles above the basin flood line and reduced the potential for nozzle damage and clogging.

The flow rate and pressure of the cleaning water supplied to the nozzles were held constant throughout each test run. Provisions to adjust the flow and pressure, using throttling valves located in each header, were made so that these parameters could be varied from run to run. The cleaning system was allowed to run for up to 15 min. The flow rate and operating pressure were recorded for both nozzle arrangements. Table 5.2.c-2 provides the test conditions for the fixed spray headers with nozzles.

Even at the highest attainable pressures and flows, the nozzles proved ineffective at cleaning the basin floor and were removed from service. The nozzles were also subject to clogging from debris found in the cleaning water supplied from the spray water channel.

*Fixed Spray Header with Orifices.* Initially, four headers were installed in two arrangements in the influent and effluent ends of basin 3. Two orifice headers were located on opposite walls at the influent end of the basin, and two orifice headers were located on opposite walls at the overflow end of the basin. Each orifice header was 6.1 m (20 ft) long and equipped with 1.6 cm (5/8 in.) orifices spaced on approximately 30.5 cm (1 ft) centers. The spacing and size of the holes were selected to provide a water flow rate of 0.5 $m^3$/min/m (40 gpm/ft) at a discharge pressure of 7,030 kg/$m^2$ (10 psi). One pair of headers was installed at a centerline elevation of +1.5 m (+5.00 ft) (influent header 1A and overflow header 2B) with the orifices aimed directly at the sidewall. These headers were designed to send cleaning water down the wall in a continuous sheet, thereby washing the basin floor and carrying solids to the velocity channel. The second pair of headers (influent header 2A and overflow header 2B) was installed at a centerline of elevation -8.50 with the header aimed directly at bottom of the wall.

Figure 5.2.c-6. Spray Nozzle Header – Section




NOT TO SCALE

Figure 5.2.c-6 Spray Nozzle Header – Section

Table 5.2.c-2. Summary of Prototype Testing Operation

**Table 5.2.c-2 Summary of Prototype Testing Operation**

| *System* | *Test Run Number* | *Test Conditions* | | | *Comments* |
|---|---|---|---|---|---|
| Tipping Buckets | 1 - 16 | Volume of Water | | | Same volume of water per bucket tip was used for all test runs. Multiple tips of full buckets were required to clean the test area. Plywood training walls were installed on TB-1A area after test run number 7. The metal training wall was removed from TB-2A area after test run number 8. Metal wall fillets were removed from TB-1B and TB-2B areas after test run number 8 |
| | | ($m^3$/m of bucket) | | 0.137 | |
| | | (gal/foot of bucket) | | 11 | |
| Flushing Gate | 1 - 6 | Volume of Water | | NA | Not operational due to various mechanical problems. |
| | 7 - 15 | Volume of Water | | | Proposed staging of sand / sand bags / concrete blocks not performed. Wave velocity measured during test runs number 11 and 12. |
| | | | ($m^3$) | 15.14 | |
| | | | (gal) | 4,000 | |
| Nozzle Headers Alt 1 - Full Cone (Banks 1A/1B) | 1 - 3 | Flow Rate [1] | | 0.567-0.757 | Large fluctuations in flow. |
| | | | (gpm) | 150-200 | |
| | | Pressure | ($kg/m^2$) | 42,186 | |
| | | | (psi) | 60 | |
| | 4 - 7 | Flow Rate [1] | (gpm) | 175-200 | 60 Adjustments made to the blow off valve after test run number 3 to alleviate large fluctuations in flow. |
| | | Pressure | (psi) | | |
| | 8 - 16 | Flow Rate | | NA | NA No further testing performed after test run number 7. Spray headers were removed as they did not provide effective cleaning. |
| | | Pressure | | | |
| Nozzle Headers Alt 2 - Flat Spray (Banks 2A/2B) | 1-7 | Flow Rate ($m^3$/min) | | 0.852 | |
| | | | (gpm) | 225 | |
| | | Pressure | ($kg/m^2$) | 42,186 | |
| | | | (psi) | 60 | |
| | 8 - 16 | Flow Rate | | NA | NA No further testing performed after test run number 7. Spray headers were removed as they did not provide effective cleaning. |
| | | Pressure | | | |
| Orifice Headers Original Configuration (Headers 1/2) | 1 - 5 | ($m^3$/min) | | 3.028 | Original header configuration only proved to be partially effective. The lower header was rotated after test run number 5. Effluent headers were removed after test run number 5. |
| | | | (gpm) | 800 | |
| | | Pressure | ($kg/m^2$) | 9140.3 | |
| | | | (psi) | 13 | |
| | 6 | Flow Rate | | NA | NA System was operated for demonstration only because the basin was previously cleaned. Header 1 was rotated and wall fillets installed after test run number 6. Header 2B was installed after test run number 6. |
| | | Pressure | | | |
| Orifice Headers Modified Configuration (Header 1) | 7-16 | ($m^3$/min) | | 1.325-3.028 | Modified Header 1 configuration only proved to be slightly more effective than the original configuration. |
| | | | (gpm) | 350-800 | |
| | | Pressure[2] | ($kg/m^2$) | 3,516-17,578 | |
| | | | (psi) | 5-25 | |
| Orifice Headers Modified Configuration (Header 2A/2B/3) | 7 | Flow Rate[3] | | 3.255/NA/NA | Header 2B and 3 not yet operational. |
| | | | (gpm) | 860/NA/NA | |
| | | Pressure[4] | ($kg/m^2$) | 11,953/NA/NA | |
| | | | (psi) | 17/NA/NA | |
| | 8 | Flow Rate[3] | | | Header 3 not yet operational. Header 2A and 2B operated together. Configuration proved more effective than the original configuration. Area between Header 2A and basin side wall not cleaned. |
| | | | (gpm) | 825/600/NA | |
| | | Pressure[4] | ($kg/m^2$) | | |
| | | | (psi) | 8/8/NA | |
| Orifice Headers Modified Configuration (Header 2A/2B/3) | 9 | ($m^3$/min) | | 2.271/2.271/1.325 | All three headers operational. Sequencing: 2B, then 2A and 2B, then 2A and finally 2A and 3. Three header configuration proved effective at cleaning the test area. |
| | | | (gpm) | 600/600/350 | |
| | | Pressure[4] | ($kg/m^2$) | 5,625/5,625/7,031 | |
| | | | (psi) | 8/8/10 | |

Table 5.2.c-2 page 2

**Table 5.2.c-2 Continued**

| *System* | *Test Run Number* | *Test Conditions* | | | *Comments* |
|---|---|---|---|---|---|
| | 10 | ($m^3$/min) | | 2.650/1.703/1.514 | All three headers operational. Sequencing: 2A and 3, then 2B. Three header configuration proved effective at cleaning the test area. |
| | | | (gpm) | 700/450/400 | |
| | | Pressure[4] | ($kg/m^2$) | 10,547/10,547/140 | |
| | | | (psi) | 62 | |
| | | | | 15/15/20 | |
| | 11 | ($m^3$/min) | | 2.082/1.325/0.757 | All three headers operational. Sequencing: 2A and 3, then 2B. Three header configuration proved effective at cleaning the test area. |
| | | | (gpm) | 550/350/200 | |
| | | Pressure[4] | | ND/ND/ND | |
| | 12 | ($m^3$/min) | | 1.703/1.798/0.757 | All three headers operational. Sequencing: 2A and 3, then 2B. Three header configuration proved effective at cleaning the test area. |
| | | | (gpm) | 450/475/200 | |
| | | Pressure[4] | ($kg/m^2$) | 0/0/2,109 | |
| | | | (psi) | 0/0/3 | |
| | 13 | ($m^3$/min) | | 2.176/1.893/0.757 | All three headers operational. Sequencing: 2A and 3, then 2B and 3, then 2A, finally 2A and 2B. Three header configuration proved effective at cleaning the test area. |
| | | | (gpm) | 575/500/200 | |
| | | Pressure[4] | ($kg/m^2$) | 0/0/3,515 | |
| | | | (psi) | 0/0/5 | |
| | 14 | Flow Rate [3] | (gpm) | 650/650/200 | All three headers operational. Sequencing: 3, then 2A and 2B. Three header configuration proved effective at cleaning the test area |
| | | Pressure [4] | (psi) | 10/10/ND | |
| | 14 2.460 | ($m^3$/min) | | 2.460/2.460/0.757 | All three headers operational. Sequencing: 3, then 2A and 2B. Three header configuration proved effective at cleaning the test area |
| | | | (gpm) | 650/650/200 | |
| | | Pressure[4] | ($kg/m^2$) | 7,031/7,031/ND | |
| | | | (psi) | 10/10/ND | |
| | 15 2.460 | ($m^3$/min) | | 2.176/1.893/0.757 | All three headers (2A, 2B, and 3) operated simultaneously. Three header configuration proved effective at cleaning the test area. |
| | | | (gpm) | 525/550/200 | |
| | | Pressure[4] | ($kg/m^2$) | 3,515/3,515/3,515 | |
| | | | (psi) | 5/5/5 | |
| | 16 | ($m^3$/min) | | 2.176/2.176/0.757 | All three headers (2A, 2B, and 3) operated simultaneously. Three header configuration proved effective at cleaning the test area. |
| | | | (gpm) | 525/525/200 | |
| | | Pressure[4] | ($kg/m^2$) | 3,515/3,515/3,515 | |
| | | | (psi) | 5/5/5 | |
| Submersible Mixer | 1-2 | NA | | | Not operational due to mechanical problems. |
| | 3 | NA | | | Test section had been cleaned by NYCDEP Plant Staff. |
| | 4-7 | The mixer would start at the beginning of an inflow and end as the basins were dewatered. | | | The mixer provided marginal resuspension of solids, with a small area of impact. |
| | 8-16 | NA | | | No further testing performed after test run number 7. The mixer was removed as it did not provide effective resuspension of solids. Later inspection of the mixer indicated that the discharge manifold was clogged. |

Notes:

1. Reported flow rates denote the range of flows for the indicated system.
2. Reported pressures denote the range of pressures for the indicated system.
3. Reported flow rates denote the average flow for the Orifice Headers (2A/2B/3).
4. Reported pressures denote the average pressure for the Orifice Headers (2A/2B/3)
5. NA - denotes not applicable because the system was not in operation.
6. ND - denotes the parameter was not recorded during testing operations.

This system was also designed to provide a sheet flow of cleaning water along the floor. Since no nozzles were utilized under this alternative, there was less concern for clogging while the headers were submerged. Cross sectional views of this original orifice header configuration are shown in Figure 5.2.c-7.

Training walls stretching the width of the basin from the velocity channel to the basin sidewalls were installed at the outside end of the basin test section. Similar to the tipping buckets, the purpose of the training walls was to prevent dispersion of the cleaning water and maintain the flow within the basin test section.

After the initial six test runs, the original orifice header configuration was determined to be ineffective at cleaning. Subsequently, modifications to the header configuration were made including the following:

- Removal of the overflow end orifice headers (for reuse in the modified configuration at the basin influent)
- Rotation of the influent orifice header 1A (elevation +1.5 m [+ 5.00 ft]) to orient the water jets nearly tangential to the sidewall.
- Rotation of the influent orifice header 2A to orient the spray downward and towards the velocity channel at a spray angle of approximately 30°s with the floor.
- Installation of two new influent orifice headers (2B and 3) on the eastern side of the test section. Header 2B was installed on the basin floor at elevation –2.6 m [-8.5 ft] and header 3 was installed on the sidewall at elevation +1.5 m (+5.00 ft). The spray on header 2B was oriented similar to header 2A while header 3 was oriented similar to header 1A.
- Installation of wall fillets at the wall-floor joints beneath influent headers 1A and 3.

Figure 5.2.c-8 provides a section showing the modified header configuration.

Flow rate, operating time and pressure was recorded for each orifice header. Table 5.2.c-2 provides the operating conditions and performance for this system.

Few operational and mechanical problems were encountered while operating this system. Occasionally rags and other debris became entangled around the supports of the orifice headers; however, this debris did not effect the cleaning operation.

*Submersible Mixer.* A single submersible 18.6 kW (25 HP) jet mixer was supplied by Davis EW of Thomasville, Georgia (now a division of US Filter). The jet mixer was installed at the influent end of Basin No. 2 using steel guide rails and lifting cables similar to those typically associated with submersible pumping equipment. The mixer was provided with an integral diffuser and air supply piping system to achieve a roll mixing effect within the basin.

Figure 5.2.c-7.  Original Orifice Header Configuration – Section A

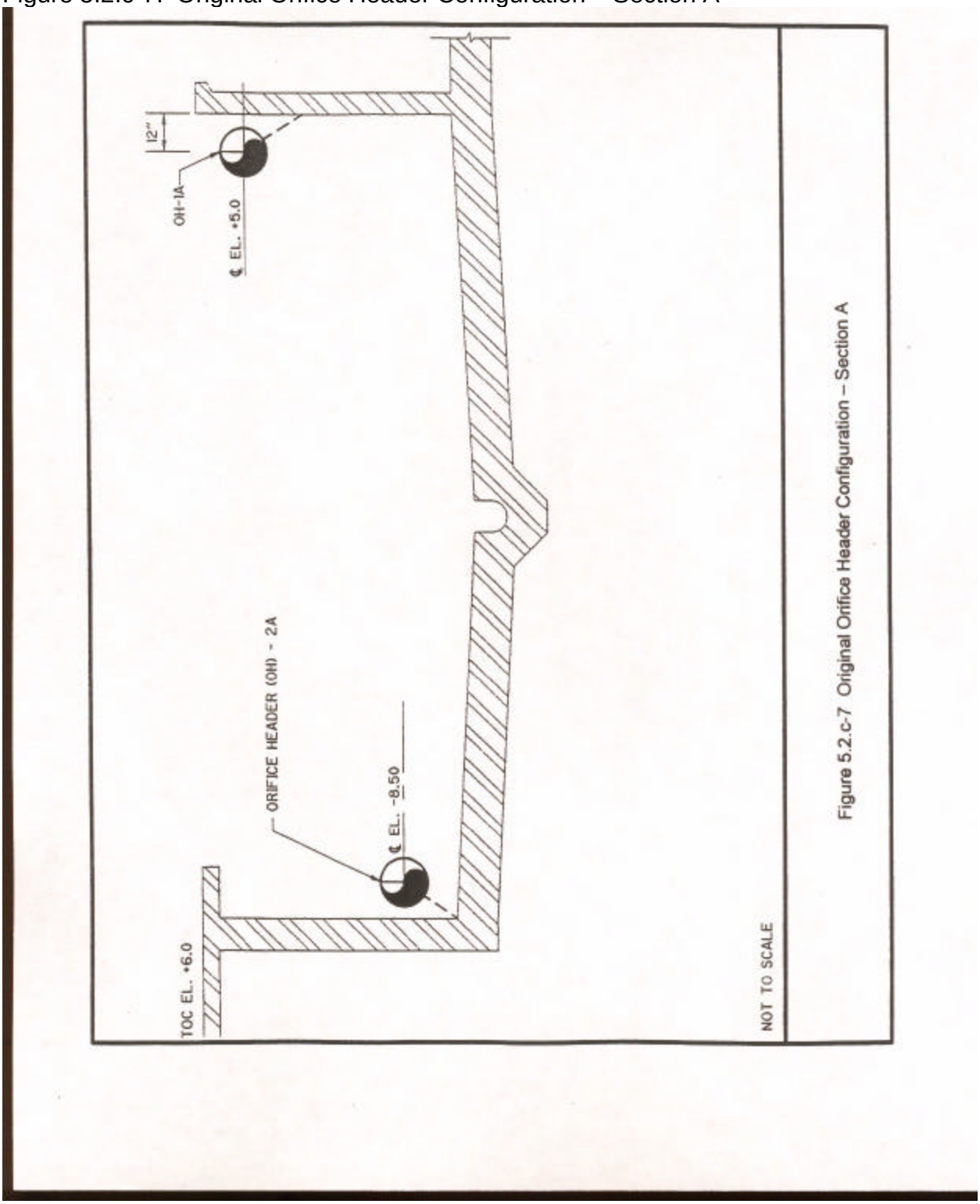
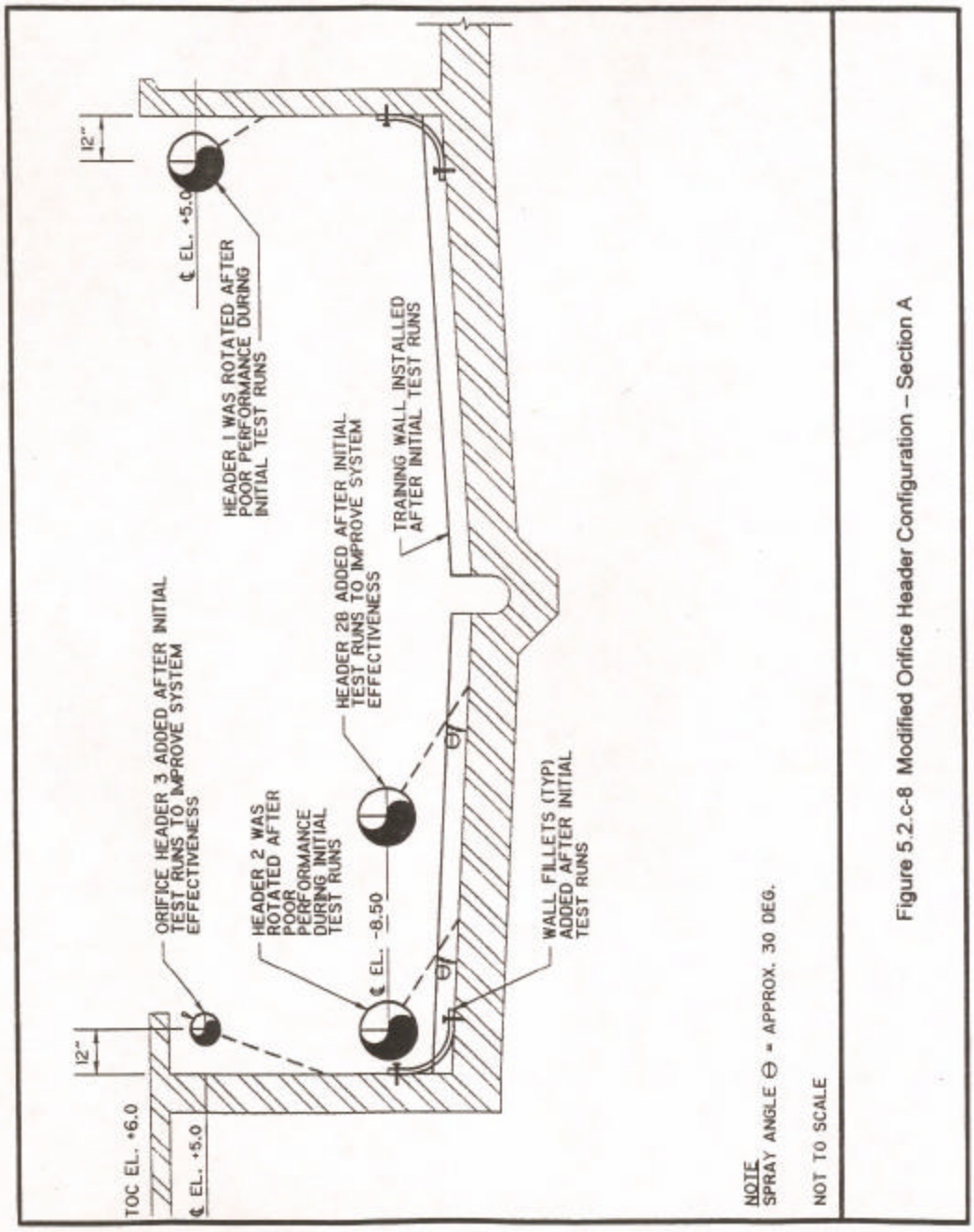


Figure 5.2.c-8. Modified Orifice Header Configuration – Section A



The jet-mixer was placed into continuous service after test run no. 3. The mixer was controlled using automatic level control floats to turn on while the basins were filling during a CSO event and off when the basins were almost completely drained. The mixer remained in operation sometimes for more than one day until the basin was drained.

The purpose of the mixer was to provide re-suspension of the solids, such that solids would be removed with the wastewater during the de-watering operation, minimizing solids deposition within the basins. During testing the mixer proved ineffective at re-suspending the solids in the test area and reducing solids deposition. The mixer was removed from service after test run no. 7. Following removal of the jet mixer from service, it was found that the diffuser and discharge piping of the mixer were almost completely clogged with leaves, rags and other debris.

Prototype Test Results

*Solids Composition and Distribution. A*s would be expected for a CSO facility, the composition and distribution of residual solids in the detention basins varied significantly between events. The measured depth of solids ranged from a trace coating (less than 1.5 cm) up to 45.7 cm (1.50 ft) at individual piles. The solids ranged in consistency from a thin watery mixture, to a viscous “clay like” consistency, to heavy grit. Test events with the greatest depth of solids and heavier grit generally followed rainstorms of high intensity. Test events with fewer solids and solids with a thinner consistency generally followed rainstorms with low intensity or low total precipitation.

In general, the influent ends of the basins contained the heavier solids, with the overflow ends characterized primarily by floatables, leaves, and a thin coating of solids. The solids depth was greater at the influent end than at the overflow end. Dead zones at the influent headwall of each basin contributed to the formation of piles or long mounds of solids in the influent test sections. The largest piles were observed on the basin side of the influent headwall columns and along the sidewall. Solids appeared to build up behind these obstructions, as scouring of solids during CSO inflow was prevented. These and other piles of solids often posed the most resistance to cleaning. Within basins no. 1 and 3, a ridge of deeper solids was observed several times along the western wall of the basin influent. At the influent end of basin No. 1, the solids depth was generally greater on the western side of the test section than the eastern side. This likely contributed to the formation of piles along the western sidewall. The difference in solids depth between the two sides of the basin No. 1 influent may have been due to a partial blockage of the influent barrel feeding the eastern side of this basin, resulting in unequal flow distribution.

Based on subjective observations, it appeared that the consistency of the residual solids was generally thicker and more cohesive when more days had transpired between the inflow event and the cleaning test as compared to when cleaning was performed immediately following an inflow. The nature of the solids consistency affected cleaning system performance, with the thicker/ cohesive solids being more resistant to cleaning.

*Benchmark of Existing Cleaning Operations.* As discussed previously, the existing traveling bridge cleaning system operations were observed to document general operations, cleaning time, and water consumption. The cleaning time, water consumption and estimated electrical cost for this system are presented in Table 5.2.c-3.

The bridges provide very effective cleaning of the basin walls and floor. Generally, several complete passes along the basin length were required to clean each basin. With the bridge mounted water cannon, solids or objects which might have been unaffected by the spray nozzles were readily moved into the velocity channel and cleaned from the basin floor. The bridges readily achieved complete basin cleaning.

The primary advantage of the traveling bridge system is that it is very effective at cleaning the basins. The degree of cleaning is only limited by the amount of time an operator expends in cleaning. Disadvantages observed with the bridge system included the following:

- Cleaning with the bridges was labor intensive, requiring staffing of each bridge during operation
- Because there is no separation from the basin airspace, the operator had the potential to be exposed to $H_2S$ levels exceeding standards
- The bridges appear to require frequent maintenance. Often during the test runs, one of the six bridges was observed to be temporarily out of service, due to problems with the bridge controls or with the electrical bus bar supplying each bridge

*Tipping Buckets.* The test results for the tipping bucket prototype cleaning system are presented in Table 5.2.c-4. This table presents tabulated results for what are considered representative solids conditions. For several test events, the preceding inflow did not result in any significant deposition of solids, or the test section may have been cleaned by tidal inflow or by plant personnel. The data for such events was not considered representative and is not included in Table 5.2.c-4. Data for the influent and overflow test sections are presented separately, with averages for the number of bucket tips per test section and projected water usage per basin calculated over each test run, each test section (influent, overflow), and over the entire basin. The average over the basin length assumes a linear distribution of solids (and associated cleaning water consumption) between the solids depth measured at the influent test section, and that measured at the overflow test section.

The tipping buckets provided very effective cleaning of the sidewalls and the basin floor. The high velocity wave produced by each bucket tip easily cleaned solids and debris from the basin floor and conveyed them to the center velocity channel. With greater depths of solids or for heavier solids or grit, more tips were required as compared to test conditions with fewer solids or with looser solids.

Table 5.2.c-3. Operating Data for Existing Traveling Bridge

**Table 5.2.c-3 Operating Data for Existing Traveling Bridge Cleaning System**

| *Test Run Number* | *Basin Number* | *Number of Passes* | *Cleaning Time (min)* | *Cleaning Water Consumption (1)* | | *Estimated Electrical Cost (2)* |
|---|---|---|---|---|---|---|
| | | | | *($m^3$)* | *(gal)* | *($/yr)* |
| 5 | 1 | ND | 65 | 246 | 65,000 | 594.88 |
| | 2 | ND | 65 | 246 | 65,000 | 594.88 |
| | 3 | ND | 45 | 170 | 45,000 | 411.84 |
| 7 | 1 | ND | 79 | 299 | 79,000 | 723.01 |
| | 2 | 2 | 79 | 299 | 79,000 | 723.01 |
| | 3 | ND | 105 | 397 | 105,000 | 960.96 |
| 8 | 1 | ND | 75 | 284 | 75,000 | 686.40 |
| | 2 | ND | 45 | 170 | 45,000 | 411.84 |
| | 3 | ND | 88 | 333 | 88,000 | 805.38 |
| | 4 | ND | 86 | 326 | 86,000 | 787.07 |
| | 5 | ND | 25 | 95 | 25,000 | 228.80 |
| | 6 | ND | 30 | 114 | 30,000 | 274.56 |
| 10 | 1 | 2 | 50 | 189 | 50,000 | 457.60 |
| | 2 | 5 | 95 | 360 | 95,000 | 869.44 |
| | 3 | 6 | 203 | 768 | 203,000 | 1,857.86 |
| | 4 | 2 | 90 | 341 | 90,000 | 823.68 |
| | 5 | 4 | 70 | 265 | 70,000 | 640.64 |
| | 6 | 2 | 40 | 151 | 40,000 | 366.08 |
| 12 | 1 | 3 | 49 | 185 | 49,000 | 448.45 |
| | 2 | 4 | 69 | 261 | 69,000 | 631.49 |
| | 3 | 4 | 75 | 284 | 75,000 | 686.40 |
| | 4 | 6 | 80 | 303 | 80,000 | 732.16 |
| | 5 | 2 | 82 | 310 | 82,000 | 750.46 |
| | 6 | 2 | 35 | 132 | 35,000 | 320.32 |
| **Average:** | | | **72** | **272** | **71,875** | **657.80** |

Notes

1. Cleaning water consumption based on a rated pump capacity of 3.785 $m^3$/m (1,000 gpm).
2. Electrical cost based 2 cleanings per week using a 29.8 kW (40 HP) pump and a unit electrical cost of $0.16/kW-hr.
3. ND - denotes the parameter was not recorded during testing operations.

Table 5.2.c-4. Tipping Bucket Cleaning System Test Results at Representative Solids Conditions

**Table 5.2.c-4 Tipping Bucket Cleaning System Test Results at Representative Solids Conditions [1]**

| *Test Run Number* | *Number of Tips Required [2]* | | *Average for Both Buckets* | | *Solids Quantity / Quality* | | *Comments* |
|---|---|---|---|---|---|---|---|
| | *Bucket 1* | *Bucket 2* | *Number of Tips [4]* | *Projected Water Usage per Basin ($m^3$)* | *Depth Range, m (Min/Max)* | *Average Depth (m)* | |
| **Basin Influent (Buckets 1A + 2A)** | | | | | | | |
| 2 | 10 | 10 | 10 | 416.35 | 0.015 / .305<br>Mounds up to 1.0 ft | 0.107 | Both buckets 90% clean after 7 tips. |
| 4 | 2 | 15 | 9 | 374.72 | 0.031 / 0.244<br>Mounds along the training wall | 0.076 | TB-2A 90% clean after 6 tips. Additional tips required to move mounds. TB-1A 90% clean after 1 tip. |
| 5 | 7 | 5 | 6 | 249.81 | 0.015 / 0.152<br>Mounds near the west side | 0.079 | Pile along training wall and velocity channel not removed |
| 6 | 17 | 8 | 13 | 541.26 | 0.031 / 0.122<br>Soupy & gritty consistency | 0.070 | TB-1A 90% clean after 9 tips, solids remain along the training wall. |
| 7 | 6 | 6 | 6 | 249.81 | 0.031 / 0.183<br>Gritty ridge | 0.076 | Both buckets 90% clean after 4 tips. |
| 10 | 8 | 14 | 11 | 457.99 | 0.031 / 0.229<br>Soupy & gritty consistency | 0.061 | Both buckets 90% clean after 6 tips. TB-2A 99% clean after 14 tips. |
| 13 | 2 | 3 | 3 | 124.91 | <0.015 / 0.046<br>Plastic / Clayey texture, large vinyl bag | 0.030 | Small amount of solids remain along training wall and inlet. TB-2A 90% clean after 2 tips. |
| 14 | 8 | 12 | 10 | 416.35 | <0.015 / 0.076<br>Mucky Solids mostly leaves | 0.046 | Presence of leaves caused solids to be heavier and harder to move. TB-1A 90% clean after 3 tips, TB-2A 90% after 7 tips. |
| 16 | NR | 30 | <30 | > 1,249.05 | <0.015 / 0.213<br>Thick clayey solids with leaves | 0.116 | Pile of matted leaves and solids near sidewall proved difficult to clean, pile moved gradually with each tip. 95% clean after 30 tips. |
| **Average [4]** | | | **11** | **457.99** | **<0.015 / 0.305** | **0.073** | |
| **Basin Overflow (Buckets 1B+2B)** | | | | | | | |
| 2 | 2 | 2 | 2 | 83.27 | <0.015 / 0.015<br>Light uniform coating | <0.015 | Test area scoured by leakage from high tide condition. Both buckets 95% clean after 1 tip. |
| 4 | 3 | 3 | 3 | 124.91 | 0.015 / 0.031<br>Partially clean, piles near training wall | 0.021 | Both buckets 90% clean after 2 tips. |
| 7 | 1 | 1 | 1 | 41.64 | <0.015 / 0.031<br>Mostly floatables | 0.015 | TB-1B 75% cleaned by the leakage from the tide before the first tip. |
| 9 | 2 | 2 | 2 | 83.27 | 0.015 / 0.122<br>Leaves and floatables | 0.021 | Both buckets 95% clean after 1 tip, some floatables remain. |
| 10 | 12 | 6 | 9 | 374.72 | <0.015 / 0.122<br>Large pile of leaves and floatables | 0.043 | TB-2B 90% clean after 3 tips. TB-1B slowly moves the large pile of floatables and leaves. |
| 13 | 2 | 1 | 2 | 83.27 | <0.015 / 0.031<br>Light film with floatables | 0.018 | 99% clean after 1 tip. |
| 14 | NR | 2 | 2 | 83.27 | <0.015 / 0.091<br>Entirely leaves and floatables | 0.046 | TB-1B test area completely cleaned by leakage from the tide. TB-2B 95% clean after 1 tip. |
| **Average [4]** | | | **3** | **124.91** | **<0.015 / 0.122** | **0.027** | |
| **Average over Basin Length [5]** | | | **7** | **291.45** | **<0.015/0.305** | **0.050** | |

Notes:

English Equivalents are provided in Appendix B.

1. Data for runs which had trace or no solids, or where basin had been previously cleaned are not representative and are not included in the analysis.
2. Number of bucket tips to achieve 99 percent clean condition.
3. NR - denotes data not representative.
4. Average number of tips is rounded up to nearest whole tip.
5. Results for influent and overflow test section data averaged to determine overall water consumption over the entire basin length.

Averaged over the basin length, 7 tips are estimated to be required per bucket, resulting in a projected water usage of 291.4 $m^3$ (77,000 gal) per basin. The range in projected water usage is from a low of 41.6 $m^3$ (11,000 gal) per basin, to greater than 1249 $m^3$ (330,000 gal) per basin. The highest projected water consumption was required to clean a pile of matted leaves mixed with solids in the influent test section during test run no. 16. In this case, the leaves seemed to bind the solids together, resulting in a single large mass that was difficult to clean. This pile was slowly eroded with each tip. After 30 tips the test was terminated while the pile still remained. Difficulty cleaning leaves was also observed in the overflow test section of run no. 10. There was a pile of matted leaves in the test section for bucket TB-1B. With each tip, the entire mass of leaves was lifted, carried a short distance, and redeposited. Ultimately, 12 tips were required to wash this pile into the center velocity channel.

The solids that had collected along the training wall proved difficult to clean and were often the limiting factor in determining when the basin was 99% clean. Elsewhere on the test section floor, the wave of wash-water was able to mix and resuspend the solids in the wash-water. However, along the training wall the solids would generally erode slowly with each tip. In the prototype installation, the wall-floor fillet on the eastern tank wall was installed flush against the sidewall in the recess between the column pilasters. As a result, over the pilaster width, there was no smooth transition for the wave between the wall and floor, and the wave would lose energy in this dead zone when it impacted the floor. The modified plywood training-wall improved the situation somewhat by eliminating the dead zone created by the sidewall pilaster. The plywood training-wall worked by preventing the deposition of solids in the area blocked by the pilaster. Solids along the modified training wall still eroded slowly with each tip. However, the amount of solids collected along the training wall was less than for the original vertical training-wall installation.

When the training-wall was removed from the test area for bucket TB-2A, the cleaning effectiveness was the same if not more effective than with the training-wall. Without the training-wall the wave could penetrate laterally into the solids, resulting in more effective resuspension of the solids at this location.

Operationally, the buckets were very simple to operate, requiring only that the water be turned on; however, difficulty in tipping first one bucket (TB-2B) then another bucket (TB-1B) was observed. Due to possible misalignment of TB-2B during installation, counterweights were added to the bucket so that it would tip automatically during filling. This worked for a time. However, later in the test program the bucket had to again be tipped manually. Periodically, the same problem was observed with bucket TB-1B.

*Flushing Gate.* The flushing gate produced a high velocity flow in the center velocity channel that extended at least 76.2 m (250 ft) down the length of the tank. The velocity of the leading edge of the wave is a function of the slope of the channel and the steep hydraulic grade created by the wave surge. The center velocity channel slopes approximately 0.63 percent towards the cross channel. The flow velocity was

calculated using Mannings Equation ranged between 1.3—1.9 m/s (4.4—6.2 ft/s) for uniform flow at typical wash-water depths of 0.15 and 0.30 m (6 and 12 in). During operation of the flushing gate, the leading wave velocity was measured at 3.4—4.3 m/s. (11-14 ft/s). The water velocity in the channel later dissipated to 1.2 m/s (4 ft/s) after approximately 40 s and 100 s at the 30.5 m and 76.2 m (100 ft and 250 ft) measuring stations, respectively. Although there were generally no significant solids within the velocity channel, this high initial flow rate would be expected to scour most if not all solids and objects from the channel.

Based on the size and configuration of the gate supplied, which was primarily designed to flush the velocity channel, the lateral extent of the high velocity flow was limited to the velocity channel. In the immediate vicinity of the gate, the high velocity flow extended laterally approximately 61—152.4 cm (2—5 ft) from the center channel. However, the high rate of discharge produced backwater within the basin effluent test area, which did not result in any effective cleaning of the test area outside of the velocity channel.

As indicated previously, significant operating problems were encountered with the flushing gate. These problems were primarily related to floatables and debris that caused problems with the closure and sealing of the gate.

*Fixed Header with Spray Nozzles.* The test data for the spray nozzle prototype system are presented in Table 5.2.c-5. The spray nozzles did not provide effective cleaning. The nozzles only cleaned the immediate area impacted by the central portion of the spray jet. Outside the central portion or cone of the spray jet, the nozzles produced a fine mist, which was ineffective at cleaning. Away from this immediate area the wash-water would form channels in the deposited solids and would not provide any further cleaning. Overall, approximately only 25% of the floor test-area was cleaned by either nozzle configuration. This degree of cleaning was achieved within the first minute of operation. The system was generally allowed to operate for 15 min. However, the added operating time did not result in any further cleaning. Since the spray nozzles did not provide complete cleaning, the cleaning times and volumes presented in Table 5.2.c-5 have been qualified to indicate that greater cleaning time and volumes would be required. The flat spray nozzles directed at the sidewall provided an adequate job of cleaning the wall only.

Based upon field observations, it appears that in order to provide adequate cleaning with this spray nozzle configuration, the headers would need to be swept back and forth across the floor during operation, similar to the existing traveling bridge operation.

*Fixed Header with Orifices.* The test data for the original and modified orifice header configurations are presented in Tables 5.2.c-6 and 5.2.c-7, respectively. The original header configurations provided only partial cleaning of the floor test area. An area extending approximately 1.5—3 m (5—10 ft) from the sidewall was completely cleaned within the first minute of operation. These solids were usually cleaned with the initial surge of wash-water.

Table 5.2.c-5. Spray Nozzle Cleaning System Test Results (2 pg.)

**Table 5.2.c-5 Spray Nozzle Cleaning System Test Results**

| Test Run Number | Test Results: Header Test Conditions: Alt 1 Flow Rate ($m^3$/min) | Alt 1 Pressure ($kg/m^2$) | Alt 2 Flow Rate ($m^3$/min) | Alt 2 Pressure ($kg/m^2$) | Required Cleaning Duration [4] (MM:SS) | Water Consumption (Alt 1 / Alt2) ($m^3$) | Solids Quantity / Quality: Depth Range, m (Min/Max) | Average Depth (m) | Comments |
|---|---|---|---|---|---|---|---|---|---|
| **Basin Overflow** | | | | | | | | | |
| 1 | 0.61 [2] | 42,186 [2] | 0.95 [2] | 42,186 [2] | NR<br>0.01524 | NR / NR | <0.015 / 0.031<br>Mostly Floatables | 0.015 | Pilot run for demonstration purposes only. Cleaning not effective, only cleans immediate area impacted by the spray pattern. |
| 2 | 0.66 | 40,780 | 0.85 | 38,671 | >15:00 | >9.94 / >12.77 | 0.015 / 0.062<br>Light uniform coating with floatables. | 0.034 | Pilot operations interrupted by the plant staff. Cleaning not effective, only cleans immediate area impacted by the spray pattern. |
| 3 | 0.66 | 40,780 | 0.85 | 35,155 | >15:00 | >9.94 / >12.77 | 0.015 / 0.031<br>Area partially cleaned by plant | 0.021 | Half of the area cleaned by the plant staff. Cleaning not effective, only cleans immediate area impacted by the spray |
| 4 | 0.66 | 42,186 | 0.85 | 42,186 | >15:00 | >9.94 / >12.77 | NA<br>Test area mostly cleaned by leakage from the tide. | NA | Small pile of leaves near the sidewall not moved by the water. Cleaning not effective, only cleans immediate area impacted by the spray pattern. |
| 5 | 0.66 | 42,186 | 0.85 | 45,702 | >15:00 | >9.94 / >12.77 | <0.015 / 0.015<br>Test area mostly cleaned by leakage from the tide. | <0.015 | Cleaning not effective. Only cleans immediate area impacted by the spray pattern. |
| 6 | 0.61 | 40,780 | 0.49 | 35,155 | >15:00 | >9.08 / >7.38 | NA<br>Test area mostly cleaned by leakage from the tide. | NA | Alt 1 flowmeter is not operating, estimated flow rate. Cleaning not effective, only cleans immediate area impacted by the spray pattern |
| 7 | 0.61 | 41,483 | ND | ND | >15:00 | >9.08 / ND | NA | NA | Alt 1 flowmeter is not operating, estimated flow rate. Cleaning not effective, only cleans immediate area impacted by the spray pattern |
| | | Average | | | >15:00 | >9.65 / >11.70 | <0.05 / 0.20 | 0.021 | |

Notes:

English Equivalents are provided in Appendix B.

1. Cleaning system was run for 15 minutes without achieving effective cleaning.
2. Flow rates and pressures were not recorded; the values reported represent target values.
3. ND - system was not operated either because basin had been previously cleaned by plant staff or because the test was interrupted by the plant staff.
4. NR - denotes data not measured or recorded during pilot operations.
5. NA - denotes solids data was not measured due to NYCDEP staff operations.

Table 5.2.c-5. Spray Nozzle Cleaning System Test Results (2nd pg)

Table 5.2.c-5 Spray Nozzle Cleaning System Test Results

| Test Run Number | Test Results | | | | | | Solids Quantity / Quality | | Comments |
|---|---|---|---|---|---|---|---|---|---|
| | Header Test Conditions | | | | Required Cleaning Duration [1] (MM:SS) | Water Consumption (Alt 1 / Alt2) ($m^3$) | Depth Range, m (Min/Max) | Average Depth (m) | |
| | Alt 1 | | Alt 2 | | | | | | |
| | Flow Rate ($m^3$/min) | Pressure ($kg/m^2$) | Flow Rate ($m^3$/min) | Pressure ($kg/m^2$) | | | | | |
| **Basin Influent** | | | | | | | | | |
| 1 | 0.61 [2] | 42,186 [2] | 0.95 [2] | 42,186 [2] | NR | >7.19 / >10.22 | 0.031 / 0.183 0.061<br>Piles of solids up to 0.06 ft. | | Pilot run for demonstration purposes only. Cleaning not effective, only clean immediate area impacted by the spray pattern. Channeling occurs after the initial flush. 20% of area clean. |
| 2 | ND | ND | ND | ND | ND | ND / ND | 0.061 / .305 0.158<br>Soupy to sandy consistancy. | | Pilot operations interrupted by the plant staff. |
| 3 | ND | ND | ND | ND | ND | ND / ND | ND ND<br>No measurable solids. | | Area cleaned by the plant staff. |
| 4 | ND | ND | 0.85 | 42,186 | >15:00 | ND / >12.77 | 0.031 / 0.122 0.064<br>Small piles of floatables. | | Alt-1 area cleaned by the plant staff. Cleaning not effective, only cleans immediate area impacted by the spray pattern. Channeling occurs after the initial flush. 10% of the area clean. |
| 5 | 0.57 | 45,702 | 0.85 | 45,702 | >15:00 | >8.63 / >12.77 | 0.024 / 0.457 0.076<br>Floatables near the velocity channel, and piles to 1.5 ft. | | Cleaning not effective, only cleans immediate area impacted by the spray pattern. Channeling occurs after the initial flush. 25% of the area clean. |
| 6 | 0.72 | 35,155 | 0.49 | 35,155 | >15:00 | >10.79 / >7.38 | 0.015 / 0.091 0.052<br>Light and soupy solids | | Scoured area reflects the impact of the mixer. Cleaning not effective, only cleans immediate area impacted by the spray pattern. |
| 7 | ND | ND | ND | ND | ND | ND / ND | 0.024 / .457 0.152 | | Spray nozzles not operated. |
| **Average** | | | | | >15:00 | >8.33 / >10.79 | 0.024 / .457 0.094 | | |

Table 5.2.c-6. Original Orifice Header Configuration Test Results

**Table 5.2.c-6 Original Orifice Header Configuration Test Results**

| Test Run Number | Test Results (per sidewall test section) | | | | Solids Quantity / Quality | | Comments |
|---|---|---|---|---|---|---|---|
| | Flow Rate ($m^3$/min) | Pressure (kg/$m^2$) | Required Cleaning Duration [1] (MM:SS) | Water Consumption ($m^3$) | Depth Range,m (min/max) | Average Depth (m) | |
| **Basin Influent (Header No. 1A @ El +5.0)** | | | | | | | |
| 1 | 3.028 [2] | 7,031 [2] | NR | NR | NR / NR | NR | Pilot run for demonstration purposes only. Cleaning not effective, severe channeling of water. |
| 2 | ND | ND | ND | ND | <0.05 / 0.60 | 0.33 | Pilot operation interrupted by the plant staff. |
| 3 | ND | ND | ND | ND | ND / ND | ND | Area cleaned by the plant staff. |
| 4 | 3.217 | 9,140 | > 15:00 | >48.26 | 0.10 / 0.80 | 0.23 | Approximately 40% clean, severe channeling of water. |
| 5 | 2.460 | 9,140 | > 15:00 | >F3336.90 | 0.05 / 0.50 Piles near training wall | 0.10 | Approximately 60% clean, severe channeling of water. |
| 6 | ND | ND | ND | ND | ND / ND | ND | Area cleaned by the plant staff. |
| | **Average** | | **> 15:00** | **>42.58** | **<0.05 / 0.80** | **0.22** | |
| **Basin Influent (Header No. 2A @ El -8.5)** | | | | | | | |
| 1 | 3.028 [2] | 7,031 [2] | NR | NR | NR / NR | NR | Pilot run for demonstration purposes only. Cleaning not effective, severe channeling of water. |
| 2 | ND | ND | ND | ND | <0.05 / 0.60 | 0.33 | Pilot operation interrupted by the plant staff. |
| 3 | ND | ND | ND | ND | ND / ND | ND | Area cleaned by the plant staff. |
| 4 | ND | ND | ND | ND | ND / ND | ND | Area cleaned by the plant staff. |
| 5 | 3.217 | 8,437 | > 15:00 | >48.26 | 0.05 / 0.50 Piles near training wall | 0.10 | Approximately 70% clean, severe channeling of water. |
| 6 | ND | ND | ND | ND | ND / ND | ND | Area cleaned by the plant staff. |
| | **Average** | | **> 15:00** | **>48.26** | **<0.05 / 0.60** | **0.22** | |
| **Basin Overflow (Header No. 1B @ El +5.0)** | | | | | | | |
| 1 | 3.028 [2] | 7,031 [2] | NR | NR | NR / NR | NR | Pilot run for demonstration purposes only. Cleaning not effective, severe channeling of water. |
| 2 | ND | ND | ND | ND | <0.05 / 0.05 | <0.05 | Pilot operation interrupted by the plant staff. |
| 3 | 1.703 | 7,734 | > 15:00 | >25.55 | <0.05 / 0.10 Mostly leaves and floatables | 0.07 | Severe channeling of water, 50% of area clean. |
| 4 | ND | ND | ND | ND | ND / ND | ND | Area cleaned by the plant staff. |
| 5 | 3.217 | 9,140 | > 15:00 | >48.26 | <0.05 / 0.05 50% of area cleaned from the leakage of the tide. | 0.05 | Approximately 70% clean, severe channeling of water. |
| 6 | ND | ND | ND | ND | ND / ND | ND | Area cleaned by the plant staff. |
| | **Average** | | **> 15:00** | **>36.90** | **<0.05 / 0.05** | **0.06** | |
| **Basin Overflow (Header No. 2B @ El -8.5)** | | | | | | | |
| 1 | 3.028 [2] | 7,031 [2] | NR | NR | NR / NR | NR | Pilot run for demonstration purposes only. Cleaning not effective, severe channeling of water. |
| 2 | 3.028 | 9,843 | > 15:00 | >45.42 | <0.05 / 0.05 | <0.05 | Cleaning not effective, severe channeling of water. |
| 3 | 1.893 | 8,437 | > 15:00 | >22.71 | <0.05 / 0.10 Mostly leaves and floatables. | 0.07 | Cleaning not effective, severe channeling of water. |
| 4 | ND | ND | ND | ND | ND / ND | ND | Area cleaned by the plant staff. |
| 5 | 3.028 | 9,843 | > 15:00 | >45.42 | <0.05 / 0.05 50% of area cleaned from the leakage of the tide. | 0.05 | Approximately 70% clean, severe channeling of water. |
| 6 | ND | ND | ND | ND | ND / ND | ND | Area cleaned by the plant staff. |
| | **Average** | | **> 15:00** | **>37.85** | **<0.05 / 0.10** | **0.06** | |

Notes:

English Equivalents are provided in Appendix B.

1. Cleaning system was run for 15 minutes without achieving effective cleaning.
2. Flow rates and pressures were not recorded the values represent target values.
3. ND - system was not operated eith because basin had been previously cleaned by plant staff or because the test was interrupted by the plant staff
4. NR - denotes data not measured or recorded during pilot operations

Table 5.2.c-7. Modified Orifice Header Configuration Test Results at Representative Solids Concentrations (3 pg.)

**Table 5.2.c-7 Modified Orifice Header Configuration Test Results at Representative Solids Conditions [1]**

| Test Run Number | Orifice Header Number | Test Results (per sidewall test section): Header Test Condition: Flow Rate ($m^3$/min) | Header Test Condition: Pressure (psig) | Required Cleaning Duration (MM:SS) | Water Consumed ($m^3$) | Projected Water Usage per Basin ($m^3$) | Solids Quantity/ Quality: Depth Range, ft (Min/Max) | Solids Quantity/ Quality: Average Depth (m) | Comments | Header Sequencing |
|---|---|---|---|---|---|---|---|---|---|---|
| ORIFICE HEADER NO. 1A (West wall @ EL +5.0) | | | | | | | | | | |
| 7 | 1 | 2.082 | 10,547-14,062 | >15:00 | > 32.903 | > 1,645.150 | 0.015/0.091 | 0.055 | Approximately 50% clean in the initial flush, severe channeling of water. Ineffective cleaning. | ND |
| 9 | 1 | 2.176 | 8,789 | >15:00 | > 29.807 | > 1,490.344 | 0.015/0.030 Soupy solids | 0.027 | Severe channeling of water. Ineffective cleaning. Area cleaned along sidewall. | ND |
| 10 | 1 | 2.271 | 8,437-17,578 | >15:00 | > 34.947 | > 1,747.345 | 0.030/0.152 Solids are clayey/mucky | 0.061 | Approximately 70% clean in the initial flush, severe channeling of water. Ineffective cleaning. | ND |
| 12 | 1 | 1.419 | 3,867-6,328 | >15:00 | > 21.389 | > 1,069.452 | <0.015/0.152 Trace coating with piles up to 0.15 ft. | 0.030 | Approximately 60% clean in the initial flush, severe channeling of water. Ineffective cleaning. Area cleaned along sidewall. | ND |
| 13 | 1 | 1.703 | 5,625 | >15:00 | > 25.129 | > 1,256.431 | <0.015/0.060 Soupy/clayey texture. No grit. Leaves and floatables present. | 0.046 | Approximately 40% clean in the initial flush, severe channeling of water. Ineffective cleaning. Area cleaned along sidewall. | ND |
| 14 | 1 | 2.139 | 9,140 | >15:00 | > 31.938 | > 1,596.892 | 0.060/0.091 | 0.076 | Approximately 15% clean in the initial flush, severe channeling of water. Ineffective cleaning. | ND |
| 16 | 1 | 2.176 | 8,437 | >15:00 | > 31.226 | > 1,561.313 | NA/NA Very soupy solids. Whitish color on surface of solids (unkown). | NA | Approximately 25% clean in the initial flush, severe channeling of water. Ineffective cleaning. Area cleaned along sidewall. | ND |
| **Average for Header No. 1** | | | | **>15:00** | **> 29.620** | **> 1,480.989** | <0.015/0.091 | 0.049 | Does not provide effective cleaning of entire floor area. Only effectively cleans within approx. 5 ft of wall. Water channels further away. | |

Table 5.2.c-7. Modified Orifice Header Configuration Test Results at Representative Solids Concentrations 2nd page

**Table 5.2.c-7 Continued**

| Test Run Number | Orifice Header Number | Test Results (per sidewall test section): Header Test Condition: Flow Rate ($m^3$/min) | Header Test Condition: Pressure (psig) | Required Cleaning Duration (MM:SS) | Water Consumed ($m^3$) | Projected Water Usage per Basin ($m^3$) | Solids Quantity/ Quality: Depth Range, ft (Min/Max) | Solids Quantity/ Quality: Average Depth (m) | Comments | Header Sequencing |
|---|---|---|---|---|---|---|---|---|---|---|
| ORIFICE HEADERS NO. 2A, 2B, AND 3 (East wall @ EL. -8.5 and +5.0) | | | | | | | | | | |
| 7 | 2A<br>2B | 3.123-3.407<br>2.271 | 10,547-14,062<br>NR | 1:30<br>NR | 6.487 | | 0.015/0.091<br>Gritty consistency | 0.055 | 95% clean after 1 min. 100% clean after 3.5 min.<br>Header 2B not operational and was not tested | #2A |
| | Total for Run No. 7 | | | 1:30 | 6.487 | 324.375 | | | | |
| 9 | 2B | 2.555 | 10,547 | 00:28 | 1.192 | | 0.015/0.003<br>Solids with soupy consistency | 0.027 | Area clean after 0.8 min. Pile remaining near influent headwall, boundary off influence<br>Peak pressure at 28 psig. Area clean within 30 sec. Small piles remain near Header 2B.<br>Increased flow to 2A to remove piles near Header 2B.<br>80% clean under Hdr. 3 after 2 min<br>95% clean under Hdr. 3 after 4.5 min | #2B, then #2A and #2b, then 2A, finally 2A and 3. |
| | 2A/2B | 2.271/2.271 | 5,273/5,976 | 01:30 | 6.813 | | | | | |
| | 2A | 2.744 | NR | 00:05 | 0.227 | | | | | |
| | 2A/3 | 2.460/1.235 | 7,031/7,031 | 01:30 | 5.678 | | | | | |
| | Total for Run No. 9 | | | 03:33 | 13.910 | 695.494 | | | | |
| 10 | 2A/3 | 2.650/1.514 | 10,547/14,062 | 00:45 | 3.123 | | 0.1/0.50<br>Solids with mucky/clayey consistency. | 0.061 | Rags stuck to wall fillet. Little cleaning near influent.<br>100% clean after 80 sec. | #2A and #3 then #2B. |
| | 2B | 1.136 - 2.650 | 10,547 | 01:20 | 5.045 | | | | | |
| | Total for Run No. 10 | | | 02:05 | 8.168 | 408.402 | | | | |
| 12 | 2A/3 | 1.703/0.852 | 0/2,109 | 00:10 | 0.458 | | <0.015/0.152<br>Trace coating except for isolated piles up to 0.15 | 0.030 | Headers 2A & 3 were 95% clean within 10 sec. Piles along cross-channel wall were remaining. Flow peaked at 900 gpm. Header #1 was initially on, later turned off.<br>Pile near wall cleaned after 80 sec.<br>Appears that 400-500 gpm per header alone or 800 gpm for 2A & 2B combined will provide effective cleaning. | #2A and #3 then #2B. |
| | 2B/3 | 1.703/0.568 | 0/2,109<br>2812 | 01:20 | 3.028 | | | | | |
| | Total for Run No. 12 | | | 01:30 | 3.486 | 174.299 | | | | |
| 13 | 2A/3<br>2A/3 | 1.703/0.852<br>2.176/0.757 | 0/3,516<br>2,812/2,812 | 02:30<br>01:02 | 6.389<br>3.032 | | <0.015/0.060<br>Soupy to clayey consistency with some floatables. Plastic trash bag near Hdr. 2B, 30% of area cleaned by inflow | 0.046 | Peak flow at 750 gpm.<br>Piles near training wall and around garbage bag took time to clean. Increased flow for header 2A to increase cleaning. Could not force garbage back to channel. | 2A & 3, then 2B & 3, then 2A, then 2A & 2B |

Table 5.2.c-7. Modified Orifice Header Configuration Test Results at Representative Solids Concentrations (3rd page)

**Table 5.2.c-7 Continued**

| Test Run Number | Orifice Header Number | Test Results (per sidewall test section): Header Test Condition: Flow Rate ($m^3$/min) | Header Test Condition: Pressure (psig) | Required Cleaning Duration (MM:SS) | Water Consumed ($m^3$) | Projected Water Usage per Basin ($m^3$) | Solids Quantity/ Quality: Depth Range, ft (Min/Max) | Solids Quantity/ Quality: Average Depth (m) | Comments | Header Sequencing |
|---|---|---|---|---|---|---|---|---|---|---|
| | 2B/3 | 1.893/0.757 | 703/2,812 | 00:45 | 1.987 | | | | Peak flow occurred at 900 gpm. | |
| | 2A | 2.839 | 8,437 | 00:50 | 2.366 | | | | Increased flow & pressure started to move garbage bag. Turned off header 3 to deliver more flow to 2A. | |
| | 2A/2B | 2.460/2.460 | 7,031/7,031 | 00:10 | 0.821 | | | | Bag moved quickly due to flow from Header 2B. | |
| | Total for Run No. 13 | | | 04:15 | 14.595 | 729.748 | | | | |
| 14 | 3 | 0.757 | NR | 00:15 | 0.189 | | <0.015/0.091 Solids generally with thick clayey/mucky consistency | 0.043 | Solids cleaned quickly. Solids after Hdr. 2A cleaned in 30 sec. Solids after Hdr. 2B cleaned in 2 min. Small floatables pile near velocity channel required add'l. time. | 2A, 2B, and 3 operated simultaneously |
| | 2A/2B | 2.460/2.460 | 7,031/8,086 | 02:00 | 9.841 | | | | | |
| | Total for Run No. 14 | | | 02:15 | 10.030 | 501.513 | | | | |
| 16 | 3,2A&2B | 1.987/1.987 0.757 | 0/4,922/4,219 | 02:10 | 10.250 | | <0.015/0.152 Uniform coating of solids at 0.015-0.060. Some piles to 0.152 | 0.091 | Approx. 90% clean after 2 mins. No further cleaning at this flowrate. After 6 min., increased flowrate on Hdrs. 2A and 2B. Residual solids cleaned within 1:30. | 2A, 2B, and 3 operated simultaneously |
| | 3,2A&2B | 2.176/2.176 0.568 | NR | 01:30 | 7.381 | | | | | |
| | Total for Run No. 16 | | | 03:40 | 17.631 | 881.527 | | | | |
| **Average for Modified Header Configuration 2A, 2B, and 3** | | | | **02:41** | **10.615** | **530.765** | **<0.015/0.152** | **0.051** | Modified headers were very effective. A max flow of 750 gpm/floor header required to move large objects (i.e., leaf bag). | |

Notes:

English Equivalents are provided in Appendix B.

1. Data for runs which trace or no solids, or where basin had been previously cleaned are not representative and are not included in the analysis.
2. Recorded times of >15:00 indicates system was run for 15 minutes without achieving effective cleaning.
3. NR - denotes data not measured or recorded during pilot operations.
5. ND - denotes solids data was not measured due to NYCDEP staff operators.
6. NA - denotes header sequencing is not applicable for Header 1.
7. Orifice Header #1 is at el. +5.0, west side of Basin 3.
8. Orifice Header #2 is at el. -8.5, east side of Basin 3.
9. Orifice Header #2B is at el. -8.5, center, east side of Basin 3.
10. Orifice Header #3 is at el. +5.0, east side of Basin 3.

Further away from the wall, the wash-water would form channels and the remaining floor area would be cleaned only very slowly, as the flow eroded the edges of the channel. Based upon field observations, it appeared that a higher velocity of wash-water was required to completely resuspend the solids over the full width of the test section and resist the formation of channels. The headers installed at elevation +1.5 m (+5.00 ft) provided adequate cleaning of the sidewall. The system was generally allowed to operate for 15 min, by which time only 30—75% of the floor test area was effectively cleaned. Since the cleaning was incomplete, the cleaning times and volumes shown in the table are qualified to indicate that actual values are greater than those shown.

Overall, the modified orifice header configuration showed significant improvement over the original orifice header configuration. The modified orifice header no. 1A provided the same degree of cleaning, approximately 30—75% of the floor test area, as the original configuration. However, the area appeared to be cleaned sooner than the original configuration. Additionally, the wall-floor fillet minimized the quantity of solids deposited along the sidewall. While no significant solids/grease buildup was observed on the basin sidewalls, the spray from modified header no. 1A provided better cleaning action than original configuration. Because of the tangential flow spray of the wash-water onto the wall, all of the water flow washed down the wall in a sheet of water. In the original configuration in which the orifices were directed more at the wall, a large portion of the flow sprayed off the wall and rained down on the basin floor.

The combination of modified headers no. 2A, 2B, and 3 provided complete and effective cleaning of the test area floor and sidewall within a short period of time. The combination of the two lower headers (2A and 2B) spraying towards the velocity channel provided a high scouring velocity and prevented the formation of channels in the flow path that would otherwise limit cleaning effectiveness. With all three headers operating, the test area was generally 95% clean within the first minute of operation. As shown in Table 5.2.c-7, the average required cleaning time for the test section was 2.68 min with a projected basin wash-water usage of 530 $m^3$/basin (140,000 gal/basin). During the prototype testing, the header operating sequence and flow rates were varied to determine the optimum operating conditions. Simultaneous operation of headers 2A and 3, followed by header 2B alone, appeared to minimize the consumption of wash-water. Simultaneous operation of all three headers was also tested, as this was most representative of the future full-scale operating scenario. This provided cleaning effectiveness equal to sequential operation in a shorter time period, but at higher projected water consumption. A wash-water flow rate of 2.3 $m^3$/min (600 gpm) per header (30 gpm/If /header) was sufficient to clean the majority of the solids. However, in test run no. 13, a partially full trash bag was present in the test area and a flow rate of 2.8 $m^3$/min (750 gpm) per header (37.5 gpm/If /header) was required to move the trash bag into the velocity channel. The purpose of header no. 3 was to clean the sidewall and the area behind header no. 2A. The flow rate on header number 3 was varied between 0.8—1.5 $m^3$/min (200—400 gpm or 10—20 gpm/ft). A flow rate of between

0.8—1.2 $m^3$/min (200—300 gpm) appeared to be sufficient to meet these cleaning objectives.
Operation of the orifice headers was simple. The biggest difficulty was modulating the flow at startup to minimize the occurrence of an initial surge of water. The steps taken to minimize this effect included slowly opening the valves at startup and tapping the feed pipe to provide air relief points. During operation, some rags and debris became entangled on the orifice header pipe supports. The amount of material caught on the supports was minor and did not affect cleaning performance. It is important to note that after a total of 6 months of testing the lower headers, no clogging of orifices was observed. This was originally identified as a concern for submerged orifices.

*Submersible Mixer.* There is no specific test data on the submersible mixer other than solids depth measurements and qualitative observations. From observations of the residual solids distribution following basin de-watering, the mixer did not appear to effectively minimize the deposition of solids. Typically, solids were scoured from a small area approximately 1.5—3.0 m (5—10 ft) in diameter immediately at the discharge of each of the mixer nozzles. However, the solids depth immediately surrounding the scoured areas was deeper than for other basins, indicating that the scoured solids were immediately redeposited away from the nozzle discharge. Often there was a deep pile or ridge of solids formed around the scoured area. Elsewhere on the test area floor, the solids were unaffected by the mixer.

The mixer did not have any effect on floatables and leaves, which remained in the test area following basin de-watering.

The discharge nozzles would periodically become clogged as indicated by decreased mixing turbulence in the tank during mixer operation. Following removal of the submersible mixer prototype, the flow distribution box and several of the nozzles and feed lines were found to be clogged with leaves and debris. This material apparently became so compacted in the distribution box that the top weld of the box cracked open.

Comparison of Technologies and Recommendations for Design

*Summary of Cleaning Effectiveness.* The results of CDM's field testing program demonstrated that either the tipping buckets or the modified orifice header system would provide satisfactory cleaning of the basin floor, equivalent in performance to the existing traveling bridge system. Each of these two systems achieved complete cleaning of the test areas.

The spray headers equipped with nozzles and the original orifice header configuration provided only partial cleaning of the test areas. This configuration was considered to be unsuitable for application at Spring Creek given the existing basin geometry. The spray headers only provided cleaning of the immediate area impacted by the nozzle spray. This resulted in cleaning only 25% of the test area. The original orifice header configuration was designed to provide sheet flow of water over the basin floor. With this

system, channeling of the flow occurred rapidly, which limited the ability of the system to clean the entire test area. Under the best of conditions, this system cleaned only up to 75% of the test area. These two systems are, therefore, not considered further. The submersible mixer did not minimize the deposition of solids in the test area and created deep piles and ridges of solids adjacent to scoured areas. Also, the mixer became clogged with leaves and debris. Therefore, the submersible mixer is not considered further.

The flushing gate was effective at producing a high velocity wave with initial velocity of 3.4—4.3 m (11—14 ft) per second in the center velocity channel at 30.5—76.2 m (100—250 ft) downstream of the gate. This velocity would be more than sufficient to scour solids from the velocity channel or the influent barrels. It was not intended to evaluate the flushing gate for cleaning of the basin floor. The gate was demonstrated on the center velocity channel as a surrogate for possible application to the influent barrels. Operational problems were encountered with floatables and debris interfering with gate closure and sealing. For full-scale application, some means of positive closure would be recommended. Also, if an alternate method of sealing without significant leakage could not be developed, an alternate water supply would be required to fill the storage volume prior to a flushing event.

The remainder of the discussion in this section will address the two successful prototype systems for basin cleaning, tipping buckets and orifice headers.

The average cleaning requirements determined from the prototype testing for the tipping bucket and modified orifice header systems are summarized in Table 5.2.c-8 and are compared against the existing traveling bridge system. The cleaning requirements for the tipping buckets and orifice headers are based upon cleaning a single 6.1 m (2 ft) length of the basin at one time, which was the length of the test areas. The cleaning time indicated for both the buckets and orifice headers could be reduced from that shown in the table by increasing the size of the cleaning zone, and consequently, increasing wash-water pumping and conveying requirements. However, for the purpose of comparing the prototype test data, the cleaning requirements for the two systems are presented on the same 6 .1 m (20 ft) basis.

As shown in the Table 5.2.c-8, the average basin cleaning time for the tipping buckets is more than twice that for either the orifice headers or traveling bridges. This is primarily due to the multiple tips required to clean each zone. For full-scale implementation of either the tipping buckets or orifice headers, the actual time to clean all six basins would not be six times the average basin cleaning time, but rather some fraction of this quantity. The total cleaning time would be dependent upon a number of factors including:

- Length and number of cleaning zones in each basin
- Number of basins to be cleaned simultaneously
- The maximum allowable or cost-effective wash-water flow rate

- The maximum allowable solids loading rate to the cross channel and climber screens

The last two factors would limit the number of cleaning zones in operation at any one time. The flow and solids loading rates selected would impact the sizing and cost of the cleaning and de-watering screening equipment. Therefore, while it is difficult to assess the minimum full-scale cleaning time without considering all the above factors, it is believed that the use of the tipping buckets will result in a significantly greater total cleaning time than either orifice headers or traveling bridges.

Table 5.2.c-8. Projected Cleaning Requirements for a Single Basin Cleaning Event (Based on 20-ft. Cleaning Zone).

| ***Cleaning Parameter*** | | ***Tipping Buckets*** | ***Mod. Orifice Headers (2A, 2B, and 3)*** | ***Traveling Bridge*** |
|---|---|---|---|---|
| Average Time (min/basin) | | 175 | 67 | 72 |
| No. of Cycles/Zone (averaged over basin length) | | 7 tips | 1 | 2-6 passes |
| Water Consumption | | | | |
| - Influent Test Area | ($m^3$/basin) | 457.99 | 530.77 | NA |
| | (gal/basin) | 121,000 | 140,229 | NA |
| - Average Entire Basin | (m3/basin) | 291.45 | NA | 272.05 |
| | (gal/basin) | 77,000 | NA | 71,875 |
| Estimated Electrical Consumption (kW-hr/basin) | | 40 | 40 | 40 |
| Level of Staffing Required for Operation | | Automated | Automated | Fully Manned |

Note: NA denotes data not available.

The performance of the tipping buckets and the modified orifice headers compare favorably with each other and with the traveling bridge system in that all three systems generally are capable of achieving complete cleaning of the basin floor and walls. In one instance, run no. 16, the tipping buckets could not completely clean a pile of matted leaves mixed with solids. After 30 tips the test was terminated as it was determined that no further cleaning could be achieved. Similarly, difficulty with the tipping buckets moving leaves was also noted in the overflow test area of run no. 10, where 12 bucket tips were required to move a pile of leaves. The orifice headers exhibited similar difficulty moving a trash bag. However, once the lower header flow rate was increased to 2.8 $m^3$/min (750 gpm) per header, the bag was swept into the velocity channel. With

the proper design flow rate, the orifice headers should be capable of providing complete cleaning.

The average wash-water consumption for the tipping buckets and the traveling bridges is similar at 270—290 $m^3$ (71,875—77,000 gal) per basin. There is no average over the entire basin length for the modified orifice headers as this system was only tested at the basin influent end. This represents the worst case solids cleaning condition. For the influent test areas, projected basin water consumption values are presented for the tipping buckets and modified orifice headers. Although, based on this test area the tipping buckets use approximately 14% less water than the orifice headers. The wash-water volumes ranged from 100 $m^3$ to greater than 1,250 $m^3$ (27,500 to >330,000 gal) per basin for the tipping buckets and 175—880 $m^3$ (46,050—232,900 gal) per basin for the modified orifice headers. This data variability would need to be accounted for in full-scale design by assuming conservative design values. Therefore, the difference in wash-water consumption between these two systems is not considered significant.

Estimates of power consumption for the three systems are presented primarily for informational purposes. Because the cleaning systems are operated only intermittently, on the order of 8 h/week, the differences in electrical costs are insignificant. As shown in the table, the tipping buckets are estimated to use the least amount of electricity while the orifice headers use the most. This is because the orifice headers are a high flow rate, moderate head system, while the buckets are a low flow and low head system.

Both the tipping buckets and orifice header system would be designed for automatic operation in full-scale application. This is an advantage over the traveling bridges that are continually manned during operation.

Cost Comparison

The estimated cost of implementing either tipping buckets or orifice headers at the Spring Creek facility are presented in Table 5.2.c-9. These estimates are based upon average solids conditions and water consumption. For the tipping buckets, water consumption was averaged over the length of each basin based upon the influent and overflow test area results. The modified orifice header system was only tested on the basin influent end. Similar to the tipping buckets, the water consumption for the overflow end of the basin would be expected to be less than the influent end, due to the distribution of solids. For this analysis, this was not taken into account. It is likely that the operating time for the orifice headers, and associated electrical cost, could be than the calculated electrical cost.

Table 5.2.c-9. Cost Comparison of Full Scale Tipping Bucket Vs. Orifice Header Cleaning Systems

| Item | Tipping Buckets | *Orifice Headers* | |
|---|---|---|---|
| | | Piping Alt 1 Pipe Gallery | Piping Alt 2 Pipe on Roof Deck |
| Direct Capital Costs | | | |
| Piping and Valves | $ 781,000 | $ 10,255,107 | $ 5,840,000 |
| Mechanical Equipment | $ 4,956,000 | $ 120,000 | $ 120,000 |
| Instrumentation and Controls | $ 10,000 | $ 16,000 | $ 16,000 |
| Electrical Equipment | $ 362,000 | $ 300,000 | $ 300,000 |
| Indirect Capital Costs | | | |
| Contractor OH&P (1) | $ 1,283,000 | $ 2,245,000 | $ 1,318,000 |
| Design / Contingency (2) | $ 916,000 | $ 1,069,000 | $ 941,000 |
| SUBTOTAL CAPITAL COSTS | $ 8,308,000 | $ 14,005,107 | $ 8,535,000 |
| Annual O&M Costs | | | |
| Electrical Costs | $ 1,500 | $ 3,000 | $ 3,000 |
| SUBTOTAL O&M COSTS | $ 1,500 | $ 3,000 | $ 3,000 |
| O&M PRESENT WORTH (3) | $ 17,205 | $ 34,410 | $ 34,410 |
| TOTAL | $ 8,325,205 | $ 14,039,517 | $ 8,569,410 |

Notes:

1. Contractor's overhead and profit are assumed to be 21% of the direct capital cost.
2. Contingency is assumed to be 15% of the direct capital cost for the Tipping Buckets and Orifice Headers Alt 2, and 10% for Orifice Headers Alt 1.
3. Present worth analysis assumes a 20-year design life and an interest rate of 6.0%.

The design assumptions used in this cost analysis were as follows:

Tipping Bucket Design Assumptions

| | |
|---|---|
| Bucket unit volume | 2.7 $m^3$/m (220 gal/ft) |
| Bucket length | 6.1 m (20 ft) |
| Number of buckets/basin | 50 |
| Average water consumption | 290 $m^3$ (77,000 gal)/basin |
| Material of construction | 316L Stainless steel |

Orifice Header Design Assumptions

| | |
|---|---|
| Number of headers/zone | 3 |
| Length of cleaning zone | 24.4 m (80 ft) |
| Orifice header diameter | 25.4 cm (10 in.) |
| Orifice pressure drop | 7030 kg/$m^2$ (10 psi) |

| | |
|---|---|
| Cleaning flow rate (for each side of basin) | |
| Wall header | 0.2 $m^3$/min/m (15 gpm/lin ft) |
| Floor headers | 0.5 $m^3$/min/m (40 gpm/lin ft) /header |
| Total cleaning flow rate | 1.2 $m^3$/min/m (95 gpm/lin ft) |
| Average water consumption | 530 $m^3$ (140,229 gal) /basin |

Conceptual level capital and electrical operating costs are presented. The total net present worth cost is calculated based upon design life of 20 years and an interest rate of 6.0%. Costs are presented for two different feed pipe layouts for the orifice header system. Alternative-I is based upon installation of the main wash-water feed headers in pipe galleries located in the former spray water channels. This alternative has been included in the 30% design phase. Due to the space constraints in the gallery and obstructions by support columns, this alternative involved significantly more fittings and pipe flanges than the second alternative. Alternative-2, the preferred alternative for the orifice headers, is based on installation of the main wash-water feed headers on top of the basin cover system. Without the obstructions of the pipe gallery, the feed headers could be installed in straight pipe runs with pre-fabricated sections and with branches running 90° off the main header. This significantly reduced the number of fittings and pipe flanges. For comparison to the tipping buckets, only the cost for alternative-2 will be considered.

As shown in Table 5.2.c-9, the estimated present worth costs to implement either the tipping bucket cleaning system or the orifice header cleaning system are equivalent. There is only approximately a 3% difference in these estimates, with the tipping bucket cost being slightly less. The operating electrical costs are not significant compared to the total capital cost. The cost for regular maintenance of the equipment is not included in the estimate. The primary maintenance items could include the following:

- Maintenance and periodic replacement of pumps and valves
- Maintenance and periodic replacement of tipping bucket bearings
- Repair/replacement of orifice headers
- Entry into basins to remove large debris items

Currently long-term (e.g., 10 or 20 year) maintenance data is not available on tipping bucket or orifice header installations in CSO facilities. It would be difficult to assign maintenance/ replacement costs for servicing of the bucket bearings or the orifice header pipes. Also, the cost for staffing during cleaning operations has not been included in the cost comparison. Both the tipping bucket and orifice header systems would be automated for full-scale application. Some minimal staffing would be required to monitor and troubleshoot the cleaning operations. However, this could largely be combined with other facility staffing responsibilities. A staffing labor estimate for either of these two systems would be dependent upon operating philosophy and configuration (e.g., sequential vs. concurrent basin cleaning, staffing level). It is expected that the staffing cost for these two alternatives would be similar and would be significantly less than the costs for the existing traveling bridges. Assumptions on the extent of O&M costs required could significantly affect the total costs. Given this uncertainty on the

O&M costs, only the electrical costs which are well defined have been included in the total present worth cost.

Implementability and Operability

Both cleaning technologies, tipping buckets and orifice headers, are readily implementable. There are no known constructability issues that would limit the applicability of one technology over the other. Both technologies provide effective cleaning. However, matted leaves posed a cleaning problem to the tipping buckets while this was not observed to be an issue with the orifice headers. Because of the higher velocity imparted by the orifice headers, this technology appears to be better suited at cleaning matted leaves, piles, and large objects.

There were two operability issues noted with the tipping buckets, including potential bucket misalignment and transient overloading of the cross channel and climber screens by waves carrying high solids loads. Misalignment of one of the test buckets (TB-2B) caused it not to tip automatically. This was retrofitted in the field by adding counterweights. Later a second bucket (TB-1B) developed a hesitation in tipping automatically. With up to 300 buckets planned to be installed in a full-scale facility, this would present a significant concern. The second issue was a concern whenever the facility received a high inflow of grit. During testing, high quantities of grit were washed into the cross channel with the initial few tips of the buckets. Because the buckets produced discontinuous flow surges separated by zero flow conditions, the grit would often settle out in the channel and immediately before the climber screen when the water velocity decreased. This caused the channel and climber screen to become clogged with grit, which then had to be removed manually.

With the orifice headers there are two operability concerns, entanglement of debris on the pipe supports and potential clogging of the orifices. Regarding the first issue, some rags/debris became entangled on the pipe supports of the prototype system during testing. However, the spray from the orifices was sufficient to prevent a buildup of solids with the rags. With proper design, the number of pipe supports and their configuration can be selected so as to minimize this effect. Periodic entry of the basins will likely be required to remove entangled material. As for potential clogging of the orifices, no clogging of the prototype system was observed during the 6 months the headers were in the basins. If clogging does occur, the orifices in the PVC headers may be easily cleaned by plant staff.

## 5.3 Converting Dry Ponds for Enhanced Treatment

Dry ponds are extensively used throughout the U.S. and other countries (EPA 1983). These ponds have been constructed to reduce peak runoff rates (peak shaving), typically with little consideration to runoff quality improvement. Dry ponds have been widely used in flood control projects to detain runoff and thus reduce peak flows and water elevations in the receiving waters. These flow reductions can also improve the aquatic habitat by reducing flushing of fish and other organisms from urban creeks (Pitt and Bissonnette 1984). Flow reductions also reduce downstream channel bank erosion and bottom scour. The use of many dry ponds in a watershed, without regard to their cumulative effect, can actually increase downstream flooding or channel scour problems (McCuen, *et al.* 1984). The delayed discharge of a mass of water from a dry pond may be superimposed on a more critical portion of the receiving water hydrograph.

Because these ponds are normally dry and only contain water for relatively short periods of time, they can be constructed as part of parking lots, athletic fields, tennis courts and other multi-use areas. The outlets are designed to transmit all flows up to a specific design flow rate, after which excess flows are temporarily backed-up. In many cases, the ponds only contain water during a few rains each year.

Several dry detention ponds were examined as part of the National Urban Runoff Program (NURP), with monitored pollutant removals ranging from insignificant to quite poor (EPA 1983). Sedimentation may occur in dry ponds, but only during the major storms when flows are retained in the pond. The deposited material must be removed after each treated rain, or it can easily be re-suspended by later rains and washed into the receiving waters. Adler (1981) found that new sediment deposits have little cohesion and without removal as part of a maintenance program, or without several feet of overlaying water, bottom scour is probable. Because of the poorly documented stormwater pollutant control effectiveness of dry detention ponds, they cannot, by themselves, be recommended as viable water quality control measures. However, they can be very effective when used in conjunction with other stormwater control practices (e.g., between a wet detention pond and an infiltration or grass filter area).

Stanley (1996) examined the pollution removal performance at a dry detention pond in Greenville, NC, during eight storms. The pond was 0.7 ha in size and the watershed was 81 ha of mostly medium density single family residential homes, with some multifamily units, and a short commercial strip. The observed reductions were low to moderate for SS (42—83%), phosphate (-5—36%), nitrate nitrogen (-52—21%), ammonia nitrogen (-66—43%), copper (11—54%), lead (2—79%), and zinc (6—38%). Stanley (1996) also summarized the median concentration reductions at dry detention ponds studied by others. In all cases, the removal of the stormwater pollutants was substantially less than would occur at well designed and operated wet detention ponds. The re-suspension of previously deposited sediment during subsequent rains was typically noted as the likely cause of these low removals. The conditions at one of the ponds in Greenville, NC were observed three years after its construction. The most notable change was that the pond bottom and interior banks of the perimeter dike were

covered with weeds and many sapling trees (mostly willows), which indicated that the interior areas were too wet to be mowed. The perforated riser was also partially clogged and some pooling was occurring near the pond outlet. It appeared that the dry pond was evolving into a wetland. The monitoring activity had been conducted a few months after the pond was constructed and was not affected by these changes. Stanley felt that the wetlands environment, with the woody vegetation, if allowed to spread, could actually increase the pollutant trapping performance of the facility. With continued lack of maintenance, the dry pond will eventually turn into a wet pond, with a significant permanent pool. The pollutant retention capability would increase, at the expense of decreased hydraulic benefits and less flood protection than originally planned.

Nix and Durrans (1996) examined the benefits of off-line stormwater detention ponds. Off-line ponds (side-stream ponds) are designed so that only the peak portion of a stream flow is diverted to the pond (by an in-stream diversion structure). They are designed to reduce the peak flows from developed areas, with no direct water quality benefits, and are typically dry ponds. Off-line ponds are smaller (by as much as 20—50%) than on-line ponds (where the complete storm flow passes through the pond) for the same peak flow reductions. However, the outflow hydrographs from the two types of ponds are substantially different. The off-line ponds produce peak outflows earlier and the peak flows do not occur for as long a period of time as on-line ponds. If located in the upper portion of a watershed, off-line ponds may worsen flooding problems further downstream, whereas downstream on-line ponds tend to worsen basin outlet area flooding. Off-line dry ponds can be used in conjunction with on-line wet ponds to provide both water quality and flood prevention benefits. Off-line ponds have an advantage in that they do not interfere with the passage of fish and other wildlife and they do not dramatically affect the physical character of the by-passed stream itself. On-line dry ponds may substantially degrade the steam habitat by removing cover and radically changing the channel dimensions. The peak flow rate created by online dry ponds reductions can also have significant bank erosion benefits in the vicinity of the pond, although these benefits would be decreased further downstream.

Case Studies

The retrofitting detention pond case studies presented in this section cover a broad range of conditions, from underground facilities for very small paved drainage areas, to large detention facilities for major drainage areas. The problems encountered in each case history are described, along with the costs and benefits, to the extent that information was available.

### *5.3.a. Sunnyvale Detention Basin, Santa Clara County, California*

South San Francisco Bay has a serious heavy metal pollution problem, especially for copper, and numerous methods are being investigated to reduce the discharges of metals to the Bay. Woodward Clyde Consultants (1994) conducted a retrofit project for the Santa Clara Valley Nonpoint Source Pollution Control Program to demonstrate the

benefits of modifying an existing dry detention pond for enhanced water quality benefits. The discussion in this section is mostly taken from that report.

According to a survey conducted in 1990 by Woodward-Clyde (1990), there are seventeen municipally owned and operated pump stations in Santa Clara Valley. These pump stations generally consist of pumps, storage units such as a sump or a detention basin, and inlet and outlet works. The sumps and detention basins reduce the required pump capacity needed to pass the peak flood flows. The pump stations provide flood protection to low lying areas that have historically subsided and are now protected by levees. The pump stations have generally operated as single-purpose flood control facilities. The pumps are designed to go on as soon as water begins to fill the basin with the goal of emptying the basin as soon as possible after the event. Therefore, no net pollutant removals are assumed to be occurring.

One retrofitting option to achieve water quality benefits is to change the pump-operating schedule in order to increase detention time and to provide for a seasonal wet pond. A preliminary evaluation of retrofitting detention basins was encouraging and a pilot study to retrofit a facility and perform testing to measure water quality benefits and costs was August 1990 through July 1993.

The following tasks were conducted as part of this study:

- Retrofit the pump station and modify pump schedules to increase water retention and thus improve SW pollutant removal,
- Conduct water quality sampling to estimate the pollutant removal effectiveness of the retrofitted detention basin, and
- Measure sediment concentrations in the basin in order to determine whether sediments are classified as hazardous waste.

The detention basin has a channel between the inlet and outlet that, prior to the modifications, encouraged short-circuiting. A gabion baffle was installed at the outlet to reduce short-circuiting and to provide better distribution of flow into the outlet. Rock was dumped into the channel leading from the inlet, and a drainage pipe that ran below the channel was blocked off. Operational changes consisted of modifying the pump schedule to create a 0.61 m (2 ft) permanent pool at the outlet and to provide temporary storage and slow release of water over the depth range of 0.61—0.73 m (2—2.4 ft).

*Site Description.* The northern portion of Santa Clara Valley has a history of subsidence caused by groundwater pumping. In order to protect subsiding areas from flooding, a system of levees and pump stations was built. The pump stations collect and pump SW runoff from these low-lying areas through the levees into the flood control channels. In order to accommodate large flows and reduce the number and pumping capacity of the pumps required, some pump stations include relatively large sumps or detention basins. An inventory of the pump stations indicated that there are nine such facilities in the Valley with relatively large detention basins (Woodward Clyde Consultants 1990). The design and operating philosophy of these systems is to: 1) attenuate the peak flow to

reduce required pump size, and 2) drain the basins as soon as possible following the storm so that flood capacity is available for subsequent storms.

An example of one of these pump stations is Sunnyvale Pump Station No. 2, located at the junction of the Milpitas/Alviso Road (Route 237) and Calabazas Creek. The Pump Station consists of four primary pumps, each rated at 1.1 $m^3$/s (39 cfs) capacity, and one auxiliary electric pump with a capacity 0.25 $m^3$/s (9 cfs). The detention basin area is approximately 1.8 ha (4.4 ac.) in area and has a capacity of approximately 37,005 $m^3$ (30 ac-ft) as shown in Figure 5.3.a-1. It receives runoff from a 187.4 ha (463 acre) catchment consisting of the following land uses: industrial park (30%), commercial (10%), and residential (60%). There is a 2.13 m (7 ft) diameter concrete reinforced pipe that drains into the basin. A second 0.91 m (3 ft) diameter line drains a 101.2 ha (250 acre) catchment (primarily open space) and bypasses the basin to the north and directly enters the pump house.

*Treatment Concepts and Retrofitting Objectives.* The pump stations could provide an opportunity to reduce nonpoint source loads entering the South Bay if they can be cost-effectively retrofitted and maintained. The primary treatment is settling of particles. Settling can be an effective treatment for some pollutants that are mostly in the particulate fraction in stormwater. The particulate fraction of locally collected stormwater exhibited a range of 36—94% (mean of 69%) for copper and 24—97% (mean of 66%) for lead. These high particulate fractions allow sedimentation to be an effective control practice.

The retrofitting scheme would increase the detention time to allow more particulates to settle out into the basin while not significantly increasing the flood risk. A goal of this retrofit was to prevent high flows from re-suspending previously settled sediments in the detention pond. Scour protection was provided by having at least a two-foot deep permanent wet pool during the wet weather season.

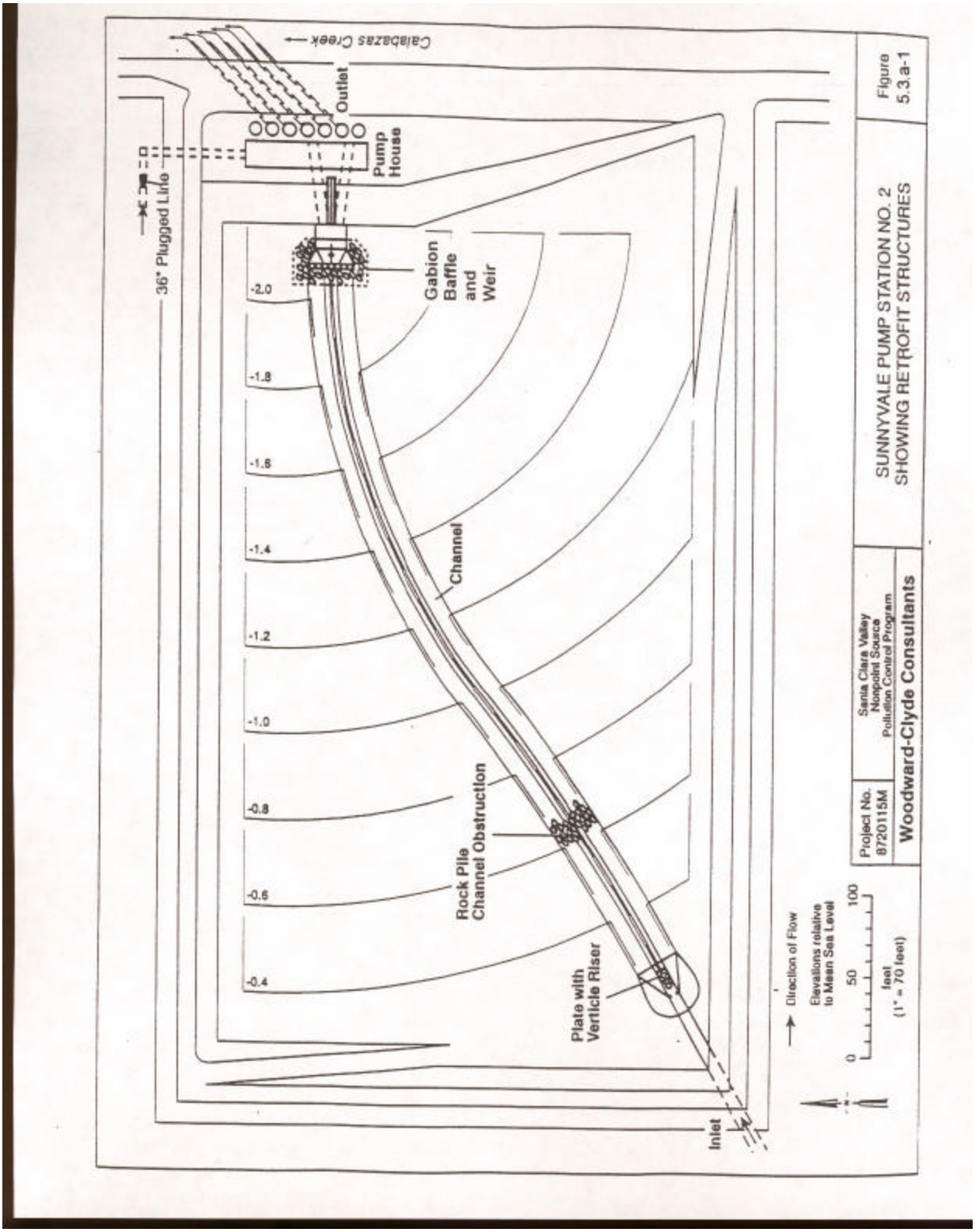
A 24—40 h hydraulic detention time for a pool several feet deep was found to be necessary to effectively settle out most of the suspended sediment in the local stormwater.

In all cases, the basin must maintain a relatively large flood control capacity and associated outlet works and pumps in order to provide the necessary flood control benefits.

Description of Sunnyvale Retrofit Activities.

*Change Pump Operational Rules to Create a Permanent Pool and Temporary Storage.* In order to create the permanent pool in the pond, the pumps were set to turn off when water levels in the basin (as measured at the outlet) dropped below 0.61 m (2 ft) In order to create temporary storage, pump settings were adjusted to phase in (and out) very slowly for depths between 0.61—0.73 m (2—2.4 ft). These operational conditions created a temporary storage depth above the permanent pool of 0.12 m (0.4 ft) with a capacity of 2158.6 $m^3$ (1.75 ac-ft). Because this is an existing flood control facility, the

## 5.3.a-1 Sunnyvale Pump Station No. 2 Showing Retrofit Structures



temporary storage depth was determined primarily based on flood control and secondarily on water quality considerations. The temporary storage depth was the maximum depth that would still allow the basin to pass the 100-year flood.

*Prevent Short Circuiting.* The pond contains a trapezoidal open pilot channel (2.44 m (8 ft) bottom width, 5.2 m (17 ft) top width, and 1.4 m (4.5 ft) depth) between the inlet and outlet. In addition to this open channel, a 0.76 m (2.5 ft) reinforced concrete pipe (RCP) is situated below the channel to convey low flows between the inlet and outlet. These conveyances effectively "short circuited" flows between the inlet and outlet, a condition that is highly unsuitable for water quality control.

In order to limit this short-circuiting, three modifications were made. At the outlet weir near the pump house, a gabion baffle was constructed around the original outlet weir to prevent short-circuiting of flows along the channel and to promote a better distribution of flow from the basin into the outlet weir. A second modification involved placing rocks into the channel near the inlet. The third modification involved covering the entrance of the 0.76 m (2.5 ft) RCP with a steel plate and vertical riser that reduced the rate at which flow would enter the drain below the trapezoidal channel.

*Plug Storm Drain that Directly Entered Pump House.* A 0.91 m (3 ft) RCP drained a 101.2 ha (250 acre) undeveloped area west of the detention basin directly to the pump house sump. This pipe was plugged with sandbags in one of the manholes upstream of the sump to prevent the runoff from this drainage area from mixing with outflow from the detention basin in the sample collection area.

*Problems Encountered.* No significant problems were encountered during the structural retrofitting of the detention basin. However, the pump control system needed major repairs in order to operate the basin within the water level tolerances required for the study. Specific problems were encountered with the liquid level sensors and transmitter (inaccurate flow monitoring because of the very low flow rates) voltage instabilities caused when certain pumps came on line, and fluctuations in the power supply. Therefore, an important aspect in evaluating the feasibility of retrofitting pump stations is the design and condition of the pump control system and the possible need for repairs and upgrading.

*Monitoring Program.* The goal of this study was to measure the total runoff and collect flow-weighted composite water samples at both the inlet and outlet of the detention basin during and after storms in order to estimate pollutant removal performance. Sediment samples were also taken to characterize basin sediments.

*Station Design and Equipment.* Automated flow and water quality monitoring stations were located at the inlet and the outlet of the basin. The inlet pipe was a 2.13 m (7 ft) diameter RCP that was quite low, placed at a level below the basin floor, and tended to be full of water during most of the wet weather season. The inlet sampling station was located 10.7 m (35 ft) upstream of the end of the pipe and consisted of a Druck pressure transducer, velocity meter, ISCO Model 3700 automatic water quality sampler and Campbell Scientific CR-10 data logger/controller. Flow volumes were estimated

using the measured velocity times the wetted area of the pipe and the flow-weighted composite samples were to be collected at the inlet based on these estimates. Initially, a Montadero-Whitney electromagnetic velocity meter was used. However, the velocities in the pipe were too low to measure with this instrument and it was replaced in March 1992 by a Detectonics I.S. Surveylogger, which relies on the Doppler effect and suspended sediment passing the instrument. When compared with estimates of anticipated runoff volumes, neither instrument appears to have measured flow velocities in a consistent and accurate manner. The primary cause appears to be the relatively low velocities in the large pipe.

The outlet sampling was conducted in the pump house where a Druck pressure transducer, an ISCO Model 2700 automatic sampler, and CR-10 datalogger/controller were installed. Sampling based on estimated flow through the pumps was planned. These estimates were based on the pump run times and pump characteristic curves (which show the relationship between flow and head for the design rpm of 700). To achieve this, the data logger was connected to the pump house control panel to determine pump run times and calculate discharge from the sump. Field visits during the 1991—92 season revealed that the pumps did not operate at the design rpm, especially during the warm-up period, resulting in inaccurate flow estimates much of the time.

*Sampling Methods*. At the inlet, a pressure sensor was used to estimate the water level in the detention basin. During each sampling event, flow was calculated as a product of velocity and area by the CR-10 microprocessor. Based on the flow estimate (which was generally poor), the CR-10 initiated and continued water quality sampling at pre-specified flow intervals. During a sampling event, instantaneous velocity and pressure were recorded each time a water quality sample was taken. Based on anticipated rainfall, the sampling algorithm in the CR-10 was designed to instruct the water quality sampler to collect twenty 500 ml sub-samples in a 10 liter borosilicate bottle over the duration of the storm event. Following the sampling event, the pressure sensor was also used to measure water level drops in the pond.

At the outlet, average hourly flowrate was estimated based on the pump run times and the pump characteristic curves (also inaccurate), and was recorded over the duration of the wet-weather season. To begin an event, field crews manually initiated the automated samplers based on anticipated flow volumes for that storm. As with the inlet, the automated samplers recorded instantaneous flow measurements when each sample was collected.

Stations were visited prior to, during, and after monitored events to ice samples, exchange sample bottles, and ensure proper equipment operation. Conductivity, pH, and temperature were measured during the site visits.

*Data Collected*. Eight storm events were sampled. For six of these events, flow-weighted composite water quality samples and hydrologic measurements were taken at both inlet and outlet stations. In most cases, only partial flow measurements were made

because equipment malfunctioned, threshold velocities were not achieved, or there were problems with the pump control system.

Due to the uncertainty in flow volume measurements, pollutant loads were not used to estimate treatment effectiveness. Instead, effectiveness was estimated based on the flow composite water quality concentration data, assuming that the inlet and outlet volumes for an event are equal.

Sediment samples were taken at three locations: in the center of the basin, near the inlet, and near the outlet. Three sets of sediment samples were collected during dry periods when the basin was empty, or nearly empty, of water (June 15, 1990, May 14, 1992, and on July 12, 1993). The first samples were obtained using a 10.2 cm (4 in) stainless steel hand auger, while the other samples were collected by scraping the top 1.27 cm (0.5 in) of sediment with a Teflon™-lined scraper.

Flooding Analysis, Storm Hydrology, Water Quality and Sediment Monitoring Results

*Flooding Analysis.* Woodward Clyde used a reservoir routing model to estimate water levels in the basin for the 100-year inflow event and for two pump operating scenarios. The first pumping scenario corresponded to the original pumping schedule used for flood control. The second scenario corresponded to the revised pumping schedule appropriate for a multipurpose flood control and water quality control facility. Based on the results of the model, the maximum water level in the basin for the 100-year flood did not change by modifying the pump operation schedule.

*Precipitation*. Rainfall was measured with a tipping bucket, which registered the time when the bucket collected 0.25 cm (0.1 in) of rainfall. The ranges of storm volumes during the sampling period were from 1—5.6 cm (0.4—2.2 in) and the storm durations ranged from 6—60 h. The Synoptic Rainfall Analysis Program (SYNOP) examined the long period characteristics of the local historical rainfall data that was collected by the NWS at the San Jose Airport (Gage No. 7821). The median event rainfall volume for the San Jose Airport gage for the period from 1948—1989 was 1.27 cm (0.5 in).

*Runoff.* Flow measurements collected at the inlet and outlet for various events were compared with rainfall to calculate the volumetric runoff coefficients. The flow measurements at both the inlet and outlet stations were not considered very reliable, as the measured runoff coefficients ranged from 0.1—1.89. Woodward Clyde estimated that the actual values would be about 0.5—0.8 for these rains and watershed characteristics.

*Comparison of Inlet Water Quality to Other Santa Clara SW Monitoring Station Data*. Laboratory chemical analyses were conducted on the water samples collected at the basin inlet and outlet stations during the six storm events. The median flow-weighted composite concentrations of total metals (cadmium, chromium, copper, lead, nickel, and zinc) from the inlet station are summarized in Table 5.3.a-1. The table shows median concentrations obtained from other Santa Clara Valley SW monitoring stations representing residential-commercial, industrial, and open land uses. The inlet concentrations of copper, lead, nickel, and zinc are higher than concentrations from

open land use, but lower than concentrations at residential-commercial and industrial land use stations. The cadmium concentration appears to be very similar to the residential-commercial land use, while the chromium concentration is closer to the open land use.

Table 5.3.a-1. Comparison of Median Metal Concentrations at Inlet to Retrofitted Basin to other Santa Clara Valley Stormwater Monitoring Station Data

| Metal | Inlet to Retrofit Basin *(n=6)* (mg/l) | Residential/Commercial Land Use Station *(n=21)* (mg/l) | Industrial Land Use Station *(n=25)* (mg/l) | Open Space Land Use Station *(n=4)* (mg/l) |
|---|---|---|---|---|
| Cadmium | 1.1 | 1.0 | 3.9 | 0.3 |
| Chromium | 12 | 16 | 24 | 11 |
| Copper | 24 | 33 | 51 | 11 |
| Lead | 38 | 45 | 91 | 2.0 |
| Nickel | 21 | 30 | 46 | 5.0 |
| Zinc | 180 | 240 | 1,150 | 5.0 |

*n= number of storm events

*Pollutant Removal Effectiveness.* Table 5.3.a-2 summarizes inlet and outlet concentrations for total and dissolved metals (cadmium, chromium, copper, lead, nickel, and zinc), SS, hardness and total oil and grease for several storm events. Based on these data, pollutant reductions were estimated as the outlet minus inlet concentration divided by the inlet concentration. The average pollutant removal effectiveness for the metals ranged from about 30—50%. The metals removal data indicated that the removal of total chromium, copper, lead, nickel and zinc were well correlated with SS removal.

*Comparison to Water Quality Objectives (WQOs).* Of these metals, total and dissolved chromium, lead and nickel did not exceed the acute WQOs. Total and dissolved cadmium exceeded the WQO in only one storm out of six monitored storms. Total copper at the inlet station exceeded WQOs in four out of six storm events. However, concentrations at the outlet station never exceeded WQOs (though the outlet concentration was essentially equal to the WQO for one event). None of the dissolved copper concentrations exceeded the acute WQOs. Total zinc concentrations at the inlet and outlet stations exceeded the acute WQOs for all six storms. Dissolved concentrations of zinc at the outlet station exceeded the WQOs in three of the six events.

*Sediment Quality*. The objectives of the sediment sampling was to characterize sediment quality in the detention basin and to compare the sediment concentrations to hazardous waste criteria. Results of these sediment samples are summarized in Table 5.3.a-3.

Table 5.3.a-2. Inlet and Outlet Observed Concentrations and Pollutant Removals

| | Cadmium (µg/L) | | Chromium (µg/L) | | Copper (µg/L) | | Lead (µg/L) | | Nickel (µg/L) | | Zinc (µg/L) | | SS (mg/L) | TH (mg/L) | O&G |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | nf | f | nf | f | Nf | f | nf | f | nf | f | nf | f | | | |
| SE 17 | | | | | | | | | | | | | | | |
| Inlet | 0.4 | <0.2 | 3.6 | 1.8 | 8.7 | 5.4 | 6.4 | 2.2 | 1.7 | <2 | 46 | 28 | 12 | 97 | 1.5 |
| Outlet | 0.2 | <0.2 | 2.7 | 1.1 | 6.8 | 4.7 | 3.4 | 1 | 1.7 | <2 | 26 | 19 | 73 | 120 | 1.4 |
| Reduction | -- | -- | 25% | -- | 22% | -- | 47% | -- | 0% | -- | 43% | -- | 39% | -- | 7% |
| SE 20 | | | | | | | | | | | | | | | |
| Inlet | 6.6 | 1.3 | 12 | 1 | 24 | 3 | 45 | 1 | 16 | 1 | 180 | 19 | 90 | 110 | 0.2 |
| Outlet | 4.8 | 2.5 | 6 | 1 | 9 | 3 | 10 | 1 | 4 | 1 | 73 | 22 | 24 | 63 | <0.2 |
| Reduction | -- | -- | 50% | -- | 63% | -- | 78% | -- | 75% | -- | 59% | -- | 73% | -- | -- |
| SE 21b | | | | | | | | | | | | | | | |
| Inlet | 1.1 | 0.2 | 18 | 1 | 24 | 2 | 53 | <1 | 25 | <1 | 180 | 5 | 140 | -- | -- |
| Outlet | 1.5 | <0.2 | 14 | 1 | 16 | 2 | 35 | <1 | 19 | <1 | 120 | 7 | 93 | -- | -- |
| Reduction | -- | -- | 22% | -- | 33% | -- | 34% | -- | 24% | -- | 33% | -- | 34% | -- | -- |
| SE 23 | | | | | | | | | | | | | | | |
| Inlet | 1 | 0.2 | 11 | <1 | 27 | 5 | 30 | 1 | 13 | 3.9 | 190 | 41 | 74 | 100 | 0.7 |
| Outlet | 0.6 | <0.2 | 8.3 | 1.4 | 12 | 4.7 | 12 | <1 | 5.8 | 2.2 | 82 | 45 | 31 | 90 | 0.5 |
| Reduction | -- | -- | 25% | -- | 56% | -- | 60% | -- | 55% | -- | 57% | -- | 58% | -- | -- |
| SE 24 | | | | | | | | | | | | | | | |
| Inlet | 1.6 | <0.2 | 21 | 1.1 | 40 | 2.1 | 76 | <1 | 42 | 9.6 | 270 | 22 | 180 | 140 | 0.6 |
| Outlet | 1.3 | 0.2 | 15 | 8.6 | 24 | 5 | 40 | 1.4 | 29 | 15 | 160 | 31 | 96 | 140 | 3.5 |
| Reduction | -- | -- | 29% | -- | 40% | -- | 47% | -- | 31% | -- | 41% | -- | 47% | -- | -- |
| SE 27 | | | | | | | | | | | | | | | |
| Inlet | 1 | 0.5 | 6.3 | 1.4 | 14 | 5.4 | 13 | <1 | 83 | 63 | 70 | 35 | 30 | 110 | 1.6 |
| Outlet | 0.6 | 0.4 | 4.9 | 1.7 | 8.9 | 4.5 | 6.6 | <1 | 25 | 20 | 47 | 26 | 15 | 220 | 1.3 |
| Reduction | -- | -- | 22% | -- | 36% | -- | 49% | -- | 70% | -- | 33% | -- | 50% | -- | -- |
| Average Reduction | -- | -- | 29% | -- | 42% | -- | 53% | -- | 51% | -- | 44% | -- | 50% | -- | -- |

nf: non-filtered (total) SS: suspended solids O&G: oil and grease
f: filtered ("dissolved") TH: total hardness, as $CaCO_3$
Removals are only given if most observations were >PQL

Table 5.3.a-3. Sediment Observations (mg/kg)

| | % TOC | Cadmium | Chromium | Copper | Iron | Lead | Manganese | Nickel | Zinc |
|---|---|---|---|---|---|---|---|---|---|
| **6/15/90 core** | -- | 2.2 | -- | 92 | -- | 36 | -- | 61 | 320 |
| **5/14/92** | | | | | | | | | |
| **surface** | 3.8 | 23 | 200 | 150 | 49,000 | 280 | 610 | 94 | 750 |
| Inlet | | | | | | | | | |
| Middle | 5.5 | 17 | 220 | 140 | 38,600 | 350 | 640 | 87 | 570 |
| Outlet | 1.9 | 35 | 140 | 47 | 47,700 | 18 | 680 | 76 | 260 |
| **7/12/93** | | | | | | | | | |
| **surface** | 2.4 | 1.0 | 170 | 110 | 34,000 | 260 | 560 | 96 | 220 |
| Inlet | | | | | | | | | |
| Middle | 0.65 | 0.2 | 120 | 37 | 36,000 | 12 | 700 | 75 | 85 |
| Outlet | 0.93 | 0.3 | 110 | 43 | 30,000 | 24 | 570 | 73 | 63 |
| **TTLC** | | 100 | 2,500 | 2,500 | -- | 1,000 | -- | 2,000 | 5,000 |

TTLC: Total Threshold Limit Concentration

In the second and third rounds of sampling, the highest concentrations for copper, nickel and zinc were found at the inlet station. Cadmium, chromium and lead were also highest at the inlet station for the July 12, 1993 sampling round. The high concentration of the majority of the metals near the inlet is consistent with other studies.

Average sediment concentrations observed in Pump Station No. 2 are compared in Table 5.3.a-4 with sediment data collected from other detention basins in the Valley and elsewhere. Results from the various basins differ substantially and indicate that sediment quality is highly site specific and varies depending on soils, catchment land use, and other factors, especially time when the samples were analyzed (for lead).

To evaluate whether the sediments were hazardous, concentrations were compared to standards established in the California Administrative Code, Title 22. Under Title 22, there are two criteria for designating solids as hazardous waste. The first criterion is that the sediment concentrations not exceed the Total Threshold Limit Concentrations (TTLC). The second criterion is that the extract obtained from the whole effluent toxicity (WET) extraction method not exceed the Soluble Threshold Limit Concentrations (STLC). For this pilot scale screening level of analysis, it was considered adequate to compare with the TTLC only. In situations where disposal is being considered, the WET extraction test should also be conducted.

None of the sediment sample concentrations collected in the Sunnyvale Pump Station basin exceeded the TTLC. The highest concentrations of cadmium, lead, and zinc were 4, 3, and 7 times lower than the TTLC, respectively. The highest concentrations reported for chromium, copper, and nickel were 11, 17 and 21 times lower than the TTLC, respectively. Based on these concentrations, sediments are not considered hazardous.

Table 5.3.a-4. Comparison of Average Sediment Concentrations from Detention Basins and Swales (mg/kg)

| Detention Basin | Cadmium | Copper | Nickel | Lead | Zinc |
|---|---|---|---|---|---|
| This Retrofit Basin | 11.2 | 88 | 80 | 140 | 324 |
| Other Santa Clara County | | | | | |
| Eastside Basin A | 0.37 | 32 | 36 | 17 | 68 |
| Eastside Basin B | 0.37 | 36 | 40 | 6 | 73 |
| Eastside Basin C | 1 | 71 | 100 | 11 | 330 |
| River Oaks | Nd | 24 | 72 | 14 | 84 |
| Fresno NURP | | | | | |
| Recharge F | -- | 37 | 32 | 713 | -- |
| Recharge G | -- | 25 | 37 | 487 | -- |
| Recharge M | -- | 55 | 53 | 1333 | -- |
| Recharge EE | -- | 25 | 22 | 297 | -- |
| Recharge MM | -- | 9.5 | 11 | 93 | -- |
| Wigington (1983) | | | | | |
| Bulk Mail Basin | 2.8 | 19 | -- | 112 | 224 |
| Kmart Basin | 0.8 | 13 | -- | 368 | 114 |
| Nightingale (1975) | | | | | |
| Detention Basin | -- | 20 | -- | 224 | 107 |
| Special Pit | -- | 23 | -- | 801 | 236 |
| Wigington (1986) | | | | | |
| Fairidge Swales | 0.26 | 4.2 | -- | 42 | 102 |
| Stratton Woods Swales | 0.18 | 10 | -- | 18 | 70 |
| Rte. 234 Rd. Swales | 0.82 | 23 | -- | 936 | 106 |

## Cost Effectiveness Evaluation

The mean annual runoff volume of 433,000 $m^3$ (351 ac-ft) was estimated based on mean annual rainfall of 33 cm (13 in.) in the vicinity of the basin, an assumed runoff coefficient of 0.7, and the 187.4 ha (463 ac.) catchment area. Mean concentrations and removal efficiencies are averages of observed data. For the metals, annual load reductions ranged from 272 grams (0.6 lbs) for cadmium to 29.5 kg (65 lbs) for zinc. For copper, the annual load reduction is estimated at 4.1 kg (9 lbs), which represents approximately 40% of the total copper that enters the basin. Table 5.3.a-5 summarizes the estimated cost effectiveness for the removal of heavy metals from the pond.

Table 5.3.a-5. Estimated Mean Annual Load reduction and Cost Effectiveness*

| METAL | Mean Inlet Concentration (mg/L) | Average Annual Removal Efficiency (%) | Load Reduction (kg/yr.) | Cost Effectiveness (kg/$1,000) |
|---|---|---|---|---|
| Cadmium | 0.002 | 35 | 0.3 | 0.03 |
| Chromium | 0.012 | 29 | 1.5 | 0.18 |
| Copper | 0.023 | 42 | 4.2 | 0.5 |
| Lead | 0.037 | 53 | 8.2 | 1.0 |
| Nickel | 0.038 | 51 | 8.2 | 1.0 |
| Zinc | 0.156 | 0.44 | 29.5 | 3.6 |
| SS | 87 | 0.50 | 18,600 | 2,270 |

*Assuming an annual runoff volume of 433,000 $m^3$ (351 ac-ft)

*Solids Accumulation and Removal*. About 18,600 kg (41,000 lbs) of SS would be collected annually in the retrofitted detention basin, which represents about one-half of the annual input of solids. Assuming a specific gravity of about 1.5 for saturated sediment collected in the basin, this would correspond to about 12.2 $m^3$ (16 cyd) of material annually. If uniformly distributed over the 1.78 ha (4.4 acre) basin, the mean annual accumulation rate would be 0.08 cm per year (0.03 in. per year). Sediments are expected to accumulate near the inlet and, in this specific case, in the pilot channel. In ten years, this accumulation rate would equal about 123.3 $m^3$ (0.1 ac-ft) compared with the capacity of the basin that is 37,000 $m^3$ (30 ac-ft). Therefore, this accumulation of sediments does not pose a risk to reducing the flood control capacity of the basin. Accumulation of at least 15.24 cm (6 in.) of sediment is required before removal is practical. This amount of sediment may take as long as 10 or 20 years to accumulate.

*Capital, Operation and Maintenance Costs*. Capital and O&M costs were estimated for the retrofitted pump station and are shown in Table 5.3.a-6. Costs were classified as capital expenditures, O&M, and disposal. Capital costs for the structural retrofitting were based on actual costs; whereas the costs for repair of the pump electronic control systems were estimated. Operations and maintenance assumes 100 hours per year labor in addition to that already being conducted to operate and maintain the facility for flood control. Disposal costs assume disposal is conducted every 10 years and include estimated future costs for landfill fees, trucking, and excavation. The total annualized cost is therefore estimated to be $8,200 for the 187.4 ha (463 acre) watershed, or about $7.3 per ha ($18 per acre) of watershed per year. The removal costs for copper were estimated to be about 0.5 kg per $1,000 (1.1 lbs per $1,000) that compares very favorably with other stormwater control alternatives, e.g. copper removal via street cleaning is approximately 0.7 kg per $1,000 (1.5 lbs per $1,000) (Pitt 1979, 1985, and 1987).

Table 5.3.a-6. Estimated Annualized Costs for Capital Expenditures and Operation

| CATEGORY | COST |
|---|---|
| 1. Capital Expenditures | |
| Structural retrofitting = $15,000 | |
| Amortized over 20 years at 8% | $1,500 |
| 2. Operations and Maintenance | |
| Inspection and repair (100 h @ $50/h) | $5,000 |
| 3. Disposal (every 10 years) | |
| Landfill (122 $m^3$ @ $38/$m^3$ | $8,000 |
| Trucking (16 trips @$75/h x 2 h/trip | $2,400 |
| Excavation (122 cyd @ $7.7/$m^3$ | $1,600 |
| Amortized over 10 years at 8% | $1,700 |
| 4. Total Cost per Year | $20,200 |

Conclusions

*Implications for Other Facilities.* According to an inventory conducted by Woodward-Clyde in 1990, nine of the existing 17 municipal pump stations in Santa Clara Valley are designed with detention basins (rather than sumps) and are, therefore, suitable for comparison with the pilot project. The detention basins range from 0.6—5.7 ha (1.5—14 ac.), with capacities of 5,550—182,556 $m^3$ (4.5—148 ac-ft). The watershed area for each pump station ranges from 10.1—405 ha (25—1,000 ac.) and the total watershed area served by all nine stations is 1,725 ha (4,260 ac.) or 17 $km^2$ (6.6 $mi^2$). This is about 2% of the 900 $km^2$ (350 $mi^2$) area of the Santa Clara Valley below the upland reservoirs. If we assume that other similar facilities could be retrofitted to achieve a performance comparable to that measured at Pump Station No. 2, the net reduction in copper load to the Bay would be about 45 kg (100 lbs). This is only about 1% of the estimated mean annual load of 6,350 kg (14,000 lbs) of copper entering San Francisco Bay.

- A 100-year flood analysis indicated that modification of the pump schedule to achieve water quality benefits did not increase the maximum 100-year elevation in the pond.
- Based on measured inlet and outlet flow composite concentrations from 6 storm events, the average pollutant removal efficiencies were: total chromium, 29%; total copper, 42%; total lead, 53%; total nickel, 51%; total zinc, 44%; and total SS, 50%.
- The removal efficiencies for chromium, copper, lead, nickel and zinc correlated well with SS removal, indicating that SS may be used as a surrogate parameter to monitor effectiveness of metals removal in detention basins.
- Metal concentrations of basin sediments were generally highest at the inlet location.

- None of the sediment concentrations exceeded the TTLC standard, indicating that the sediments are not hazardous.
- The estimated mean annual load reduction of metals ranged between 2.7—29.5 kg (0.6—65 lbs), depending on the metal. The mean annual load reduction for copper was 4.1 kg (9 lbs)
- The amortized annual capital and O&M cost for retrofitting the Sunnyvale Pump Station No. 2 is estimated at $8,200. The cost-effectiveness removal rate for copper is 0.5 kg/$1,000 (1.1 lb/$1,000).

- Solids accumulation rates are very low and are estimated to be approximately 123.3 $m^3$ (0.1 ac-ft) over 10 years. Given that the basin has a capacity of 37,000 $m^3$ (30 ac-ft), increased deposition caused by retrofitting does not increase flood risk.

Implications for Management.

- The total watershed area in Santa Clara Valley served by the nine pump stations with retention basins is approximately or 17 $km^2$ (6.6 $mi^2$) (only 2% of the total area of the Valley downstream of the reservoirs). Thus, even if an improved treatment performance could be obtained from these basins, the total load reduction to the Bay would be minimal. For example, the load reduction of copper would only be 45.4 kg (100 lbs), which is less than 1% of the estimated mean copper load to the Bay.
- Since pump stations are relatively easy to retrofit, water quality benefits could be achieved by simply changing the pumping schedule.
- If a retrofitting program is to be pursued, it would be important to ensure that the pump control equipment is operational and well maintained, and that staff are well trained in its use.

### *5.3.b. Monroe Street Detention Pond, Madison, Wisconsin*

Introduction and Site Description

This case study considers the benefits of retrofitting the outlet works of an existing wet detention pond. The original pond was creating severe downstream erosion in the channels between the pond and the receiving water, and the pond storage volume was not effectively being used for either flood control or water quality benefits. The outlets were modified and the pond has undergone extensive monitoring to confirm the water quality benefits of the retrofit.

The US Geological Survey (USGS), in cooperation with the Wisconsin Department of Natural Resources (WDNR) investigated the Monroe St. wet detention pond located in Madison, WI (House, *et al.* 1993). The University of Wisconsin originally constructed the pond to protect the water quality and ecology of Lake Wingra and surrounding wetlands from stormwater. The pond is located on the downstream side of Monroe Street at the

outlet of a storm sewer that drains a 0.96 $km^2$ (237 acre) urbanized area. Land use in the watershed area consists mostly of single-family residences and commercial strip development, with some institutional uses (i.e., schools and churches). The average basin slope is 2.2%.

The Monroe Street pond has a surface area of 5,670 $m^2$ (1.42 acre), a maximum depth of 2.3 m (7.5 ft) and an average depth of 1.1 m (3.6 ft) at normal pool elevation. The shape of the pond is round to oval with a small island. The inlet side is nearest to Monroe Street and the two outlets are on the far side away from Monroe Street. The pond has a surcharge storage volume above the normal pool elevation that is capable of holding the 10-year, 24 h storm-runoff volume without overtopping the containment berm around the pond. The pond has two outlets, each controlled by 90° V-notch weirs that drain to channels leading to Lake Wingra. The weirs are located in 2.4 m (8 ft) diameter concrete vaults, with 0.8 m (2.5 ft) concrete pipes leading to the pond. The initial primary outlet configuration consisted of two 2.4 m (8 ft) long rectangular weirs located in the vaults, made with concrete block walls. The original flow capacity of these two weirs was enormous, being about 1.4 $m^3/s$ (50 cfs) at 0.3 m (1 ft) head and 7 $m^3/s$ (250 cfs) at 0.9 m (3 ft) of head. The outlets in the pond are submerged and the bottom of the pond consists of a clay layer that inhibits infiltration of water from or into the pond.

As noted above, the discharges from the pond were little attenuated from the inflow velocities and severe channel erosion was occurring in the wetlands, negating the sediment trapping benefits of the pond. The emergency spillway appeared to never have been used since construction, even with several massive storms. In fact, the pond elevation barely fluctuated. The water quality benefits of the existing dry pond were assumed to be negligible.

The outlets were therefore modified in 1995 to reduce the downstream erosion problems by removing several courses of concrete blocks and installing 90° V-notch weirs made of plate steel in each vault. The pond normal water level was dropped about 15.3 cm (6 in.) with a lowered invert. The new primary outlets have total flow capacities of about 0.14 $m^3/s$ (5 cfs) at 0.3 m (1 ft) head and 2.3 $m^3/s$ (80 cfs) at 0.9 m (3 ft) head. The pond surface fluctuates more now, and the emergency spillway has been active every few years. Most significantly, the downstream channels are now stable.

The pond was designed for an expected 90% event mean concentration (EMC) removal for SS (particulate residue). The ratio of pond to drainage area is 0.6%. This percentage is close to the value (0.4—0.8%) required for 5 micron control, which generally corresponds to a 90% reduction of SS.

The Monroe Street pond may have more water quality monitoring data than any pond in the country. A total of 64 events were extensively monitored between February 1987 and April 1988. The monitored rains varied from 2 to more than 82 mm during this period. Periodic water quality and flow monitoring has also continued at this pond since 1988.

Method of Investigation

Water-quality data were collected by the U.S. Geological Survey (House, *et al.* 1993) using programmable automatic refrigerated water samplers installed at the inflow and outflow sites of the pond. The outflow data was collected at two locations, east and west. The samplers were programmed to obtain flow-proportional samples for each storm. These samples represent the flow-averaged constituent concentrations during a runoff event. These samples were removed from the samplers, preserved, and shipped to the Denver USGS laboratory for analysis within 24 h of being collected. The samples were analyzed for SS, volatile total solids, total and dissolved chemical oxygen demand (COD), total chloride, total and dissolved phosphorus, phosphate total and dissolved forms of TKN, nitrates, and total and dissolved forms of copper, zinc, and lead.

Precipitation data were also recorded at 5 min intervals during the storm events using a recording rain gage located at the pond site. Storm runoff (pond inflow) was monitored at the box culvert that was the terminus of the 0.96 $km^2$ drainage area. Discharge rates and flow volumes passing through the culvert were determined by use of a flow velocity sensor and water level indicator installed inside the culvert. The velocity and depth sensors were connected to a data logger that recorded the water level and velocity data and computed discharge rates based on the culvert geometry.

Data Analysis and Observations

The pond inlet and outlet pollutant concentrations were analyzed to determine the pollutant reduction. Various relationships between inlet and outlet concentrations were investigated by statistical analysis, including Particulate Pollutant Strength (PPS) and percent controls. Each statistical process is described in the following paragraphs. The basic data are contained in the USGS report. (House, *et al.* 1993).

*Hydrograph/Flow Calibration*. An important part of the Monroe Street project was validating the DETPOND wet detention pond water quality model that was used to design the retrofit of the outlet structures (Pitt and Voorhees 1995). The first step in the validation was to check flow volumes and peak flow rates, and the complete hydrographs.

Fifteen storm events were used to validate the flow portions of the DETPOND program. The program predicted outflow flow values from the inflow hydrographs using the modified pulse routing method. The outfall predictions (at five-minute intervals) were compared to the observed outfall flow values. The predicted outflow hydrographs very closely matched the corresponding observed outflow hydrographs. In addition to comparing the general shape of the discharge hydrographs, the outflow total discharge volume, peak discharge flow rate, SS removal, and outflow particle size distribution were also compared for validation. The predicted outflow volumes and peak discharges very closely matched the observed outflow conditions.

*Observed Influent and Effluent Pollutant Concentrations*. Table 5.3.b-1 lists the influent and effluent conditions observed at the pond. The number of observations, and the mean, maximum, and average concentrations are shown. The standard deviation (Std.

Dev.) and coefficient of variation (COV) are given. The Mann Whitney $\alpha$ values, which indicates whether the influent and effluent values are statistically significantly different, are also summarized.

Table 5.3.b-1. Summary of Observed Influent and Effluent Pollutant Concentrations at Monroe St. Pond

| RESIDUE | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Number of Observations | Minimum | Max. | Mean | Std. Dev. | COV * | Mann Whitney |
| Total Solid (mg/L) | | | | | | | |
| IN | 64 | 76 | 5810 | 526 | 816 | 1.55 | 0.0003 |
| OUT | 57 | 63 | 1180 | 276 | 259 | 0.94 | |
| Suspended Solid (mg/L) | | | | | | | |
| IN | 64 | 21 | 1330 | 230 | 257 | 1.11 | 0.0000 |
| OUT | 57 | 4 | 140 | 32 | 28 | 0.87 | |
| Filtered Residue (mg/L) | | | | | | | |
| IN | 64 | | | 303 | 789 | 2.6 | 0.0758 |
| OUT | 57 | | | 244 | 244 | 1.0 | |
| Volatile Residue (mg/L) | | | | | | | |
| IN | 62 | 28 | 376 | 92 | 63 | 0.68 | 0.0000 |
| OUT | 55 | 0 | 232 | 48 | 30 | 0.63 | |
| Total Chlorides | | | | | | | |
| | Number of Observations | Minimum | Max. | Mean | Std. Dev. | COV | Mann Whitney |
| IN | 65 | 0.8 | 3100 | 120 | 430 | 3.58 | .0032 |
| OUT | 57 | 2.1 | 570 | 90.9 | 135.5 | 1.49 | |
| COD | | | | | | | |
| Total COD, (mg/L) | | | | | | | |
| IN | 63 | 21 | 370 | 90 | 63.4 | 0.70 | 0.000 |
| OUT | 57 | 16 | 350 | 37 | 44 | 1.2 | |
| Particulate COD, (mg/L) | | | | | | | |
| IN | 56 | 8 | 327 | 53 | 53 | 1.0 | 0.000 |
| OUT | 52 | -3 | 281 | 16 | 38.5 | 2.4 | |
| Filtered COD, (mg/L) | | | | | | | |
| IN | 56 | 10 | 160 | 38 | 32.3 | 0.85 | 0.0019 |
| OUT | 52 | 9 | 69 | 21.4 | 11 | 0.51 | |

* Coefficient of Variation – standard deviation divided by the mean.

Table 5.3.b-1. Summary of Observed Influent and Effluent Pollutant Concentrations at Monroe St. Pond, Continued

| RESIDUE | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Number of Observations | Minimum | Max. | Mean | Std. Dev. | COV | Mann Whitney |
| PHOSPHORUS | | | | | | | |
| Total Phosphorus, (mg/L) | | | | | | | |
| IN | 65 | 0.16 | 1.7 | 0.56 | 0.39 | 0.69 | 0.000 |
| OUT | 59 | 0.13 | 1.6 | 0.26 | 0.19 | 0.73 | |
| Particulate Phosphorus, (mg/L) | | | | | | | |
| IN | 60 | 0.08 | 1.53 | 0.33 | 0.27 | 0.82 | 0.000 |
| OUT | 53 | 0.04 | 1.52 | 0.16 | 0.2 | 1.25 | |
| Filtered Phosphorus, (mg/L) | | | | | | | |
| IN | 60 | 0.06 | 1.33 | 0.23 | 0.28 | 1.2 | 0.0001 |
| OUT | 53 | 0.02 | 0.27 | 0.1 | 0.06 | 0.6 | |
| PHOSPHATES | | | | | | | |
| Total Phosphates, (mg/L) | | | | | | | |
| IN | 56 | 0.026 | 1.22 | 0.2 | 0.27 | 1.35 | 0.0002 |
| OUT | 51 | 0.004 | 0.25 | 0.08 | 0.06 | 0.75 | |
| COPPER | | | | | | | |
| Total Copper, (µg/L) | | | | | | | |
| IN | 64 | 10 | 130 | 50 | 14 | 0.28 | 0.592 |
| OUT | 57 | 3.5 | 60 | 46 | 12.43 | 0.27 | |
| Particulate Copper, (ug/L) | | | | | | | |
| IN | 60 | 7 | 120 | 43 | 13.93 | 0.32 | 0.0652 |
| OUT | 52 | .5 | 49 | 42.3 | 12.02 | 0.28 | |
| Filtered Copper, (ug/L) | | | | | | | |
| IN | 60 | 3 | 16 | 6.35 | 3.32 | 0.52 | 0.0035 |
| OUT | 53 | 3 | 13.5 | 4.5 | 2.17 | 0.48 | |
| LEAD | | | | | | | |
| Total Lead, (ug/L) | | | | | | | |
| IN | 64 | 18 | 420 | 85 | 52.4 | 0.61 | 0.100 |
| OUT | 57 | 4 | 100 | 68 | 39 | 0.57 | |
| Particulate Lead, (ug/L) | | | | | | | |
| IN | 59 | 15 | 417 | 83 | 54 | 0.65 | 0.1800 |
| OUT | 52 | 1 | 97 | 66 | 39 | 0.59 | |

Table 5.3.b-1. Summary of Observed Influent and Effluent Pollutant Concentrations at Monroe St. Pond, Continued

| RESIDUE | | | | | | | |
|---|---|---|---|---|---|---|---|
| | Number of Observations | Minimum | Max. | Mean | Std. Dev. | COV | Mann Whitney |
| Filtered Lead, (ug/L) | | | | | | | |
| IN | 59 | 3 | 12 | 3.5 | 1.7 | 0.49 | 0.54 |
| OUT | 53 | 3 | 4 | 3.0 | 0.17 | 0.06 | |
| ZINC | | | | | | | |
| Total Zinc, (ug/L) | | | | | | | |
| IN | 64 | 35 | 1100 | 152 | 136 | 0.89 | 0.0450 |
| OUT | 57 | 30 | 1400 | 191 | 265 | 1.39 | |
| Particulate Zinc, (ug/L) | | | | | | | |
| IN | 58 | -3.0 | 980 | 103.5 | 131 | 1.26 | 0.0000 |
| OUT | 47 | -15.5 | 630 | 60.3 | 101 | 1.68 | |
| Filtered Zinc, (ug/L) | | | | | | | |
| IN | 57 | 9.0 | 160.0 | 51 | 34 | 0.66 | 0.0020 |
| OUT | 48 | 20.0 | 1218 | 150 | 215 | 1.43 | |

The coefficient of variation (COV) is calculated by dividing the standard deviation by the mean and normalizes the standard deviation. A high value indicates that the data spread is wide, requiring many data observations to obtain a precise estimate of the EMC.

COV varied from 3.58 for inlet chlorides, which may not be significant due to seasonal variations, to 0.06 for outlet filtered lead. All filtered lead had lower COV values which indicated small concentration variations for samples.

The Mann-Whitney test is a non-parametric analysis comparing two sets of data. The null hypothesis used in the Mann-Whitney is inlet pollutant concentration minus outlet pollutant concentration equals zero. Generally, an $\alpha$ value < 0.05 indicates that the sample sets are from two different populations (significantly different at the 95% confidence level). The following constituents had significant $\alpha$ values indicating that the pond significantly affected the concentrations:

Total solids
Suspended solids
Volatile solids
Chlorides
COD (all forms)
Phosphorus (all forms)
Phosphate
TKN (total and particulate)
Nitrate
Copper (filtered)
Zinc (all forms)

*Particulate Pollutant Strength.* PPS is the ratio of a particulate pollutant concentration to the suspended solid concentration expressed in mg/kg. PPS was calculated for each pollutant with a particulate form. The small particles in stormwater have greater PPS values than the large particles. Therefore, all pollutants had higher outlet than inlet PPS values due to preferential removals of large particles in the detention pond, leaving relatively more small particles in the discharge water. A wide difference between influent and effluent PPS values indicates that the main components of the contaminant (such as for TKN and phosphorus) are associated with fine particles.

*Control of Pollutants.* The reduction of pollutants was calculated from the difference in pollutant concentrations in the inlet and outlet water for each event, as shown on Table 5.3.b-2. As expected, control was higher for all particulate forms of the constituents than for filtered forms, except for copper. Filtered constituents (<0.45 μm) have characteristics like colloidal particles and will tend to be transported through the wet detention pond relatively unchanged. A well designed wet detention pond will remove 70—90% of SS, 70% of COD, 60—70% of nutrients and 60—95% of the particulate forms of the heavy metals.

Table 5.3.b-2. Summary Table of Pollutant Control*

| **Parameter** | **10%** | **50%** | **90%** |
|---|---|---|---|
| Suspended solids | 35 | 87 | 97 |
| Total Residue | <0 | 52 | 86 |
| Volatile Residue | <0 | 41 | 76 |
| Filtered Residue | <0 | <0 | 56 |
| | | | |
| Particulate COD | 15 | 80 | 95 |
| Total COD | 29 | 60 | 84 |
| Filtered COD | <0 | 24 | 80 |
| | | | |
| Particulate Phosphorus | -20 | 60 | 80 |
| Total Phosphorus | <0 | 47 | 81 |
| Filtered Phosphorus | <0 | 43 | 83 |
| | | | |
| Particulate TKN | -40 | 40 | 80 |
| Total TKN | <0 | 45 | 75 |
| Filtered TKN | <0 | 12 | 68 |
| | | | |
| Particulate Zinc | - 117 | 70 | 95 |
| Total Zinc | <0 | 31 | 69 |
| Filtered Zinc | <0 | <0 | 59 |

* Copper and Lead observations were mostly below the detection limits and are therefore not shown.

Particle Size Distributions and Short-Circuiting

Seven events were studied to find the short-circuiting “n” factors using observed and predicted particle size distributions in effluent water. Particle size distributions were measured using the Sedigraph method at the USGS Denver laboratory. This technique measures settling rates of different size suspended solid particulates down to 2 microns. The value of n is calculated using the concentrations of large particles that are found in the effluent. In ideal settling, no particles greater than the theoretical critical size (about 5 μm for Monroe Street) should appear in the effluent. However, there is always a small number of these larger particles in the effluent. It is generally assumed that short-circuiting is responsible for these large particles. The measured values for n were one, or less, indicating a high degree of short-circuiting in the pond. However, these observations were possibly affected by scour of bottom deposits near the subsurface effluent pipes. The maximum effect of short-circuiting on pond performance is shown in Table 5.3.b-3, showing the average reduction in SS removals for different n values, compared to the best performance (n value equal to 8):

Table 5.3.b-3. Average Suspended Solids Removal for variable n values for the Monroe St. Ponds

| n value | % SS removal (average) | Reduction in % SS removal compared to n=8 |
|---|---|---|
| 8 | 85 | |
| 3 | 84 | 1 |
| 1 | 80.7 | 4.3 |
| 0.5 | 78.5 | 6.5 |
| 0.2 | 59 | 26 |

The median value of n observed was about 0.35, indicating a reduction in SS capture efficiency of no more than about 10%. The effects of this short-circuiting, has a minimal effect on the SS percentage removals. The Monroe Street pond provided an average SS reduction of 87%, compared to the design goal of 90%. The small difference (3%) between actual and design SS reduction indicates that the short-circuiting has a negligible effect on actual performance.

Monroe St. Pond Retrofitting Conclusions

The retrofitting of the Monroe Street wet detention pond was very successful. Changing the outlet structures from large rectangular weirs to V-notch weirs significantly reduced effluent flows, reduced downstream channel erosion and improved water quality benefits of the pond.

All effluent concentrations were lower than associated influent concentrations, except for chlorides. The measured short-circuiting factor indicated a severe short-circuiting problem, but that could be a false indication due to minor scour near the effluent works

in the pond. Data suggest that short circuiting does not adversely impact treatment efficiency. The Monroe Street pond is meeting all reasonable expectations in both downstream channel protection and in contaminant capture.

### *5.3.c. Birmingham, Alabama Dry-Ponds*

This retrofit case study examines a dry detention pond at a new apartment complex in Birmingham, Alabama. This is an example of a typical small pond installed to meet local regulations for peak flow control. The pond is not even expected to operate satisfactory even as a peak flow rate pond based on review of the current hydrological assessment. This analysis makes suggestions for retrofitting this pond to a wet pond and to improve the overall benefits of the device. It also demonstrates how a combination of water quality and flow analysis computer programs can be used together to evaluate and design effective multiple benefit SW controls. The water quality benefits of the existing dry pond were assumed to be negligible for this analysis.

Redesign of Dry Pond for Water Quality Benefits

There are a multitude of options for retrofitting the existing dry pond: excavation below the outlet, raising the outlet, or some combination of the two. However, since relocating and/or reconstructing the existing outlet structures to gain additional water volume below the outlet would be quite expensive we will only consider excavating below the existing outlet located at 219.5 m (720 ft) mean sea level (msl). This redesign will only require excavation within the existing space with a minimum of re-grading.

There are several factors to consider when designing a wet pond, among these are depth, safety, and storage criteria. Pitt (1998) recommends 0.9—1.8 m (3—6 ft) of storage below the invert of the lowest outlet, 219.5 m (720 ft). In addition, safety requires a shallow ledge, submerged 0.3 m (1 ft) below the permanent storage area. The redesign presented here will include the preferred depth guideline of 1.8 m (6 ft), requiring excavation from 219.5—217.6 m (720—714 ft), and a minimal three-foot ledge at 219.2 m (719 ft) to match existing grades. The side slope will be 1:1 from 219.5 m (720 ft) to the ledge at 219.2 m (719 ft), and also 1:1 from the interior of the ledge to 217.6 m (714 ft). Assuming a prismatic cross-section, the additional wet storage to be constructed below the 219.5 m (720 ft) of elevation is approximately 250 $m^3$ (0.204 ac-ft).

*Safety Criteria*. Pitt (1998) recommends a gradual slope, 1:1 to 1:4, on the approach to the pond's normal high water mark, 221.6 m (727 ft) in this case. The existing pond may be dangerous, as the side slope is approximately 1:2 and quite long about 23 m (75 ft). As an additional safeguard, a barrier should be constructed to prevent access to the pond, preferably some type of barrier vegetation. This apartment complex is a new development with the majority of the construction occurring in 1997. Therefore, some of the landscaping is not complete as yet. There are some shrubs and small trees planted around the perimeter, however these do not completely surround the pond. In fact, the western slope, which is the longest and steepest approach to the pond, is unfortunately the area that does not have barrier vegetation. Since the stormwater runoff is not

planned for water contact activities because of high levels of pollutants, e.g. bacteria, some proactive effort should be made to discourage the public from making contact with the water.

*Peak Reduction Factors (PRF)*. The dry pond barely reduces the peak outflow rates from the contributing area. The 100-year storm's runoff rate is expected to be reduced from 4.3 $m^3/s$ (153 cfs) to about 4.1 $m^3/s$ (145 cfs) by constriction at the pond's outlet structures and by storage in the pond.

$$PRF = 1 - Q_o/Q_i. \quad (10)$$

Where: $Q_o$ = effluent
$Q_i$ = influent

The PRF for this event is only 0.05, corresponding to a 5% reduction of the inflow hydrograph in the pond. For the 50-year and 25-year storms, the PRFs are 0.06 and 0.07, respectively. This pond is likely inadequate for any measurable peak flow reduction. The pond also likely exceeds the maximum stage of 233.4 m (733 ft). This is especially a problem because the water coming through the spillway flows directly onto a busy street with high speed traffic.

*Upflow and Critical Settling Velocities*. The most important rain events to control for water quality improvement are the small events that occur with great frequency, or those that produce less than approximately 3.18 cm (1.25 in.) of runoff. For the smaller rain events, the DETPOND simulations demonstrated that the retrofitted pond would satisfy the maximum up flow velocity (or critical settling velocity) maximum of 0.0004 mm/s (0.00013 fps) which is necessary for 5 micron particle control. Analyses also indicated that removal of 90% of SS was easily obtained on all but the largest rain events. The analyses using DETPOND and the 1976 Birmingham rain file (a typical year) showed that the annual average suspended control was 86%.

*Pond's Water Quality Storage*. A reasonable goal is to have the pond's water quality storage be equal to the runoff associated with 3.18 cm (1.25 in.) of rain for the land use, development, and cover of the watershed served by the pond (Pitt 1998). The composite CN for the watershed was estimated to be 75. This corresponds to approximately 1 cm (0.40 in) of runoff, from a storm of 3.18 cm (1.25 in.) (Pitt 1987). Therefore the minimum active pond storage (between the invert elevation of the lowest outlet and the secondary outlet discharge devices) required should be a least 1727 $m^3$ (1.4 ac-ft). Since the retrofitted pond's water quality storage is about 826 $m^3$ (0.67 ac-ft), this requirement is not met.

*Pond's Surface Area Requirements*. A pond's surface area should be sized as a percent of watershed's area based on land use, development, cover, and the particle size control desired. The apartment site has residential and woodland land uses. Pitt (1998) recommends 0.8% of the watershed area for 5 micron particle control in a commercial area (based on NURP, National Urban Runoff Program, studies). Therefore the

minimum surface area of the pond should be at least 0.13 ha (0.33 ac.), which is close to the normal water surface area of the retrofitted pond of .125 ha (0.31 ac.).

*Possible Problems*. Experience and observations of similar ponds in this region of the country have shown that the shallow areas near the shore of the retrofitted pond might become a breeding area for mosquitoes. The shallow shoreline might allow too much aquatic vegetation to grow into the pond's interior area causing congestion and periodically dying and subsequently increasing the BOD. In addition, the aquatic plants located on the shallow shelf could grow in and around the outlet and reduce its output capacity. Reptiles (including snakes), waterfowl, rodents (i.e., muskrats) might become too plentiful in the pond area and become nuisances. Decaying vegetation may result in strong odors in the summer, and these odors might have a negative impact on residents.

Background Information Related to Site Evaluation

The peak inflow hydrograph values were determined by HydroCAD's soil conservation service (SCS) TR-20 methodology. For the site, a SCS Type III rainfall intensity distribution frequency (IDF) curve was selected. Rainfall depths for the 100-year, 50-year, and 25-year storms were approximately 22 cm (8.6 in.), 20 cm (7.8 in.), and 18 cm (7.1 in.) respectively. Time of concentration for the watershed was also calculated using HydroCAD's built-in TR-20 methods; Tc = 24.3 min for the apartment complex area, and Tc = 33.8 for the Woodland area.

*Land Use, Development, Cover, Soils Type, and CNs.* SCS soil maps for the area were examined, and it was determined that the site consisted of Nauvoo-Sunlight complex, with 15—25% slopes, and Townley silt loam, with 12—18% slopes. The SCS Hydrologic Soil Groups for these soils are type B and type C respectively. Research conducted at the University of Alabama Birmingham has shown that development, due to construction disturbances, compaction, and soil mixing, can significantly reduce the actual infiltration rates from those assumed. Therefore, the CN assigned to the developed area was for the worst case, type D soil. However, the undeveloped woodland area, mostly Nauvoo soil, was assigned a CN based upon the type B soil. Therefore for the developed area of 10.5 ha (26.4 ac.), a composite CN of 87 was assigned. This composite CN of 87 is a weighted average at the sub-basin CNs. The basins include 6.5 ha (16 ac.) of residential land use with one-eighth-acre lots, SCS soil type D, and 65% impervious cover (CN = 92), plus the remaining 4.2 ha (10.4 ac.) of open lawns with good grass cover (> 75%), and type D soil (CN = 80). For the woodland area of 6.1 ha (15 ac.), a CN of 55 was assigned, corresponding to woods with good hydrologic condition and type B soils.

Analysis of Design Storms

*HydroCAD™*. The HydroCAD Stormwater Modeling System (version 4.53) was used to analyze the pond for hydraulic characteristics. This computer software allows for calculating hydrographs, based upon design storm and watershed characteristics, and then routing these through a drainage system composed of subcatchments, reaches,

and ponds. The program does not consider the dead storage below the first outlet, assuming that this is always full of water, therefore the flow characteristics of both the existing pond and the retrofitted pond are identical.

The subcatchment component is used to model a given drainage area or watershed. In this case, there are two subcatchments: subcatchment 1 refers to the 10.5 ha (26.4 ac.) of the apartment complex, and subcatchment 2 consists of the 6.1 ha (15 ac.) of woodland area that drains to the complex. The program uses built-in SCS TR-20 hydrology methods for determining the hydrograph characteristics. Next, the hydrograph is routed through a series of defined reaches and/or ponds. In this case, there are two hydrographs, one from each of the two subcatchments, which are routed through a single pond.

The pond component of this model is described using a stage v. surface area curve, shown in Figure 5.3.c-1 and Table 5.3.c-1. In addition, the model requires descriptions of the outlet structures.

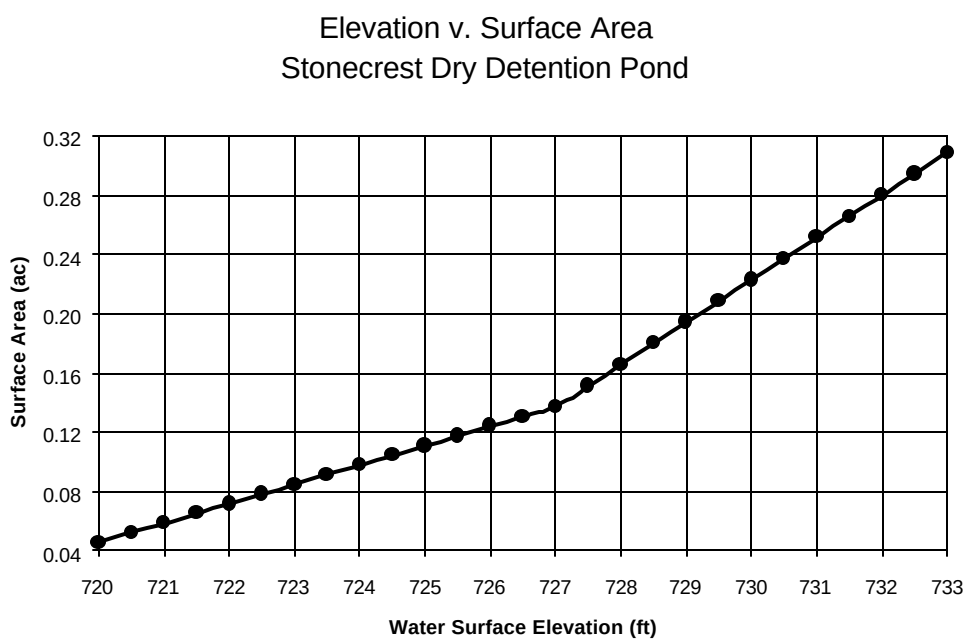



Figure 5.3.c-1. Stage v. Surface Area Curve.

Table 5.3.c-1. Outlet Device Descriptions

| # | Route | Invert | Outlet Device |
|---|---|---|---|
| 1 | Primary | 219.5 m (720 ft) | 61 cm (24 in) culvert<br>n=0.013, length=56.4 m (185 ft), slope=0.02 m/m, Ke=0.5, Cc=0.9 |
| 2 | to #1 | 219.5 m (720 ft) | 30.5 cm (12 in) orifice |
| 3 | to #1 | 221.6 m (727 ft) | sharp-crested rectangular weir<br>length = 3.7 m (12 ft)<br>height = 20.3 cm (8 in)<br>(square concrete box with cap) |
| 4 | Secondary | 223.1 m (732 ft) | 3 m (10 ft) broad-crested rectangular weir emergency spillway |

Ke = 0.5 represents the entrance loss for a flush entrance
Cc = 0.9 represents the coefficient of discharge for the culvert

The HydroCAD simulations were run for three, 24 h, SCS Type III design storm frequencies: 25-year 18 cm (7.1 in.), 50-year 20 cm (7.8 in.), and 100-year 22 cm (8.6 in.). Table 5.3.c-2 summarizes the model's output for these three storms, showing the hydrographs peaks and volumes. As expected, an examination of the last two columns shows that the most significant contribution to the hydrograph flowing into the pond comes from the apartment complex area. The surface area of the complex is larger than that of the woodlands and land use, soil characteristics, and other hydrologic characteristics of the complex all contribute to generating much more and rapid runoff per unit area.

Table 5.3.c-2. Subcatchment Summaries for Design Storms

| Subcatch-ment No. | Description | Design Storm Frequency (yr) | Rainfall (cm) | Rainfall (in) | Peak Flow ($m^3/s$) | Peak Flow (cfs) | Volume ($m^3$) | Volume (ac-ft) |
|---|---|---|---|---|---|---|---|---|
| 1 | apartment complex | 25 | 18 | 7.1 | 2.91 | 102.6 | 13,310 | 10.79 |
| 2 | woodland | 25 | 18 | 7.1 | 0.54 | 19.13 | 3,035 | 2.46 |
| 1 | apartment complex | 50 | 20 | 7.8 | 3.24 | 114.4 | 14,850 | 12.04 |
| 2 | woodland | 50 | 20 | 7.8 | 0.67 | 23.59 | 3,690 | 2.99 |
| 1 | apartment complex | 100 | 22 | 8.6 | 3.62 | 127.9 | 16,615 | 13.47 |
| 2 | woodland | 100 | 22 | 8.6 | 0.82 | 28.94 | 4,490 | 3.64 |

Next the hydrographs were routed through the pond. Table 5.3.c-3 summarizes the results. As previously mentioned, the peak reductions are quite low (5—7%), and the peak discharge lag is only a few minutes. The predicted peak water surface elevation in the pond is higher than the maximum elevation in the pond, 223 m (733 ft) (column 3),

indicating flooding. This is a dangerous situation because it means that the water is flowing uncontrollably over the dam. This could damage the emergency spill way and cause erosion of the dam itself. These events occur even at the lowest storm frequency modeled 25-year. It would appear that the pond is inadequate for the amount of runoff generated by these storms.

Table 5.3.c-3. Pond Results of HydroCAD simulations

| Design Event* | Peak Elev.** | | Peak Storage | | Peak $Q_{in}$ | | Peak $Q_{out}$ | | Peak $Q_{outlet}$† | | Peak $Q_{emerg}$‡ | | Atten. | Lag Time |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | (m) | (ft) | $m^3$ | ac-ft. | $m^3/s$ | cfs | $m^3/s$ | cfs | $m^3/s$ | cfs | $m^3/s$ | cfs | % | (min) |
| 25-yr. | 233.5 | 733.4 | 2,603 | 2.11 | 36.1 | 118.3 | 33.6 | 110.2 | 15.4 | 50.5 | 18.2 | 59.7 | 7 | 8.1 |
| 50-yr. | 233.7 | 733.8 | 2,726 | 2.21 | 40.9 | 134.2 | 38.4 | 126.1 | 15.6 | 51 | 22.9 | 75.2 | 6 | 7.2 |
| 100-yr. | 223.8 | 734.1 | 2,850 | 2.31 | 46.5 | 152.6 | 44.2 | 144.9 | 15.7 | 51.6 | 28.5 | 93.4 | 5 | 5.8 |

* Design storms are type III 24 h for Shelby County (SCS methods).

** Flood elevation is at 223.4 m (733 ft).

† Peak flow through the first and second outlets to a 0.6 m (2 ft) diameter culvert.

‡ Peak flow in the emergency spillway (flowing onto Bowling Drive).

*DETPOND Modeling.* DETPOND generally uses a simplified triangular hydrograph for individual storms in long-term simulations (several decades) for water quality evaluations, so this model is not suitable for modeling synthetic rainfall distributions, such as those appropriate for design storms. However, HydroCAD is capable of these types of simulations, and one can input the hydrograph generated by HydroCAD into DETPOND using the “user defined hydrograph” option. Therefore, the 25-year, 50-year, and 100-year hydrographs generated using HydroCAD’s TR-20 methods were input, and these results are presented in the Table 5.3.c-4. A comparison of the hydraulic results from HydroCAD (Table 5.3.c-4) shows that these results are similar. For instance during the 25-year event, HydroCAD calculated a maximum flow rate into the pond of 36.1 $m^3/s$ (118.3 cfs), while DETPOND’s calculations resulted in 36.0 $m^3/s$ (118.0 cfs). Even under these worst-case conditions, the pond is removing approximately 50% of the SS.

Table 5.3.c-4. DETPOND Summary for Design Storms

| Design Event | Max. Stage (m) | Max. Stage (ft) | Max. Inflow ($m^3/s$) | Max. Inflow (cfs) | Max. Outflow ($m^3/s$) | Max. Outflow (cfs) | Max particle size discharged (μm) | Avg. Min Particle Size Controlled (μm) | % SS Removed |
|---|---|---|---|---|---|---|---|---|---|
| 25 yr | 223.6 | 733.5 | 36.0 | 118.0 | 30.5 | 100.1 | 95 | 26.5 | 50.8 |
| 50 yr | 223.7 | 734.0 | 40.9 | 134.2 | 36.0 | 118.0 | 95 | 28.1 | 48.6 |
| 100 yr | 223.9 | 734.5 | 46.5 | 152.6 | 41.6 | 136.6 | 95 | 29.7 | 46.7 |

## Analysis Using True Rainfall Characteristics

DETPOND evaluates long-term continuous simulations using many years of actual rainfall data. Rain files are created by determining the rain depth, rain duration, and interevent duration for multiple storms. DETPOND then routes a simple, triangular hydrograph for each event, and routes it through the pond to evaluate the SS removal. DETPOND simulations were conducted using rain files created from the 1976 monitoring data, and also the 1952 through 1989 rain record. The 1976 file is representative of a typical year, and was used to enable more model runs because of shorter run time. The larger file was then run to continuously evaluate the pond based on almost forty years of actual storm data. There are only 23 events, out of a total of 4,107 in the large 40 year rain file in which the pond stage rises to the level of the second outlet. All of the DETPOND simulations were conducted using the retrofitted pond, as water quality benefits in the existing dry pond were assumed to be negligible. The results of the simulations using the 1976 file are presented in Table 5.3.c-5.

Table 5.3.c-5. Water Quality Output Summary for 1976 Rain File

| Statistic | Rain Depth (cm) | Rain Depth (in) | Rain Duration (h) | Inter-Event Duration (ds) | Rain Intensity (cm/h) | Rain Intensity (in/h) | Max Pond Stage (m) | Max Pond Stage (ft) | Flow-weighted Particle Size | Approx. Part. Res. Control* | Peak Reduction Factor | Event Flushing Ratio** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mean | 1.27 | 0.50 | 12.01 | 1.81 | 0.10 | 0.04 | 1.94 | 6.30 | 4.26 | 86.3% | 0.07 | 1.75 |
| Std. Dv. | 1.91 | 0.75 | 10.77 | 2.36 | 0.15 | 0.06 | 0.16 | 0.51 | 4.23 | 13.5% | 0.07 | 2.41 |
| COV | 3.84 | 1.51 | 0.90 | 1.30 | 3.76 | 1.48 | 0.03 | 0.08 | 0.99 | 0.2% | 1.00 | 1.37 |
| Minimum | 0.03 | 0.01 | 1.00 | 0.00 | 0.00 | 0.00 | 1.84 | 6.00 | 0.00 | 57.3% | 0.01 | 0.00 |
| Max. | 9.75 | 3.84 | 45 | 11.68 | 0.79 | 0.31 | 2.73 | 8.84 | 15.70 | 100% | 0.31 | 9.34 |

* Approximate Particle Residue Control (SS).

** Event flushing ratio – the storm runoff volume divided by the water volume contained in the pond at the beginning of the event.

The control for the 5 $\mu$m particles corresponds to approximately 90% SS control based on the mean values of the columns of Flow Weighted Particle Size and Approximate

Particle Control. On average, in a typical year, the pond will collect particle sizes 4.26 microns and greater in size. The average rain depth is 1.27 cm (0.5 in.), and the average duration is 12 h. For the smallest storms, the pond is achieving greater than 95% control, and for the largest storm in 1976, the SS control is still 57%.

Figure 5.3.c-2 shows the maximum pond stage. Axis labels denote the elevation above the pond bottom (1.8 m [6 ft] corresponds to 219.5 m [720 ft] msl elevation, the invert of the first outlet device) versus the percent particle control. There is an expected trend; the control decreases with increasing stage.

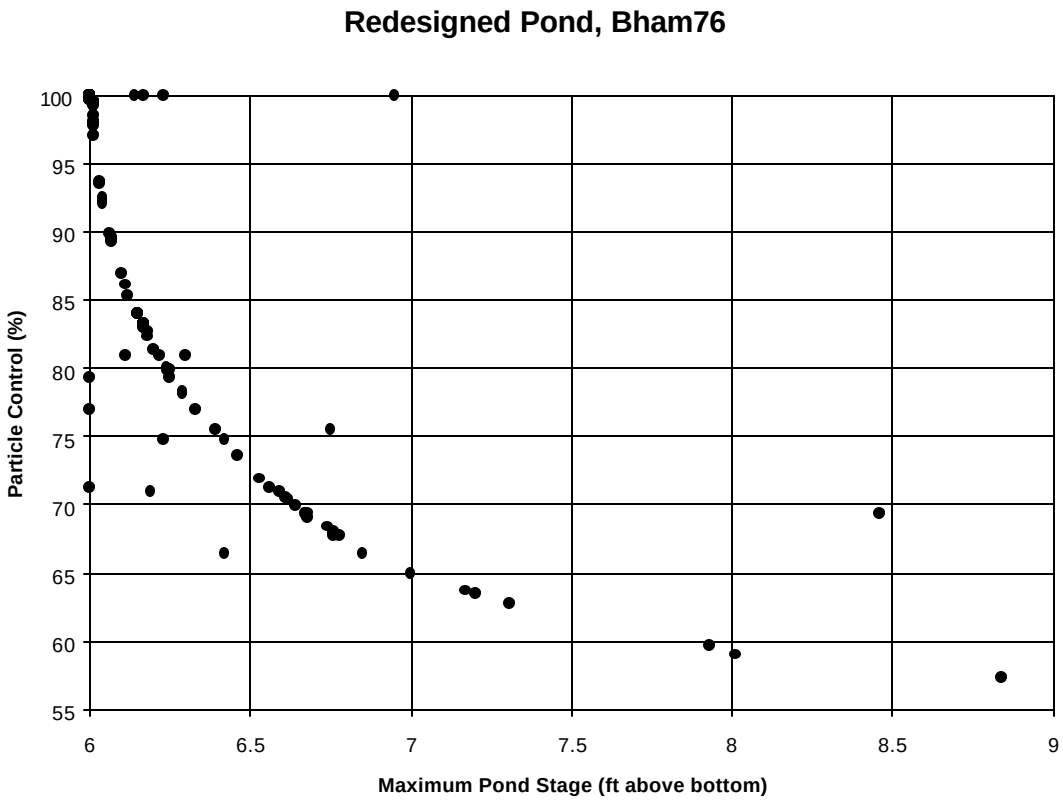



Figure 5.3.c-2. Pond Stage v. Particle Residue Control

Figure 5.3.c-3 shows the water quality performance of the retrofitted pond (percent particulate control) versus the rain depth in inches. There is considerably more scatter in this plot, but one can still observe a trend. Generally, control decreases as rain depth increases, as expected. The results are similar for Figure 5.3.c-4 depicting control versus rain intensity.

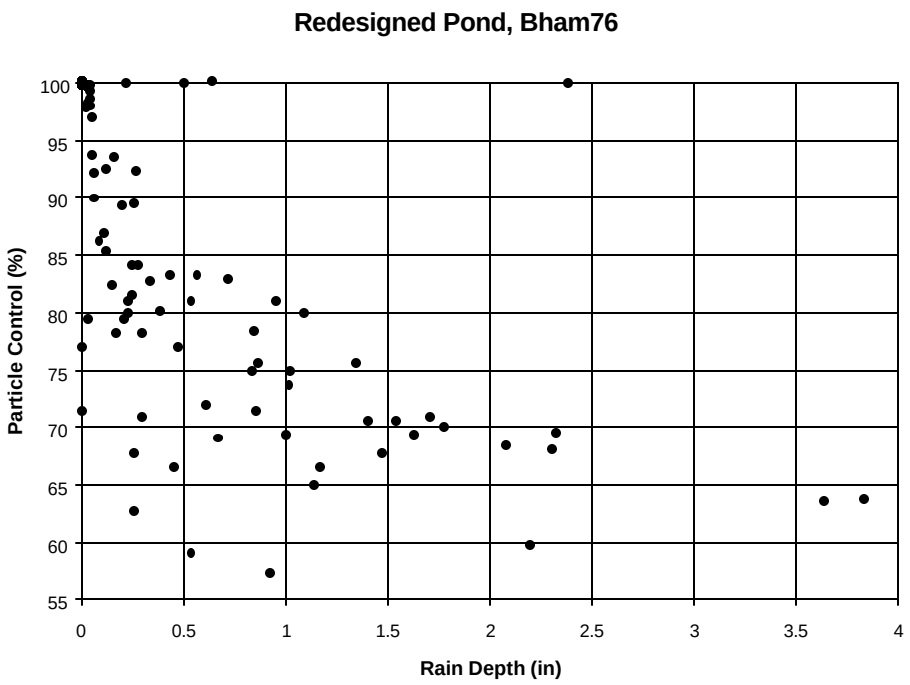



Figure 5.3.c-3. Rain Depth v. Particle Residue Control

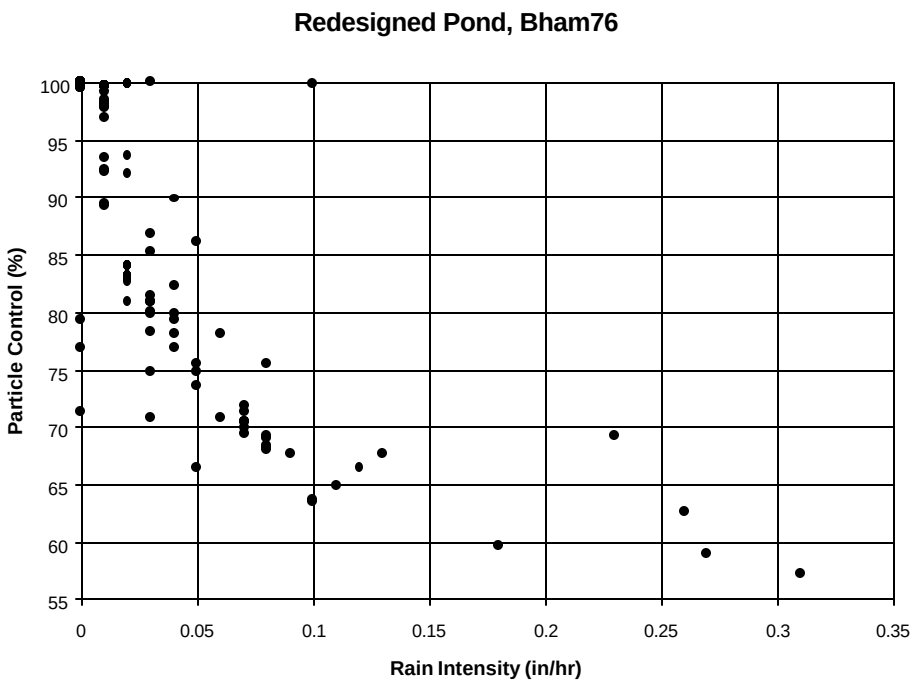



Figure 5.3.c-4. Rain Intensity v. Particle Residue Control

## Long-term Simulation of 1952-1989 Rains

Table 5.3.c-6 shows the statistics for the 4,107 rain events that occurred in Birmingham from 1952-1989. Notice that the minimum and maximum values are different than those from the 1976 simulations, but the mean values are quite similar. The most important value in this table is the mean particle control, 80%. For 1976, the control was approximately 86%. This is not significantly different. With close to forty years of rains, the redesigned pond still averages 80% SS removal.

Table 5.3.c-6. Water Quality Output Summary for 1952—1989 Rain File

| Statistic | Rain Depth (cm) | Rain Depth (in) | Rain Duration (h) | Inter-Event Duration (ds) | Rain Intensity (cm/h) | Rain Intensity (in/h) | Max Pond Stage (m) | Max Pond Stage (ft) | Flow-weighted Particle Size | Approx. Part. Res. Control* (%) | Peak Reduction Factor | Event Flushing Ratio** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mean | 1.27 | 0.50 | 6.31 | 2.57 | 0.10 | 0.04 | 1.94 | 6.30 | 6.43 | 80.13 | 0.13 | 1.91 |
| Std. Dv. | 1.91 | 0.75 | 6.88 | 3.54 | 0.15 | 0.06 | 0.16 | 0.51 | 4.91 | 14.36 | 0.10 | 2.44 |
| COV | 3.84 | 1.50 | 1.09 | 1.38 | 3.76 | 1.48 | 0.03 | 0.08 | 0.76 | 0.18 | 0.76 | 1.28 |
| Minimum | 0.03 | 0.01 | 1.00 | 0.00 | 0.00 | 0.00 | 1.84 | 6.00 | 0.20 | 48.50 | 0.00 | 0.00 |
| Max. | 34.49 | 13.58 | 93 | 44.31 | 0.79 | 0.31 | 2.73 | 8.84 | 23.2 | 99.8 | 0.57 | 9.96 |

* Approximate Particle Residue Control (SS).

** Event flushing ratio – the storm runoff volume divided by the water volume contained in the pond at the beginning of the event.

## Conclusions

As described above, retrofitting a dry pond to a wet pond can provide improvements to the overall benefits of the device. Average SS removal for particles of 5 μm or greater is expected to approach 80% removal annually. Additionally, a combination of water quality and flow analysis computer programs can be used together to evaluate and design effective stormwater controls and SW control structures can provide both water quality protection and flood control.

## 5.4 Retrofitting WWF Storage Tanks for DWF Augmentation

The operation of a WWTF requires the accommodation of wide fluctuations in flow due to naturally occurring variations in the generation of wastewater and the associated effects of I/I. Diurnal variations in flow rates are typically observed with higher flow rates occurring during the daytime when water use is high and lower flow rates at night when water use is low.

Wastewater equalization can dampen these diurnal variations to achieve a relatively constant hydraulic loading to the WWTF. Equalization basins are normally located within the WWTF following grit and screening facilities; however, equalization is sometimes located in the collection system to relieve overloaded trunk sewers.

Equalization can relieve overloaded treatment facilities in existing treatment plants and can improve the efficiency, reliability, and operability of these facilities. Equalization may also be a more cost-effective solution to modifying existing facilities. Retrofitting existing WWF storage tanks to serve a second function as DWF equalization tanks may prove to be a cost-effective alternative to the modifying existing treatment facilities or the construction of new equalization tanks at the treatment plant. Two desktop analyses will be evaluated to demonstrate this:

- Retrofit of the existing Erie Boulevard Storage System (EBSS) in Syracuse, NY
- Retrofit of the existing Spring Creek Auxiliary Wastewater Pollution Control Plant in New York, New York.

### *5.4.a. Erie Boulevard Storage System, Syracuse, New York*

Facility Description

The EBSS was constructed as part of the Best Management Practice (BMP) improvements in 1982. The existing large diameter storm sewer running underneath Erie Boulevard 2x3 m (7.5x10.5 ft) was retrofitted with automated sluice gates and level sensors to entrap CSOs that discharge to this structure. These discharges are temporarily stored in the EBSS until the Syracuse Metropolitan Wastewater Treatment Facility (METRO) has capacity to accept the WWF. The 18,950 $m^3$ (5 MG) volume of the EBSS can adequately contain the discharge from the 90 percentile storm in its tributary area. Operational problems with control equipment prevented the system from functioning as intended. Reactivation of the EBSS as a WWF storage facility is targeted for completion by July 2002.

As shown in Figure 5.4.a-1, the EBSS receives flow from three major sources:

1. Burnet Avenue Trunk Sewer (BATS) overflows, including the James Street Relief Sewer (JSRS).
2. Fayette Street Trunk Sewer (FSTS) overflows.
3. Storm Sewers at the upstream end of the system as part of the natural drainage basin.

Figure 5.4.a-1. EBSS Layout

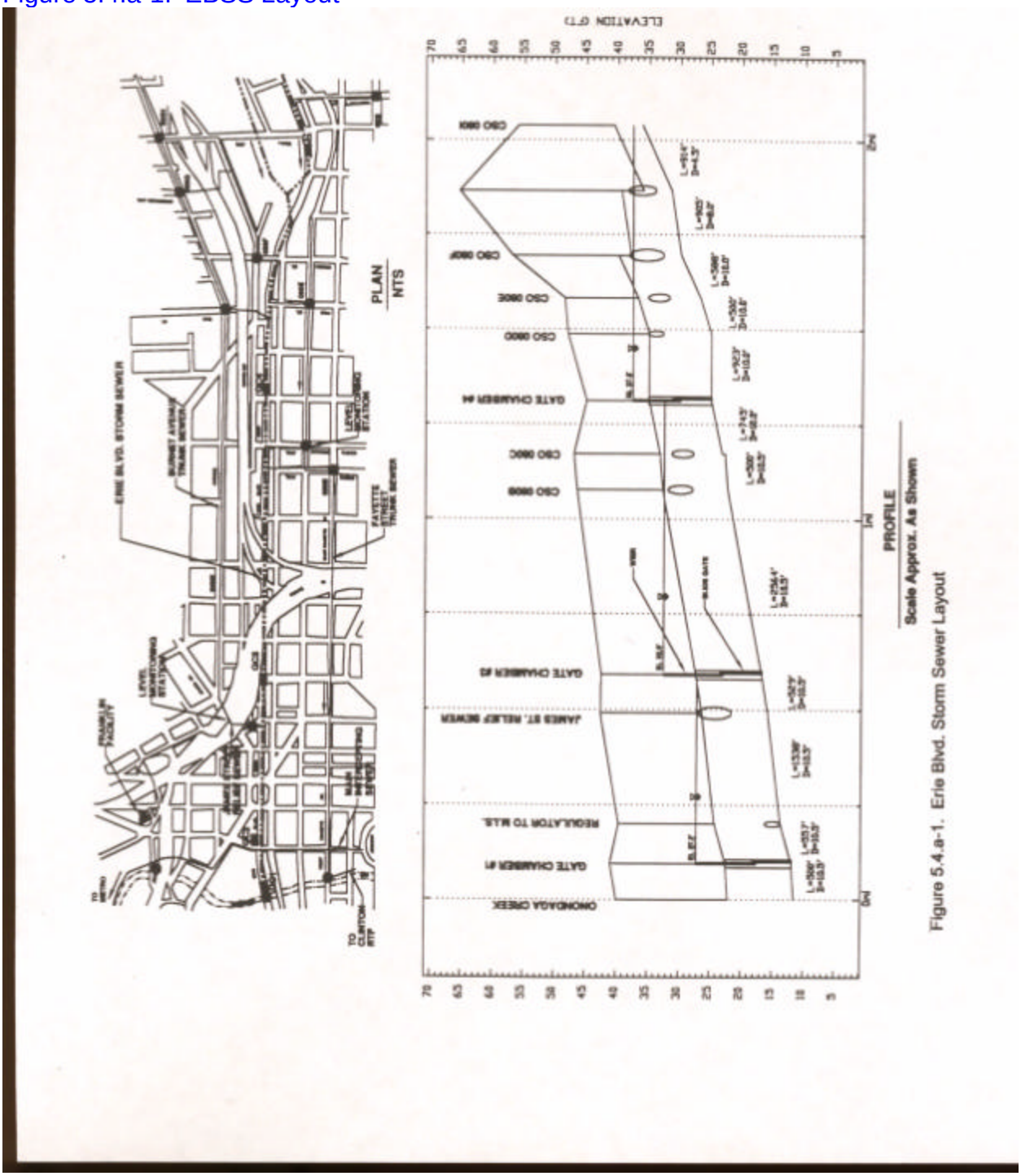



Figure 5.4.a-1. Erie Blvd. Storm Sewer Layout

The EBSS will be automatically controlled through the use of state-of-the-art SCADA control system. This system consists of aboveground sluice gate control structures for each of the four control gates that operate based on level sensors in the influent sewers. The SCADA system also controls operation of the inflatable dam that regulates flow from the BATS, the major source of flow to the EBSS.

Three sluice gates (Gates 1, 3, and 4) on the EBSS establish maximum water surface elevations as shown on Figure 5.4.a-1. Gate Chamber 2 is located on the regulator sewer that connects the EBSS to the main interceptor sewer (MIS) that conveys flow to METRO. This control gate operates based on the water level in the MIS.

Upon the initiation of WWF into the EBSS, it is filled sequentially through the closure of Gate Chambers 1, 3, and 4. Flow fills each basin sequentially until the 18,950 $m^3$ (5 MG) volume is exceeded, at which time, flow exits the EBSS through the high-water overflow at Gate Chamber 1. Gate Chamber 2 is closed except for dewatering of the EBSS following a wet-weather event as described below.

Stored volume behind Gate Chamber 1 is dewatered first by opening Gate Chamber 2 to convey flow to the MIS. Gate Chamber 3 is dewatered following dewatering of Gate Chamber 1 by opening Gate Chamber 3, and then Gate Chamber 4 is opened allowing the stored volume to flow into the lower chambers to the MIS for ultimate treatment at METRO. Gate Chamber 1 remains closed during this phase. Gate Chamber 1 is opened upon final dewatering of the system and EBSS is returned to standby mode.

This facility in its intended wet-weather operational mode will be used to evaluate the hypothetical potential for retrofitting the EBSS to serve a second function for DWF equalization. Equalization of DWF in the EBSS could prove to be beneficial to the treatment plant by eliminating the diurnal flow variability, providing METRO a relatively constant DWF component from the EBSS tributary area.

Proposed Retrofit

A portion of the DWFwill be directed from the BATS to the EBSS through the JSRS by the use of regulator structure located in the BATS immediately downstream of the JSRS. This structure is proposed as part of the EBSS reactivation to direct WWF into the EBSS. This structure contains a bottom-outlet orifice device to allow the DWF to be conveyed downstream through the BATS. The proposed retrofit will modify this orifice to allow only a portion of the dry-weather to pass downstream while regulating the higher DWF into the EBSS. DWF in excess of the orifice capacity will be directed into the EBSS where it will be stored and regulated to the MIS using a new vortex valve that would be retrofitted into Gate Chamber 2, for ultimate treatment at METRO.

The storage volume between Gate Chambers 1 and 3 targeted for use for DWF equalization is 8,680 $m^3$ (1.5 MG) based on the storage volume below the invert elevation of the JSRS. This will allow the remaining 13,250 $m^3$ (3.5 MG) of the 18,950 $m^3$ (5 MG) EBSS capacity (2,650 $m^3$ (0.7 MG) remaining capacity between Gate

Chambers 1 and 3 and 10,600 $m^3$ (2.8 MG) upstream of Gate Chamber 3) to be available for storage of WWF.

Monitoring of the BATS downstream of the JSRS conducted from July 29, 1998 through August 6, 1998 found the DWF to vary between 0.16-0.69 $m^3/s$ (5.7-12.8 cfs) with an average of 0.22 $m^3/s$ (7.6 cfs). It is proposed to divert DWF in excess of 0.15 $m^3/s$ (5.2 cfs) to the EBSS for equalization to provide, on average, 8,680 $m^3$ (1.5 MG) of flow to be equalized per day utilizing the capacity of the previously described DWF volume. Stored DWF will be routed to METRO through a new vortex valve that will be retrofitted into the existing Gate Chamber 2 with a capacity of 0.065 $m^3/s$ (2.3 cfs). This low release rate will allow effective equalization of DWF and utilization of the proposed 8,680 $m^3$ (1.5 MG) dry-weather storage volume. Gate Chamber 2 is directly connected to the MIS and will serve as the drain of the entire EBSS during wet-weather operation.

Stormwater Management Model Results

The EPA SWMM Version 4 assessed the impact of the retrofit by comparing the use of the EBSS under pre- and post-retrofit conditions. A simplified model was simulated hourly rainfall, based upon a previously calibrated model.

The data consisted of 30 years of hourly rainfall data form the NWS station at Hancock Field in Syracuse, NY for the years from 1962-1991 and were used as input to the simplified RUNOFF model. The projected flows were used to simulate long-term overflow using the TRANSPORT block of SWMM. The TRANSPORT model was constructed as a simplified network consisting of overflows and regulator pipes. The simplified model was used to project the annual wet-weather capture from the EBSS tributary area.

Table 5.4.a-1 demonstrates that the EBSS operated in a wet-weather mode provides a considerable annual capture of WWF. The retrofitting of the EBSS for DWF equalization reduces the ability to capture and retain WWF. Table 5.4.a-1 shows that this retrofit reduces the overall WWF capture from 80% to 75%.

Table 5.4.a-1. Average Annual Capture Volume from the EBSS Under Pre- and Post-Retrofit Conditions (1962 – 1991)

| EBSS CONDITION | Average Annual Total WWF in the Combined Sewer System ($m^3$) | (MG) | Average Annual WWF Captured ($m^3$) | (MG) | Percent Volume Capture |
|---|---|---|---|---|---|
| Existing Wet-Weather Operation | 1,071,465 | 283 | 859,450 | 227 | 80% |
| Dry-Weather Retrofit Operation[(1)] | 1,071,465 | 283 | 802,650 | 212 | 75% |

[(1)] Assumes dry-weather portion of storage is fully utilized during wet-weather events to represent worst case

Design Considerations

Retrofitting the EBSS to provide equalization of a portion of the DWF from the BATS required careful consideration of several key design factors, as follows:

- Hydraulic Loadings
- Flushing Requirements
- Operational and Maintenance Requirements

These design factors are described in further detail in this section.

*Hydraulic Loadings.* It is proposed to divert DWF in excess of 0.15 $m^3/s$ (5.2 cfs) to the EBSS for equalization. This would result in an average of 8,680 $m^3$ (1.5 MG) of flow to be equalized per day. DWF in excess of the available 8,680 $m^3$ (1.5 MG) storage volume will be directed to METRO through the existing BATS.

*Operational Requirements*. The only adjustable control point necessary for this retrofit is a pinch valve located on the dry-weather orifice device. The dry-weather orifice will be automatically adjusted when flow in the BATS exceeds the maximum DWF of 0.69 $m^3/s$ (12.8 cfs) or when the dry-weather storage volume is fully utilized. This will be done by fully opening the pinch valve on the dry-weather orifice to allow the entire DWF to be conveyed downstream through the BATS. This will be controlled through the EBSS SCADA system. The pinch valve will be engaged in the normal operational mode to only allow passage of the lower portion of the DWF.

*Flushing Requirements*. Accumulated solids deposition will increase during equalization of DWF, however, the layout of the EBSS serves to provide considerable flushing following wet-weather events. This is accomplished by staged release of Gate

Chambers 3 and 4 to provide a wave of water to direct accumulated solids downstream for conveyance to METRO through the MIS. This system will be an effective method for the removal of accumulated solids from the DWF equalization.

*Operation and Maintenance Requirements*. The use of EBSS for DWF equalization will result in increased O&M costs. The following list summarizes the areas of expected O&M cost increases:

- Labor to perform inspection of the facilities
- Power for the control system

Table 5.4.a-2 summarizes the expected increase in annual O&M costs resulting from the use of the EBSS for DWF equalization.

Table 5.4.a-2. Estimated O&M Costs

| Description | Estimated Annual Cost |
|---|---|
| Process Control Adjustments | $3,000 |
| Clean-Up of Wet-Weather Treatment Structures | $10,000 |
| Power | $2,000 |
| Total Estimated O&M Cost | $15,000 |

Project Costs

The projected capital costs for retrofitting the EBSS for DWF equalization are provided in Table 5.4.a-3.

Table 5.4.a-3. Estimated EBSS Retrofit Capital Costs

| Description | Estimated Cost[(1)] |
|---|---|
| Flow Diversion Structure Retrofit | $10,000 |
| Flow Regulator Structure Retrofit | $15,000 |
| Electrical Controls | $35,000 |
| Instrumentation and SCADA System | $55,000 |
| Total Project Cost | $140,000 |

[(1)] Includes Design Engineering, Construction Engineering, Legal, and Fiscal Expenses, does not include contingencies and right-of-way costs

### *5.4.b. Spring Creek AWCP, New York, New York*

The Spring Creek Auxiliary Water Pollution Control Plant (AWPCP) is a CSO wet-weather detention facility with approximately 49,200 $m^3$ (13 MG): 37,900 $m^3$ (10 MG) in basin storage and 11,400 $m^3$ (3 MG) in influent barrel storage. The details of the Spring Creek facility are provided in Section 5.2 of this document.

Proposed Retrofit

A portion of the DWF will be directed from the Autumn Avenue regulator in Brooklyn to basin 6 to minimize the diurnal fluctuations of DWF from Autumn Ave. regulator drainage area that is tributary to the 26 Ward WPCP. The DWF will be conveyed to AWPCP to allow a specified level of DWF to be conveyed to the 26 Ward WPCP.

The storage volume in basin 6 targeted for use for DWF equalization is 3,790 $m^3$ (1.0 MG) or approximately one-half of the storage volume. This will allow the remaining 34,000 $m^3$ (9.0 MG) of the 37,900 $m^3$ (10 MG) AWPCP capacity to be available for storage of WWF.

O'Brien & Gere Engineers previously performed an extensive evaluation of the 26 Ward WPCP throughout the past 15 years. Their work has shown the average DWF from the Autumn Avenue regulator to vary between 1.06-2.07 $m^3/s$ (37-73 cfs) with an average of 1.5 $m^3/s$ (53 cfs). It is proposed to divert DWF in excess of 1.33 $m^3/s$ (47 cfs) to AWPCP Basin No. 6 for equalization to provide, on average, 3,790 $m^3$ (1.0 MG) of flow to be equalized per day. Stored DWF will be routed to the 26 Ward WPCP through a new dry-weather pump station with a capacity of 0.28 $m^3/s$ (10 cfs). This low release rate will allow effective equalization of DWF and utilization of the proposed 3,790 $m^3$ (1.0 MG) dry-weather storage volume.

The proposed retrofit will consist of a physical modification to the existing diversion structure by installation of sluice gate on the 1.5 m (60 in.) diameter dry-weather outlet of the regulator structure. This sluice gate will be controlled by a flow meter located downstream of the sluice gate in the 1.5 m (60 in.) diameter interceptor sewer to limit flow to 1.33 $m^3/s$ (47 cfs). The regulator chamber will also be modified to include a weir structure to convey DWF in excess of 47 cfs to Bay Number 6 for equalization. Level sensors in the dry-weather equalization tank will also be used to control the dry-weather sluice gate when the 1 MG volume is utilized.

The dry-weather sluice gate will also be designed to open fully when sewage overflows the Autumn Ave. regulator structure into Spring Creek to maximize flow to the treatment plant by the used of a level sensor located at the invert elevation of the overflow weir.

Stormwater Management Model (SWMM) Results

The SWMM Version 4 assessed the impact of the retrofit by comparing the use of the Spring Creek AWPCP under pre- and post-retrofit conditions. A simplified model was use to simulate hourly rainfall, based upon a previously calibrated model.

The data consisted of hourly rainfall data from the National Oceanographic and Atmospheric Administration rain gauge No. 305803 at J.K.F. International Airport in New York, NY for the 1985. This data was used as input to the simplified RUNOFF model. The projected flows were used to simulate long-term continuous simulation using the TRANSPORT block of SWMM. The TRANSPORT model was constructed as a simplified network consisting of the overflow and regulator pipes. The simplified model was used to project the annual wet-weather capture from the Spring Creek AWPCP under pre- and post-retrofit conditions.

Table 5.4.b-1 demonstrates that the AWPCP presently discharges a considerable amount of WWF to Spring Creek. The retrofitting of the AWPCP for DWF equalization reduces the ability to capture and retain WWF only to a small extent. Table 5.4.b-1 shows that this retrofit increases the annual WWF by 12% or approximately 50,000 $m^3$ per year (13.2 MG per year).

Table 5.4.b-1. Wet-Weather Discharge Volume from the Spring Creek AWPCP Under Pre- and Post-Retrofit Conditions for 1985

| Spring Creek Storage Volume | Wet-Weather Discharge for 1985 ($m^3$) | (MG) |
|---|---|---|
| Existing (37,850 $m^3$ [10 MG]) | 433,005 | 114.4 |
| Retrofitted (34,065 $m^3$ [9 MG]) (1) | 482,967 | 127.6 |
| Increase In Wet-Weather Discharge | 49,962 | 13.2 |
| Percent Increase | 12% | 12% |

(1) Assumes dry-weather portion of storage is fully utilized during wet-weather events

Design Considerations

Retrofitting the AWPCP to provide equalization of a portion of the DWF from the Autumn Avenue regulator required several design factors:

- Hydraulic Loadings
- Operational Requirements
- Flushing Requirements
- Operational and Maintenance Requirements

These design factors are described in further detail in this section.

*Hydraulic Loadings* – It is proposed to divert DWF in excess of 1.33 $m^3/s$ (47 cfs) to AWPCP for equalization. This would result in an average of 8,680 $m^3$ (1 MG) of flow to be equalized per day. DWF in excess of the available 8,680 $m^3$ (1 MG) storage volume will be directed to the 26 Ward WPCP.

*Operational Requirements* – The two primary control points for this retrofit are the new DWF sluice gate that will be installed in the Autumn Ave. regulator and the new DWF pump station that will be used to convey equalized flow to the 26 Ward WPCP.

The dry-weather sluice gate will be automatically adjusted when flow in the 1.5 m (60 in.) diameter interceptor exceeds 1.33 $m^3/s$ (47 cfs) or when the 1 MG dry-weather storage volume is fully utilized.

New level sensors will be used to control operation of the DWF pump. These controls will be set to energize the pump station when flow is sensed in the storage tank and to turn the pump off when the tank is empty.

*Flushing Requirements* – Accumulated solids deposition will increase during equalization of DWF, however, a flushing system will remove accumulated solids following wet-weather events. The design of new flushing system for the Spring Creek AWPCP is presently being evaluated to provide more efficient removal of accumulated solids. The selected system will be an effective method for the removal of accumulated solids from the DWF equalization.

Operation and Maintenance Requirements – An increased O&M burden will be created for the use of Spring Creek for DWF equalization. The following list summarizes the areas of expected O&M cost increases:

- Electrical Power to operate the dry-weather pump station and the orifice control
- Labor to perform inspection of the facilities
- Labor to perform flushing of accumulated solids

Table 5.4.b-2 summarizes the expected increase in annual O&M costs resulting from the use of Spring Creek for DWF equalization.

Table 5.4.b-2. Estimated O&M Costs

| Description | Estimated Annual Cost |
|---|---|
| Electrical Power | $3,000 |
| Labor (Maintenance of Facilities) | $10,000 |
| Labor (Tank Flushing) | $25,000 |
| Total Estimated O&M Cost | $38,000 |

Project Costs

The projected capital costs for retrofitting Spring Creek for DWF equalization are provided in Table 5.4.b-3.

Table 5.4.b-3. Estimated Spring Creek DWF Retrofit Capital Costs

| Description | Estimated Cost[1] |
|---|---|
| Regulator Structure Retrofit | $205,000 |
| DWF Pump Station and Force Main | $170,000 |
| Process Control System | $45,000 |
| Instrumentation and SCADA System | $65,000 |
| Total Project Cost | $485,000 |

[1] Includes Design Engineering, Construction Engineering, Legal, and Fiscal Expenses, Does not include contingencies and right-of-way costs

***Conclusions***

Retrofitting the EBSS and Spring Creek AWPCP were evaluated for the purpose of serving a dual role as wet-weather and dry-weather equalization facilities without drastically reducing the ability of the storage tank to capture WWF. The primary benefit of equalization provided by the DWF storage is the dampening of peak flow rates that would otherwise result at the treatment plant. This retrofit will increase the treatment capacity for a treatment plant that is operating at or near design capacity and will also allow the primary and secondary treatment facilities to operate at a more uniform rate to provide consistent treatment efficiency throughout a typical day. However, there is adequate hydraulic and biological capacity the Syracuse METRO and 26 Ward treatment facilities.

This retrofit option may also provide a reduction in operation of high flow pumps, where appropriate, as a result of eliminating the high diurnal flows. This could result in a reduction in O&M at the treatment plant; however, the O&M required at the equalization facility may offset any benefit at the treatment plant.

## 5.5 SSO Control Using Storage

### *5.5.a. Hypothetical Overflow Retention Facility Retrofit to an Existing Sanitary Sewer System*

This desktop analysis involves a hypothetical retrofit to a sanitary sewer system to eliminate SSOs from an existing regulator structure. This desktop analysis was developed from a real life project from a confidential source. Overflow from this regulator occurs during and following rainfall events due to a well-established I/I problem. The I/I problem was assessed as part of a sewer system evaluation survey (SSES) that identified and quantified various sources of I/I with the collection system. A rehabilitation program was implemented in accordance with the findings of the SSES to provide removal of the primary I/I sources. This program also requires the elimination of the existing regulator overflow.

Sewer system modeling was performed using the Environmental Protection Agency's SWMM Version 4. The previously calibrated model was expanded to identify flow conditions at the regulator following the implementation of the I/I rehabilitation program. The model was used to identify peak overflow rates, volumes and hydraulic grade lines for various overflow abatement alternatives.

**Project Approach**

A 0.9 m (36 in.) diameter interceptor and an 0.5 m (18 in.) diameter interceptor enter the regulator chamber. A 0.9 m (36 in.) diameter changes to a 1 m (42 in.) diameter interceptor then exits the chamber and continues to the WWTF. The existing interceptor and other downstream piping configurations restrict the flow to the treatment plant. An 0.5 m (18 in.) diameter overflow also exits the chamber that discharges to the receiving water body when wet-weather events cause the chamber to overflow.

The regulator overflow was likely developed to relieve a persistent flooding problem in the vicinity of the regulator. The interceptor network in the vicinity of the regulator presently surcharges during the two-year design storm resulting in approximately 12,100 $m^3$ (3.2 MG) of surface flooding. The existing overflow relieves surcharge and minimizes flooding. The peak hydraulic grade line cannot increase beyond surcharge with the overflow closed. Closing the overflow would result in pronounced basement and street flooding in the areas upstream of the existing overflows and would not increase the peak flow to the treatment plant from the interceptor. The high head losses are associated with the overflow and limitations of the interceptor network that includes two double-barrel siphons downstream of the regulator overflow. The alternative of constructing a new sewer parallel to the existing sewer to convey flow to the treatment plant was not considered feasible given long length of sewer, deep rock excavation, to long siphons, and a shoreline construction.

The two-year frequency, two-hour duration design storm was selected in order to determine what control was necessary in terms of treatment, transportation or storage of the regulator overflows. Based on the SWMM model, during a two-year storm, a

peak rate of approximately 28,000 $m^3$/d (7.4 MGD) overflows to the receiving water with a total overflow volume of 6,400 $m^3$ (1.7 MG). One recommended abatement alternative is to use a 6,400 $m^3$ (1.7 MG) ORF to eliminate overflows up to the two-year design storm and to treat overflows in excess of the storage capacity with high-rate disinfection.

The WWTF is presently permitted to treat 64,000 $m^3$/d (17 MGD). The monthly average flow during a two-year monitoring period of the SSES was 76,000 $m^3$/d (20 MGD). As the plant already exceeds its flow limit, recommendations to increase the peak WWF to the plant were not made, but to store the WWF during wet events with a slow release to the WWTF following the storm.

Overflow Retention Facilities

The proposed retrofit consists of a 6,400 $m^3$ (1.7 MG) ORF located between the regulator chamber and the existing outfall to capture the flow associated with the two-year design storm. Overflow events producing less than 6,400 $m^3$ (1.7 MG) are retained and returned to the treatment plant when influent flows subside. Overflow events producing in excess of the available storage volume are disinfected using chlorine and dechlorinated before being discharged to the receiving water. The retained volume of 6,400 $m^3$ (1.7 MG) is returned to the treatment plant through the interceptor sewer utilizing an overflow return pump.

The overflow retention facilities include the following components:

*ORF Tanks*. The ORF tanks will consist of an in-ground reinforced concrete tank with a dimension of 30.5 m (100 ft) wide by 73.2 m (240 ft) long with a 3 m (10 ft) depth to provide a useable retention volume of 6,400 $m^3$ (1.7 MG).

*Overflow Return*. A 10.2 cm (4 in.) overflow return pump will be used to return the captured volume to the WWTF through the existing interceptor sewer within 48 h after the occurrence of an overflow event.

*Disinfection*. The disinfection system will utilize sodium hypochlorite introduced into a contact tank sized for 5 min detention time at peak flow rate of 28,000 $m^3$/d (7.4 MGD) and requires the use of mechanical mixers to induce energy to provide effective disinfection. The mixing velocity gradient and chlorine dose were calculated on the basis of estimated influent and effluent fecal coliform levels for sanitary sewer overflows. The high-rate disinfection system was sized by estimating disinfectant dosage and mixing energy required to assure the inactivation of fecal coliform. Based on the optimum velocity gradient calculated, horsepower requirements for mixers were determined. From this analysis, it was determined that an optimum chlorine dosage of 20 mg/l and approximately 1.1 kW (1.5 HP) of total mixing energy are required to insure proper disinfection. A 5 cm (2 in.) disinfection tank pump will be used to return the disinfection tank volume to the WWTF through the existing interceptor sewer within 48 h after the occurrence of an overflow event.

*Dechlorination*. Dechlorination, which reduces the chlorine residual prior to discharge, will be accomplished by injecting sodium bisulfite in a 10 $m^3$ (2,600 gal) tank with a high-rate mixer to provide detention time of 30 s at a peak flow of 28,000 $m^3$/d (7.4 MGD). The dechlorination facilities are metered automatically as a function of overflow flow rate to minimize chlorine residual.

*Overflow Metering*. Overflows will be metered following discharged from the dechlorination tank through an open channel-type metering device located in the outfall pipe.

*Control Building*. A 63.2 $m^2$ (680 $ft^2$) structure will be provided as the control building for this facility. This structure will include process control equipment and be used to house the chemical storage tanks.

Stormwater Management Model Results

The EPA SWMM Version 4 was used to assess the benefit of the retrofit by comparing annual WWF volume presently discharged to the volume captured by the retention tank. A simplified model was developed to simulate hourly rainfall for the hypothetical collection system

The data used for these projections consisted of 15 years of hourly rainfall data for the years from 1961-1985. These data were used as input to the simplified RUNOFF model that projected flows that were used to simulate long-term overflow using the TRANSPORT block of SWMM. The TRANSPORT model was constructed as a simplified network consisting of hypothetical overflows and regulator pipes. The simplified model was used to project the annual volume presently discharged and the percent of the annual volume captured by the proposed storage facility.

Table 5.5.a-1 demonstrates that the 6400 $m^3$ (1.7 MG) storage facilities provide considerable annual capture of WWF compared to that was presently discharged without treatment.

Table 5.5.a-1. Average Annual Overflow Volume from the Pre- and Post-Retrofit Facilities (1971 – 1985)

| | Average Annual Overflow Volume | |
|---|---|---|
| | $m^3$ | (MG) |
| Pre-Retrofit WWTF Overflow | 70,400 | 18.6 |
| Post-Retrofit (1) | 18,000 | 4.7 |
| Percent Capture | 75% | |

Although the ORF was sized on the two-year design storm, the system annually captures only 75% of the WWF that is presently discharged. This occurs as a result of the regulator chamber modification required to reduce the hydraulic grade line at the

regulator and relieve surcharge and street flooding in the vicinity of the regulator chamber. Therefore, the ORF will be active more frequently than overflow presently occurs from the existing overflow.

Design Considerations

The retrofitting of a sanitary sewered collection system to eliminate overflows through storage and disinfection required the following design considerations:

- Hydraulic Loadings
- Treatment Objectives
- Operational Requirements
- Chemical Storage and Feed Requirements
- Sludge Handling Requirements
- Operations and Maintenance Requirements

*Hydraulic Loadings* - Hydraulic loadings were developed for the ORF to provide retention of WWF for subsequent treatment at the WWTF following a wet-weather event and to provide disinfection of WWF beyond the 6,400 $m^3$ (1.7 MG) ORF capacity.

*Treatment Objectives* – The objective of this retrofit is to eliminate overflow from the sanitary sewer system up to the two-year design storm and to provide disinfection of WWF beyond the two-year design storm level.

Operational Requirements - The ORF are designed to operate on a continuous basis include:

- WWF retention
- High-rate disinfection of flow in excess of ORF capacity
- Chemical Storage and Feed Requirements - The ORF utilize two chemicals:
- Liquid Sodium Hypochlorite for disinfection
- Liquid Sodium Bisulfite for dechlorination

The liquid sodium hypochlorite system is based on commercial-grade liquid sodium hypochlorite (15% solution) which contains the equivalent of 150 kg of chlorine/$m^3$ of solution (1.25 lbs of $Cl_2$/gal of solution). Two metering pumps are designed to deliver a dosage of 20 mg/L at the peak design flow with one unit as a backup. The liquid sodium hypochlorite is stored in a bulk storage tank with an effective working volume of 11.4 $m^3$ (3,000 gal). The available chlorine in sodium hypochlorite solution declines with age. The anticipated sodium hypochlorite storage time will be 50 days.

The liquid sodium bisulfite storage system is based on commercial grade liquid sodium bisulfite (38% solution) that contains appropriately 503.2 kg of sodium bisulfite/$m^3$ of solution (4.2 lbs of sodium bisulfite/gal of solution). Two metering pumps are designed to deliver sodium bisulfite dosages necessary to dechlorinate effluent with 1.5 mg/L of sodium bisulfite per mg/L of chlorine residual. Approximately 1.6 kg of sodium bisulfite will be used per kg of chlorine residual. A bulk storage tank with an effective working volume of 5.7 $m^3$ (1,500 gal) will be used for sodium bisulfite storage.

The sodium hypochlorite and sodium bisulfite metering pumps are flow paced. An ultrasonic type level sensor is used for effluent flow measurement.

*Sludge Handling Requirements* – Accumulated sludge will be returned to the WWTF during the pump back of retained WWF and the associated tank flushing following tank draining. Tank flushing will be accomplished manually using a 7.6 cm (3 in.) municipal water service.

*Operation and Maintenance Requirements* - The implementation of the ORF will create an O&M cost. The areas of expected O&M burden include:

- Chemicals for disinfection and dechlorination
- Power for tank draining and chemical feed systems
- Water for tank flushing.

The estimated annual O&M costs are provided in Table 5.5.a-2.

Table 5.5.a-2. ORF O&M Costs

| Annual Chemical Costs | Cost |
|---|---|
| Sodium Hypochlorite | $ 8,200 |
| Sodium Bisulfite | $ 3,800 |
| O&M Labor | $10,000 |
| TOTAL COST | $22,000 |

Project Costs

The estimated construction costs for the ORF are provided in Table 5.5.a-3.

Table 5.5.a-3. Estimated ORF Construction Costs

| Item Description | Quantity | Unit | Unit Cost | Installation Factor | Cost |
|---|---|---|---|---|---|
| **Site Work** | | | | | |
| Common Excavation | 1,529 | $m^3$ | $26 | 1.0 | $40,000 |
| Rock Excavation-Blasting Method | 574 | $m^3$ | $131 | 1.0 | $75,000 |
| Select fill | 459 | $m^3$ | $29 | 1.0 | $13,200 |
| Sheeting | 743 | $m^2$ | $377 | 1.0 | $280,000 |
| Site Dewatering | 1 | Ea | $20,000 | 1.0 | $20,000 |
| New Pavement & Restoration | 785 | $m^2$ | $24 | 1.0 | $18,800 |
| Fencing Restoration | 61 | m | $98 | 1.0 | $6,000 |
| General Site & Bldg Restoration | 1 | Ea | $12,000 | 1.0 | $12,000 |
| | | | | **Subtotal** | **$465,000** |
| **Structural & Architectural** | | | | | |
| CIP Concrete-Base | 214 | $m^3$ | $523 | 1.0 | $112,000 |
| CIP Concrete-Walls | 191 | $m^3$ | $1,046 | 1.0 | $200,000 |
| CIP Concrete-Roof | 199 | $m^3$ | $1,177 | 1.0 | $234,000 |
| Control Building, Brick & Block | 63 | $m^2$ | $1,507 | 1.0 | $95,200 |
| | | | | **Subtotal** | **$641,200** |
| **Mechanical** | | | | | |
| Chemical Metering Pumps | 2 | Ea | $3,500 | 1.5 | $10,500 |
| Submersible Mixers-1.1 kW | 5 | Ea | $3,700 | 1.7 | $31,450 |
| Oder Control Blowers-11.2 kW | 1 | Ea | $4,500 | 1.7 | $7,650 |
| ORF Tank Dewatering Pump-10.2 cm | 1 | Ea | $3,500 | 1.7 | $5,950 |
| Disinfection Tank Dewatering Pump-5.1 cm | 1 | Ea | $2,200 | 1.7 | $3,740 |
| Back Flow Preventer-7.6 cm | 1 | Ea | $2,000 | 1.5 | $3,000 |
| Backwater Valve-53.3 cm | 1 | Ea | $1,500 | 1.5 | $2,250 |
| | | | | **Subtotal** | **$64,540** |
| **Special Construction** | | | | | |
| Carbon Absorption Unit | 1 | Ea | $60,000 | 1.3 | $78,000 |
| Chemical Storage Tank (Chlorination) | 1 | Ea | $3,000 | 1.5 | $4,500 |
| Chemical Storage Tank (Dechlorination) | 1 | Ea | $1,500 | 1.5 | $2,250 |
| Flow Meter | 1 | Ea | $10,000 | 1.5 | $15,000 |
| SCADA Tansmitter | 1 | Ea | $15,000 | 1.5 | $22,500 |
| Jib Crane | 2 | Ea | $2,000 | 1.0 | $4,000 |
| Misc. Safety Equipment | 1 | Ea | $5,000 | 1.0 | $5,000 |
| | | | | **Subtotal** | **$131,250** |
| **Site Piping & Utilities** | | | | | |
| 106.7 cm Interceptor Relocation | 18.3 | m | $2,625 | 1.0 | $48,000 |
| 91.4 cm Interceptor Relocation | 55 | m | $1,637 | 1.0 | $90,000 |
| 61 cm ORF Inf/Effl. Sewer | 9.1 | m | $1,150 | 1.0 | $10,500 |
| 45.7 cm Interceptor Relocation | 4.6 | m | $984 | 1.0 | $4,500 |
| 182.9 cm Storm Sewer Relocation | 67 | m | $1,970 | 1.0 | $132,000 |
| Bypass Pumping Interceptor | 1 | Ea | $20,000 | 1.0 | $20,000 |
| Diversion Structure | 1 | Ea | $31,000 | 1.0 | $31,000 |
| Storm Sewer Junction Manhole | 1 | Ea | $28,000 | 1.0 | $28,000 |
| Special Manhloe for Sewer Crossing | 1 | Ea | $34,000 | 1.0 | $34,000 |
| 10.2 cm Force Main | 6 | m | $328 | 1.0 | $2,000 |
| 7.6 cm Facility Water Service | 67 | m | $263 | 1.0 | $17,600 |
| Wet Tap for Water Main Service | 1 | Ea | $3,000 | 1.0 | $3,000 |
| Temporary Overflow Bypass | 9.1 | m | $656 | 1.0 | $6,000 |
| | | | | **Subtotal** | **$426,600** |
| **Construction Cost Subtotal** | | | | | **$1,728,590** |
| **Other Contracts** | | | | | |
| Electrical | 15% | PCT | $1,728,590 | | $259,289 |
| Heating & Ventilating | 7% | PCT | $1,728,590 | | $121,001 |
| Plumbing | 2% | PCT | $1,728,590 | | $34,572 |
| **Subtotal Constuction Costs** | | | | | $2,143,452 |
| **Estimating Contingency** | 25% | PCT | $2,143,452 | | $535,863 |
| **Total Construction Cost** | | | | | $2,679,315 |
| **Engineering & Legal** | 20% | PCT | $2,679,315 | | $535,863 |
| **TOTAL CONSTRUCTION COST** | | | | | **$3,220,000** |

## Conclusions

Retrofitting SSOs with ORF can provide a cost-effective alternative in lieu of expansion of the sanitary collection system to convey the volume of overflow to the treatment plant for primary and secondary treatment. This alternative requires the availability of a sufficient amount of land to build such a facility.

## 5.6 Retrofitting for Industrial Wastewater Control in a Combined Sewer System

EPA has identified maximizing flow to the POTW in combined sewer systems as one of the nine minimum controls for CSO abatement (EPA 1995). The objective of this control is to reduce the magnitude, frequency, and duration of CSOs that flow untreated into receiving waters. Industrial wastewater is often characterized by high-strength pollutants that can be detrimental to receiving waters. The control of industrial wastewater in a combined sewer system can reduce the impact of the CSO discharges on receiving waters. A desktop analysis of the Rockland WWTF in Rockland, ME, was evaluated to demonstrate the technique.

### *5.6.a. Rockland Wastewater Treatment Facility, Rockland, Maine*

The city of Rockland, Maine owns and operates a secondary WWTF that discharges to Rockland Harbor, a Class SC water body. The WWTF was originally constructed in 1978 with upgrades in 1994 and 1998. The Rockland WWTF provides secondary treatment using conventional activated sludge. The WWTF was designed to handle a monthly-average flow of 11,000 $m^3$/d (2.9 MGD) and an instantaneous-peak flow of 22,000 $m^3$/d (5.7 MGD). The WWTF serves a CCS. Rockland's existing collection system services most of the residential population, commercial properties, and industrial properties within the limits of the city of Rockland. The system also serves the neighboring community of Rockport and the Samoset Resort. The system contains three types of sewers:

- combined sanitary and storm
- separate sanitary
- storm

Nine pumping stations exist serving various areas of the collection system. At present there are approximately 21 km (13 miles) of gravity sewer ranging from 15 cm (6 in.) to 76 cm (30 in.) in diameter. Pipe materials include stone and brick (dating back to the Civil War era) as well as more recent installations of vitrified clay, asbestos-cement, concrete, cast iron, and PVC pipe. According to the 1998 CSO Facilities Plan, there are presently two permitted CSOs located within the system. In addition, five other CSOs were located during the Facility Planning effort that began in 1993. In an average year, 187,000 $m^3$ (49.3 MG) are discharged from these seven CSOs.

A majority of the wastewater currently entering the Rockland sewer system is domestic sewage from residential and commercial sources. The WWTF also receives a significant flow and loading from the FMC Corporation Food Ingredients Division facility. This facility manufacturers carrageenan, a seaweed based food additive. The WWTF is designed to receive approximately 67% of the organic loading and approximately 22% of the DWF from the FMC Facility. Annual average wastewater flows to the WWTF are 7,570 $m^3$/d (2.0 MGD) and average daily $BOD_5$ loadings are 1,565 kg/d (3,445 lbs/d).

Prior to implementation of the CSO Abatement Program, the WWTF used the following unit processes to provide preliminary, primary and secondary treatment:

- Three raw sewage pumps with a capacity of 10 $m^3$ (2,600 gal) per min each,
- One 92 cm (36 in.) comminutor with a manual bypass screen,
- Two primary clarifiers (each at 14 m [45 ft] with a total surface area of approximately 295 $m^2$ [3,180 $ft^2$]). Primary sludge is conveyed to two cyclone classifiers to remove grit prior to thickening,
- Six mechanically aerated activated sludge aeration tanks with a total volume of 6,590 $m^3$ (1.74 MG),
- Two secondary clarifiers (each at 17 m [55 ft] with a total surface area of approximately 440 $m^2$ [4,750 $ft^2$ ]),
- A chlorine contact tank with a volume of approximately 345 $m^3$ (91,000 gal),
- A dechlorination chamber with a total volume of approximately 15 $m^3$ (4,000 gal).

Figure 5.6.a-1 shows the existing site layout prior to implementing the CSO Abatement Program. Figure 5.6.a-2 shows a schematic of the existing WWTF prior to retrofitting with a description of the numbered overflow points. When influent peak flows would reach 18,930 $m^3$ (5 MGD), the WWTF influent pumps would be throttled and excess flow bypassed to the outfall downstream of the chlorine contact tank resulting in untreated, undisinfected discharges.

These overflows are discharged to Rockland Harbor via permitted CSO #002 (located in the wetwell of the WWTF influent pump station Figure 5.6.a-2). The wetwell receives the FMC Industrial Wastewater plus WWTF sidestream flows from the primary sludge gravity thickener overflow, waste activated sludge DAF thickener overflow, and sludge dewatering system vacuum filter filtrate. The industrial wastewater and WWTF sidestream flows are mixed with the combined sewage upstream of the influent pump station wetwell. Consequently, these high-strength wastes are included in the overflow discharged via CSO #002.

In March 1998, a CSO Facilities Plan was completed. The CSO Facilities Plan was approved by the State of Maine Department of Environmental Protection (DEP) and EPA. The approved plan recommended a three-phase program to address CSO discharges.

Phase I of the CSO Abatement Program included rehabilitation of the Lindsey Brook Sewer. This sewer is located within the streambed for Lindsey Brook and has been determined to be a considerable source of I/I during wet weather or high groundwater periods. The Lindsey Brook Sewer is also suspected to be a source of exfiltration during periods of dry weather.

Figure 5.6.a-1.  Existing Site Layout Prior to Retrofit

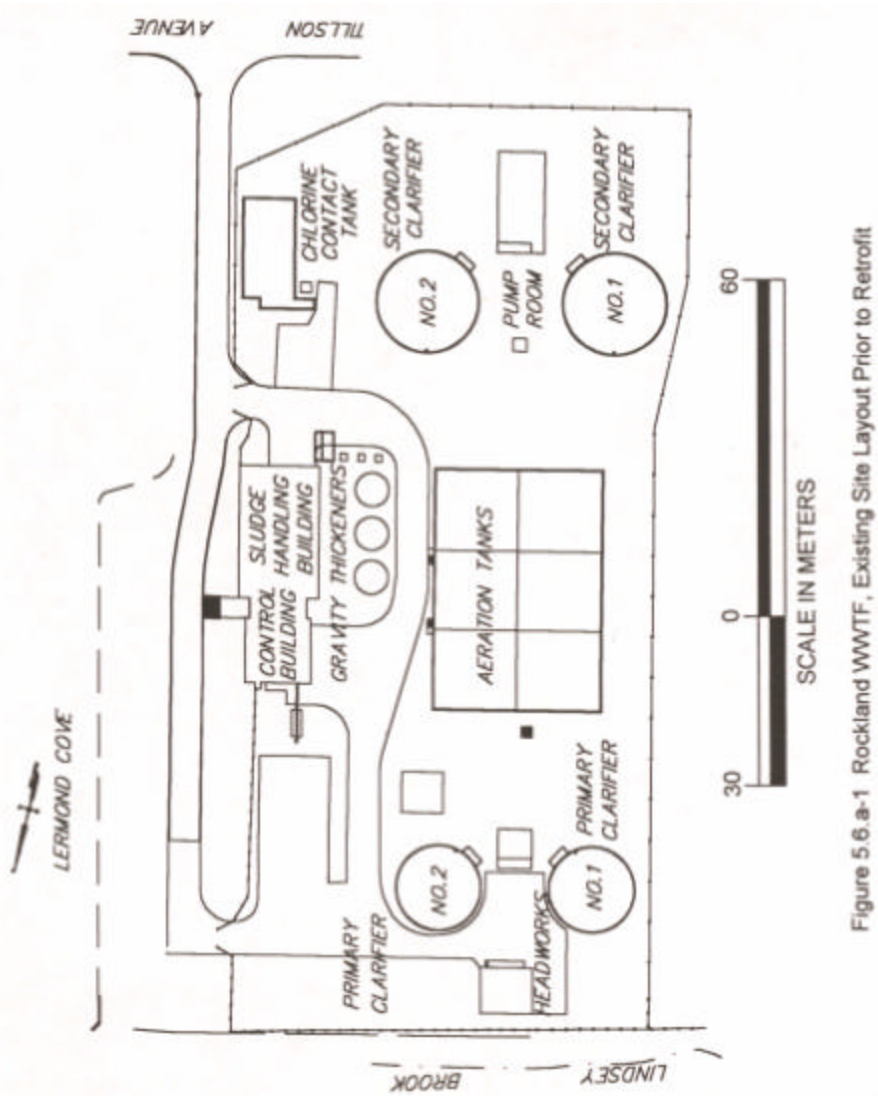
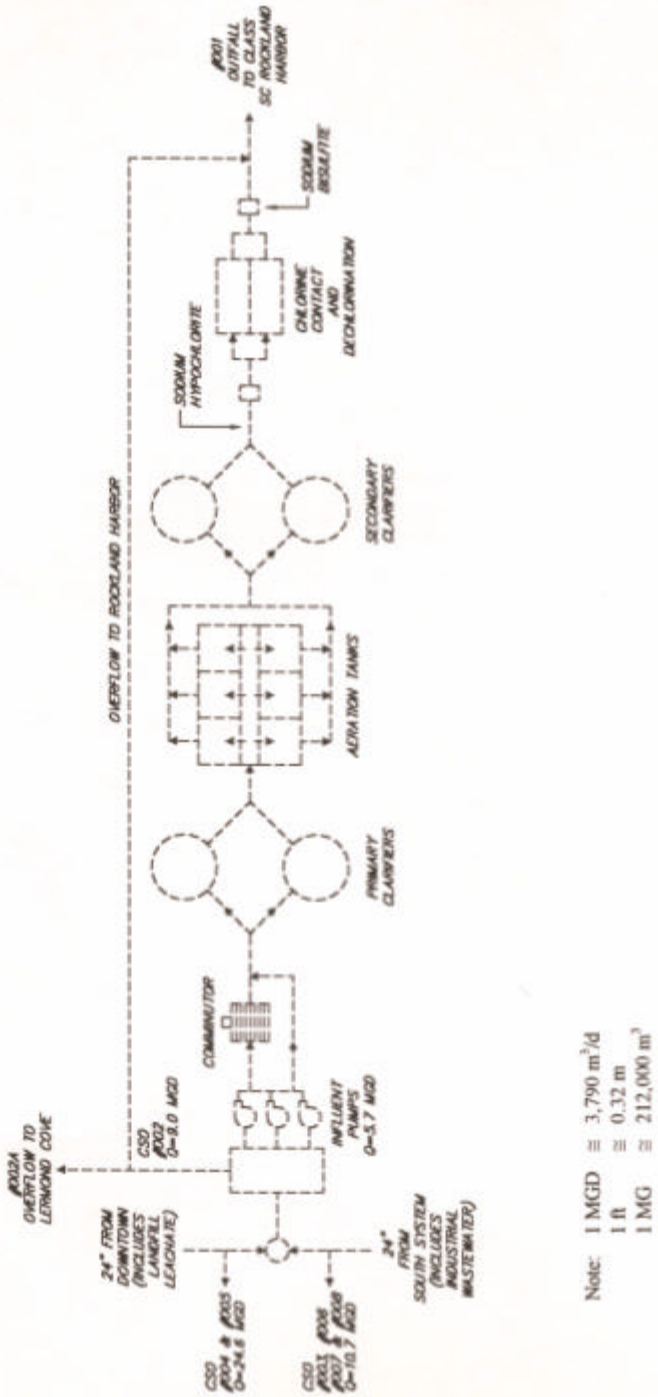



Figure 5.6.a-1 Rockland WWTF, Existing Site Layout Prior to Retrofit

Figure 5.6.a-2. Process Flow Schematic Prior to Retrofit




Note: 1 MGD ≅ 3,790 $m^3/d$
1 ft ≅ 0.32 m
1 MG ≅ 212,000 $m^3$

Figure 5.6.a-2 Rockland WWTF, Process Flow Schematic Prior Retrofit

Phase II of the CSO Abatement Program included upgrading the WWTF to handle additional WWF. Phase II also includes new consolidation conduit to intercept overflows from the second and third largest CSO's (#004 and #005) that are located approximately 92 m (300 ft) upstream of the WWTF on Main Street (Route 1) in downtown Rockland. Phase II also includes a new force main to separate industrial wastewater produced at the FMC Food Ingredients Division Facility from the City interceptor sewer and provide a direct connection to the Rockland WWTF. Finally, Phase II will redirect WWTF sidestream flows to the primary and secondary treatment units downstream of the influent pumping station.

Phase III of the CSO Abatement Program includes selected sewer separation in the south portion of the Rockland CCS to address the remaining CSO discharges.

This case study will focus on Phase II Improvements at the Rockland WWTF. Phase II Improvements include:

- A new 76 cm (30 in) diameter consolidation conduit from CSO #004 and CSO #005 to the WWTF.
- A new flow diversion structure (SS#1) at the WWTF.
- Conversion of the existing headworks into a submersible pumping station to handle DWF and WWF. Two dry weather pumps are provided. Each dry weather pump has a capacity of 15 $m^3$ (4,000 gal) per min. Three wet weather pumps are provided. Each wet weather pump has a capacity of 30 $m^3$ (7,800 gal) per min.
- Construction of a new headworks with mechanically cleaned bar screen, screenings compactor, manually cleaned bar rack, grit chamber, recessed impeller grit pump, cyclone grit classifier, and 31 cm (12 in.) parshall flume.
- Construction of two new upflow style flow distribution structures for the existing primary and secondary clarifiers to improve hydraulic flow splitting.
- Construction of a new vortex separator to provide high-rate primary treatment for projected peak wet weather CSO flows of 127,560 $m^3$/d (33.7 MGD).
- Construction of a new CSO Disinfection and Dechlorination Structure to provide high-rate disinfection and dechlorination for projected peak WWFs of 114,690 $m^3$/d (30.3 MGD); 127,560 $m^3$/d (33.7 MGD) peak WWF minus 12,870 $m^3$/d (3.4 MGD) vortex underflow.
- Constructions of a new 20 cm (8 in.) diameter force main for industrial wastewater. This force main would convey industrial wastewater to the existing primary clarifiers, existing aeration tanks, or a new 1,100 $m^3$ (0.29 MG) equalization tank. The equalization tank was created by refurbishing one (1) unused existing aeration tank. The equalization tank utilizes diffused aeration for mixing. The equalization tank contains two submersible pumps (1 primary and 1 standby). Each pump has a variable speed drive to allow output to be adjusted.

- Construction of new plant drain piping to eliminate discharge of WWTF sidestreams to the influent sewer. The gravity thickeners for primary sludge are eliminated by replacing the recessed impeller primary sludge pumps with new positive displacement units designed to remove thickened sludge directly from the primary clarifiers. The waste activated sludge filtrate drain was redirected to the existing aeration tank inlet channel. The sludge dewatering filtrate drain was redirected to the primary clarifier influent.

Figure 5.6.a-3 shows a site layout of the Rockland WWTF following construction of the Phase II CSO Abatement Program. Figure 5.6.a-4 shows a process flow schematic of the Rockland WWTF following Phase II of the CSO Abatement program. These improvements will allow high-rate primary treatment and disinfection for peak wet-weather CSO flows at 128,000 $m^3$/d (33.7 MGD). This provides treatment for CSO #002, #004, and #005 that represent approximately 90% of the total annual overflow volume from the Rockland CCS.

Stormwater Management Model Results

The EPA SWMM Version 4 was used in the facility plan to predict overflow rates and volumes at all of Rockland's CSOs. The SWMM is a relatively complex mathematical model. To model the various physical processes associated with stormwater runoff and CSO discharge, a large amount of input data is required. Drainage basin characteristics (such as size, average ground slope, width, percent imperviousness, roughness, surface storage, soil infiltration and gutter or pipe flow) must be supplied to the model.

The RUNOFF and EXTRAN blocks of SWMM were used in this study. The model was constructed using records of the physical system: pipe sizes, slopes, materials, lengths and elevations. Each CSO diversion structure was modeled. The existing Park Street Pumping Station was also modeled. Then drainage area characteristics were modeled for such factors as total acres, percent imperviousness, slope and dimensions. Once the SWMM model was mathematically constructed, it was calibrated using the extensive rainfall and CSO quantity data gathered during 1993, 1994 and 1995.

Unlike the calibration process, where actual "real-time" rainfall data are used, SWMM projections are made using two types of rainfall data:

- Synthetic rainfall hyetographs (or designs storms) for determining peak rates of overflow.
- Long-term rainfall records from NOAA for determining statistically based overflow volumes.

The former data are used in the evaluation of pipes and conduits, pumps and CSO treatment facilities where peak rates of discharge are used as the basis of design. The latter data are used to design storage facilities and to test the long-term effectiveness of overall CSO abatement programs.

Figure 5.6.a-3. Site Layout Following Retrofit

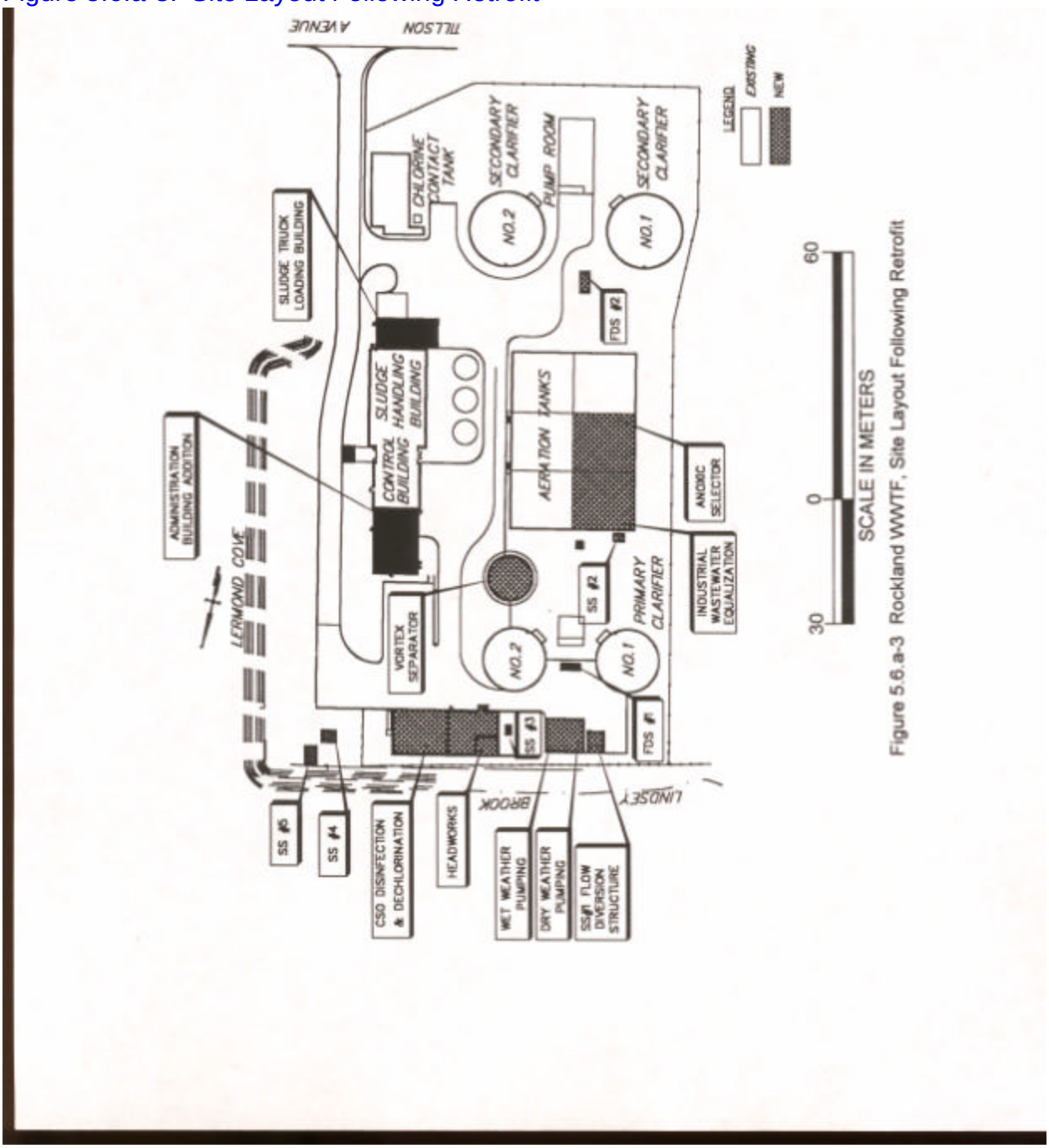
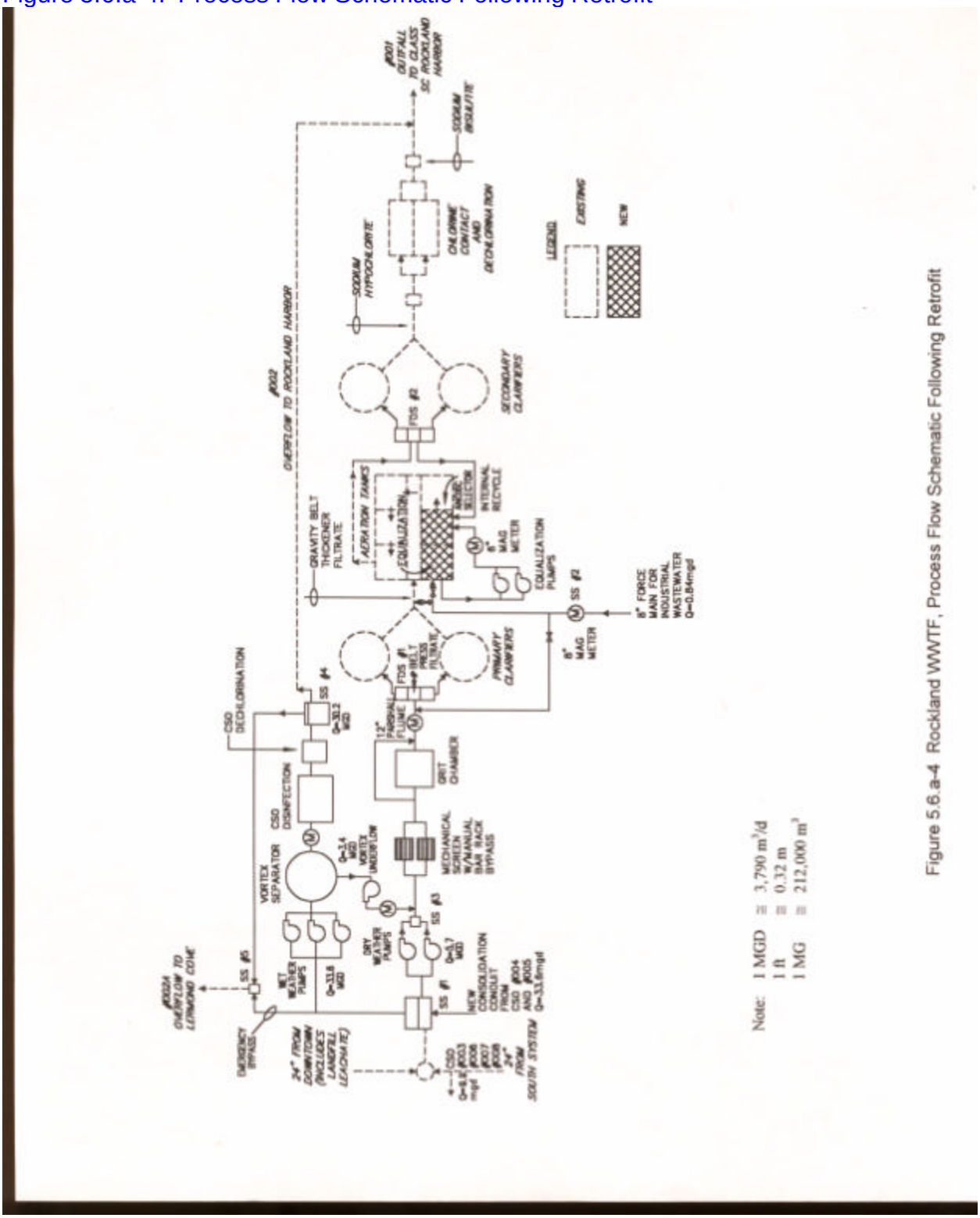



Figure 5.6.a-3 Rockland WWTF, Site Layout Following Retrofit

Figure 5.6.a-4. Process Flow Schematic Following Retrofit




Note: 1 MGD ≅ 3,790 m³/d
1 ft ≅ 0.32 m
1 MG ≅ 212,000 m³

Figure 5.6.a-4 Rockland WWTF, Process Flow Schematic Following Retrofit

For the city of Rockland CCS, the predictions were made using design storms with return frequencies of 0.5, 1, 2 and 5 years. A two-hour storm duration was selected for the design to reflect an average of time of concentration of the Rockland system. The SWMM Modeling identified several important hydraulic characteristics of Rockland's CCS:

- CSO #005 has the highest projected peak overflow rate, CSO #002 (WWTF Bypass) has the second highest peak overflow rate.
- CSO #002 (WWTF Bypass) discharges the greatest overflow volume with the North Subsystem overflows (CSO #004 and #005) discharging the second greatest volume.

Table 5.6.a-1 shows the predicted overflow activity in terms of average annual volume of overflow per year. These data were developed using the SWMM model and continuous simulation. Tables 5.6.a-2 and 5.6.a-3 show the projected peak overflow rates for each CSO using the design storms.

Table 5.6.a-1. SWMM Modeling Summary - Annual Overflow Volumes

| Drainage Area Description | CSO ID Number | Estimated Annual CSO Discharge ($m^3$/yr.)[1] | Estimated Annual CSO Discharge (MG/yr.)[1] |
|---|---|---|---|
| WWTF Bypass | 002 | 110,900 | 29.3 |
| North/Lindsey/Downtown | 004 and 005 | 54,000 | 14.4 |
| South | 003, 006, 007 and 008 | 21,200 | 5.6 |
| | TOTAL | 186,100 | 49.3 |

[1]Using long-term simulation based upon 21 years of hourly rainfall data form the NWS station in Portland, ME for the years from 1971 to 1991.

Table 5.6.a-2. SWMM Modeling Summary - Projected Peak Flows

| | Estimated Peak Flow ($m^3$/d) | | | |
|---|---|---|---|---|
| CSO ID Number | 0.5 Year Storm[1] | 1 year Storm[1] | 2 year Storm[1] | 2 year Storm[1] |
| 002 | 23,850 | 31,800 | 33,000 | 34,100 |
| 003 | 5,300 | 5,300 | 5,300 | 5,300 |
| 004 | 0.0 | 10,100 | 23,100 | 37,100 |
| 005 | 47,700 | 53,800 | 54,900 | 56,000 |
| 006 | 380 | 6,440 | 14,000 | 21,200 |
| 007 | 6,440 | 6,440 | 6,440 | 6,440 |
| 008 | 1,900 | 3,400 | 5,300 | 7,600 |

[1] Using 2 h duration.

Table 5.6.a-3. SWMM Modeling Summary - Projected Peak Flows

| Estimated Peak Flow (MGD) | | | | |
|---|---|---|---|---|
| CSO ID Number | 0.5 Year Storm[1] | 1 year Storm[1] | 2 year Storm[1] | 2 year Storm[1] |
| 002 | 6.3 | 8.4 | 8.7 | 9.0 |
| 003 | 1.4 | 1.4 | 1.4 | 1.4 |
| 004 | 0.0 | 2.9 | 6.1 | 9.8 |
| 005 | 12.6 | 14.2 | 14.5 | 14.8 |
| 006 | 0.1 | 1.7 | 3.7 | 5.6 |
| 007 | 1.7 | 1.7 | 1.7 | 1.7 |
| 008 | 0.5 | 0.9 | 1.4 | 2.0 |

[1]Using 2 h duration.

Design Considerations

The following is a list of key design considerations for the Rockland WWTF retrofit:

- Impacts of High Strength Wastes
- Hydraulic Loadings
- Treatment Objectives
- Operational Requirements
- Chemical Storage and Feed Requirements
- Sludge Handling Requirements
- Operations and Maintenance Requirements
- Construction Sequencing and Site Constraints

Each of these considerations is described in this section.

*Impacts of High-Strength Wastes.* One of the nine minimum CSO controls is the review and modification of pretreatment programs to assure that CSO impacts are minimized. The elimination of the industrial wastewater connection from FMC Food Ingredients Division to the city of Rockland CCS, and construction of a separate force main to the WWTF will satisfy this objective.

The industrial wastewater design loading to the Rockland WWTF is approximately 2,990 kg (6,583 lbs) per day. This is approximately 67% of the total organic loading of 4480 kg (9,871 lbs) that the WWTF receives daily. Following the implementation of Phase II improvements, this industrial wastewater can be directly discharged to three locations at the Rockland WWTF:

- Existing Primary Clarifier Influent at New Flow Distribution Structure No. 1
- Existing Aeration Tank Inlet Channel
- Equalization Tank

The industrial wastewater enters the WWTF via a 20 cm (8 in.) diameter force main connected to a new pre-cast concrete valve vault (SS #2). SS #2 contains valves to direct flow to the three discharge locations. A 20 cm (8 in.) magnetic flow meter measures flow. If the flow is directed to the equalization tank, then it is mixed using a fine bubble diffused air system. The design airflow is 30 scfm per thousand cubic feet. The air blower has a variable frequency drive allowing adjustments to the airflow. The WWTF staff is able to mix the equalization basin at a lower rate, if desired. A total of 324 membrane diffusers each with a maximum airflow of 3.5 scfm are provided in the equalization basin.

Two submersible pumps (1 primary and 1 standby) convey industrial wastewater from the equalization basin to the WWTF. Each pump has a rated capacity of 2.3 $m^3$ (600 gal) per min at a total dynamic head of 4.5 m (14 ft). The pumps are driven with 3.7 kW (5 HP) motors controlled with variable frequency drives that allow the WWTF staff to adjust the flow of industrial wastewater to the process. The discharge from the submersible pumps can be directed to two locations:

- Existing Primary Clarifiers via new Flow Distribution Structure No. 1
- Existing Aeration Tank Inlet Channel

A 15 cm (6 in.) diameter magnetic flow meter is provided on the submersible pump discharge allowing the rate of industrial wastewater flow to be monitored by the WWTF staff.

Other high-strength wastes eliminated from the CSO discharge with the Phase II Improvements are the sidestream flows from the sludge processing system. The sidestream flows represent a relatively small portion of the daily load. Approximately 127 kg (279 lbs) per day are discharged (approximately 3% of the overall design organic loading of 4480 kg (9,871 lbs) per day). However, these loadings are concentrated within a few hours during the day, which magnifies their effects.

The sludge thickener filtrate is produced in a 3.5-4 h period and represents approximately 8% of the hourly organic loading while the sludge de-watering filtrate is produced in a 3.5-4 h period and represents approximately 12% of the hourly organic loading. Overall, the sidestreams represent approximately 20% of the hourly organic loading at the WWTF when the units are in service. The sludge thickening sidestream was redirected to the existing Aeration Tank Inlet Channel while the sludge dewatering sidestream was redirected to the existing Primary Clarifier influent at new Flow Distribution Structure No. 1 with the Phase II Improvements.

*Hydraulic Loadings.* Pollutant and hydraulic loadings were developed for the wet-weather treatment units at the Rockland WWTF during the Facilities Planning and Design stages. The primary objective of the wet-weather treatment units is to remove floatable and settleable solids to promote effective disinfection. The primary objective of the industrial wastewater equalization basin is to remove high-strength industrial wastewater from the CCS to allow full secondary treatment of these wastes at all times.

Table 5.6.a-4 shows the design criteria for the wet-weather treatment processes at the Rockland WWTF.

Table 5.6.a-4. Wet-Weather Treatment Design Criteria

| | Description and Criteria | Phase II Design Value | |
|---|---|---|---|
| A. | Flows and Loadings | | |
| | WWTF Peak Monthly Avg. Flow | 10,100 $m^3$/d | 2.90 MGD |
| | WWTF Peak Daily Flow | 12,500 $m^3$/d | 3.30 MGD |
| | WWTF Instantaneous Maximum | 21,600 $m^3$/d | 5.70 MGD |
| | Total Annual CSO Volume | 186,600 $m^3$ | 49.3 MG |
| | CSO Annual Volume at WWTF | 188,100 $m^3$ | 43.7 MG |
| | CSO North/West Subsystem Peak (5 yr., 2 h) | 127,550 $m^3$/d | 33.7 MGD |
| | CSO North/West Subsystem Peak (2 yr., 2 h) | 110,900 $m^3$/d | 29.3 MGD |
| | CSO North/West Subsystem Peak (1 yr. 2 h) | 96,500 $m^3$/d | 25.5 MGD |
| | CSO North/West Subsystem Peak (1/2 yr. 2 h) | 71,500 $m^3$/d | 18.9 MGD |
| B. | Influent Pumping - Dry Weather | | |
| | No. of Pumps (each) | 2 | |
| | Type | Non-clog Submersible | |
| | Flow | 15 $m^3$/min | 4,000 gpm |
| | TDH | 9 m | 30 ft |
| | Power (each) | 37.3 kW | 50 HP |
| C. | Screenings Handling System | | |
| | Type | Mechanical Screen | |
| | No. of units (each) | 1 | |
| | Channel Width | 91.0 cm | 36 in. |
| | Bar Screen Clear Spacing | 2.5 cm | 1.0 in. |
| | Bar Size | 6.5 x 1.0 cm | 2-1/2 x 3/8 in. |
| | Headloss, half clogged | 22.9 cm | 9.0 in. |
| D. | Grit Removal | | |
| | Type | Pista | |
| | No. of Units (each) | 1 | |
| | Diameter | 3.7 m | 12.0 ft |
| | Hopper Diameter | 1.5 m | 5.0 ft |
| | Type of pump | Recessed Impeller | |
| | No. of pumps (ea.) | 1 | |
| | Flow | 1 $m^3$/min | 260 gpm |
| | TDH | 9 m | 30 ft |
| | Horsepower (each) | 10 | |
| | Type of De-watering | Cyclone | |
| | Pressure | 4,922 kg/$m^2$ | 7.0 psig |
| | Screw Diameter | 30.5 cm | 12.0 in. |
| E. | Wet Weather Pumping | | |
| | No. of pumps (each) | 3 | |
| | Type | Non-Clog Submersible | |

Table 5.6.a-4. Wet-Weather Treatment Design Criteria, continued

| | Description and Criteria | Phase II Design Value | |
|---|---|---|---|
| | Flow each | 30 $m^3$/min | 7,800 gpm |
| | TDH | 8.5 m | 28 ft |
| | Power each | 74.6 kW | 100 HP |
| | Maximum Speed (rpm) | 720 | |
| | Minimum Efficiency (%) | 76 | |
| F. | Vortex Separator | | |
| | No. of Units (each) | 1 | |
| | Type | EPA | |
| | Diameter each | 10.7 m | 35 ft |
| | Sidewater Depth each | 2.7 m | 8.75 ft |
| | 5 year Hydraulic Loading | 1,430 $m^3/d/m^2$ | 35,000 $gpd/ft^2$ |
| | 2 year Hydraulic Loading | 1,240 $m^3/d/m^2$ | 30,400 $gpd/ft^2$ |
| | 1 year Hydraulic Loading | 1,080 $m^3/d/m^2$ | 26,500 $gpd/ft^2$ |
| | ½ year Hydraulic Loading | 780 $m^3/d/m^2$ | 19,600 $gpd/ft^2$ |
| | Influent Pipe Size | 107 cm | 42 in. |
| | Effluent Pipe Size | 107 cm | 42 in. |
| | Underflow Pipe Size | 35.5 cm | 14 in. |
| | Underflow Rate, Minimum | 6435 $m^3$/d | 1.7 MGD |
| | Underflow Rate, Maximum | 12,870 $m^3$/d | 3.4 MGD |
| | No. of Underflow pumps | 2 | |
| | Type of pump | Self Priming Non-Clog | |
| | Pump Flow (each) | 4.5 $m^3$/min | 1,170 gpm |
| | TDH | 4.3 m | 14 ft |
| | Motor power each | 7.5 kW | 10 HP |
| | Minimum Efficiency (%) | 55 | |
| | Maximum Pump Speed (rpm) | 860 | |
| G. | CSO Disinfection And Dechlorination | | |
| | No. of Units (each) | 1 | |
| | Volume | 400 $m^3$ | 105,300 gal |
| | Length to Width Ratio | 40:1 | |
| | Chlorine Detention Time – 5 yr. | | |
| | Q = 114,685.5 $m^3$/d (30.3 MGD) (min) | 5.0 | |
| | Chlorine Detention Time – 2 yr. | | |
| | Q = 98,031.5 $m^3$/d (25.9 MGD) (min) | 5.8 | |
| | Chlorine Detention Time – 1 yr. | | |
| | Q = 83,648.5 $m^3$/d (22.1 MGD) (min) | 6.7 | |
| | Chlorine Detention Time – ½ yr. | | |
| | Q = 58,667.5 $m^3$/d (15.5 MGD) (min) | 9.2 | |
| | Dechlorination Tank Volume | 19 $m^3$ | 5,100 gal |
| | Dechlorination Detention Time – 5 yr. | | |
| | Q = 114,685.5 $m^3$/d (30.3 MGD) (s) | 15 | |
| | No. of $NaHSO_3$ pumps (each) | 2+1 standby | |
| | Capacity of $NaHSO_3$ Pumps (each) | 0.4 $m^3$/h | 105 gph |
| | No. of Sodium Bisulfite Pumps (each) | 1+1 standby | |
| | Capacity of $NaHSO_3$ Pump (each) | 0.1 $m^3$/h | 25 gph |

*Treatment Objectives.* Sampling and monitoring performed during the development of the CSO Abatement Program indicated that Rockland Harbor does not meet current water quality criteria for enterococcus bacteria during wet weather. Currently, the State of Maine allows a maximum geometric mean enterococcus concentration of 14 colonies per 100 ml and a daily maximum enterococcus concentration of 94 per 100 ml to comply with Standards established for Class SC waters. The primary focus of the phase II Improvements are:

- Increasing the quantity of CSO conveyed to the WWTF for treatment and disinfection.
- Segregating high-strength industrial wastewater and WWTF sidestreams from the overflow at the influent pump station.
- Providing high-rate primary treatment to remove floatable and settleable solids from the CSO to facilitate disinfection.

The CSO Abatement Program recommended a plan to treat CSO discharges to meet the current maximum daily water quality limits. The high-rate primary treatment and disinfection processes were selected as the most economical method to meet the treatment objectives to meet water quality objectives for this project.

*Operational Requirements.* The Phase II Improvements to the Rockland WWTF which are designed to operate on a continuous basis include:

- Dry Weather Pumping Station
- Industrial Wastewater Equalization
- Mechanical Screening
- Grit Removal
- Primary Treatment

The wet-weather pumping system, high-rate primary treatment (vortex separator) high-rate disinfection, and dechlorination systems are intended to operate in an event-based mode during periods of excessive WWF. Under normal dry weather conditions flow from the north and south CCS enter a new diversion structure (SS #1) upstream of the influent pumping station. SS #1 contains a side overflow weir separating the overflow from the normal sewage flow through the structure. As sewage flows increase during a wet-weather event, the new submersible dry weather pumps will be throttled to control flow to the secondary WWTF. As water levels rise in the dry-weather pump wetwell flow will automatically be diverted to the overflow chamber. These overflows are conveyed to the wetwell of the wet-weather pumping system. When levels in the wet-weather wetwell rise, the wet-weather pumps will automatically start and convey flow to the new vortex separator via a 1.0 m (42 in.) force main. Once flow through the vortex separator has been established, a level switch located at the inlet to the high-rate CSO Disinfection and Dechlorination Structure will start the vortex underflow pumping system. The vortex underflow pumps are rated to deliver 4.50 $m^3$ (1,170 gal) per min at

4.5 m (14 ft) total dynamic head. Each pump has a 7.5 kW (10 HP) electric motor with a variable frequency drive allowing the WWTF staff to adjust the rate of vortex underflow.

The level switch located at the inlet of the CSO Disinfection and Dechlorination Structure will also start the chemical metering pumps (sodium hypochlorite for disinfection and sodium bisulfite for dechlorination) and the tank mixers.

An ultrasonic flow meter is provided at the 160 cm (64 in.) broad crested rectangular weir at the inlet of the CSO Disinfection and Dechlorination Structure. The flow meter records CSO flows, and provides a 4-20 mA signal to control the speed of the sodium hypochlorite and sodium bisulfite chemical metering pumps. Flow pacing of the chemical feed rate will improve disinfection and dechlorination efficiency, and reduce overall chemical consumption.

At the end of the wet weather event automatic level controls will stop the submersible wet-weather pumps. When the influent flow stops, the level in the vortex separator will fall as it is drained with the vortex underflow pumping system. The chemical metering pumps and tank mixers automatically stop when the level in the CSO Disinfection and Dechlorination Structure falls below the pre-selected set point. The WWTF staff would then drain the CSO Disinfection and Dechlorination Structure and wash down the structures to prepare for the next event.

*Chemical Storage and Feed Requirements.* The CSO Abatement facilities will utilize two chemicals:

- Liquid Sodium Hypochlorite for disinfection
- Liquid Sodium Bisulfite for dechlorination

The liquid sodium hypochlorite system is designed for commercial-grade liquid sodium hypochlorite (15% solution) which contains the equivalent of 150 kg of chlorine/$m^3$ (1.25 lbs $Cl_2$/gal) of solution. The metering pumps can deliver a dosage of 25 mg/L at the peak design flow with one unit out of service. The bulk storage tank has an effective working volume of 4 $m^3$ (1,000 gal). This tank is interconnected with the existing 15 $m^3$ (4,000 gal) sodium hypochlorite bulk storage tanks used for secondary effluent disinfection, giving the WWTF a total of 19 $m^3$ (5,000 gal) of hypochlorite storage. The available chlorine in sodium hypochlorite solution declines with age, the anticipated sodium hypochlorite storage time will be 52 days at the design flow of 10,980 $m^3$/d (2.9 MGD).

The liquid sodium bisulfite storage system is designed for commercial grade liquid sodium bisulfite (38% solution) which contains appropriately 500 kg sodium bisulfite/$m^3$ (4.2 lbs $NaHSO_3$/gal) of solution. The metering pumps can deliver sodium bisulfite dosages necessary to dechlorinate effluent with a chlorine residual of 5.0 mg/L. Approximately 1.6 kg of sodium bisulfite will be used per kg of chlorine residual. The bulk storage tank has an effective working volume of 4 $m^3$ (1,000 gal). This tank is interconnected with the existing 8 $m^3$ (2,000 gal) sodium bisulfite tank used for

secondary effluent dechlorination, giving the WWTF a total of 12 $m^3$ (3,000 gal) of sodium bisulfite storage.

*Sludge Handling Requirements.* Sludge handling requirements increase during periods of WWF. The most pronounced increase occurs during the initial period of WWF (first flush).

A solids balance was performed to estimate the quantity of sludge produced during a one yr frequency, two hr duration design storm. The daily sludge quantities would increase as follows:

- Estimated Sludge Quantity 2,870 kg (6,326 lbs) per d
- One-Year Storm Estimated Sludge Quantity 3,640 kg (8,029 lbs) per d

The sludge quantity is estimated to increase by approximately 27% during a one-year storm event. The overall increase can be assimilated if sludge pumping and processing schedules are adjusted during the remainder of the week. The following is a summary of weekly sludge quantities during a period with a one-year duration, two-hour frequency design storm:

- Initial Estimated Sludge Quantity (kg/week) 20,090
- Projected Sludge with One-Year Design Storm (kg/week) 20,860

The estimated weekly sludge quantity would increase by approximately 4% during a week with a one-year storm event.

*Operation and Maintenance Requirements.* The implementation of the Phase II Improvements will result in increased O&M costs. The following list summarizes the areas of expected O&M cost increases at the WWTF:

- Labor to perform sampling, laboratory analyses, reporting, wet-weather process adjustments, and clean-up following an event.
- Power for in-plant pumping, sludge processing and chemical feed systems.
- Chemicals for disinfection, dechlorination, and polymer for additional sludge quantities.
- Sludge disposal tipping and transport fees.

Table 5.6.a-5 summarizes the expected increase in O&M costs at the WWTF resulting from the Phase II Improvements.

Table 5.6.a-5. Estimated O&M Costs Phase II CSO Improvements

| Description | Estimated Annual Cost |
|---|---|
| Sampling | $600 |
| Laboratory Analyses | 2,500 |
| Report Preparation | 1,200 |
| Process Control Adjustments | 1,000 |
| Clean-Up of Wet-Weather Treatment Structures | 4,500 |
| Power | 700 |
| Sodium Hypochlorite | 3,100 |
| Sodium Bisulfite | 1,600 |
| Polymer for Sludge De-watering | 300 |
| Sludge Disposal Transport and Tipping Fees | 4,000 |
| Total Estimated Cost | $19,500 |

*Construction Sequencing and Site Constraints.* The existing Rockland WWTF site is crowded with little room for new facilities in the area where the influent sewers enter the WWTF. Therefore, it was necessary to construct some of the CSO Abatement Facilities in locations previously used for other purposes. The following is a listing of special sequencing and site considerations related to the CSO Abatement Program at the WWTF:

- A storage garage was demolished to facilitate construction of the vortex separator.
- There was limited space available for wet-weather pumping. The existing preliminary treatment building (which contained the influent pumps) was converted into a submersible pumping station for DWF and WWF. The dry weather pumps are installed in the existing wetwell. The wet-weather pumps are installed in the existing pump room that is converted into a wetwell for the submersible wet-weather pumps. The modifications will require temporary bypass pumping of flow into the WWTF during construction.
- New junction structures are required on the existing outfall pipe (SS #4 and SS #5) to increase the hydraulic capacity. An emergency relief discharging to Lermond Cove (adjacent to the WWTF site) will be preserved.

A substantial completion time of 510 calendar days was specified for the Phase II Improvements.

Project Costs

The bids for the Phase II Improvements were opened on July 8, 1998. Costs for the CSO related components of the project are listed in Table 5.6.a-6. Costs include construction requirements and project services. Project services include design engineering, construction engineering, legal, and fiscal expenses. Project services costs do not include contingencies, right-of-way expenses, or other miscellaneous items.

Table 5.6.a-6. Project Costs -Phase II CSO Improvements

| Description | Cost[1] |
|---|---|
| General Conditions | $175,000 |
| Site Work and Yard Piping | 375,000 |
| Consolidation Conduit | 140,000 |
| Special Structure No. 1 | 90,000 |
| Dry Weather/Wet Weather Pumping Station Renovation | 180,000 |
| Headworks/CSO Disinfection and Dechlorination Structure | 1,080,000 |
| Vortex Separator | 240,000 |
| Special Structure No. 2 | 40,000 |
| Equalization Tank Renovation | 58,000 |
| Industrial Wastewater Force Main | 22,000 |
| Flow Distribution Structure No. 1 | 60,000 |
| Flow Distribution Structure No. 2 | 65,000 |
| Special Structure No. 4 | 60,000 |
| Special Structure No. 5 | 55,000 |
| Yard Electrical Ductbanks and Lighting | 90,000 |
| Instrumentation and SCADA System | 70,000 |
| Total Construction Cost | 2,800,000 |
| Project Services[2] | 600,000 |
| Total Project Cost | 3,400,000 |

[1]Based on July 8, 1998 Bid Prices
[2]Includes Design Engineering, Construction Engineering, Legal, and Fiscal Expenses. Does not include contingencies and right-of-way costs.

Conclusions

The retrofit of an unused aeration tank and the construction of a separate force main from the FMC Corporation Food Ingredients Division facility to the WWTF provides a cost-effective alternative for the elimination of a established portion of high-strength wastes from CSO discharges at the Rockland WWTF. This retrofit, combined with other WWTF improvements directed at CSO abatement, will direct high-strength waste from the FMC facility to the dry-weather headworks, eliminating the potential for CSO discharge of industrial wastewater. This retrofit was part of an overall CSO Abatement Program necessary to address system-wide CSO discharges.

## 5.7 Bringing Outdated/Abandoned POTWs Back Online as Wet-Weather Flow Treatment Facilities

The wastewater treatment facilities remaining following upgrades or construction of new POTWs are often abandoned or demolished. These outdated or abandoned facilities could possibly serve a useful function as part of the new treatment facilities depending on their condition and location with respect to the new treatment facilities and the combined sewer system. These facilities may be utilized as wet-weather storage or flow equalization facilities.

### *5.7.a. Auburn WWTF, Auburn, New York*

Background

The city of Auburn Water Pollution Control Plant was initially constructed during the 1930s and provided primary treatment. In 1970, the original primary treatment facilities were upgraded to provide secondary treatment. Since that time, several improvements have been completed.

The WWTF includes preliminary treatment (screening and grit removal), primary sedimentation, trickling filtration, final sedimentation, and chlorination. Sludge treatment facilities include gravity sludge thickeners, belt filter press for sludge dewatering, and incinerators for sludge disposal. The ash from the incineration process is landfilled.

The Auburn WWTF services the city of Auburn and the surrounding Towns of Aurelius, Fleming, Sennett, and Owasco. Portions of the existing sewer system consist of combined sewers that carry both sanitary sewage flow and storm sewer flow in the same pipe.

In December 1989, the city of Auburn entered into a consent order agreement with the New York State Department of Environmental Conservation (NYSDEC). The consent order required the city of Auburn to complete an Infiltration/Inflow Analysis, a CSO Analysis, BMP Program, and a Comprehensive Performance Evaluation (CPE) for the city of Auburn Water Pollution Control Plant.

Phase I improvements to the city of Auburn WWTF were performed between 1993-1995, as summarized below:

1. New preliminary treatment facilities consisting of mechanical bar screens housed in a new influent building, aerated grit channels, and new influent flow metering.
2. New primary treatment facilities, including new primary settling tanks and sludge handling pumps and rehabilitation of the existing- settled sewage pumps and controls.

3. A new ORF utilizing the abandoned primary settling tanks, with chlorination (using sodium hypochlorite) and dechlorination (using sodium bisulfite) facilities and flow metering equipment for the ORF overflow.

4. Trickling filter rehabilitation, including replacement of rotary distribution arms, rehabilitation of existing recirculation pumps and controls, and installation of recirculation flow metering equipment.

5. New septage receiving facilities, including an enclosed septage receiving station, mechanical septage screening, underground septage storage tank, and septage handling equipment.

6. Odor control facilities to treat ventilated air from influent building, screenings and grit loading area, septage receiving station and septage storage tank, and belt press room of the existing administration building.

Construction costs for the Phase I improvements were estimated to be $9,600,000. Figure 5.7.a-1 shows the WWTF flow schematic prior to construction of the Phase I improvements

The following summarizes the retrofit of the abandoned primary treatment tanks for use as overflow retention facilities for the treatment of WWF in excess of the treatment plant capacity.

Previous Studies

As a requirement of the consent order with New York State, the city of Auburn has completed studies on I/I and CSO abatement.

*Infiltration and Inflow Study.* In January 1991, the city of Auburn completed an I/I study of the wastewater collection system. The purpose of this study was to develop a recommended plan of corrective action to eliminate excessive I/I within the city's sanitary sewer system. The study recommended the removal of sewer system conveyance restrictions and the elimination of direct inflow sources. The results of the I/I study indicated that infiltration was not excessive and that it was more cost effective to transport and treat, than to eliminate. The study also suggested that approximately 31,420 $m^3$/d (8.3 MGD) of peak inflow could cost effectively be removed.

*CSO Abatement.* In July 1993, the city of Auburn completed a CSO abatement study/comprehensive water collection system plan. The CSO abatement study included the evaluation of alternatives for the elimination, storage, and/or treatment of CSOs that discharge to the Owasco Outlet within the city of Auburn. The CSO study identified the alternative of centralized high rate treatment as the most cost effective and environmentally sound alternative for the city of Auburn's collection system. This alternative includes the construction of a new interceptor to increase the capacity of the existing North Owasco and South Interceptors, and the construction of a high rate treatment facility and diversion structure at the existing WWTF. The CSO study recommended the implementation of the North Interceptor improvements as described

## 5.7.a-1 Existing Wastewater Treatment Flow Diagram

in the North Interceptor Replacement Sewer Basis of Design Report dated April 1993. Also included in the CSO study was an estimate for the future (year 2010) total peak flow to the WWTF of 191,000 $m^3$/d (50.46 MGD). Of this total flow, 44,970 $m^3$/d (11.88 MGD) was estimated for total base flow plus infiltration.

*Comprehensive Performance Evaluation.* In June 1992, an interim CPE report for the city of Auburn was completed. This interim report developed the conceptual WWTF improvements required to meet the anticipated reclassification of Owasco Outlet and revised effluent limits.

Project Approach

The CSO report recommended the construction of peak flow storage facility on the North Interceptor and the construction of parallel interceptors to convey peak flows to the WWTF. This report had also recommended the construction of new overflow facilities, such as a swirl concentrator, to treat flows above 78,400 $m^3$/d (20.7 MGD). These facilities were completed and operation began in July 1996.

In lieu of the swirl concentrators to treat peak flows at the WWTF, an ORF was evaluated. The ORF would treat flows above the 95,000 $m^3$/d (25 MGD) WWTF capacity and be constructed from the abandoned primary settling tanks, which became available after the treatment plant was upgraded. The reuse of this tankage represented a significant cost saving to the city of Auburn.

Overflow Retention Facilities

The abandoned primary settling tanks were rehabilitated to function as an ORF. The basis of design for the ORF is summarized in Table 5.7.a-1 as described below.

Table 5.7.a-1. Basis of Design of the Overflow Retention Facilities

| Overflow Retention Tanks | Peak hourly flow 96,500 $m^3$/d (25.5 MGD) |
|---|---|
| Number of Units | 5 (abandoned primary settling tanks) |
| Total Surface Area | 782 $m^2$ (8,416 $ft^2$) |
| Maximum SOR | 123 $m^3/d/m^2$ (3,025 $gpd/ft^2$) |
| Equipment | |
| Overflow Return Pumps | |
| Number of Units | 4 |
| Type | Centrifugal |
| Return Duration | 24 h |
| Overflow Influent Metering | |
| Type | Open Channel |
| Flow Element | Weir |
| Flow Meter | Ultrasonic |
| Range | 0—11,355 $m^3$/d ( 0—3 MGD) |

Table 5.7.a-1. Basis of Design of the Overflow Retention Facilities, continued

| | |
|---|---|
| High-Rate Disinfection | |
| Number of Units | 1 (abandoned primary settling tank #6) |
| Contact Tank Volume | 318 $m^3$ (11,220 $ft^3$) |
| Detention Time | 4.7 min at peak hourly flowrate |
| Tank Dimensions (existing) | 5.9 m x 20.9 m x 2.6 m |
| (*Width x Length x Side Water Depth)* | (19.5 ft x 68.5 ft x 8.4 ft) |
| Mixing Energy | 22.4 kW (30 HP) |
| Chlorination Chemical Feed Equipment | |
| Type | Liquid Sodium Hypochlorite |
| Dosage | 25 mg/l |
| Dechlorination | |
| Detention Time | 12 s at peak hourly flow |
| Contact Tank Volume | 13 $m^3$ (458 $ft^3$) |
| Tank Dimensions | 1.5 m x 3.6 m x 2.4 m |
| *(Width x Length x Side Water Depth)* | (5 ft x 11.85 ft x 7.74 ft) |
| Equipment | |
| Dechlorination Chemical Feed | |
| Type | Liquid Sodium Bisulfite |
| Dosage per mg/l of chlorine residual | 1.5 mg/l |
| Mixers | |
| Number of Units | 1 per tank |
| Type | Constant Speed |
| Tank Flushing System | |
| Source | Chlorinated Plant Effluent |
| Overflow Effluent Metering | |
| Type | Open Channel |
| Flow Element | Open Channel |
| Flow Element | Weir |
| Flow Meter | Ultrasonic |
| Range | 0—113,550 $m^3$/d (30 MGD) |

Influent flows in excess of the 95,000 $m^3$/d (25 MGD) WWTF capacity are directed to the retention facility through a controlled diversion structure. The maximum SOR provided by using five of the six abandoned primary settling tanks as an overflow retention tank(s), is 123 $m^3/d/m^2$ (3,025 gal/d/$ft^2$) at the estimated peak hourly flow of 96,500 $m^3$/d (25.5 MGD). Overflow events producing less than approximately 4,100 $m^3$ (1.083 MG), which includes volume disinfection contact chamber, are retained and returned to the treatment process when influent flows subside. Overflow events producing in excess of the available storage volume are disinfected using chlorine and

dechlorinated before being discharged to the Owasco Lake Outlet. The retained volume is returned to the aerated grit chambers utilizing four overflow return pumps. Figure 5.7.a-2 shows the flow schematic of the Auburn WWTF following construction of the ORF.

The overflow retention facilities include the following:

*Modifications to the Abandoned Primary Settling Tanks.* The primary sludge longitudinal and cross collectors were removed to use the abandoned primary settling tanks as an ORF. Deteriorated concrete surfaces were repaired and weirs were installed to allow flow between tanks. A high volume tank flushing system was provided that uses chlorinated plant effluent to wash the tank walls and bottom following an overflow event.

*Overflow Return*. The old primary sludge pump was replaced with overflow return pumps. The new pumps are suitable for pumping grit and primary sludge. In order to prevent odors, the entire overflow retention tank volume is returned to the treatment process within 24 h after the occurrence of an overflow event provided influent flow rates have subsided.

*Disinfection.* Overflow events producing stormwater in excess of the available storage volume are disinfected using a high-rate disinfection system. The system consists of a contact tank sized for a 4 min detention time at peak hourly flows of 96,520 $m^3$/d (25.5 MGD) and requires the use of mechanical mixers to induce energy to provide effective disinfection. The mixing velocity gradient and chlorine dosage were calculated on the basis of estimated influent and effluent fecal coliform levels for CSOs, detention time, concentration of TKN, BOD, and an estimated value for pH. The high-rate disinfection system was sized by estimating disinfectant dosage and mixing energy required to assure the inactivation of total coliform. Based on the optimum velocity gradient calculated, horsepower requirements for mixers were determined. From this analysis, it was determined that an optimum chlorine dosage of 25 mg/l and approximately 22.4 kW (30 HP) of total mixing energy are required to insure proper disinfection.

The abandoned Primary Settling Tank #6 was modified for use as the high-rate disinfection contact tank. The chlorination facilities are housed in a new structure constructed over a portion of the new overflow retention tank. This structure houses the chlorination and dechlorination equipment. The chlorination equipment was designed to be automatically flow-paced according to the overflow flow rate.

*Dechlorination*. Dechlorination facilities were installed to reduce the chlorine residual before discharge to the Owasco Outlet. The dechlorination facilities are metered automatically as a function of overflow flow rate and will be designed to minimize chlorine residual. The dechlorination tank contains a high-rate mixer designed for a detention time of 30 s at peak hourly flows of 96,500 $m^3$/d (25.5 MGD). This tank was constructed within a portion of abandoned Primary Settling Tank #6.

*Overflow Metering.* Overflows discharged to the Owasco Lake Outlet are metered after being discharged from the dechlorination tank through an open channel-type metering device located in a separate concrete structure.

Figure 5.7.a-2. Conceptual Wastewater and Sludge Treatment Flow Diagram

Stormwater Management Model Results

The EPA SWMM Version 4 was used to predict the annual capture of WWF that previously discharged at the WWTF and is now treated through the ORF. A simplified model of the WWTF drainage area was used to determine the impact of pre- and post-retrofit. This analysis is based on 21 years of hourly rainfall from the NWS Station at Hancock Field in Syracuse, NY. Table 5.7.a-2 demonstrates that use of the ORF resulted in a considerable annual capture of WWF that was previously discharged without treatment.

Table 5.7.a-2. Average Annual Overflow Volume from the Pre- and Post-Retrofit Facilities (1971 – 1991)

| Condition | Average Annual Overflow Volume ($m^3$) | Average Annual Overflow Volume (MG) |
|---|---|---|
| Pre-Retrofit WWTF Overflow | 29,000 | 7.65 |
| Post-Retrofit Overflow[(1)] | 7,000 | 1.85 |
| Percent Capture | 76% | |

[(1)]Overflows receive preliminary treatment and disinfection prior to discharge

The ORF annually captures 22,000 $m^3$ (5.8 MG) of wet-weather overflow at the WWTF that is returned to the WWTF influent for treatment following storm events.

Design Considerations

The retrofitting of the abandoned primary settling tank to provide enhanced wet-weather treatment through storage and disinfection required careful consideration of several factors. The following is a list of key design considerations for this retrofit:

- Pollutant and Hydraulic Loadings
- Treatment Objectives
- Operational Requirements
- Chemical Storage and Feed Requirements
- Sludge Handling Requirements
- Operations and Maintenance Requirements

Each of these considerations will be described in this section.

*Pollutant and Hydraulic Loadings.* Pollutant and hydraulic loadings were developed for the ORF during the Facilities Planning stage. These values were refined and updated as the project progressed through the preliminary engineering and detailed design

stages. The primary objective of the ORF is to provide preliminary treatment and disinfection of WWF beyond the 95,000 $m^3$/d (25 MGD) WWTF capacity. The basis of design for the ORF is shown on Table 5.7.a-1.

*Treatment Objectives.* The rationale for this retrofit was meeting the revised effluent limits from the WWTF in the Owasco Lake outlet. Preliminary treatment and high-rate disinfection were selected as the most economical method to meet the treatment objectives for this project.

*Operational Requirements.* The ORF are designed to operate on a continuous basis include:

- Preliminary Treatment
- High-Rate Disinfection

Each of the five ORF tanks fill uniformly during an overflow event. After the wet-weather flows subside, the flow retained in the ORF is pumped to the WWTF headworks for treatment. WWTF staff manually controls this operation. The tanks are flushed by a manually controlled system using chlorinated plant effluent.

*Chemical Storage and Feed Requirements.* The ORF utilize two chemicals:

- Liquid Sodium Hypochlorite for disinfection
- Liquid Sodium Bisulfite for dechlorination

The liquid sodium hypochlorite system design is based on commercial-grade liquid sodium hypochlorite (15% solution) which contains the equivalent of 160 kg of chlorine/$m^3$ (1.25 lbs $Cl_2$/gal) of solution. The metering pumps are designed to deliver a dosage of 25 mg/L at the peak design flow with one unit out of service. The liquid sodium hypochlorite is stored in a bulk storage tank with an effective working volume of 10 $m^3$ (2,500 gal). The available chlorine in sodium hypochlorite solution declines with age. The anticipated sodium hypochlorite storage time will be 51 days initially, decreasing to 25 days at the future design flow.

The liquid sodium bisulfite storage system design is based on commercial grade liquid sodium bisulfite (38% solution) that contains appropriately 503 kg of sodium bisulfite/$m^3$ (4.2 lbs $NaHSO_3$/gal) of solution. The metering pumps are designed to deliver sodium bisulfite dosages necessary to dechlorinate effluent with 1.5 mg/L of sodium bisulfite per mg/L of chlorine residual. Approximately 1.6 kg of sodium bisulfite will be used per kg of chlorine residual. A bulk storage tank with an effective working volume of 3.8 $m^3$ (1,000 gal) is provided.

The sodium hypochlorite and sodium bisulfite metering pumps are flow paced. An ultrasonic type level sensor is used for effluent flow measurement. A broad-crested rectangular weir 3 m (10 ft) in length is provided as the primary measuring device.

*Sludge Handling Requirements.* Accumulated sludge will be handled through the pump back of retained WWF and the associated tank flushing following tank draining.

*Operation and Maintenance Requirements.* The implementation of the ORF will result in increased O&M costs. The following list summarizes the areas of expected O&M cost increases at the WWTF:

- Chemicals for disinfection and dechlorination
- Power for tank draining and flushing, and chemical feed systems.

The estimated annual O&M costs are provided in Table 5.7.a-3.

Table 5.7.a-3. ORF O&M Costs

| Annual Chemical Costs | Cost |
|---|---|
| Sodium Hypochlorite | $7,000 |
| Sodium Bisulfite | $3,000 |
| O&M | $8,000 |
| Total | $18,000 |

Project Costs

Project costs for the capital costs for the ORF are provided in Table 5.7.a-4.

Table 5.7.a-4. ORF Chlorination/dechlorination Capital Costs

| Chlorination | Cost |
|---|---|
| Chemical Feed Pumps and Controls | $20,000 |
| Storage Facilities | $14,000 |
| Installation | $21,000 |
| Subtotal | $55,000 |
| Dechlorination | |
| Chemical Feed Pumps and Controls | $20,000 |
| Storage Facilities | $16,000 |
| Residual Analyzer System | $10,000 |
| Installation | $19,000 |
| Subtotal | $65,000 |
| Total Cost | $120,000 |

The conventional treatment alternative for this retrofit would be to construct wet-weather storage facilities to equalize WWF for ultimate treatment at the WWTF. This alternative would require the construction of a 4,100 $m^3$ (1.083 MG) ORF. The estimated cost to construct this facility based upon EPA cost guidelines for CSO control technologies (EPA 1992) is $3.816 million derived from the following equation:

$$\text{Cost} = 3.577\ V^{0.812} \quad (1)$$

where,

Cost = Average construction cost, millions of dollars

V = Storage volume, MG

The ORF retrofit is a simple solution, requiring minimal O&M cost, resulting in a total cost that is a fraction of the cost of constructing new wet-weather storage facilities. The ORF retrofit provides an average annual 76% wet-weather volume capture. The cost for this retrofit is $120,000 dollars compared to the $3,816,000 cost for conventional storage facilities.

## Conclusions

Retrofitting the abandoned primary treatment tanks for use as ORF was a success, in large part due to the availability and condition of the existing facilities. The abandoned primary tanks were in good condition and required only minor modifications for use as an ORF. The ORF retrofit is a cost-effective alternative in cases where abandoned primary tanks are available in good condition for retrofitting. This type of retrofit may be a less cost-effective solution for cases in where the existing tanks are in poor condition requiring a substantial cost to upgrade the tanks to serve a functional role as an ORF.

## Section 6 – References

### REFERENCES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## APPENDIX A

## BIBLIOGRAPHIC DATABASES

**Wilson Applied Science & Technology Abstracts (WBA)**

H.W. Wilson Company
950 University Ave.
Bronx, NY 10452

Phone: (718) 588-8400
800-367-6770
Fax (718) 538-2716

Contact: Technical Support Department.
E-mail: techmailinformationo.hwwiison.com (Internet).

**Type**: Bibliographic,

**Content**: Contains abstracts to all articles cited in the *Applied Science Technology Index* (described in a separate entry) from March 1993 to date. Includes an index of non-English-language periodicals English abstracts are provided. Sources include 400 English language scientific and technical publications.

**Subject Coverage**. Applied science and technology, including such areas as aeronautics and space science, chemistry, computer technology, construction, data processing, energy, engineering, the environment, fire prevention, food, geology, machinery, marine technology and oceanography, mathematical metallurgy, meteorology, mineralogy, optics, petroleum and gas, physics, robotics, solid state technology, telecommunications, textiles, transportation, and water and waste.

**Language**: English.

**Geographic Coverage**: International.

**Time Span**: October 1983 to date (indexing coverage); March 1993 to date (Abstracts).

**Updating**: Approximately 1,000 records per week.

Compendex*Plus
Engineering Information, Inc. (Ei)
Castle Point on the Hudson
Hoboken, NJ 07030

Phone: (201) 216-8500,
800-221-1044
Fax: (201) 216-8532

**Type**: Bibliographic.

**Content**: Contains more than 3.63 million citations, with abstracts, from more than 2,500 journals, reports, books, and conference proceedings covering fields of engineering and technology. Sources include approximately 2500 journals, reports, books, and conference proceedings. Conference review records contain conference code numbers that are links to complete coverage of all individual papers. Corresponds in part to *The Engineering Index Monthly and Engineering Index Annual.*

**Subject Coverage**: All fields of engineering, including related areas of app science, management, and energy. Specific fields covered include civil engineering, construction and materials, transportation, waterworks, sanitary, engineering, bioengineering, ocean engineering, mining, petroleum, fuel technology, metallurgical engineering, mechanical engineering, aerospace, engineering, automotive engineering, electronics and electrical engineering, control engineering, chemical engineering, nuclear engineering, and computers and robotics engineering.

**Language**: English.

**Geographic Coverage**: International.

**Time Span**: 1970 to date.

**Updating**: Weekly; 13,000 records a month.

**Environmental Sciences & Pollution Management Database**
Cambridge Scientific Abstracts (CSA) Phone: (301)961-6750
7200 Wisconsin Ave., Ste. 601 800-843-7751
Bethesda, MD 20814-4823 Fax: (301)961-6720

**Type**: Bibliographic.

**Content**: Contains more than 64,000 citations, with abstracts, to literature covering the environmental sciences. Covers some 1498 core journals, monographs, and conference proceedings, as well as 5500 secondary sources, drawn from the primary sources of abstracts journals such as: Ecology, Pollution Digest of Environmental Impact Statements, toxicology, Health & Safety Science, Bacteriology, Environmental Engineering, and others.

**Subject Coverage**: Environmental sciences, including agricultural biotechnology, aquatic environments, bacteriology, ecology, environmental biotechnology, environmental engineering, environmental impact statements, industrial hygiene, microbiology applied to industrial and environmental issues, pollution (land, air, water, noise, and solid waste), risk assessment, safety science, toxicology, water quality, and water resource issues.

**Language**: English.

**Geographic Coverage**: United States.

**Time Span**: 1983 to date.

**Updating**: Monthly. 76,DOO records/year.

**National Center for Environmental Publications and Information**
United States Environmental Protection Agency
Washington, D.C.
http://www.epa.gov/ncepihom/index.html

**Type**: National Publications Catalog

**Content**: 5,500 titles in paper and/or electronic.

**Subject Coverage**: Repository for all EPA documents.

**Language**: English.

**Time Span**:

**Updating:** Yearly.

**ProCite**

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**Type**: Bibliographic.

**Content**: 3,700 wet-weather references.

**Subject Coverage**: The database was prepared by Robert Pitt for the Wet Weather Floe Design for the Future project under contract by the United States Environmental Protection Agency.

**Language**: English.

**Time Span**: 1899-1997

**Updating**: Database constructed in 1997.

# APPENDIX B

# SELECTED TABLES WITH ENGLISH EQUIVALENT UNITS

Table 5.1.b-2. Comparison of ACTIFLO and Existing Primary Treatment Systems

| Parameter | Existing System | ACTIFLO | Net Savings with ACTIFLO |
|---|---|---|---|
| Total Capacity | 240 MG | 240 MG | - |
| Number of Units | 8 | 6 | - |
| Unit Capacity | 30 MGD | 40 MGD | - |
| Area Required | 14,307 $ft^2$ (per unit)<br>114,453 $ft^2$ (total) | 945 $ft^2$ (per train)<br>5,670 $ft^2$ (total) | 94% per unit<br>95% total |
| Process Volume | 143.066 $ft^3$ (per unit)<br>1,114,530 $ft^3$ (total) | 20,790 $ft^3$ (per unit)<br>124,740 $ft^3$ (total) | 85% per unit<br>89% per total |
| Hydraulic Retention Time | 51 min | 5.6 min | 89% |