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# A study on vortex separator technology to control combined sewer overflow

Nawshin Rummnan
*Ryerson University*

# A STUDY ON VORTEX SEPARATOR TECHNOLOGY TO CONTROL COMBINED SEWER OVERFLOW

By
Nawshin Rumman
B. Sc. Eng. (Civil), Bangladesh Institute of Technology, 2002

A project report
Presented to Ryerson University in partial fulfillment of the
Requirements for the degree of Master of Engineering
In the Program of Civil Engineering

Toronto, Ontario, Canada, 2005

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Nawshin Rumman
Department of Civil Engineering
Ryerson University

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Nawshin Rumman
Department of Civil Engineering
Ryerson University

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The undersigned certify that they have read, and recommend to the School of Graduate Studies for acceptance, a project entitled "A Study on Vortex Separator Technology to Control Combined Sewer Overflow" submitted in partial fulfillment of the requirements for the degree of Master of Engineering in the program of Civil Engineering.

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| | Supervisor<br>Department of Civil Engineering |
| | Department of Civil Engineering |
| | Date |

**A Study on Vortex Separator Technology to Control Combined Sewer Overflow**

By: Nawshin Rumman

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Department of Civil Engineering, Ryerson University

## ABSTRACT

A combined-sewer overflow (CSO) is a significant contributor of contamination to surface waters. During a rain event, the flow in a combined sewer system (CSS) may exceed the capacity of the intercepting sewer leading to a wastewater treatment plant, thus releasing a mixture of storm water and raw sanitary wastewater into the receiving water. As CSOs contain untreated domestic, commercial, and industrial wastes, as well as surface runoff, many different types of contaminants can be present. Because of these contaminants and the volume of the flows, CSOs can cause a variety of adverse impacts on the physical characteristics of surface water, impair the viability of aquatic habitats, and pose a potential threat to drinking water supplies. The resulting short-term problems are poor aesthetics (floatables, turbidity, oil and grease) and beach closure due to increased harmful bacteria levels. The long term impacts include reduced dissolved oxygen in receiving waters, eutrophication and sediment contamination. Since CSO is considered to be a major source of water quality impairment for the receiving waters, much attention has been directed to reducing the quantity and quality of CSO discharged to meet the Ministry of Environment guidelines. There are several approaches to control the quantity and quality of CSO. The selection of a particular treatment technology depends on various factors such as site conditions, CSO characteristics, receiving water quality requirements. One of the emerging options is the vortex separator technology for High Rate Treatment (HRT) facilities at overflow location. There are many devices for CSO control in different trade names where vortex separator technology has been used (e.g. EPA Swirl Concentration, FluidSep™, Storm King™ , CDS®). This study articulates the different CSO control technologies with emphasized on vortex separator technology. The City of Niagara Falls HRT pilot project for CSO control to the Niagara River is presented as a case study in this report. The performance of two HRT devices –Storm King™ and CDS® are evaluated in this pilot project. Analytical Probabilistic Model has been used as a tool in this study to evaluate the potential pollution reduction at the Niagara Falls CSO system.

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- The City of Niagara Falls
- The Regional Municipality of Niagara
- National Water Research Institute
- CDS Technologies Inc.
- Hydro International Plc.
- R.V Anderson Associates Ltd.
- Ryerson University

# DEDICATION

To my family for their love and support

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# CHAPTER ONE

## INTRODUCTION

### 1.1 Background

Combined sewer systems (CSSs) are wastewater collection systems designed to carry storm water (surface drainage from rainfall or snowmelt) and sanitary sewage consisting of domestic, commercial, and industrial wastewater in a single pipe to a treatment facility. Rain and snowmelt enter the sewers as direct runoff through catch basins and roof drains as well as by groundwater infiltration into sewer pipes. During dry weather, CSSs convey the wastewater to the Waste Water Treatment Plant (WWTP). In periods of rainfall or snowmelt, total wastewater flows can exceed the capacity of the CSS and treatment facilities. To prevent sewer backups and basement flooding, and to protect the sewage treatment plant against hydraulic overloading, CSSs incorporate overflow structures that discharge some of the combined sewage directly to surface water bodies, such as lakes, rivers, estuaries, or coastal waters (Schmidt et al. 1997). If combined pipes do not have overflows, untreated wastewater could back-up into homes and businesses, and cause flooding in the streets. When the untreated wastewater and storm water do overflow into a stream, this is called a Combined Sewer Overflow (CSO), which can be a major source of water pollution in communities served by CSSs. The point where the overflow enters a stream is called CSO outfall. Figure 1.1 schematically shows the CSS and how the CSO occurs in wet weather condition.

As CSOs contain untreated domestic, commercial, and industrial wastes, as well as surface runoff, many different types of contaminants can be present. Contaminants may include pathogens, oxygen-demanding pollutants, suspended solids, nutrients, toxics, and floatable matter. Because of these contaminants and the volume of the flows, CSOs can cause a variety of adverse impacts on the physical characteristics of surface water, impair the viability of aquatic habitats, and pose a potential threat to drinking water supplies. The resulting short term problems are poor aesthetics (floatables, turbidity, oil and grease) and beach closure due to increased harmful bacteria levels. The long term impacts include reduced dissolved oxygen in receiving waters, eutrophication and sediment contamination.

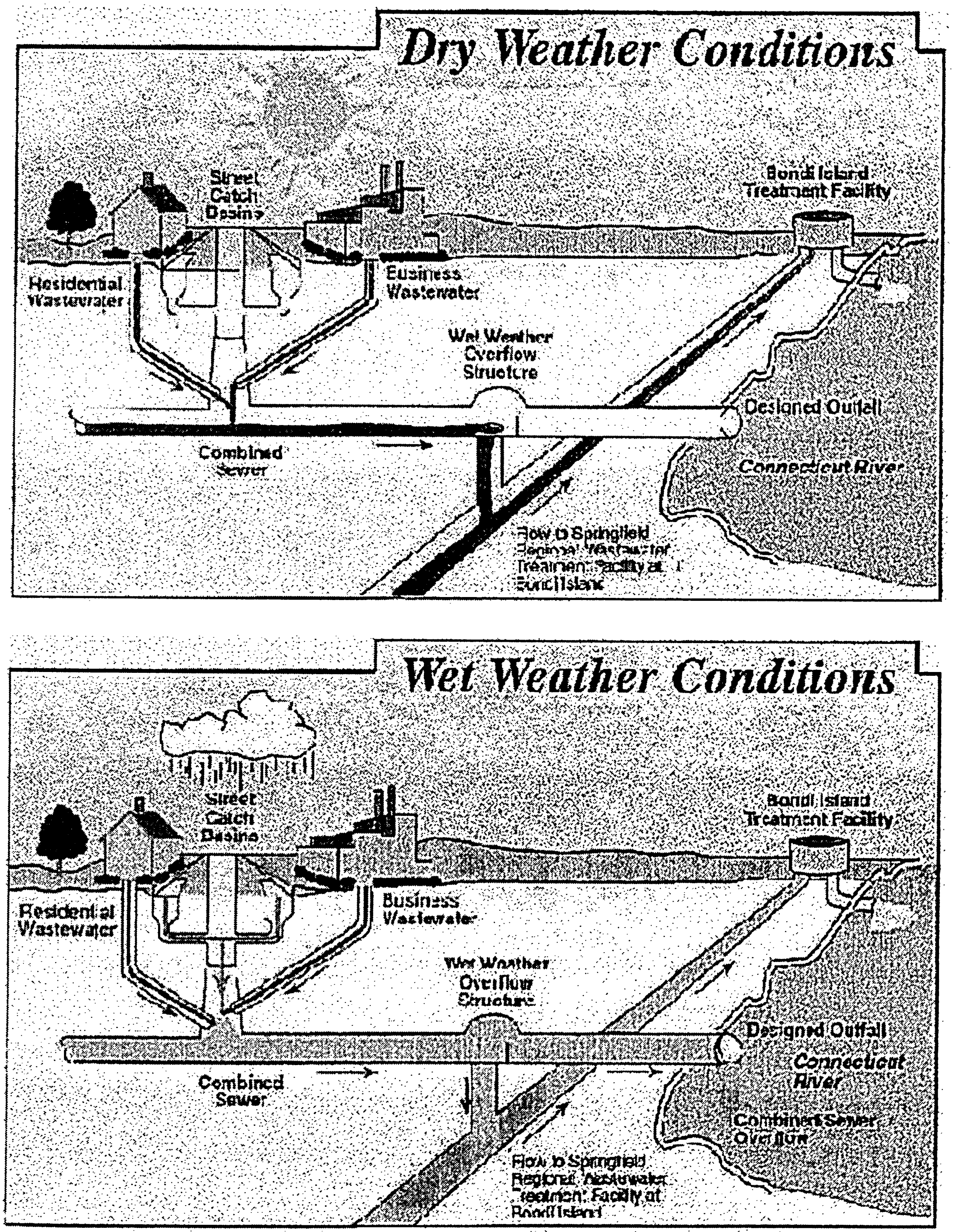



**Figure 1.1 How CSO Occur in Combined Sewer System** (Source: Springfield Water and Sewer Commission, 2005)

Since CSO is considered to be a major source of water quality impairment for the receiving waters, much attention has recently been directed to reducing the quantity and quality of CSO discharged. The frequency of CSO events and the volume of wastewater discharged may be minimized by separation of storm and sanitary sewers and by the construction of new collector sewers, in conjunction with modification or enlargement of the major sewage treatment plants to accept greater flows. Wet-weather flow may also be stored within the existing sewer system where capacity exists, or stored in new tanks or tunnels, for subsequent treatment. However, these expensive options are not always feasible and cannot cope with all storms. HRT facilities at overflow locations may be a practical, economical alternative (or addition) to the construction of new sewers and storage (MOEE, 2000). These satellite treatment systems would be designed to achieve suspended solids removal and possibly disinfection, using physical and chemical treatment operations. High-rate physical/chemical satellite treatment facilities are not expected to achieve effluent qualities equivalent to conventional (secondary) wastewater treatment processes. In general, physical/chemical treatment processes can be designed to produce a wide range of effluent qualities, using a variety of unit operations. However, the cost and complexity of the physical/chemical processes increase with increasing effluent quality. Satellite treatment plants would be expected to consist primarily of solid/liquid separation operations and have no significant removal capacity for dissolved pollutants. Furthermore, for cost-effective operation, most of the contaminants to be removed must be associated with "settleable" solids rather than the finer, colloidal particles (Schmidt et al. 1997).

Even though construction of new combined sewer systems was abandoned almost half a century ago, the older portions of many Canadian municipalities are served by the combined sewer system. With the support of the Ontario Ministry of Environment (MOE) and the Government of Canada's Great Lakes Sustainability Fund (GLSF), a number of municipalities have carried out Pollution Prevention and Control Planning (PPCP) studies to deal with water quality problems resulting from combined sewer overflows and urban stormwater runoff. Like most of the older part of many Canadian municipalities, the City of Niagara Falls (NF) faces the CSOs problem to the Niagara river, specially at Muddy Run overflow location. The Muddy Run CSO consists of the Muddy Run pumping station's overflow and overflows from the Central Pumping station, which is ultimately discharged through a drop shaft near Muddy Run pumping station to the

Niagara river. Approximately 600,000 $m^3$ of untreated CSO is discharged on an annual basis at this site over an average 34 events. As per Procedure F-5-5, the Ontario Ministry of the Environment requires the City of Niagara Falls and the Regional Municipality of Niagara to capture 90% of the wet weather flow and remove carbonaceous biochemical oxygen demand (CBOD) and suspended solids in the overflow by 30% and 50% respectively. To meet these criteria, the City of Niagara Falls has commissioned a series of studies to investigate cost-effective technologies for controlling combined sewer overflows to the Niagara River. One of the preferred options is the application of vortex technologies. Vortex separators offer substantial costs savings compared to more conventional treatment alternatives such as storage, retention-treatment basins, high rate sedimentation, and high rate screening/filtration. The City of Niagara Falls (NF), Ontario Great Lakes Renewal Foundation (GLRF), Government of Canada's Great Lakes Sustainability Fund (GLSF), Ontario Ministry of Environment (MOE), National Water Research Institute (NWRI), Ryerson University (RU), and the Regional Municipality of Niagara (RMN) are in the process of implementing a HRT pilot study to evaluate the performance of two commonly available vortex separator treatment technologies named Storm King™ Vortex Separator and Continuous Deflective Separation (CDS™) technologies.

In this report, the above project is taken as a case study for performance evaluation of vortex separator technology in CSO pollution control point of view. The average annual overflow volume and frequency, and pollution mass load are determined by the Analytical Probabilistic Model (APM). As the pollutant removal efficiency of Storm King™ and CDS™ unit at the study site are not completed yet, so in this report, the predicted overall pollutant concentration (for TSS ) after provide treatment is calculated individually both for the two devices based on their TSS removal efficiency claimed in their corresponding manuals.

## 1.2 Structure of the Report

The outline of this report includes the objectives and scope of the study, concluding Chapter one. Chapter two includes a literature review on the issues with currently used CSO control strategies. This chapter also includes a review on vortex separator technologies with emphasized on Storm King™ Vortex Separator and Continuous Deflective Separation (CDS™) technology. The characteristics of settling solids are determined by different settling test procedures, which are discussed in chapter three. In Chapter four, the concept and the derivation of Analytical Probabilistic Model are briefly discussed. A brief description of the City of Niagara Falls HRT pilot project site which is taken as a case study in this report are presented in chapter five. In chapter six, methodology undertaken for the development and implementation of the monitoring program are discussed. The impacts of HRT facilities on water pollution concentration are analyzed in chapter seven. Finally, Chapter eight concludes the project report with recommendations.

### 1.3 Study Objectives and Scope

Combined sewer overflows (CSOs) are recognized as significant sources of water quality problems. Even though construction of new CSSs was abandoned about half a century ago, the environmental problems associated with CSOs from existing CSS persist to this day. The control of CSOs has proven to be extremely complex. This complexity stems partly from the difficulty in quantifying CSO impacts on receiving water quality and the site-specific variability in the volume, frequency, and characteristics of CSOs. The objectives of this study are to understand the causes and associated problem of CSOs, and to review currently practice various short-term and long-term CSO control technologies- emphasized on vortex separator technologies. The City of Niagara Falls pilot project is taken as a case study for this report, where the performance of two HRT devices- Storm King and CDS vortex separator technologies are evaluated. The analytical probabilistic model is used to determine the average annual pollution mass discharged to the receiving water with and without the HRT technologies applied at strategic locations of a CSS.

# CHAPTER TWO

## LITERATURE REVIEW

This chapter provides an overview of CSO control strategy with existing short- term and long-term CSO control technologies. A comprehensive literature pertaining to vortex separation technologies for CSOs control are discussed in this chapter with special emphasized on Storm King® Overflow with swirl cleanse Cleanse™ Screen produced by Hydro International Plc. and Continuous Deflective Separation (CDS™) produced by CDS Technologies Inc.

### 2.1 CSO Control Strategy

Historically, the control of CSOs has proven to be extremely complex. This complexity stems partly from the difficulty in quantifying CSO impacts on receiving water quality and the site-specific variability in the volume, frequency, and characteristics of CSOs. In addition, the financial considerations for communities with CSOs can be significant (USEPA, 1995). A CSO control program has the overall objective of preventing/controlling/reducing the water quality impacts produced by combined sewer overflow. A control program is different than a CSO facilities plan. A control program considers a wider range of measures than facilities plan including operational controls such as maximizing combined sewer flows captured and taken to treatment within existing facilities. Pollution prevention measures, such as public education programs and industrial pre-treatment programs, both aimed at reducing or eliminating hazardous contaminants in CSO, are also important parts of an abatement program. The capital facilities planning component is of course central to the overall abatement program development. It is the capital facilities aspect of the program that specifies the type, size and location of structural control measures. Typically, this is the most costly part of the program to implement.

CSO abatement program development should be a systematic process. The first step should consider the goals and objectives of the program. The process then builds by developing an understanding of the behavior of the subject collection/treatment system and its interaction with the natural environment. The next step is then to evaluate and select a preferred set of operational and structural control or abatement measures designed to meet the program objectives. The final element is the preparation of an implementation plan presenting the sequence, timing, and cash flow requirements, for specific facilities development and other non-structural program elements. The implementation plan must also deal with facilities permitting, regulatory approvals and post-construction monitoring. Figure 2.1 presents an overview of the phasing structure of a CSO abatement program.

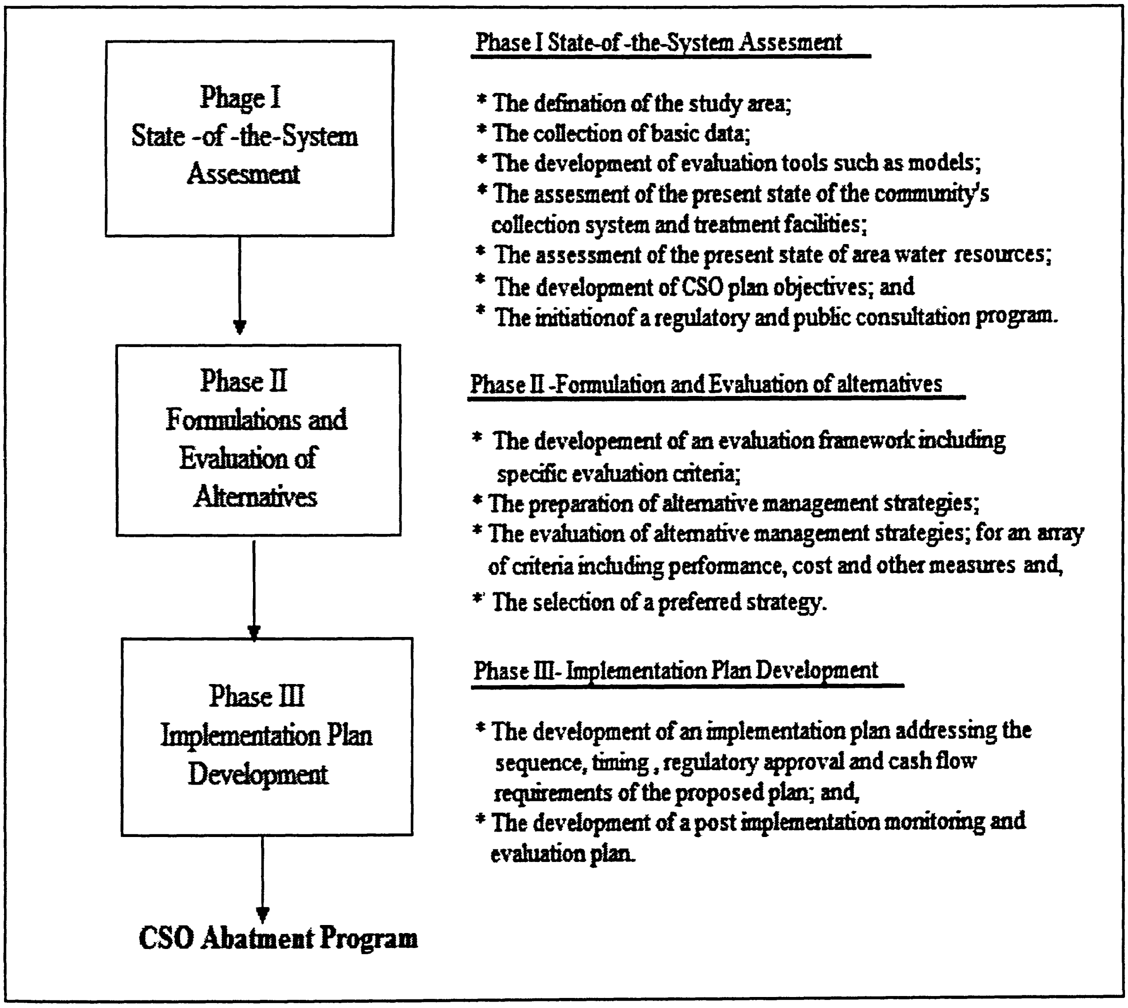



**Figure 2.1 CSO Abatement Program** (Source: CSO manual)

The identification and formulation of alternatives is perhaps the key step in producing a cost-effective and comprehensive CSO abatement program. The process needs to be systematic and well documented so that upon reviews, the basis for alternative selection is clear and traceable.

In some jurisdictions, the regulatory authorities specify both the abatement program objectives as well as the control technologies. For example, the long term goal of the CSO abatement program in British Columbia is complete elimination of all CSO through sewer separation. Some regulatory authorities (e.g. Ontario province and US) require a minimum program of nonstructural or operational control measures. In the US, these measures are termed the Nine Minimum Controls as discussed in the following section. A community embarking upon the preparation of a CSO abatement program should, therefore, clearly identify these regulatory expectations since they will obviously influence the alternatives considered and the preferred strategy selected.

An important consideration in the overall development of a CSO abatement program and in the selection of a CSO treatment strategy is an understanding of the underlying regulatory framework. In Canada, the CSO control approach varies from province to province. The Ontario ministry of Environment "Guidelines for the Design of Sanitary Sewerage Systems, July 1985" states "All new sewer construction within the Province of Ontario should be of the 'separate' type, with all forms of storm and groundwater flow being excluded to the greatest possible event. New 'combined' sewer systems will not be allowed " However, existing combined sewers may undergo rehabilitation or be replaced by new combined sewers provided the municipality or operating authority has met the Ministry requirements as stated in Procedure F-5-5, "Determination of Treatment Requirements for Municipal and Provincial Combined and Partially Separated Sewer Systems." The Procedure F-5-5 is a supporting document for Guideline F-5, "Levels of Treatment for Municipal and Private Sewage Treatment Works Discharging to Surface Waters". Specifically, Procedure F-5-5 is a policy document for controlling CSOs that requires municipalities to meet the minimum CSO controls as presented below (MOE, 1997)

- Eliminate CSOs during dry weather periods except under emergency conditions
- Establish and implement pollution prevention programs that focus on pollutant reduction activities at source.

- Establish and implement proper operation and regular inspection and maintenance programs for combined sewer system in order to ensure continued proper system operation
- Establish and implement a floatables control program to control coarse solids and floatable materials
- Maximize the use of the collection system for the storage of wet weather flows which are conveyed to the sewerage treatment plant for treatment when capacity is available
- Maximize the flow to the sewerage treatment plant for the treatment of wet weather flows
- During a seven-month period commencing within 15 days of April 1, 90% of wet-weather flow is to be treated to primary treatment equivalency, which is defined as a seasonal average of at least 50% removal of total suspended solids (TSS) and 30% removal of 5- day carbonaceous biochemical oxygen demand ($BOD_5$). Furthermore, for satellite treatment facilities the effluent TSS concentration should not exceed 90 mg/l for more than 50% of the time (Li, 2004).
- The interim effluent bacteriological quality criterion for treated combined sewage is a monthly geometric mean not exceeding 1000 E. coli per 100 ml.
- Controlling to achieve not more than two overflow events per season (June 1 to September 30).

## 2.2 Nine Minimum Controls (NMC)

To facilitate implementation of the CSO Control Policy, United States Environmental Protection Agency (EPA) has prepared a guidance documents "EPA-Combined Sewer Overflows-Guidance for Nine Minimum Controls" that can be used in planning and implementing CSO controls that will ultimately comply with the requirements of the Clean Water Act. The NMC are controls that can reduce CSOs and their effects on receiving water quality, do not require significant engineering studies or major construction, and can be implemented in a relatively short period (e.g., less than approximately two years) prior to the implementation of long-term control measures. The NMC are as follows (EPA, 1995):

- Proper operation and regular maintenance programs for the sewer system and CSO outfalls
- Maximum use of the collection system for storage
- Review and modification of pretreatment requirements to ensure that CSO impacts are minimized
- Maximization of flow to the POTW for treatment
- Elimination of CSOs during dry weather
- Control of solid and floatable materials in CSOs
- Pollution prevention programs to reduce containments in CSOs
- Public notification to ensure that the public receives adequate notification of CSO occurrences and CSO impacts
- Monitoring to effectively characterize CSO impacts and the efficacy of CSO controls.

## 2.3 Commonly Used CSO Control Technologies

A number of approaches can be considered to deal with the CSO problem. The frequency of CSO events and the volume of wastewater discharged may be minimized by separation of storm and sanitary sewers and by the construction of new collector sewers, in conjunction with modification or enlargement of the major sewage treatment plants to accept greater flows. Wet-weather flow may also be stored within the existing sewer system where capacity exists, or stored in new tanks or tunnels, for subsequent treatment. However, these expensive options are not always feasible and cannot cope with all storms. HRT facilities at overflow locations may be a practical, economical alternative (or addition) to the construction of new sewers and storage facilities (Schmidt et al. 1997).

There are six major CSO control technologies based on the fact that they are currently in wide use and have been demonstrated to be effective in reducing CSO flows and / or pollutant loads. A brief discussion of each is provided below:

## In-System Controls/ In-Line Storage

One of the more readily implementable and cost effective approaches to achieving immediate reductions in CSO volumes is to utilize the available storage and conveyance capacity of existing collection systems and the available treatment capacity at the Publicly Owned Treatment Works (POTW). This control approach is only feasible if sufficient capacity is available in the collection system and at the treatment plant. A number of "in-system" technologies or strategies can contribute to maximizing in-line storage, maximizing flows to the POTW, and reducing overflow volumes, including (EPA, 1993):

- Collection system inspection and maintenance.
- Tide gate maintenance and repair.
- Reduction of surface inflow.
- Adjustment of regulator settings.
- Enlargement of undersized pipes to eliminate flow restrictions.
- Removal of obstructions to flow, such as sediments.
- Polymer injection to reduce pipe friction.
- In-system flow diversions through existing system interconnections.
- Adjustment and/or upgrade of pumping station operations.
- Partial separation of storm drains connections from combined sewers.
- Infiltration removal.

## Off-Line Near-Surface Storage/Sedimentation

Off-line, near-surface storage/sedimentation facilities consist of tanks that store and/ or treat combined sewer flows diverted from combined trunk sewers and interceptors. These facilities provide storage up to the volume of the tanks, as well as sedimentation treatment for flows that pass through the facilities in excess of the tank volume. Coarse screening, floatables control, and disinfections are commonly provided. The phrase "near-surface" means that these facilities are constructed at depths that allow the use of traditional open-cut excavation techniques, as opposed to the deep tunnel facilities.

### Deep Tunnel Storage

Deep tunnel storage often is considered as an alternative to near-surface storage/ treatment facilities where space constraints, potential construction impacts, and other issues challenge the feasibility of near-surface facilities. This technology provides storage and conveyance of storm flows in large tunnels constructed well below the surface with little disturbance to existing surface features, which can be very beneficial in congested urban areas. A typical deep tunnel system includes the following features (EPA, 1993):

- Regulators, to divert and control storm flows to the tunnel system
- Consolidation conduits, to convey flows from regulators to the tunnel system
- Coarse screening, to remove large debries and protect downstream pumps
- Vertical drop shafts, to deliver flow to the tunnel and dissipate energy
- Air separation chambers, to allow release of air entrained in the drop shafts
- Tunnel, sized to store and convey flows from a given design condition.
- Access shafts, for maintenance personel and equipment
- Vent shafts, for balancing air pressure
- Dewatering system, to pump volume stored in the tunnel to the POTW once conveyance system and treatment capacity is restored
- Odor control systems at certain venting locations

### Coarse Screening

This technology provides coarse solids removal, as well as a degree of floatables removal. Coarse screening typically is provided upstream of other control technologies, such as storage facilities or vortex units that are applied as off-line treatment units. Coarse screening equipment, consisting of vertical or inclined steel bars spaced evenly across a channel, with or without mechanical raking apparatus, is installed at CSO control facilities, both for the protection of downstream equipment and to provide floatables removal.

**Disinfection**

This process inactivates or destroys microorganisms in CSO, most commonly through contact with chlorine, although a variety of disinfection technologies are available without chlorine. Some of the more common technologies include gaseous chlorine, liquid sodium hypochlorite, chlorine dioxide, ultraviolet radiation, and ozone. For disinfection of CSOs liquid sodium hypochlorite is the most common technology.

**Swirl/Vortex Technologies**

These devices provide flow regulation and solids separation by including a swirling motion within a vessel. Solids are concentrated and removed through an underdrain, while clarified effluent passes over a weir at the top of the vessel. This technology originally was applied in England in the 1960's, and since has evolved into a number of configurations, which will be discussed elaborately in Section 2.4 of this report.

**Real-Time Control and In-System Storage**

In Real-Time Control (RTC) system the combined sewer process data such as water level, flow, pollutant concentration, etc. are continuously monitored in the system and, based on these measurements, regulators are operated during the actual flow and/or treatment process. RTC is a custom-designed management program for a specific urban sewerage system. During a storm event, RTC performs three main functions: it routes flows, maximizes the use of existing storage within the sewerage system, and eliminates/reduces untreated overflows. The system continually adjusts the level of CSO gates and storage facilities according to changing rainfall and flow conditions. This ultimately maximizes the use of storage within the CSS and minimizes combined sewer overflows. A series of rain gauges and flow sensors provide constant measurements for real-time rainfall and flow forecasting. The data is recorded and transmitted to a central computer every 5 to 10 minutes. The information is then fed into a computer which simulates the existing CSO system and predicts flow rates and depths at important points within the system. The computer helps to minimize CSOs to local receiving waters by determining which gates should be opened, for how long, and when additional adjustments may be necessary. This allows the maximum amount of wastewater to be treated by the WWTP. Once the computer has determined the optimal positions for gates and storage tanks, the signal is transmitted, and the devices are adjusted accordingly (Stirrup, 1996).

RTC systems are generally comprised of the following elements (CSO mannual):

- Sensors that measure the state of the system with respect to flows, levels and the status of various controllable elements such as gates;
- Rainfall measurement through ground based rain gauges and/or through radar measurements;
- A communications network to tie together the sensors, rainfall measurements and controllable elements with one or a number of operator interface stations;
- Computing elements, these may be centralized or located locally;
- System management and supervisory software (SCADA) that is designed to facilitate data collection, data storage, operator interface, data analysis and display, setting of set points, alarms and control of controllable elements such as pumps and gates;
- System computational software, which may support a wide range of functions from hydraulic analysis to real time forecast of rainfall, system flows and control action trajectories; and,
- Human operators who supervise the system and may modify the control actions proposed by the system management software.

RTC can be implemented in range of complexity from locally based control to a fully global system with forecast capability. RTC is not, however, a stand alone alternative. There needs to be sufficient capacity within the storage, transport and treatment facilities so that RTC application can bring about improvements in capacity utilization. If a system is already overloaded, RTC application may have very little practical benefit.

Besides of those technologies, a variety of practices and control techniques can be utilized to supplement the application of a control technology at a CSO discharge location. Example of such control practices are dissolved air floatation, fine screens and microstrainers, high rate filtration, and biological treatment.

## 2.4 Vortex Separator Technology

Vortex solids separators are compact devices that provide flow regulation and a rough level of suspended solids (SS) and floatables removal. The technology was first applied in England and has been in use for over 30 years. In the early 1960s, Bernard Smisson first incorporated a cylindrical vortex-type CSO regulator and settleable- solids concentrator into the Bristol, England sewerage system (Andoh et al. 2002). Hydro Research and Development Ltd. of Great Britain continued research into vortex separators during the 1970s and produced the Storm King™ dynamic separator or the Hydro Dynamic Separator. This unit reportedly improved head loss and solids transport characteristics. In the early 1970s, the U.S. EPA completed a series of projects to develop and demonstrate swirl settleable solids removal technology for North American application. The projects resulted in the EPA swirl CSO regulator and settleable solids concentrator (hereafter referred to as swirl) and other concentrator devices, including the EPA swirl degritter, a variation that effect settleable -solids separation but is not used to regulate flow. Research conducted by Dr. Hans Brombach in Germany in the mid 1980s resulted in the development of a vortex separator, which is marked as the Fluidsep™. The objective was to develop a vortex vessel that could operate at high hydraulic loads and provide substantial removal of settleable and floatable solids. In addition, the German researchers wanted to maximize the detention storage of the vessel for small storms and allow the vessel to act as a solids separator for larger storms (Field et al. 1997).

The vortex separator devices provide three functions: flow regulation, settleable solids concentration and floatables capture. Although each types of vortex separators is configured differently, the operation of each unit and the mechanism for solids separation are similar. Settleable solids concentration is achieved by a combination of gravity settling and inertial separation due to the circular flow pattern. The concentrated stream is discharged in the underflow. Floatable materials are influenced by the same forces and are generally trapped at the surface of the devices. Flow regulation is a prime function of the swirl. Flow regulation is achieved by installation of the device in-line in the sewer system. Dry-weather flows (DWF) pass through the unit. The volume of the device, if large enough, provides storage of wet weather flow (WWF) to attenuate flow to a downstream treatment plant. Excess WWF is discharged in the overflow. Flow entering the unit is directed around the perimeter of a cylindrical shell, creating a swirling, vortex flow pattern. The swirling action throttles the influent flow, and causes solids to be concentrated at the bottom of the unit. The throttled under flow containing the concentrated solids passes out through a foul sewer outlet in the bottom of the unit, while the clarified supernatant passes out through the top of the unit. The under flow is typically discharged to the downstream interceptor for treatment at the POTW. Various baffle arrangements capture floatables in the supernatant. The floatables are carried out in the under flow when the unit drains, once storm flows subside. The mechanism for solids separation is created by the flow patterns within the unit. Flow initially follows a path around the perimeter of the unit after one revolution; the flow is deflected into an inner swirl pattern, which has a lower velocity then the outer swirl. Gravity separation occurs as particles follow a "long path" through the outer and inner swirl. The quiescent inner swirl, as well as tangential breakaway of particles from the cyclonic flow field and drag forces along the walls, bottom, and in the shear zone between the inner and outer swirl, all contribute to solids separation. Secondary currents direct particles across the floor of the unit towards the foul sewer outlet (Moffa, 1997).

Despite design and application differences, the main intent of these technologies is the same: to separate settleable solids from the storm flow by a vortex or swirling flow field. To effectively apply swirl and vortex capabilities in a combined sewerage or storm water drainage system, the control functions, applicability, and idiosyncrasies of their individual designs must be clearly understood. Factors which are essential to the successful application of swirl/vortex devices are:

- Consistent and appropriate flow measurement, wastewater sampling and characterization protocols.
- Appropriate data management techniques, particularly the calculation of efficiency
- An understanding of swirl/vortex mechanisms, with realistic performance expectations
- Appropriate application or placement in the sewerage system.

### 2.4.1 Advantages of Swirl/Vortex Technologies

The advantages of swirl/ vortex technologies include:

- The lack of moving parts and the ability to operate at high hydraulic loads, resulting in a small footprint and low cost.
- No power required.
- The relatively small volume of these devices makes them much less expensive than sedimentation tank.
- The swirl/vortex units are designed for continuous discharge of the underflow, thereby requiring no sludge handling facilities.
- The underflow (foul- orifice) diameter is usually large enough to avoid blockage such that pretreatment is not required.

### 2.4.2 Disadvantages of Swirl/Vortex Technologies

The disadvantages of swirl/ vortex technologies include:

- The potential need for post-storm cleaning due to shoaling of solids.
- Underflow pumping, which increases operation and maintenance costs, may be required depending upon the local gradient and the depth of the unit.
- Compared to more conventional treatment operation e.g., sedimentation, swirl/vortex units produce a relatively dilute underflow rather than concentrated grit/sludge residuals.
- If the underflow is not returned to the sewerage system, the relatively large volumes of underflow require storage and possibly further treatment.

### 2.5 Brief Description of Various Swirl/Vortex Separators

The swirl/vortex devices are similar in general operating principle although there are design and application differences. Currently, there are four types of vortex separators available for CSO applications:

- The US EPA Swirl flow regulator/concentrator (swirl) a public domain device;
- Fluidsep® vortex chamber, marketed by John Meunier;
- Storm King®, marketed by Hydro International Limited; and,
- CDS®, marketed by CDS Technologies Inc.

The US EPA Swirl concentrator and Fluidsep vortex separator are briefly discussed in the following two sections. Storm King and CDS vortex separators are discussed elaborately in Section 2.6 and 2.7 respectively.

#### 2.5.1 EPA Swirl Concentrator

The U.S. EPA Municipal Environmental Research Laboratory in Cincinnati, Ohio, in conjunction with the LaSalle Hydraulics Laboratory in Montreal, Quebec, and the General Electric Company in Philadelphia, Pennsylvania, conducted a series of hydraulic and mathematical modeling studies in the 1970s to develop and demonstrate swirl concentration technology. This work resulted in the design of the US EPA Swirl CSO flow regulator/concentrator, shown schematically in Figure 2.2.

The main function of the inlet ramp is to introduce the flows to the unit tangentially at the chamber floor, so that the "long path" maximizing solids separation may be developed. The deflector wall, or the extension of the interior wall of the inlet ramp, deflects flow, inwards. A scum ring is provided for the removal of floating solids from the overflow. An overflow weir and weir plate provide a connection to a central down shaft that carries the overfloyw. The underside of the weir traps floatables. The main function of spoilers is to reduce the rotational energy of the liquid above the weir plate, and between the scum ring and weir, for the purpose of increasing the overflow capacity and improving the separation efficiency. The floatables are directed into a channel crossing the weir plate to a vertical vortex cylinder located at the wall of the overflow down shaft. A foul sewer outlet provides an exit orifice to direct the peak dry-weather flow and separated combined sewage to the interceptor, while the primary floor re-directs dry-weather flows to the sewer outlet to avoid dry-weather solids deposition. The supernatant is allowed to overflow the central circular weir into the down shaft for storage, further treatment, or discharge to the receiving water. A secondary overflow weir is also provided for extremely high flows to improve the unit's performance during this condition.

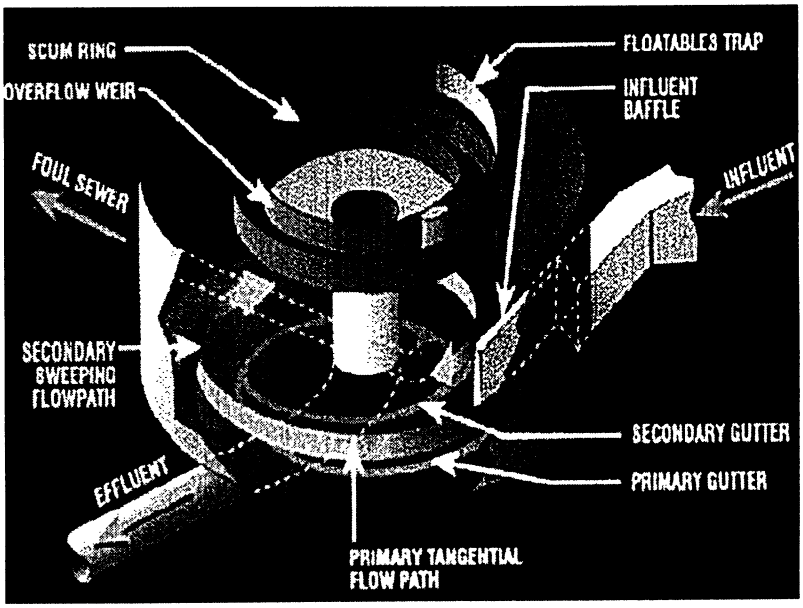



**Figure 2.2 EPA Swirl Concentrator** (Source: Environmental Engineering Associates, 2002)

### 2.5.2 Fluidsep™

The Fluidsep™ vortex separator was designed by a German firm Umwelt and Fluid- Technik (UFT). The Fluidsep™ vessel encourages free vortex flow since it has no centrally located baffles or influent deflector plate. Free vortex creates less turbulence and has a less disrupted flow pattern than the swirl concentrator. The Fluidsep™, shown schematically in Figure 2.3, is a rotationally symmetric vortex chamber with conical bottom sloping towards the centre of the chamber. It does not have gutters on floors, unlike the swirl and Storm King® units, and it incorporates a unique conical-shaped guiding baffle to minimize the overflow of particles carried upwards by secondary flow currents.

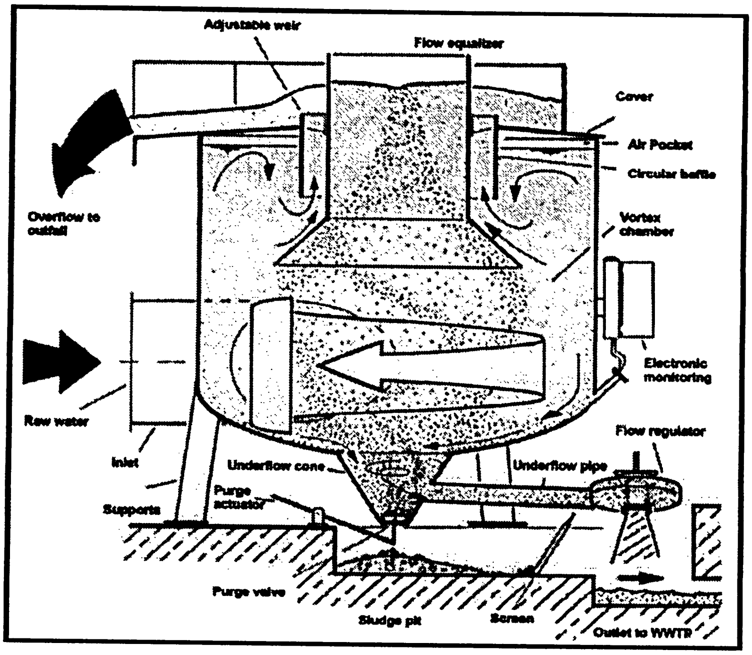



**Figure 2.3 Cross Section of a Fluidsep™ Vortex Separator** (Source: John Meunier Inc. 2002)

In an in-line installation, the dry weather flow passes freely by on the sloped bottom towards the central evacuation cone and then through a flow regulator. During a small storm event, the unit acts as a storage element. For more intense or more durable storms, the unit overflows through its central overflow weir, which is made of two plunging cylindrical treatment baffles providing a double crown arrangement. The overflow water is evacuated through the ring-shaped opening formed by these two treatment baffles. When the unit is filled, a pocket of air is formed under the unit's cover that catches the floatables, where they are retained until the unit gets back to dry-weather conditions. The rotational movement induces the creation of a vortex separation in the tank. The resulting flow pattern is non-turbulent and favourable to the separation of suspended solids, which settle and are pulled by the centrifugal currents towards the wall of the separator. Once the particles are caught along the walls, they fall to the structure bottom and into the evacuation cone. From there, they are carried out with the underflow through the regulator.

Under certain flow conditions the lower cone of the Fluidsep™ vortex separator can be fitted with an underflow drain, to help separate the coarse elements from the separator or flush out entrapped material.

## 2.6 Storm King™

The Storm King™ was designed by the British firm Hydro Research and Development. The Storm King™ Dynamic Separator is a cylindrical vessel with a sloping bottom. It contains a number of internal baffles and plates designed to control the flow patterns and to promote the

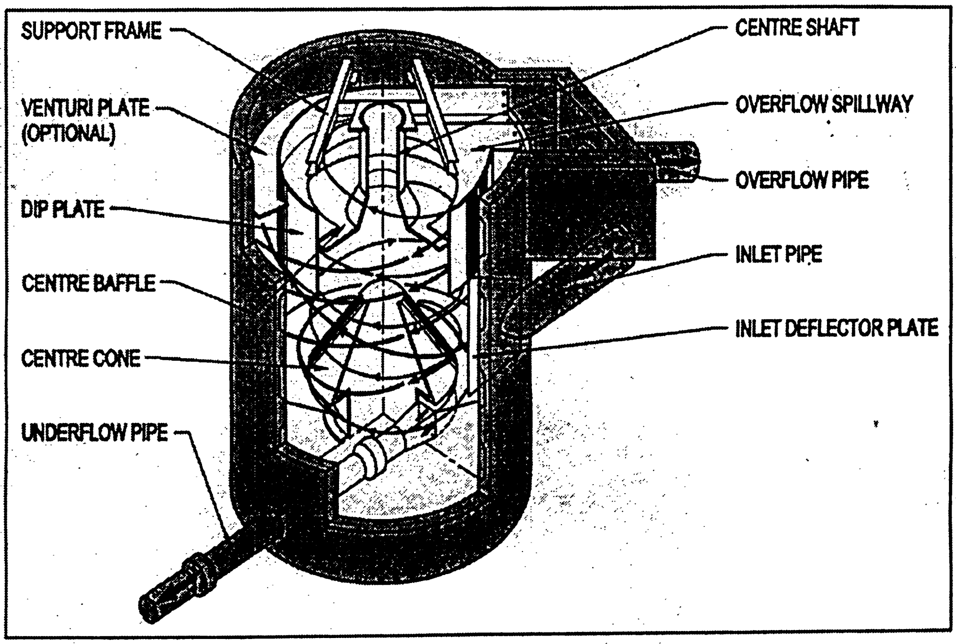



**Figure 2.4 A Typical Storm King Dynamic Separator** (Source: Hydro International Plc. 2004)

settling of solids and capture of floatables, as shown in Figure 2.4. The raw liquid is feed tangentially into the side of the vessel at about mid-height, creating a flow pattern, which rotates about the vertical axis of the unit. Heavier particles settle out by gravity enhanced by inertial forces set up by the complex flow regime within the chamber. As the flow rotates about the vertical axis, settleable solids are directed towards the base of the chamber where they are collected and removed by either gravity or pumping. This foul is then passed forward to treatment. Floatable material is captured in the outer section of the chamber. The main flow is directed away from the perimeter and back up the middle of the chamber as a narrower spiralling column that rotates at a slower velocity than the outer downward flow. Figure 2.5 shows the simplified view of flow patterns in a Storm King Overflow device. The interface between the outer downward circulation and the inner upward circulation is the shear zone, where a difference in velocity encourages further solids separation. Floatables are captured between the shear zone and the chamber wall. By the time the flow reaches the top of the chamber, it is virtually free of solids and is discharged through a spillway to the receiving waters.

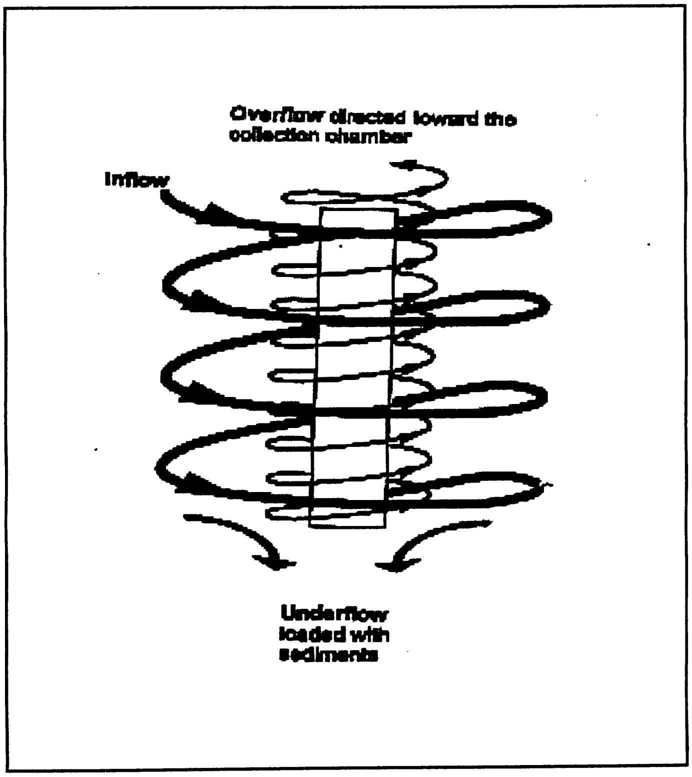



**Figure 2.5 Simplified Views of Flow Patterns in a Storm King Overflow Devices**
(Source: Hydro International Plc. 2004)

More recently, an additional feature has been added (Swirl-Cleanse™ screen) to the Storm King® unit to improve the capture of solids. In the next section the mechanism of screening system, collected from the Hydro International Plc's CSO product manual, are discussed.

### 2.6.1 Storm King® with Swirl-Cleanse™ Screen

The Storm King ® Overflow with Swirl-Cleanse™ Screens is essentially a Storm King ® Overflow incorporating Hydro International plc's proprietary self cleansing, self-activating, non-powered screening system that provides one of the best possible combinations of non-powered self cleansing devices for the treatment of CSOs and other intermittent discharges. Designed as the ultimate CSO treatment device, the Storm King® with Swirl-Cleanse™ does it all, from the screening and removal of gross solids and floatables greater than 4 mm in two directions to the removal of sediments, settleable solids and associated pollutants e.g. TSS, BOD, COD. The self-cleansing mechanism operates by holding back screened water in the outlet channel.

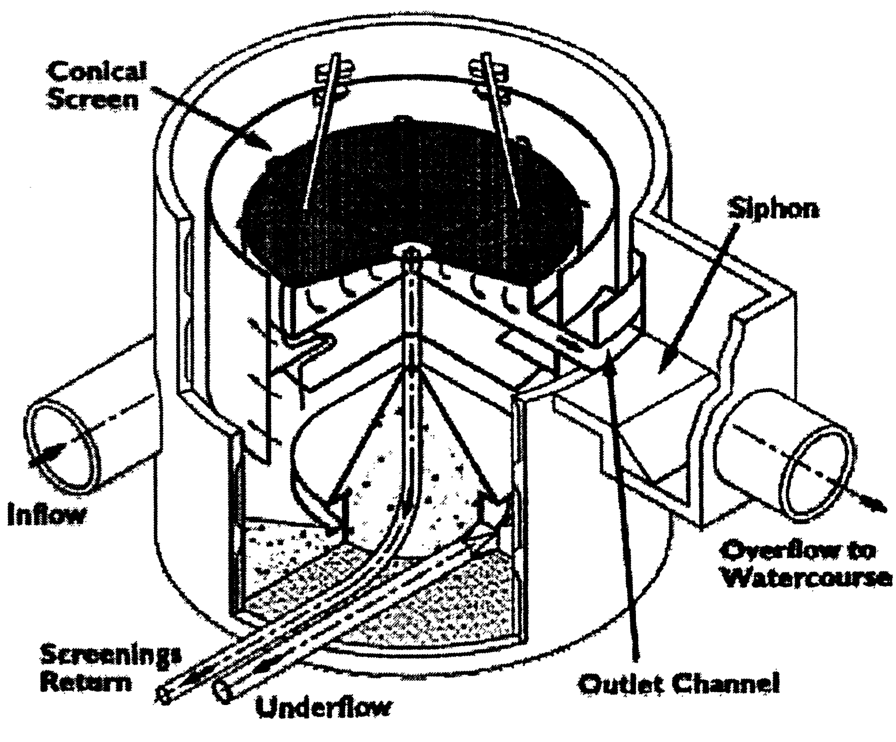



**Figure 2.6 A Typical Storm King with Swirl Cleanse Screen** (Source: Hydro International Plc. 2004)

A swirling motion is created which backwashes the screen and conveys the captured debris towards the central screening return pipe that conveys it to the foul sewer leading to the treatment plant. The efficiency of the self-cleansing action, and the self-activating function, are generated by a unique siphon mechanism built into the Overflow. As shown in Figure 2.6, the main unit is a standard Storm King® Overflow onto which a conical run-down screen, a central vertical screenings return pipe, and a backwash siphon are added. The backwash siphon is located in a chamber immediately downstream from the overflow channel. It is also required an additional Reg-U-Flo® Vortex valve to control the screenings return flow.

### 2.6.2 The Self Cleansing Screening Process

The swirl- Cleanse™ Screen is a self- cleansing physical barrier of relevant aperture size usually 4mm to 6mm, employing a hydraulically operated periodic backwash process to keep the screen clean. Figure 2.7 and Figure 2.8 show the cutaway view and top view of the self cleaning screen system respectively. The swirl- Cleanse™ Screen works by holding back screened water in the outlet channel. During operation, the water passes over and through the screen, trapping solids greater than the mesh size on its surface. Initially, the screened water is prevented from being discharged by a level control device on the overflow that acts as an open or closed valve. This is usually a siphon.

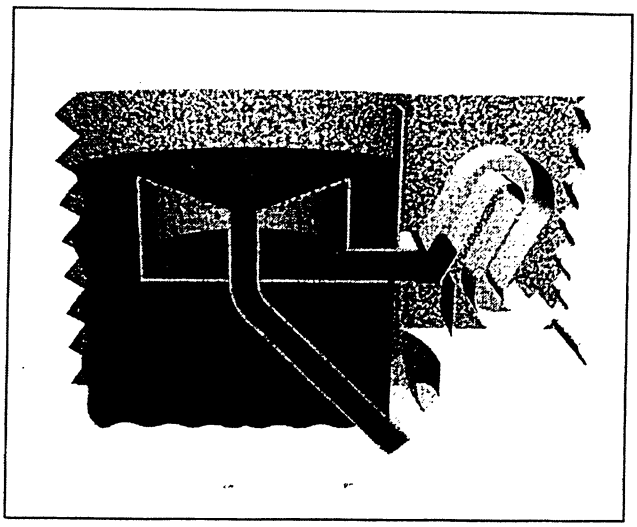


**Figure 2.7 Cutaway View of the Self-Cleansing Screening System** (Source: Hydro International Plc. 2004)

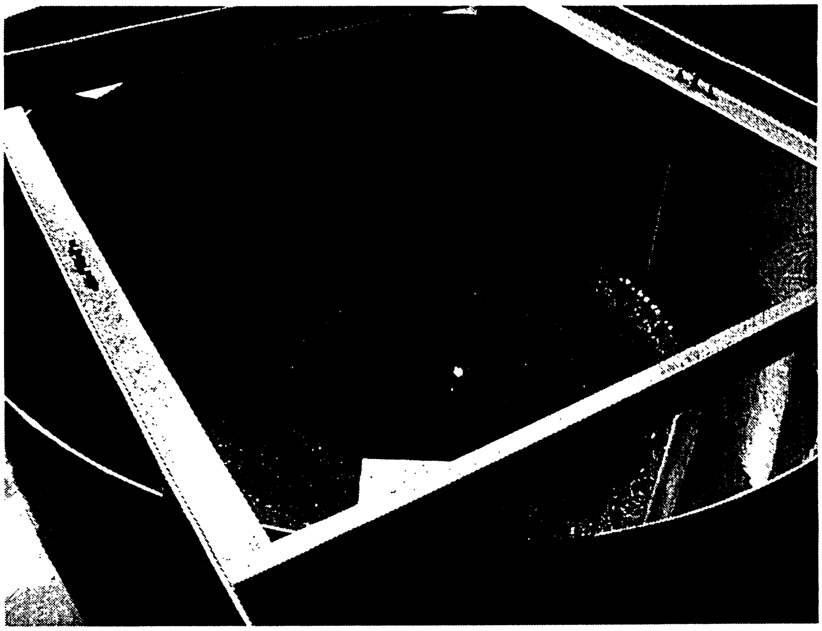


**Figure 2.8 Top View of the Self Cleansing Screening System** (Source: Photograph taken from Niagara Falls HRT Pilot Study site)

The water level beneath the conical screen rises with time, eventually flooding through the mesh and over the screen's surface to enhance the backwashing action. When the water level reaches the top of the screen, the backwash is complete. The siphon then primes, discharging the screened effluent to the receiving watercourse. The water level in the chamber then drops below the apex of the screen, the siphon is interrupted and the cycle repeats. Figure 2.9 demonstrates the backwashing cycle.

During the backwashing cycle, the rising water level removes the deposited materials in three ways: Firstly, the rise in water level increases the air pressure under the screen, effectively 'blowing' air through the screen mesh. This effect becomes more pronounced as the water level rises and air is forced out from beneath a curtain of water. This mechanism is useful for cleaning the top perimeter of the screen. Secondly, the water rising back up through the screen lifts materials off the screen. The relatively low open area of the screen holes increases the velocity at which the water passes through the screen. Thirdly, the water discharges to the central waste pipe in a form of a vortex that spreads around the surface of the screen. This vortex has a scouring effect on the debris, helping to carry it towards the central waste pipe.

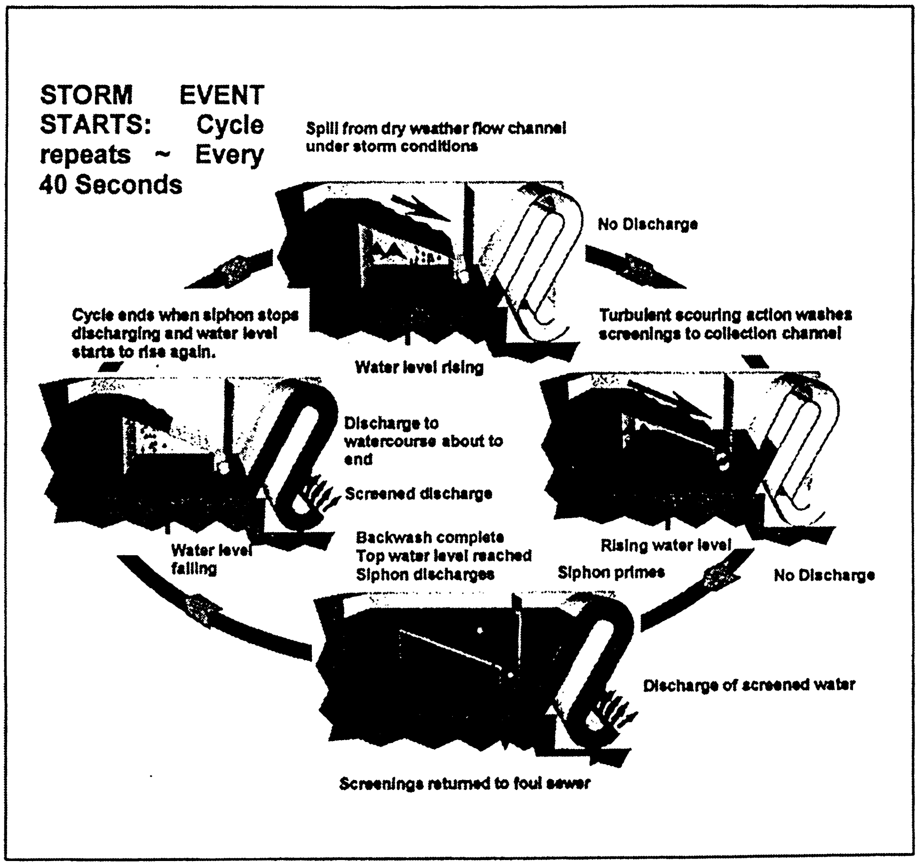



**Figure 2.9 Screen Backwashing Cycle** (Source: Hydro International Plc. 2004)

### 2.6.3 Design Variables of Storm King Overflow Unit

The principal parameter for Storm King$^{TM}$ vortex separator unit design is the applied hydraulic surface loading rate, settleability of influent solids, and the effective residence time within the unit. The surface loading rate relates to the overflow rate or velocity in conventional sedimentation theory. It is generally followed that the higher surface loading rates (short residence times) can be applied when the solids in the influent stream are readily settleable. For influent with poor settling characteristics, lower loading rates and longer residence times may be necessary to achieve the desired levels of solids removal. For a typical for Storm King™ unit for gross solids removal in a CSO environment, the maximum recommended surface loading rate ranges from 19 L/sec/m$^2$ to 30 L/sec/m$^2$). The Storm King™ unit is designed on the basis of peak flow. Therefore, for a given flow rate and hydraulic loading rate relating to desired performance levels, the diameter of the unit can be determined from (Hydro International plc. 2004):

$$\text{Plan area of unit} = \text{Flow rate/ Surface loading rate}$$

The aspect ratio of the Storm King™ unit determines its overall height for a given diameter unit. The aspect ratio is defined as the ratio of the barrel depth to the diameter of the unit. The typical aspect ratio for Storm King™ is 0.5. The general observation being that for a given diameter, the bigger aspect ratio is more efficient. Furthermore greater aspect ratios result in larger volume unit with longer retention time or lower volumetric loading rate. A disadvantage is that a bigger aspect ratio results in a deeper unit and higher construction costs. Lower aspect ratio can be accommodated where the Storm King™ unit is used with screening system.

### 2.6.4 Field Monitoring Studies of Storm King™ Units

**Scarborough, Ontario, Canada**

In 1994, a pilot plant was constructed in the City of Scarborough, at an overflow to Massey Creek, a tributary of the Don River. The plant was configured to operate with wet weather and dry weather sewage, as well as synthetic suspensions. The objective of the study was to evaluate and demonstrate technologies for the treatment of CSOs. The principal process unit was a 3-m diameter Storm King® vortex separator. The 3-m size was selected because it was considered to be large enough to minimize problems of scale-up and small enough to be transportable for use at other sites. Other units placed on the site were a head tank; circular flow clarifier, cross-flow inclined plate clarifier, inclined rotary drum screen, and a mobile filtration pilot plant. Over the two operating seasons in 1994-1995, a total of 71 events were monitored. The wastewater found at the test site, contained a considerable quantity of poorly settleable suspended solids and an apparent industrial component that made it prone to foaming. The average quiescent settling test results over the three seasons indicated that approximately 35% of the wet-weather suspended solids and 45% of the dry-weather suspended solids were non-settleable at a surface loading rate (test threshold) of 0.3 m/h. Experimental results from the first two seasons of operation demonstrated that the vortex separator was more effective for total suspended solids (TSS) removal than the circular clarifier at the same (steady-state) surface loading rate. Also, the total efficiency of the unit was similar to that obtained under quiescent settling conditions in the long column tests. The results indicate that primary treatment efficiency, defined as minimum of 50% removal of TSS, could be attained, on average, from the vortex separator without coagulants at a surface load of approximately 5 m/h when treating CSO suspensions. When coagulant is added, an SOR of up to 9 m/h can be achieved while maintaining a 50% TSS removal.

**Stoke Canon (1984-1990)**

The Stoke Canon Storm King™ Overflow is a 3m freestanding unit installed in 1984. This overflow has been monitored by the UK's South West Water plc and found to comply with the 100 mg/L BOD and 60 mg/L suspended solids standards, with average BOD and SS removals of 35% and 60% respectively.

**Columbus, Georgia, USA**

A pilot study conducted between 1992 and 1993, evaluated the effectiveness of the Storm King™ Overflow compared with a conventional flow through mixing sedimentation basin. This was done in terms of both solids removal and disinfection. The results showed the Storm King™ Overflow vortex system to be up to 10 times more effective for the removal of total suspended solids and other pollutants, and approximately three times as effective at disinfection, compared to the mixed basin system. Following the results of this pilot study, two satellite CSO treatment sites were constructed in1995 at Columbus, Georgia involving the use of Storm King™ Overflow units for sedimentation, contracting and sediment removal.

**University of Sheffield**

The University of Sheffield performed a study on the overall performance of a 3.4 m diameter Storm King™ Overflow with Swirl-Cleanse ™ screen at the National CSO test facility at Hoscar WWTW near Wigan. The system was evaluated by means of capturing solids in 6mm mesh sacks. Overall 12 tests were carried out covering inlet flow rate ranging from 70 to 200 L/s. The work essentially concluded that the system was able to meet the '6 mm in two dimensions' solids removal requirement, and demonstrated that the backwashing mechanism utilized by the system was able to prevent blinding of the screen for the duration of the test (Hydro International plc. 2004).

### 2.7 Continuous Deflection Separation (CDS®)

Continuous Deflection Separation (CDS), a technology invented in 1992 by two Australians, Paul Blanche and Steve Compton. Continuous deflection separators are cylindrical devices constructed in any size necessary to treat storm water from catchments as small as a single parking lot (1 cfs), or as large as an entire drainage basin (300 cfs). As shown in Figure 2.10, it consists of a cylindrical chamber containing a circular, perforated (screen like) plate through which the storm water passes.

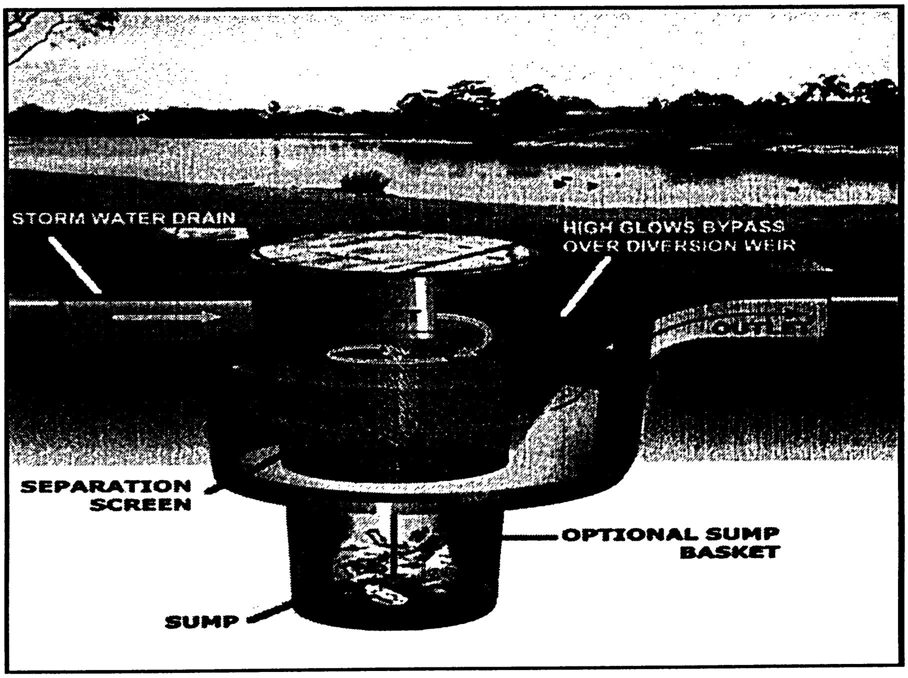



**Figure 2.10 Typical CDS™ Unit** (Source: CDS technologies Inc. 2003)

CDS technology utilizes a non-blocking, non-mechanical screening process to remove pollutants from storm water flow and combined sewer overflows. As mentioned in their product manual, CDS units capture fine sands and solids and are capable of removing more than 80% of annual TSS from storm water. Additionally, CDS units remove 100% of floatables and 100% of all particles, which are equal to or greater than one-half the size of the screen opening. The unit removes 93% of all particles, which are one-third the size of the screen opening, and 53% of all particles one-fifth the size of the screen opening.

CDS technology uses the hydraulic energy of storm water runoff/CSO entering the unit to gather and trap pollutants. The inflow and associated pollutants are diverted from a drainage pipe into the unit; it begins a circular motion that allows the water to pass through a perforated screen and exits via the outlet pipe, while forcing the pollutants to swirl toward the center of the cylinder. As the movement of the pollutants slows, most tend to settle into a central sump where they are no longer affected by the moving water above them (CSO manual). Floatable pollutants simply continue to swirl around the center of the cylinder until flow through the unit stops, or until the floatables are removed, but they cannot escape or be flushed back into the storm water drain. The swirling flow within the separation chamber behaves in the manner of a solid body in rotation. Therefore, objects in the flow that has a density greater than water will be forced outward and would be pressed against the perforated screen if it were not for the tangential flow around the chamber that continuously sweeps the screen and prevents blockage. Thus the units are carefully designed to ensure that the tangential force at every point around the chamber screen is always greater than other forces acting upon pollutants, which would otherwise tend to block the screen. Depending on the nature of the applications, the solids in the sump can be removed using a vector truck, a removable basket or an automated pumping system. CDS® units can be placed underground and are appropriate for situations where space is limited. In CSO applications where a CDS® unit is underground, the collected solids can be returned to the sewer by gravity or by using an underflow pump.

In contrast to vortex separators, the CDS® unit does not rely on secondary currents to concentrate solids in the centre of the chamber. Rather, debris and particles are held captive in the chamber by the special deflective screen as shown in Figure 2.11. The screen apertures are

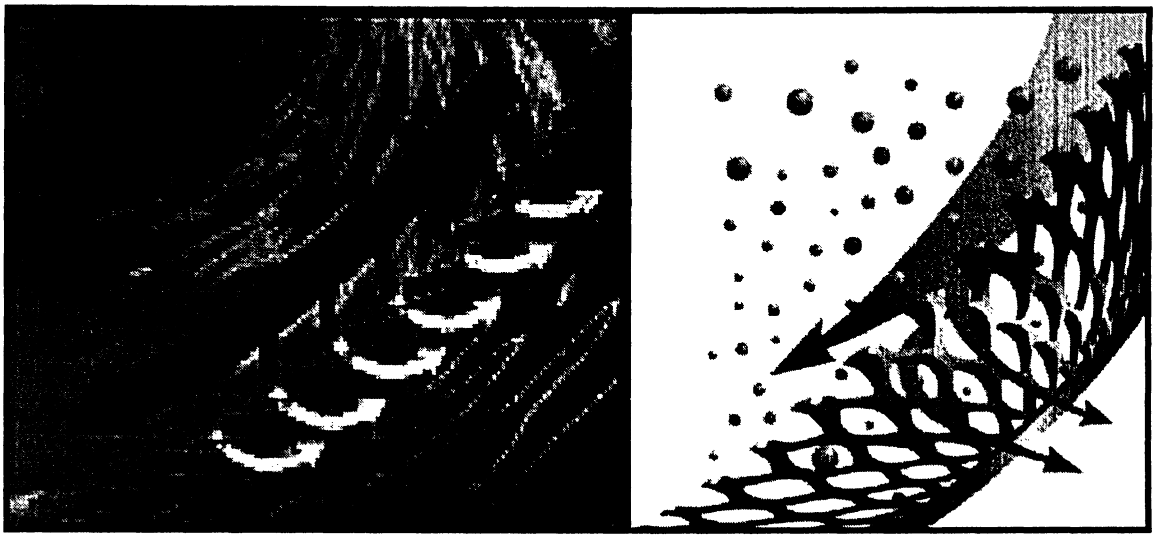


**Figure 2.11 Deflective Characteristic of CDS® Screen** (Source: CSO Manual, 2004)

elliptical and are aligned with the longer ellipse axis in the vertical direction. The width of the shorter ellipse axis is a design specification that depends on the wastewater characteristics. The available width range of screen aperture is 0.6 mm to 4.7 mm. Typically, a screen aperture of 4.7 mm is used for CSO applications or for runoff with large amount of gross trash and debris. Stormwater applications tend to use smaller sizes such as 1.2 mm. CDS Technologies® manufactures pre-cast units with treatment capacity between 31 and 1416 L/s, diameter ranging from 1.8 m - 5.3 m and a sump capacity of 4.0 - 10.8 $m^3$. Cast-in-place units with peak treatment capacity up to 8.5 $m^3/s$ are also available. Table 2.1 presents the capacities and some physical features of CDS® units suitable for CSO applications (CDS Technology Inc. 2003).

### 2.7.1 Hydraulic Design and Analysis of CDS™ Unit

Based on the pollutograph, a CDS™ unit can be designed for the flow that mobilizes the gross pollution in the catchment. The recommended design flows for the CDS™ CSO units are typically those with a return period of 3 to 6 months. These flows are normally in excess of those required to generate movement of pollutants typically associated with "first flush" event. However, should higher flows be identified as movers of pollution in a particular watershed, CDS$^{TM}$ capacity should be increased accordingly. Flows that are within the CDS design capacity are treated in the unit. If runoff flows are greater than the design flow, they are split in a weir diversion box, with the CDS capacity flow passing through the processor, while the excess flow spills over the diversion weir and continues downstream. After the CDS$^{TM}$ design flow has been determined, the appropriate standard model can be selected from the Table 2.1. The approximate height of the weir can be established by determining the hydraulic grade line ($HGL_{d/s}$) in the system immediately downstream of the CDS$^{TM}$ unit and adding the CDS head loss ($h_{cds}$) for the selected unit. The sum of the above represents the $HGL_{u/s}$ required at the entrance to the diversion weir (CDS product manual).

$$HGL_{u/s} = HGL_{d/s} + h_{cds}$$

Then the height of the CDS diversion weir is:

$$\text{Weir height} = HGL_{u/s} - \text{Invert Level}$$

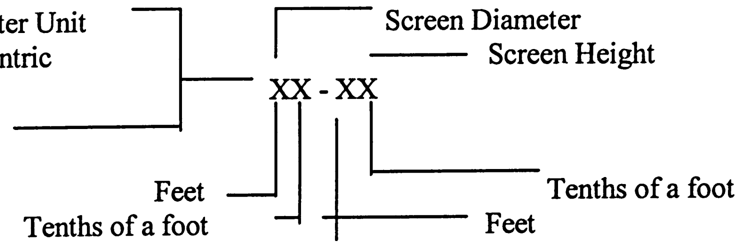
Based on laboratory measurements and analysis, it has been established that the actual head loss under system design flow will not exceed $1.3 \times V^2 / 2g$ in a well-designed diversion structure, where V is the design flow velocity in the system when the pipe is flowing. CDS Technologies recommends that the head loss across the weir be limited to no more than 1.4 times the CDS unit head loss at its design flow to ensure that it continues to operate properly during the conveyance system's peak flows. The effects of the diversion weir primarily influence the rise in the water surface under the conveyance system design flow. The actual effect can be controlled by properly designing the weir length and clear height above the weir to take advantage of the potential energy that can be developed in the system without inducing flooding upstream.

**Table 2.1 Specifications of CDS Systems***

| Model Designation | | | Treatment Capacity Range | | Screen Diameter\ Height (ft) | Sump Capacity ($yd^3$) | Depth Below Pipe Invent (ft) |
|---|---|---|---|---|---|---|---|
| | | | cfs | MGD | | | |
| Precast | Inline | PMIU20_15(Drop-in-inlet) | 0.7 | 0.5 | 2.0\1.5 | 0.5 | 4.2 |
| | | PMSU20_15_4 | 0.7 | 0.5 | 2.0\1.5 | 0.5 | 3.5-4 |
| | | PMSU20_15 | 0.7 | 0.5 | 2.0\1.5 | 1.1 | 5.1 |
| | | PMSU20_20 | 1.1 | 0.1 | 2.0\2.0 | 1.1 | 5.7 |
| | | PMSU20_25 | 1.6 | 0.7 | 2.0\2.5 | 1.1 | 6.0 |
| | | PMSU30_20 | 2.0 | 1.3 | 3.2\2.0 | 2.1 | 6.2 |
| | | PMSU30_30 | 3.0 | 1.9 | 3.0\3.0 | 2.1 | 7.2 |
| | | PMSU40_30 | 4.5 | 3.0 | 4.0\3.0 | 5.6 | 8.6 |
| | | PMSU40_40 | 6.0 | 3.9 | 4.0\4.0 | 5.6 | 9.6 |
| | Offline | PSWC30_20 | 2.0 | 1.3 | 3.0\2.0 | 1.9 | 6.0 |
| | | PSW30_30 | 3.0 | 1.9 | 3.0\3.0 | 1.8 | 7.0 |
| | | PSWC30_30 | 3.0 | 1.9 | 3.0\3.0 | 2.1 | 7.0 |
| | | [redacted] | | | | | |
| | | PSWC30_30 | 6.0 | 3.9 | 4.0\4.0 | 1.9 | 9.6 |
| | | PSW50_42 | 9.0 | 5.8 | 5.0\4.2 | 1.9 | 9.6 |
| | | PSWC56_40 | 9.0 | 5.8 | 5.6\4.0 | 1.9 | 9.6 |
| | | PSW50_50 | 11.0 | 7.1 | 5.0\5.0 | 1.9 | 10.3 |
| | | PSWC56_53 | 14.0 | 9.0 | 5.6\5.3 | 1.9 | 10.9 |
| | | PSWC56_68 | 19.0 | 12.0 | 5.0\6.8 | 1.9 | 12.6 |
| | | PSWC56_78 | 25.0 | 16.0 | 5.6\7.8 | 1.9 | 13.6 |
| | | PSW70_70 | 26.0 | 17.0 | 7.0\7.0 | 3.9 | 14.0 |
| | | PSW100_60 | 30.0 | 19.0 | 10.0\6.0 | 6.9 | 12.0 |
| | | PSW100_80 | 50.0 | 32.0 | 10.0\8.0 | 6.9 | 14.0 |
| | | PSW100_100 | 64.0 | 41.0 | 10.0\10.0 | 6.9 | 16.0 |
| Cast in Place | CSW150_134 | | 148.0 | 95.5 | 15.0\13.4 | 14.1 | 19.6 |
| | CSW200_164 | | 270.0 | 174.0 | 20.0\16.4 | 14.1 | 22.6 |
| | Csw240_160 | | 300.0 | 194.0 | 24.0\16.0 | 14.1 | 21.2 |

Note: Shaded model indicates the selected model for HRT pilot study of City of Niagara Falls

PMSU = Precast Manhole Storm Water Unit
PSWC = Precast Storm Water Concentric
PSW = Precast Storm Water
CSW = Cast in Place Storm Water




*(Source: Li, 2004)

### 2.7.2 CDS Variable Components

As per CDS product manual, the variable components in a CDS unit within a model family are the screen height, screen aperture, sump diameter and depth.

**Screen Height**

The screen height is important within a model family because it controls the design flow that can pass through the unit without clogging the screen. In general, screen heights can vary between 60 to 150 % of the screen diameter.

**Screen Aperture**

The screen aperture is important because it sets the capture parameter for settleable pollutants. The standard screen for storm water applications is 4700 µm for coarse screening. A 2400 µm is also available where there is a need to separate sediments than those removed by the 4700 µm screen.

The CSO Pilot Study (HydroQual, 2002) performed at Rockland County, New York, evaluated the effectiveness of a CDS unit to treat raw wastewater similar to CSO in solids characteristics. Two screens of 1200 µm and 600 µm, substantially smaller than the CDS® technology typically used (2400 µm) for floatables removal, were investigated. TSS removals averaged 10% to 30% for the two sizes, respectively. The smaller screen was found to blind at its surfaces, while the 1200 µm screen retained the desired self-cleaning capability.

**Sump**

The sump is another variable that can be adjusted for site-specific conditions and utility preference. Each model family is equipped with a standard sump. However, the diameter and depth can be adjusted to meet site-specific requirements.

Sump cleanout is a critical component of a successful CDS operation. The methods for maintenance and cleanout are generally specific, dependent on the preferences of a given agency.

At the utility's discretion, a unit can be cleaned using a vacuum truck or a small clamshell bucket, or a basket can be provided to fit a standard sump.

### 2.7.3 CDS Unit Maintenance

According to the CDS technical manual, it is recommended that the unit be inspected periodically to assure its condition to handle anticipated runoff. The inspection for new installation should be after every runoff event for the first 30 days. During the wet season, the unit should be inspected at least once every 30 days. At least once a year, the unit should be pumped down and the screen carefully inspected for damage and to ensure that it is properly fastened. The standard maintenance cycle should be a minimum of once a year.

### 2.7.4 Field Monitoring Studies of CDS Units

**Melbourne, Australia**

Cooperative Research Center (CRC) for Catchment Hydrology conducted a field study of a CDS system (4.7 mm Aperture Screen) in Melbourne, Australia. The CDS system was installed at a 4 ft diameter drain that caught runoff from a 125 hectare watershed. The watershed had one-third commercial and two-thirds residential land uses. During a six months monitoring period, almost all gross pollutants were trapped by the CDS. At high flow conditions, the CDS trapped 74% total suspended solids. However, the mean nutrient removal efficiency during high flows was found to be about 36%.

**Louisville and Jefferson County, Kentucky**

The Louisville and Jefferson County Metropolitan Sewer District (MSD) began the testing of a combined system of CDS and Trojan UV treatment for controlling fecal coliforms in CSO. Twenty-four samples were taken over 5 days spread over two months. Except the first day, all samples showed a reduction of total coliforms and E. Coli. For instance, the E. Coli had 3.4 log reduction.

**Brevard County**

In July 1997, Brevard County's Stormwater Utility Program installed a CDS® unit. This location served a drainage basin of 24.87 hectares (62.45 acres) of mixed industrial, commercial, and vacant land. Over an 18 month period 5 storm events were monitored. The time of concentration to the site was 63 minutes, with a 10 years flow of 1,557 L/sec (55 cfs) and mean annual flow of 1,177 L/s (38.2 cfs). In Brevard County, the 10 years storm was 20.1 centimeters (7.9 inches) of rainfall and the mean annual storm was 13.97 centimeters (5.5 inches) of rainfall. There was no base flow at this location. A diversion weir was placed in front of the culvert giving an off-line design which effectively diverted flows under 254 L/sec (9 cfs) through the CDS® unit. It was estimated that the CDS® unit provided an average removal efficiency of 52% for total suspended solids and 31% for phosphorus respectively.

# CHAPTER THREE

## SETTLEABILITY TEST PROCEDURES

Understanding the nature and settling characteristics of CSO solids assists in selecting the appropriate CSO treatment process. A number of different methods have been used in the past to characterize settleability of CSOs and wastewater. This chapter briefly describes the different settleability test procedures based on the interim report "Evaluation of CSO Treatability for the City of Welland" prepared by Exall et al. (2004).

### 3.1 Test Procedures

From the viewpoint of treatability, solids in CSO can be classified in a number of ways, but primarily as suspended or filterable solids. Suspended solids can be subdivided into settleable and nonsettleable solids, and filterable solids can be subdivided into colloidal or dissolved solids. The settling characteristics of suspended solids in combined sewer overflow and stormwater runoff is the determination of the effectiveness of using high rate treatment for the removal of suspended solids from these wastewater. The conventional long column settling test is an established procedure for the quantification of settling rates in the flocculant regime. Flocculant settling behaviour results from interactions between the suspended particles. Weak agglomerates of flocs are formed as particles of different sizes and settling rates collide while settling (MOEE, 2000). The result is a gradual increase in settling rates with time and with depth. Consequently, the test procedure must incorporate the actual times and depths applicable to the treatment unit which is being simulated. The testing procedure includes three main steps:

- Event identification and sampling of CSOs
- Conducting settling column testing
- Water sample analysis to determine particle size distribution.

Several researchers have compared the traditional and alternative methods in order to determine which technique is the most suitable for assessing the treatability of wet-weather flows (Exall et al. 2004). Since the testing methods for CSO characterization are not standardized, four such methods were included in this study. Three of the methods use settling columns, including Aston column, Brombach column and the U.S. EPA multi-port long column; the fourth one is a new elutriation method using an elutriation apparatus. The individual methods used are described below.

### 3.2 Aston Column

The Aston column was developed at Aston University, UK with the objective of characterizing not only settling solids (sinkers), but also floating solids (floaters). The Aston column is constructed of acrylic (2.2 m long and 5 cm ID), has a volume of approximately 6 L, and is supported by central gimbals allowing 180° rotation in the vertical plane to facilitate sampling of settled and floating solids (Figure 3.1). At each end of the column, ball valves isolate terminal cells, which separate the sampling volume from the rest of the column. The test procedure is as follows:

- The column is first filled with thoroughly mixed wastewater sample (approximately 6 L) at ambient lab air temperature (~20°C). With the outside valves closed, the column is rotated several times, the inside valves are exercised to purge trapped air, and sewage is topped-up as required to fill the column.
- The column remains undisturbed in the starting vertical position during a 3-hour initial settling period. After the initial settling period, the two inside valves are closed, and water with floaters and sinkers collected during the initial period is removed from the outer cells A and B (see Figure 3.1). The initial floaters (cell A) are saved for further analysis;
- The sinkers from cell B are thoroughly mixed in a small volume of tap water, poured into the top cell A, the bottom cell B is filled with tap water and the column is returned to the starting position.
- In sequence, the inside top valve (#2) and bottom inside valve (#3) are opened, releasing the re-introduced sinkers into the central column section for settling over a 2.5 hour period.

- At the pre-selected time intervals (1, 3, 5, 10, 20, 30, 40, 60, 90, 120, and 150 minutes), settled solids are collected.
- At the end of each sampling interval, valve 3 is closed (isolating cell B), and valve 4 is opened to collect the sample. The column is then inverted, cell B is refilled with tap water, the valves are exercised to purge entrained air, and the column is rotated back to its starting position. Valve 3 is then opened to capture settled sediment and stays open until the next sampling interval.
- At the end of the test, the final floaters, sinkers, and non-settled sample volumes are collected, and the apparatus is flushed. All samples including the flush are analyzed for TSS. The relationship between the solids captured and the time interval is used to generate a settling curve. The settling velocity is determined from the time interval and the depth of the settler. The characteristics of the settling solids are determined from the solids captured at the different intervals. Finally the results are used to check the mass balance of the test procedure.

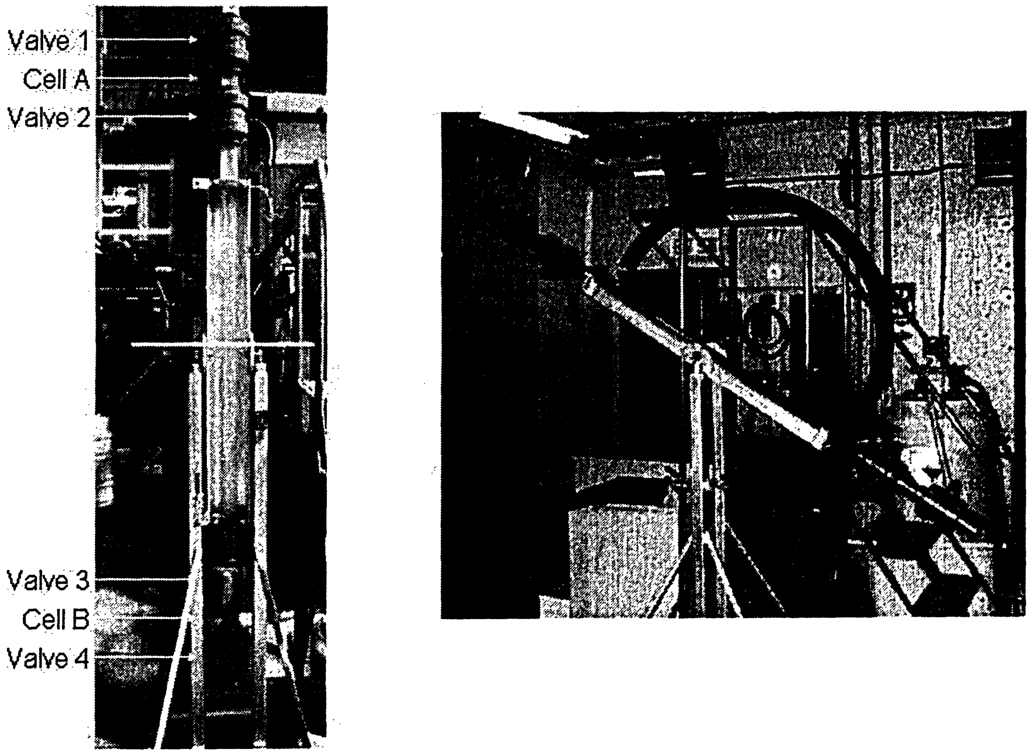



**Figure 3.1 Aston Column Components and Column Rotation** (Source: Exall et al. 2004)

### 3.2.1 Mass Balance Calculation for Aston Column

Let the original mass of the particles in the raw sample used in the Aston column be $M_R$, and the masses of the sinkers and floaters collected after the initial three hour period be $M_S$ and $M_F$ respectively. The masses of samples collected at different sampling times during the settling experiment are denoted by the symbol, $M_i$, where (i=1, 2 ...). Let the mass of the floaters collected at the end of the settling experiment be $M_{FE}$, and the mass of the non-settled fraction in the column at the end of the test be $M_{NS}$. The mass collected during the flushing operation is denoted by $M_{flush}$. Using these symbols, the total mass of particles measured at different stages of the operation was calculated as:

$$\text{Total mass of particles measured } (M_M) = M_S + M_F + \sum M_i + M_{FE} + M_{NS} + M_{flush} \qquad (3.1)$$

The above mass was compared with the original raw sample mass $M_R$, and a mass balance error (MBE) as a percentage was calculated as follows:

$$\text{MBE} = \frac{M_M - M_R}{M_R} X100 \qquad (3.2)$$

### 3.2.2 Calculation of Settling Velocity Distribution for Aston Column

The settling velocity distribution was calculated using $M_i$ values as follows:
$M_i$ collected at $T_i$ gives the mass of the particles that have settling velocity in the range between $L/T_i$ (where L is the length of the column), and $L/T_{i-1}$. Expressing $M_i$ as a percentage of $M_M$, a cumulative percentage of particles that have a settling velocity less than a certain value can be calculated as shown in Table 3.1. A typical settling velocity distribution measured using the Aston column is shown in Figure 3.2

**Table 3.1 Calculation of Settling Velocity Distribution Using Aston Column Data***

| Settling velocity, $SV_i$, in mm/s | % of particles with settling velocity less than corresponding $SV_i$ |
|---|---|
| $L/T_1$ | 100 - (% of mass collected at $T_1$) |
| $L/T_2$ | Above value - (% of mass collected at $T_2$) |
| . <br> . | . <br> . |
| $L/T_{11}$ | Above value – (% of mass collected at $T_{11}$) |
| $-L/T_{11}$ | Above value – $(M_{NS}/M_M)*100$ |
| $-L/T_{10}$ | Above value – $(M_{FE}/M_M)*100$ |
| $-L/T_1$ | Above value – $(M_F/M_M)*100$ |

* Source: Exall et al. 2004.

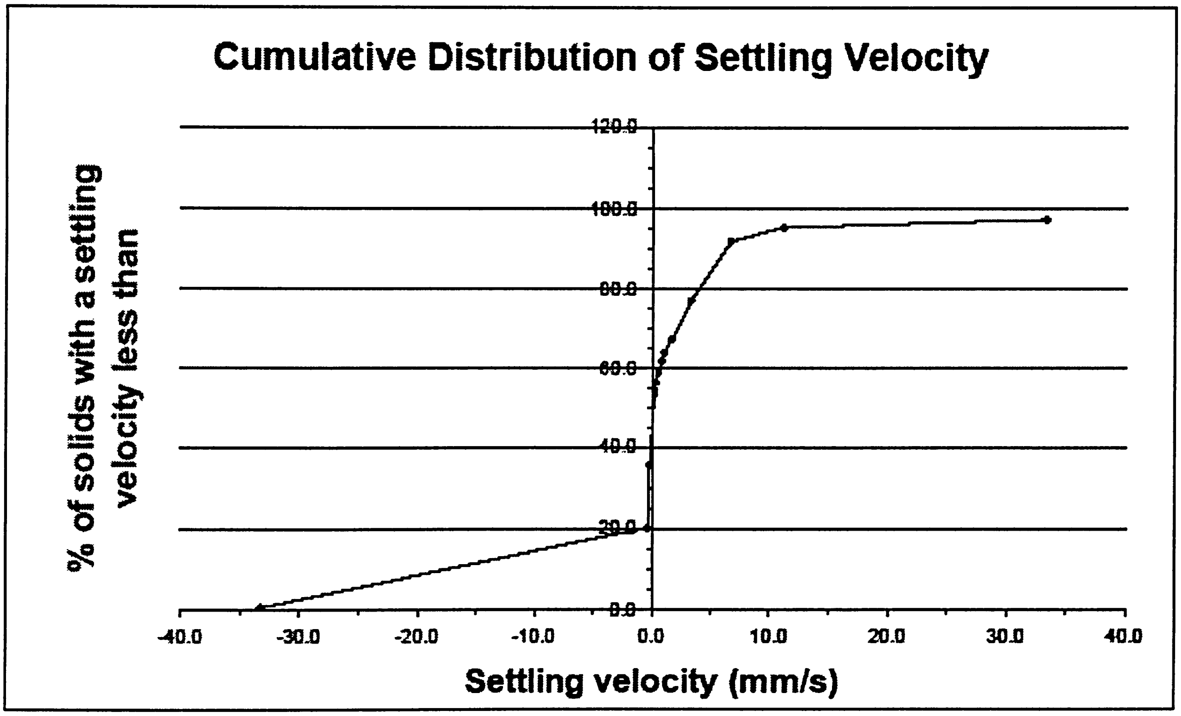



**Figure 3.2 A Typical Settling Velocity Distribution Measured using the Aston Column**

(Source: Exall et al. 2004)

### 3.3 Brombach Settling Column

The Brombach column, named after its creator Prof. Hahns Brombach of Umwelt-Und Fluid Technik in Bad Nergentheim Germany, has been used extensively to characterize the settleability of CSOs. The column consists of an upper reservoir (500 mL), with an offset sample delivery cylinder, middle, transparent column section (approximately 5 cm ID x 49 cm), and an Imhoff cone (100 mL) attached to the column bottom. Samples are collected from the cone using a silicone tube with a pinch-clamp (Figure 3.3). The Brombach method involves the following:

- 1 L well mixed sample of the wastewater at ambient lab air temperature is first filled into the column and allowed to settle for two hours in the settling column, an Imhoff cone. After this period, the settled sludge volume index (SVI) (ml/L) is determined as the volume of solids accumulated in the Imhoff cone (measured in mL) divided by the sample volume (1 L).
- The bottom sludge is then withdrawn and saved for further testing. The remaining wastewater in the column (the non-settling fraction) is drained, and the column is refilled with tap water at the ambient lab air temperature. The bottom sludge is then mixed with tap water to obtain 75 mL of slurry, which is then poured into the offset sample delivery cylinder in the upper reservoir of the column.
- To initiate the second phase of the settling test, the sample delivery cylinder is slid sideways, until aligned with the top opening of the settling column, and the slurry is released from the cylinder into the settling column.
- The material is then allowed to settle, and 25 ml samples are withdrawn from the Imhoff cone drain tube at logarithmic time intervals (e.g. 15 sec, 30 sec, 60 sec, 2 min, 4 min, 8 min, 15 min, 30 min, 60 min, and 120 min.) to define the relative amounts of settleable mass of the original sludge.
- After each sample withdrawal, the upper water reservoir is replenished with an equivalent volume of tap water to maintain a constant hydraulic head in the column. The final non-settling volume and column flush are sampled and analyzed to verify mass balance for the test.

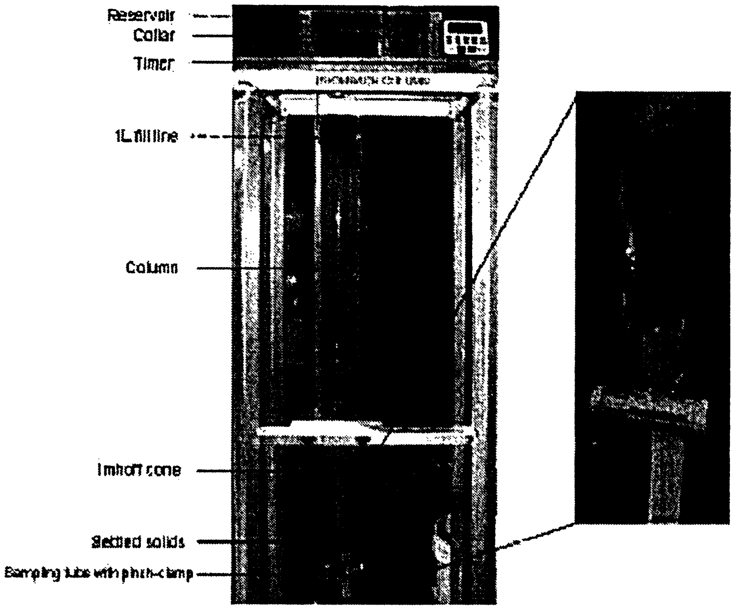



**Figure 3.3 Brombach column: Overall View and a Detailed View of the Imhoff Cone**
(Source: Exall et al. 2004)

### 3.3.1 Mass Balance Calculation for Brombach Settling Column

Let the masses of initially non-settled, initial column flush, final non-settled and final column flush be denoted by $M_{NSI}$, $M_{flush1}$, $M_{NS2}$, $M_{flush2}$ respectively. Let the masses of sampled particles be $M_i$ (i =1, .. 10), and the mass of particles in the original raw sample be $M_R$ . Using these symbols, the total mass of particles measured at different stages of the Brombach column use is calculated as:

Total mass of measured particles ($M_M$) = $M_{NSI} + M_{flush1} + \sum M_i + M_{NS2} + M_{flush2}$ (3.3)

The above mass was compared with the original raw sample mass $M_R$, and a mass balance error (MBE) was calculated using Equation (3.2).

### 3.3.2 Calculation of Settling Velocity Distribution for Brombach Column

The procedure used to calculate settling velocity distributions using data from the Brombach column is similar to the one used for the Aston column data. Table 3.2 gives the details. A typical settling velocity distribution measured using the Brombach column is shown in Figure 3.4

**Table 3.2 Calculation of Settling Velocity Distribution for Brombach Column Data***

| Settling velocity, $SV_i$, in mm/s | % of particles with settling velocity less than corresponding $SV_i$ |
|---|---|
| $L/T_1$ | 100 - (% of mass collected at $T_1$) |
| $L/T_2$ | Above value - (% of mass collected at $T_2$) |
| . . | . . |
| $L/T_{10}$ | Above value – (% of mass collected at $T_{11}$) |

* Source: Exall et al. 2004.

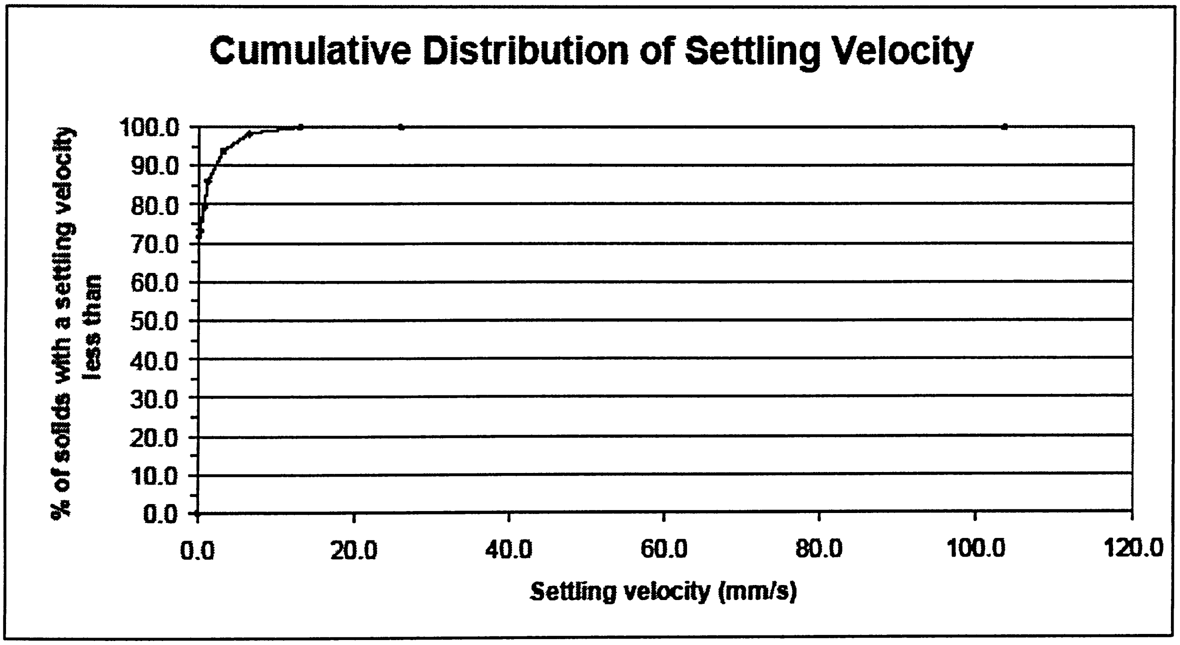



**Figure 3.4 A Typical Settling Velocity Distribution Measured using the Brombach Column** (Source: Exall et al. 2004).

### 3.4 EPA Settling Column

The U.S. EPA column is also known as the "long" column. It is usually constructed of clear acrylic, in lengths ranging from 1.8 to 2.5 m, and fitted with evenly spaced side ports for sample withdrawal (Figure 3.5) and a drain valve at the bottom. During the tests, it is quickly filled from the top with well mixed sewage. At set time intervals (2, 4, 8, 16, 30, 60, 120 min), samples are collected from the top, centre and bottom side ports. As successive samples are withdrawn, the total depth of sewage in the column is reduced, which necessitates corrections of calculated settling rates for these changes. As the test progresses, larger sample volumes may have to be withdrawn to maintain an appropriate accuracy of TSS determinations. At the conclusion of the test, a sample of the settled solids (accumulated on the bottom of the column), unsettled solids (remaining in the column), and a residual column flush are collected, analyzed for TSS, and the corresponding masses are used in mass balance calculations.

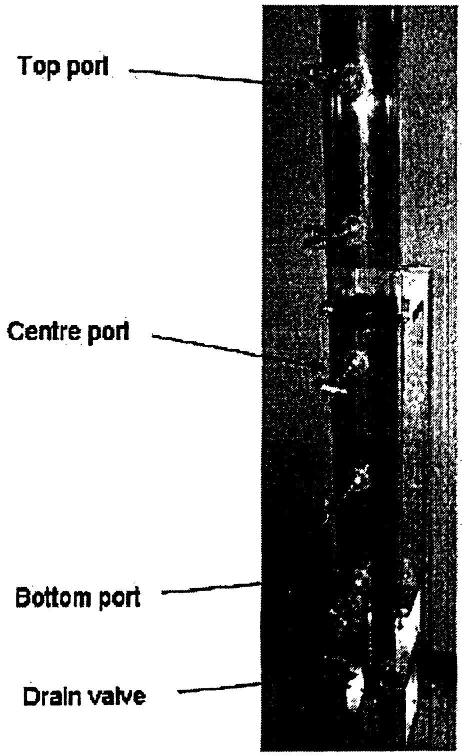



**Figure 3.5 EPA Settling Column** (Source: Exall et al. 2004).

### 3.4.1 Mass Balance Calculation for EPA Settling Column

Let the mass of the raw sample used in the EPA column be $M_R$ , and the masses of nonsettled, settled and the flush portions of the particles be $M_{NS}$ , $M_S$ , and $M_{flush}$ respectively. The masses of the solids collected during sampling are denoted as $M_{i,j}$ (i = top, middle and bottom ports and j = 1-7, and this includes the mass wasted from each port when sampling). The total mass of particles measured during the operation of the column is denoted as $M_M$ , and is given as follows:

$$M_M = M_{NS} + M_S + M_{flush} + \sum M_{i,j} \qquad (3.4)$$

Using $M_M$ and $M_R$, an error in mass balance for this method was calculated according to Equation 3.2.

### 3.4.2 Calculation of Settling Velocity Distribution for EPA Settling Column

Samples collected at the top, middle and bottom sampling ports give the concentration of solids at different time intervals at these three locations. Knowing the distances from the free surface to these sampling locations and the sampling times, three different settling velocities can be calculated and the masses of solids exceeding these three settling velocities can be computed by knowing the concentrations of solids in three overlapping portions of the column. In calculating the concentrations of the solids in different portions of the column, average values were computed using the measured concentrations at different elevations. The settling velocity and the percentage of mass of particles that have settling velocities less than the specified value were sorted and plotted into a cumulative settling velocity distribution. A typical distribution measured using the EPA column is shown in Figure 3.6. The points are the measured data and the line represents an analytical expression that gives the best fit to the data.

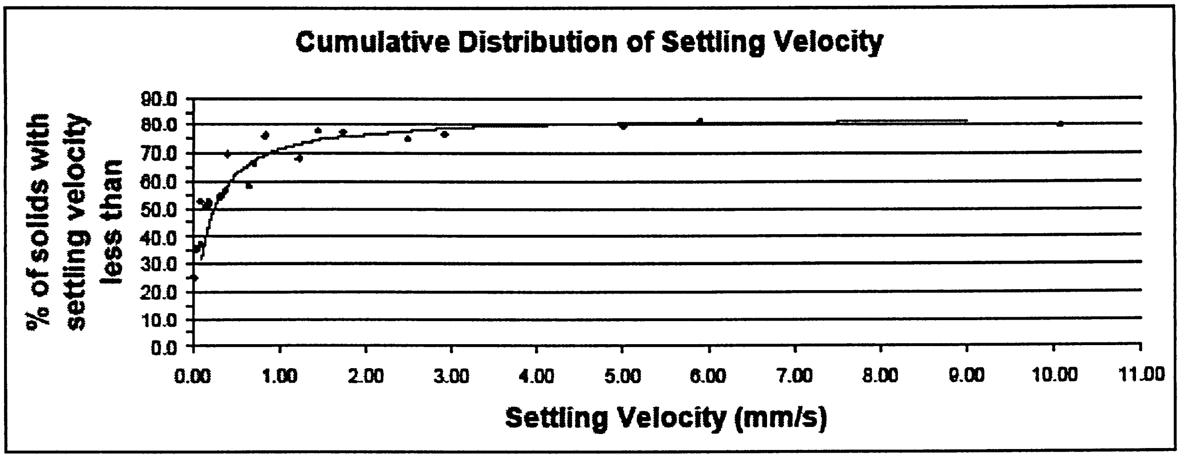



**Figure 3.6 A Typical Settling Velocity Distribution Using the EPA Settling Column**

(Source: Exall et al. 2004).

### 3.5 Elutriation Apparatus

Elutriation apparatus method (EAM) provides an alternative approach to conventional static settling column tests. In EAM, the particles are exposed to dynamic interaction while settling, and this more accurately reflects the type of settling which would occur in a conventional full-size flow-through settling basin. The method is adapted from a water elutriation process, which was originally proposed, by Walling and Woodward (1993) to measure particle size distribution of riverine suspended sediment. The original apparatus developed by Walling and Woodward (1993) consisted of four cylinders with diameters 25 mm, 50 mm, 100 mm and 200 mm, and arranged sequentially in the ascending order of their diameters. The river water was drawn through these cylinders by a pump, which was placed at the downstream side of the cylinders. The river water was routed through these cylinders in such as way that it entered the cylinders near the bottom and exited near the top. Such an arrangement allowed the river sediment that has settling velocity higher than the upward velocity of the water to settle in a particular cylinder. Since the diameters of the cylinders were progressively increasing, sediment with different settling velocities settled in different cylinders. By measuring the amount of sediment in each cylinder, the settling velocity distribution was deduced.

Krishnappan *et al.* (2004 in press) used such a system and developed a protocol for measuring the settling velocity distribution of CSO solids. The apparatus consists of eight cylinders (instead of four) to provide higher resolution of settling velocity distributions (first seven columns) and to trap the floatable material (8th cylinder). The configuration of the apparatus is shown in Figure 3.7. Columns 1 through 8 are filled with distilled water at the start of the experiment. The internal diameters of settling columns 1 through 8 are: 25, 34, 49, 70, 105, 143, 197 and 197 mm. In the present test procedure, CSO samples are split into two 25 L carboys (a total of 50 L of sample is eluted) and mixed by impellers. A Y-connector combines the delivery lines from the two carboys, so that their streams become completely mixed prior to entering the first column. This configuration was designed to duplicate the effect of an online mixing process such as polymer addition, which is often used to improve settleability of CSOs. As the CSO sample enters the column at the bottom, it begins to rise towards the outflow tube located at the top of the column. Particles or flocs with settling velocities greater than the upward flow velocity are retained within the column, and particles with settling velocities smaller than the upward flow velocity are carried through into the next column. As the upward flow velocities in each successive column become progressively slower, finer and finer solids settle. Finally, column 8 at the downstream end of the apparatus is designed to collect floatable materials by having reversed flow field, in a downward direction (Figure 3.5). Floatable materials are retained in the top portion of the column, and all other materials with settling velocities smaller than those collected in column 7 will pass through to the effluent carboys. The masses of solids collected in each column (and effluent carboys) are determined using a conventional TSS analysis (Exall et al. 2004).

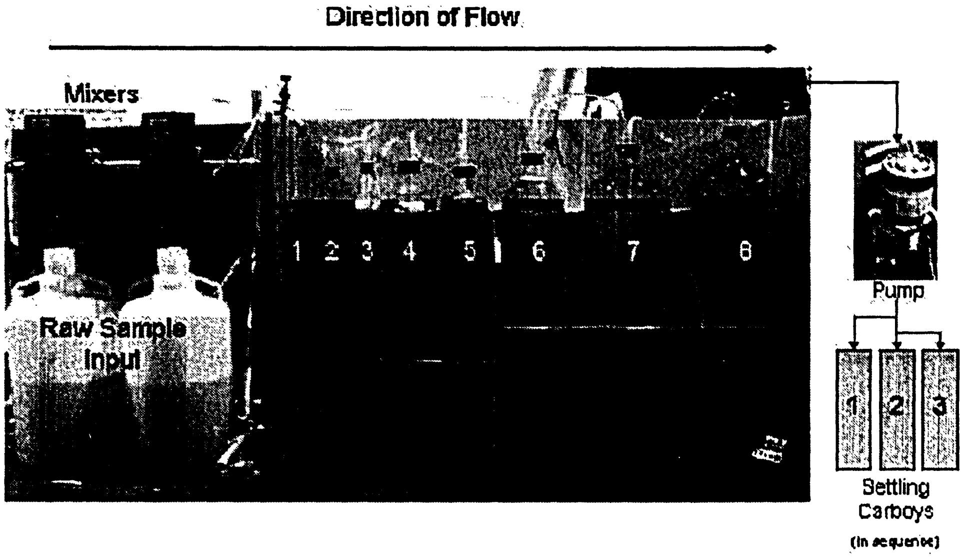



**Figure 3.7 Elutriation Apparatus Configuration** (Source: Exall et al. 2004).

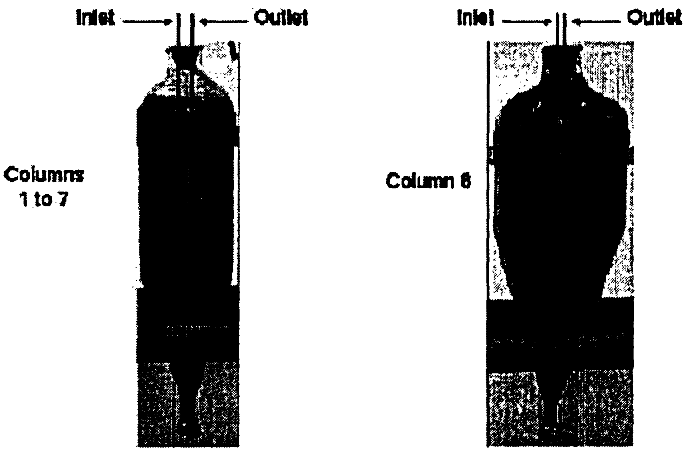



**Figure 3.8 Elutriation Apparatus Flow Arrangements: Upward Flow in Columns 1-7 and Downward Flow in Column 8** (Source: Exall et al. 2004).

### 3.5.1 Mass Balance Calculation for Elutriation Apparatus

The masses of particles in the two raw sample input carboys are measured before and after the operation of the elutriation apparatus. The difference gives the mass of particles routed through the apparatus during the test. This mass is then compared with the masses collected in all eight columns and three collecting flasks. From this comparison, a mass balance error is computed.

### 3.5.2 Calculation of Settling Velocity Distribution for Elutriation Apparatus

From the value of the flow rate through the apparatus, the flow velocities in individual columns can be computed. Particles collected in a particular column have settling velocities larger than the flow velocity in that column. Therefore, knowing the settling velocities in all the columns, and the masses of particles collected in these columns, a cumulative settling velocity distribution is calculated. A typical distribution measured using the Elutriation apparatus is shown in Figure 3.9.

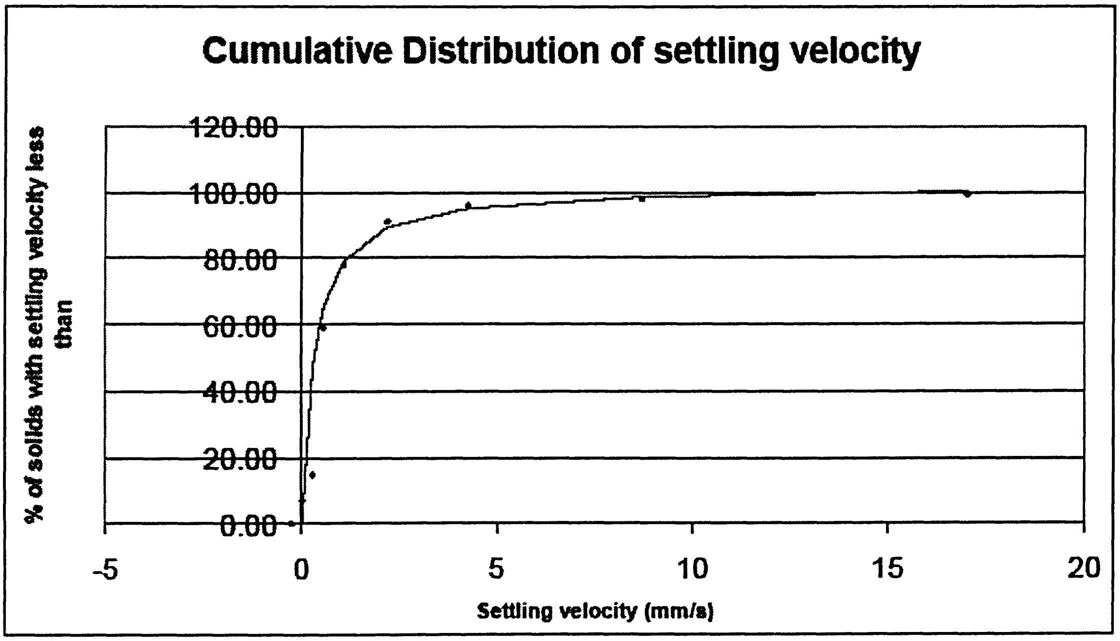



**Figure 3.9 A Typical Settling Velocity Distribution Measured Using the Elutriation Apparatus** (Source: Exall et al. 2004).

# CHAPTER FOUR

## ANALYTICAL PROBABILISTIC MODEL

Urban drainage system performance analysis is essential to plan, design and operate cost-effective drainage system alternatives both for the construction of new system and for the rehabilitation of existing system. Numerous models have been developed for the analysis of stormwater drainage system. Analytical Probabilistic Model is one of them. A brief description of this model based on research of Li and Adams (2000) is presented in this chapter.

### 4.1 Modelling Concept

To protect the society and the environment adequately from stormwater impacts such as flooding, erosion and receiving water pollution while minimizing the resources required doing so; stormwater management analysis models have become essential to the task. Such models require adequate representations of both the hydrologic and hydraulic behavior of drainage systems in order to size and configure system control elements in a cost-effective manner. From a hydrological perspective, the estimation of runoff derived from precipitation is required. From a hydraulic perspective, the transport or routing of these flows through various drainage system elements, such as conveyance devices and storage facilities, is necessary (Adams et al. 2000). A major problem in the planning and design of the engineered elements of the system is to establish the size, configuration, and operation of these elements to best meet the performance objectives of the drainage system. Since the performance objectives are generally expressed in terms of frequency of occurrence, it is necessary to describe the meteorological input probabilistically. For this purpose, analytical probabilistic models were applied to analyze the runoff quantity/quality control performance of various combinations of storage and treatment systems. These analytical probabilistic models are developed with derived probability distribution theory whereby the input meteorology to the catchment is described by probability density functions (pdf's) of the meteorological characteristics, which are transformed, by hydrologic/hydraulic functions to pdf's of the system performance variables. The resulting pdf's are then used to determine both average performance conditions and frequencies of extreme conditions.

The approach used to model the rainfall-runoff-overflow transformation is illustrated for a single catchment in Figure 4.1. Initial rainfall fills the depression storage ($S_d$) on the catchment and a

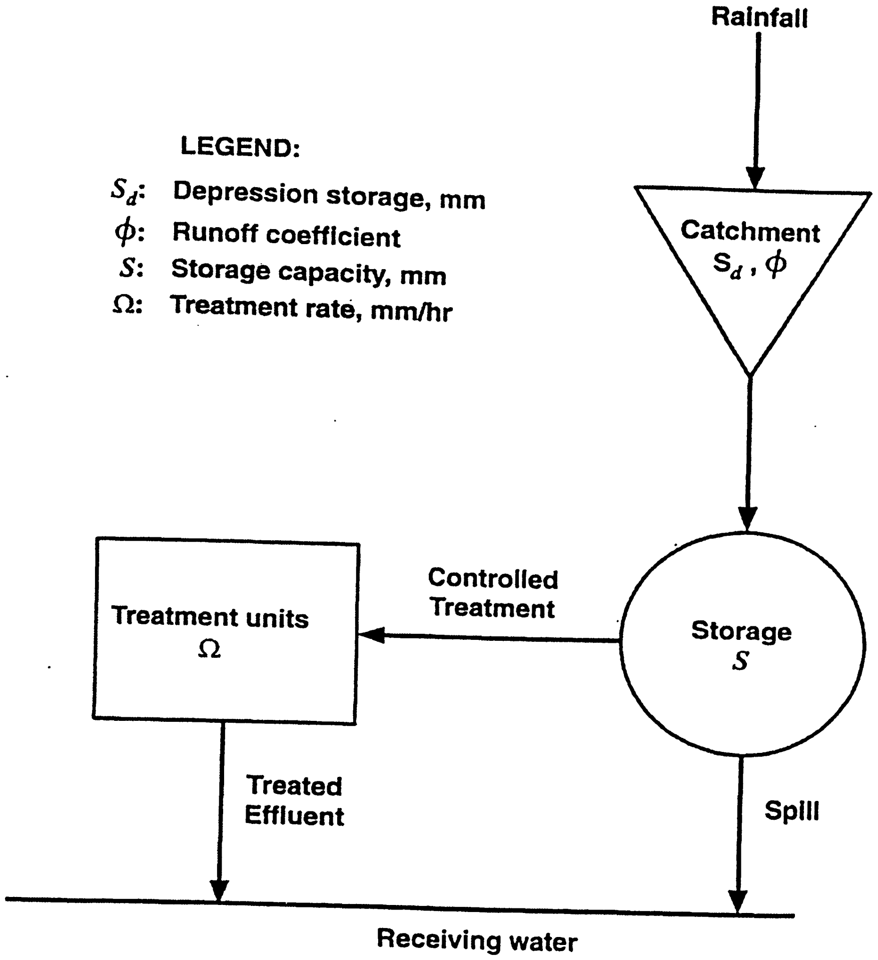



**Figure 4.1 Schematic Model of Urban Drainage System** (Li and Adams, 2000)

fraction ($\Phi$) of the remaining rainfall becomes runoff which is concentrated at a downstream storage treatment site. If the runoff volume over its duration is less than that available in the storage system (S) and that processed by the controlled outflow capacity to a treatment system ($\Omega$), no overflow occurs and the only source of pollution is the treatment plant effluent. For larger runoff events, part of the runoff is overflowed from the storage reservoir into the receiving water while the captured runoff is treated and released as effluent. Thus, the total pollution load for the overflow condition includes contributions from both storage overflow and treatment system effluent. In order to model the total pollution load to the receiving water, both the overflow and non-overflow conditions must be taken into consideration (Li and Adams, 2000).

Long-term quantity and quality control performance of a runoff control measure can be specified by the following measures:

1) quantity control performance measures such as the average annual percent of runoff volume controlled ($C_r$) and average annual number of overflows ($N_s$); and
2) quality control performance measures such as the average annual percent of runoff pollution mass controlled ($C_p$) and average annual number of overflows ($N_s$).

The U.S. Environmental Protection Agency and the Ontario Ministry of Environment have specified Cr and $N_s$, in their criteria for combined sewer overflow control.

## 4.2 Derivation of Models

Derivation of the models for quantity and quality control performance involves four steps: (l) transformation of input rainfall volume to runoff volume; (2) transformation of runoff volume to overflow from a storage-treatment system; (3) transformation of runoff volume to runoff pollution mass load; and (4) transformation of runoff pollution load to total pollution load in the receiving water from a storage-treatment system.

### 4.2.1 Rainfall-Runoff Transformation

A continuous rainfall record can be divided into discrete rainfall events by applying an interevent time definition (IETD). Rainfall pulses which are separated by a time interval greater than the IETD are considered to be separate events. Once this distinction is made, a point rainfall record is divided into discrete events; the events can be statistically analyzed to determine the magnitudes of rainfall characteristics; namely, the volume (v), duration (t), average intensity (i), and interevent time (b). The rainfall record then contains a time series of magnitudes for each of the above characteristics. Adams et al. (1986) found that these rainfall event characteristics can be described by exponential probability density functions (Pdfs) as follows:

$$f(w) = ze^{-zw} \;;\quad z = 1/E[w] \tag{4.1}$$

in which f(w) is the probability density function (pdf) of w; w is the rainfall event characteristic, i.e. v, b, t, i; E[w] is the expected value of w; and z is the reciprocal of E[w]. Kauffman (1987) compiled the above rainfall parameters for thirty-eight long term rain gauge stations in Canada. The transformation of a rainfall event to a runoff event is given by the following:

$$v_r = \begin{cases} 0 & \text{if } v \leq S_d \\ \Phi(v - S_d) & \text{if } v > S_d \end{cases} \tag{4.2}$$

in which $v_r$ is the runoff event volume (mm), v is the rainfall event volume (mm), $S_d$ is the depression storage (mm), and $\Phi$ is the runoff coefficient (dimensionless). With the pdf of rainfall event volume (4.1) and the rainfall-runoff transformation relationships (4.2), the cumulative distribution function (cdf) of runoff event volume ($Fv_r(v_r)$) is derived analytically. Figure 4.2 schematically shows the transformation function as depicted in Equation (4.2).

According to the rainfall-runoff model, rainfall events will not cause runoff events if their volume is less than that of the depression storage. As a result, there is an impulse probability that no runoff will occur which is equal to the probability that a given rainfall event's volume does not exceed depression storage and is represented by the shaded area in Figure 4.2. This impulse probability is given by

$$pv_r(0) = \Pr ob[V_r = 0] = \Pr ob[V \leq S_d] = \int_{v=0}^{S_d} fv(v)dv = \int_{v=0}^{S_d} \zeta e^{-\zeta v} dv = 1 - e^{-\zeta S_d} \tag{4.3}$$

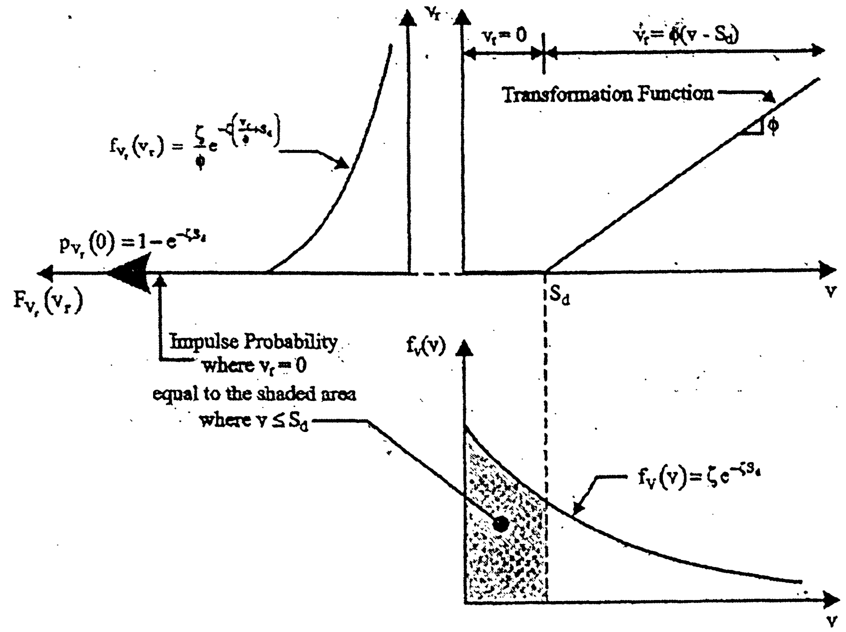



**Figure 4.2 Transformation of PDF of Rainfall Volume to PDF of Runoff Volume**

(Source: Adams & Papa, 2000)

The remainder of the cdf of runoff volume exists over the range where runoff occurs ($V_r$), which corresponds to the range where the volume of a rainfall event is greater than the depression storage value (v> $S_d$). That is

$$
\begin{aligned}
Fv_r(v_r) &= \Pr ob[V_r \le v_r] = \Pr ob(V_r = 0) + \Pr ob[S_d < V \le \frac{v_r}{\Phi} + S_d] \\
&= \Pr ob[V_r = 0] + \int_{S_d}^{(v_r/\Phi)+S_d} \mathrm{fv(v)dv} = 1 - \mathrm{e}^{-\zeta[(v_r/\Phi)+S_d}
\end{aligned}
\quad (4.4)
$$

The expected value of runoff event volume ($E\,(V_r)$, mm), is given as

$$E[V_r] = 0.pv_r(0) + \int_{v_r=0}^{\infty} v_r fv_r(v_r)dv_r = \frac{\Phi}{\zeta} e^{-\zeta S_d} \quad (4.5)$$

and the average annual runoff volume (R, mm) can then be determined as follows:

$$R = \theta E[V_r] = \theta \frac{\Phi}{\zeta} e^{-\zeta S_d} \quad (4.6)$$

in which $\zeta$ ($mm^{-1}$) is the reciprocal of the expected value of rainfall event volume and $\theta$ is the average annual number of rainfall events. Eq.4.6 characterizes the annual runoff quantity from a catchment as a function of rainfall statistics ($\zeta$, $\theta$) and catchment land use characteristics ($\Phi$, $S_d$)

### 4.2.2 Runoff-Overflow Transformation

The probability of overflow volume per rainfall event from a downstream storage and treatment system is derived by considering the change in storage contents as indicated in Figure 4.3. Assuming the storage reservoir at the end of previous event (si, mm) to be full (si=S) and the runoff duration to be approximately equal to that of the rainfall duration, the storage contents are depleted at a controlled release rate $\Omega$ (mm/h) until the present rainfall event arrives at time b (h) (i.e., $b < S/\Omega$ as in Figure 4.3 (a) or $b \geq S/\Omega$ as in Figure 4.32 (b)). The overflow volume (p, mm) is given by

$$p = \Phi(v - S_d) - \Omega t - \Omega b; \quad b < \frac{S}{\Omega} \tag{4.7}$$

in which t is the event duration (h).

The volume of rainfall (v), which causes an overflow volume of p, is then given by

$$v = \frac{p + \Omega t + \Omega b}{\Phi} + S_d \tag{4.8}$$

If the present rainfall event arrives after the storage reservoir is completely empty (i.e., $b \geq S/\Omega$), the overflow volume (P) is given by

$$p = \Phi(v - S_d) - \Omega t - S \; ; \quad b \geq \frac{S}{\Omega} \tag{4.9}$$

The volume of rainfall (v) which causes an overflow volume of, p, is then given by

$$v = \frac{p + \Omega t + S}{\Phi} + S_d \tag{4.10}$$

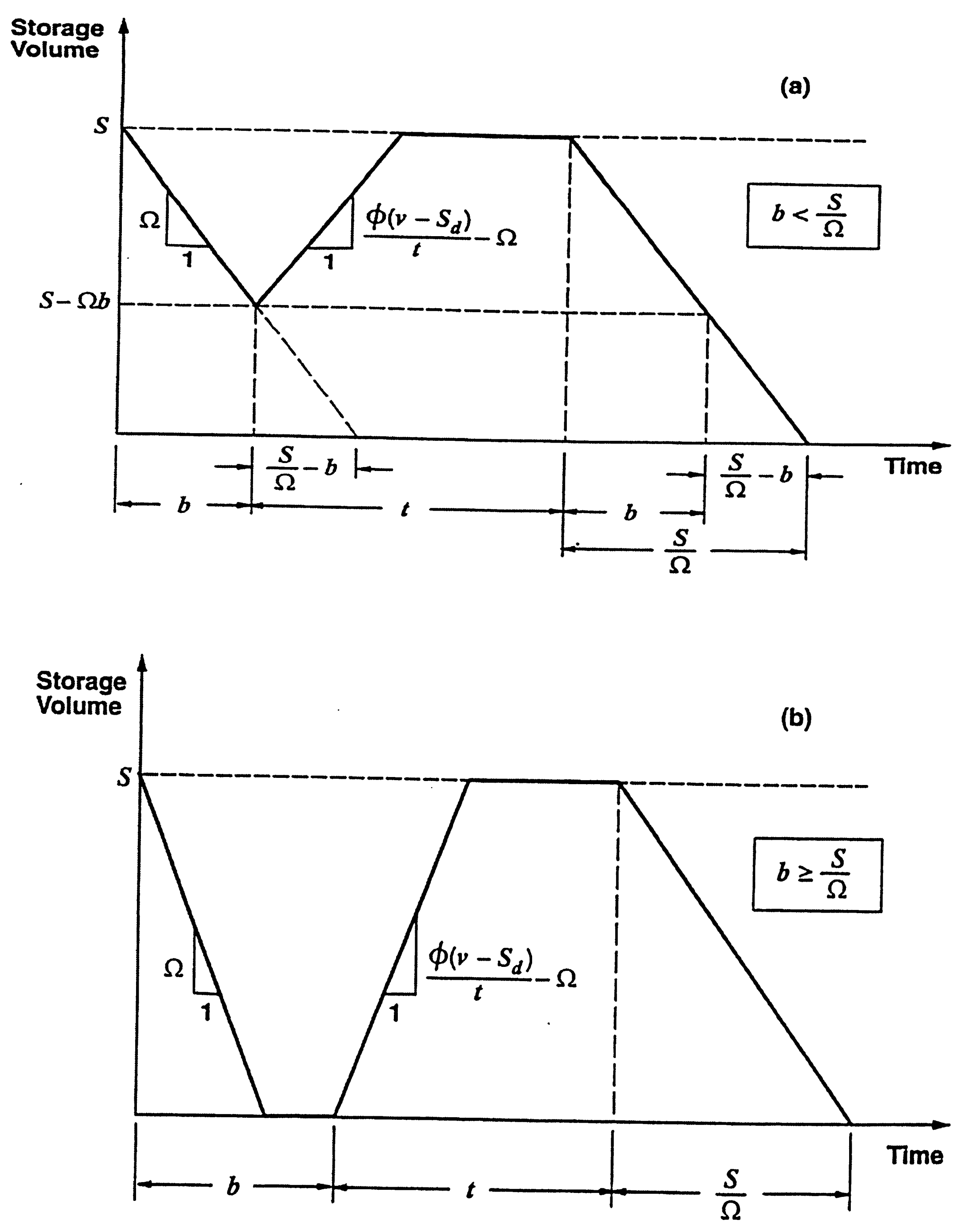



**Figure 4.3 Time Histories of Reservoir Content** (Source: Adams & Papa, 2000)

The overflow conditions can then be summarized as follows:

$$p=\begin{cases}0; & t\ge 0;\ b<\dfrac{S}{\Omega};\ 0\le v\le\dfrac{\Omega t+\Omega b}{\Phi}+S_d\\ \Phi(v-S_d)-\Omega(t+b); & t\ge 0;\ b<\dfrac{S}{\Omega};\quad v>\dfrac{\Omega t+\Omega b}{\Phi}+S_d\\ 0; & t\ge 0;\ b\ge\dfrac{S}{\Omega};\ 0\le v\le\dfrac{\Omega t+S}{\Phi}+S_d\\ \Phi(v-S_d)-\Omega t-S; & t\ge 0;\ b\ge\dfrac{S}{\Omega};\quad v>\dfrac{\Omega t+S}{\Phi}+S_d\end{cases}\tag{4.11}$$

Equation 4.11 is then mapped onto the joint probability distribution space of rainfall volume (v), duration (t), and interevent time (b). The probability per rainfall event of an overflow volume equalling or exceeding some volume p (mm) is denoted as (Gp (p)) and is derived by integrating the joint probability space above the overflow condition surface as follows

$$\begin{aligned}G_p(p)=&\int_{t=0}^{\infty}\int_{b=0}^{\frac{S}{\Omega}}\int_{v=\frac{p+\Omega(t+b)}{\Phi}+S_d}^{\infty} f_{V,B,T}(v,b,t)dtdbdv+\\ &\int_{t=0}^{\infty}\int_{b=\frac{S}{\Omega}}^{\infty}\int_{v=\frac{p+\Omega t+S}{\Phi}+S_d}^{\infty} f_{V,B,T}(v,b,t)dtdbdv\end{aligned}\tag{4.12}$$

Where $f_{V,B,T}(v,b,t)$ is the joint probability density function of rainfall volume, interevent time, and duration. Assuming the variables to be statistically independent, it is given by

$$f_{V,B,T}(v,b,t)=f_V(v)f_B(b)f_T(t)=\lambda\psi\zeta e^{-\lambda t-\psi b-\zeta v}\tag{4.13}$$

Substituting Equation 4.13 into Equation 4.12 and performing integration, the probability per rainfall event of any overflow volume equaling or exceeding a value p, is then given by

$$G_p(p)=\left[\frac{\frac{\lambda}{\Omega}}{\frac{\lambda}{\Omega}+\frac{\zeta}{\Phi}}\right]\left[\frac{\frac{\psi}{\Omega}+\frac{\zeta}{\Phi}e^{-(\frac{\psi}{\Omega}+\frac{\zeta}{\Phi})S}}{\frac{\psi}{\Omega}+\frac{\zeta}{\Phi}}\right]e^{-\frac{\zeta}{\Phi}(p+\Phi S_d}\tag{4.14}$$

in which $\lambda$ ($h^{-1}$) is the reciprocal of the mean rainfall event duration; $\Psi$ ($h^{-1}$) is the reciprocal of the mean interevent time; and $\zeta$ ($mm^{-1}$) is the reciprocal of the mean rainfall event volume.

The probability per rainfall event of any overflow of any magnitude (i.e., p > 0) is then given by

$$G_p(0)=\left[\frac{\frac{\lambda}{\Omega}}{\frac{\lambda}{\Omega}+\frac{\zeta}{\Phi}}\right]\left[\frac{\frac{\psi}{\Omega}+\frac{\zeta}{\Phi}e^{-(\frac{\psi}{\Omega}+\frac{\zeta}{\Phi})S}}{\frac{\psi}{\Omega}+\frac{\zeta}{\Phi}}\right]e^{-\zeta S_d}\tag{4.15}$$

The probability density function of overflow volume per rainfall event is then derived by differentiating its cumulative density function (cdf), which is given by:

$$f_p(p)=\begin{cases}1-G_p(0); & \mathrm{p}=0\\ \frac{1}{\Phi}G_p(0)\zeta e^{-\zeta\frac{p}{\Phi}}; & \mathrm{p}>0\end{cases}\tag{4.16}$$

The expected magnitude of overflow per rainfall event (E [P], mm) is given by

$$E[P]=\int_{p=0}^{\infty} pf_p(p)dp=\frac{\Phi}{\zeta}G_p(0)\tag{4.17}$$

Therefore the average annual uncontrolled spill volume, $P_u$ is given by

$$P_u=\theta E[P]=\theta\frac{\Phi}{\zeta}G_p(0)\tag{4.18}$$

Long term average system performance measures *Cr* and *Ns* are then given by

$$C_r = \left[1 - \frac{\theta E[P]}{R}\right] \times 100\% = \left[1 - G_p(0)e^{\zeta S_d}\right] \times 100\% \tag{4.19}$$

$$N_s = \theta G_p(0)$$

Both *Cr* and *Ns* are functions of the rainfall parameters ($\lambda$, $\Psi$, $\zeta$, $\theta$), the catchment hydrologic parameters ($\Phi$, $S_d$) and the control system variables ($S$ and $\Omega$).

### 4.2.3 Transformation of Runoff Volume to Runoff Pollution Mass Load

The runoff event pollution load ($L_r$, mass per unit area) is the product of runoff event volume ($V_r$) and the event flow-weighted mean concentration (C, mass per unit volume) given by

$$L_r = V_r * C \tag{4.20}$$

and its expected value by

$$E[L_r] = E[C] * E[V_r] + COV[V_r, C] \tag{4.21}$$

in which E [C] is the expected value of C per rainfall event; E [V] is the expected value of V, per rainfall event; and COV [$V_r$, C] is the covariance of V, and C. If $V_r$ and C are independent, their covariance is zero. According to the U.S. Nationwide Urban Runoff Program (U.S. EPA, 1983), C was found to be generally uncorrelated with $V_r$ other analyses by Wallace (1980) also indicate that the correlation between pollutant concentration and runoff volume is weak. Therefore, the expected event runoff load (E [Lr]) may be approximated by

$$E[L_r] = E[C] * E[V_r] \tag{4.22}$$

And the average annual runoff pollution load (Lr) can be estimated by

$$L_R = \theta * E[L_r] \tag{4.23}$$

The event mean concentration approach described above requires the determination of the expected event concentration, E [C). If no runoff quality data are available for a catchment, E [C] may be selected from the literature (e.g., U.S. EPA 1983; Driver and Lystrom 1986). However, it is important that the expected event concentration be selected in relation to land use characteristics, geographical location, hydrology and drainage system characteristics.

### 4.2.4 Transformation of Runoff Pollution Mass Load to the Total Pollution Mass Discharge Load

In order to determine the total runoff pollution load discharged to the receiving water, the water quality changes through the runoff control systems (e.g. storage and treatment facilities) must be consider in addition to the runoff hydraulics. It is assumed that the runoff have a uniform pollutant concentration equal to the event mean concentration (EMC) is conveyed from the catchment and routed through storage to utilize the treatment efficiency of storage facility ($\eta_S$) as shown schematically in Figure 4.4. The runoff that is processed at the storage facility's controlled release rate ($\Omega$) may receive additional treatment from a treatment plant or overflow treatment

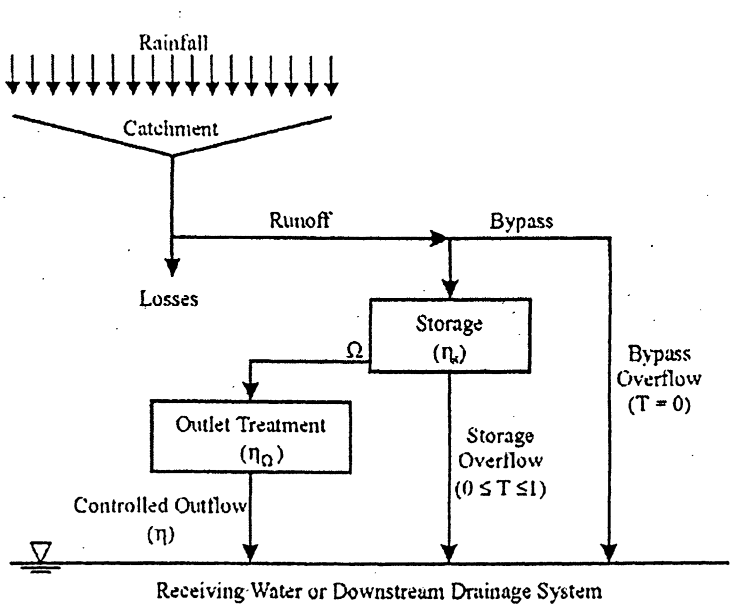



**Figure 4.4 Urban Runoff Storage/Treatment Systems** (Source: Adams & Papa, 2000)

device, which operates at or below its capacity, treating runoff at an efficiency $\eta_\Omega$. The combined pollutant removal efficiency is then given by:

$$\eta = \eta_S + \eta_\Omega (1 - \eta_S) \qquad (4.24)$$

When the storage reservoir is full, the outlet is operating at its capacity ($\Omega$) and the rate of inflow (runoff intensity) to the reservoir exceeds the outlet capacity and overflow occurs. Spills may be routed in one of two ways:

- They may be processed through the storage device and may receive a fraction (T) of the removal efficiency offered by the storage facility or
- They may be bypassed upstream of the storage facility, thus receiving no treatment (T=0), and discharged directly to the receiving water.

The factor T reflects the possibility that spills may receive some but not completes treatment in storage and can therefore take on values in the range $0 \leq T \leq 1$. Considering the average annual runoff volume (R) in Equation 4.6 and average annual spill volume ($P_u$) in Equation 4.18, the long term annual fraction of pollution controlled by the system is then given by:

$$C_P = [\,(R - P_u) * \eta + P_u * T * \eta_S] / R. \qquad (4.25)$$

In above equation no allowance is made for the possible loss of efficiency during runoff conditions or for the possible gain in efficiency from the more continuous operation of the treatment facility through the use of storage. Thus the efficiency of the treatment facility is assumed constant.

# CHAPTER FIVE

## CASE STUDY- THE CITY OF NIAGARA FALLS HRT PILOT PROJECT

This chapter will discuss the purpose of the High Rate Treatment (HRT) pilot project of the city of Niagara Falls with brief description of the existing sewer system of the study area.

### 5.1 Background

The City of Niagara Falls has commissioned a series of studies to investigate cost-effective technologies for controlling the Muddy Run combined sewer overflows from the Central Pumping Station service area to the Niagara River. One of the preferred options is the High -Rate Treatment (HRT) facility by the application of vortex separator near Muddy Run trunk. The City of Niagara Falls (NF), Ontario Great Lakes Renewal Foundation (GLRF), Government of Canada's Great Lakes Sustainability Fund (GLSF), Ontario Ministry of Environment (MOE), National Water Research Institute (NWRI), Ryerson University (RU), and the Regional Municipality of Niagara (RMN) are in the process of implementing a High-Rate Treatment Pilot study to evaluate the performance of two commonly available treatment technologies namely Storm King Vortex Separator and Continuous Deflective Separation (CDS). The loading condition for each HRT device is limited to a maximum 140 L/s and the local influent characteristics. The objectives of verification testing are to determine the (Li, 2004):

- Performance of each HRT device relative to the manufacturer's stated range of equipment capabilities;
- Range of operating conditions and the ease of operation of the equipment;
- Impact of influent characteristics on the performance of the equipment;
- Impact of the equipment operating cycle and operations and maintenance performance.

**5.2 Description of Existing Sewer System**

In 1996, CH2M Gore & Storrie Limited examines the existing combined and stormwater sewer system in Niagara Falls as part of the sewer system analysis and CSO abatement study for the City of Niagara Falls. As per their study, Figure 5.1 schematically shows the existing sewer system contributing the Muddy Run pumping station and Central pumping station service area. The Ontario Street Trunk and the Muddy Run Trunk intercept wastewater flows collected by the municipal collection system. These flows are then conveyed by gravity to the Central Pumping Station. The Central Pumping Station also receives pumped flows from Muddy Run Pumping

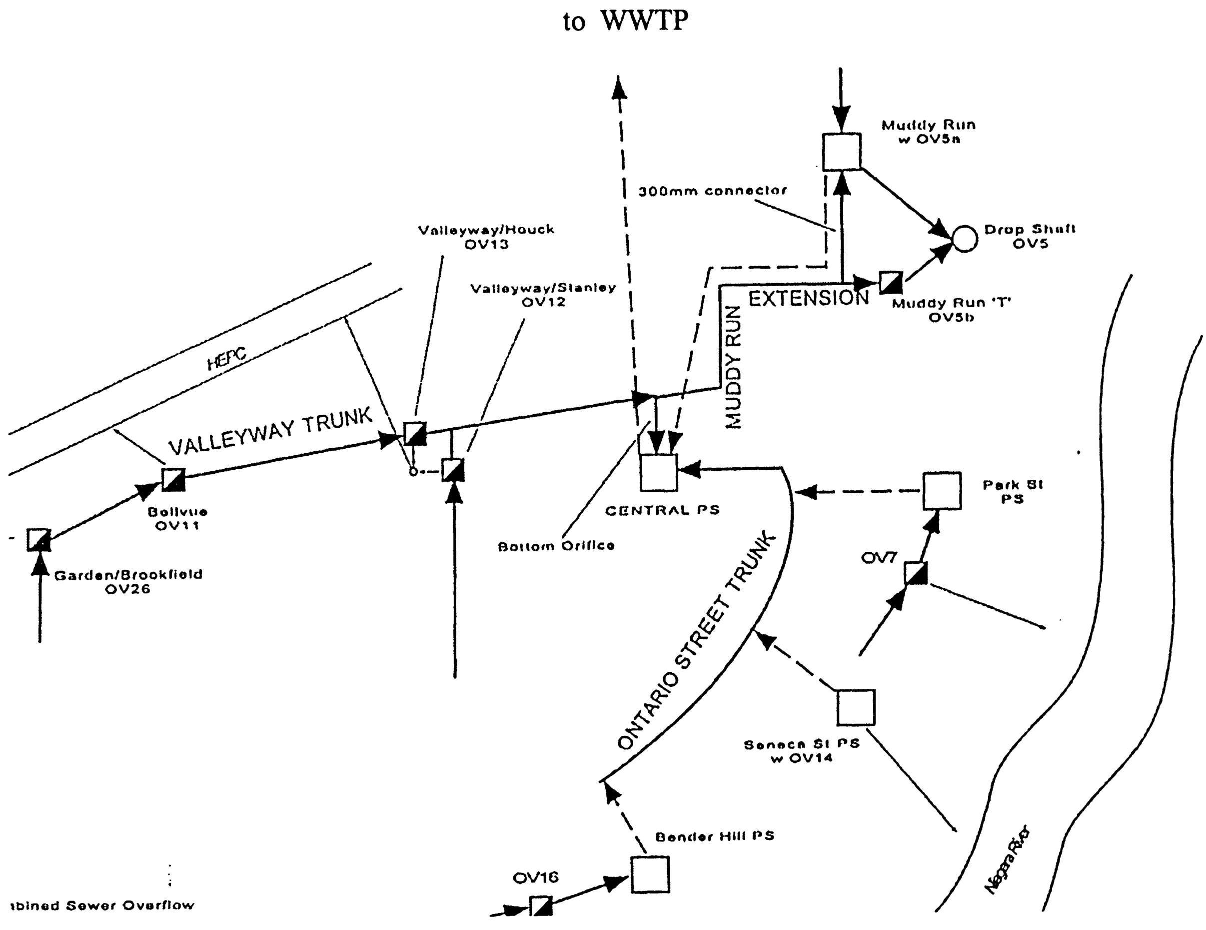



**Figure 5.1 Schematic View of the Existing Sewer System of Central Pumping area**

(Source: CH2M Gore & Storrie Limited, 1996)

Station via a forcemain located within the Muddy Run Trunk Sewer. The Bender Hill, Seneca Street and Park Street Pumping Stations pump flows to the Ontario Street trunk Sewer. These pumping stations serve as the terminal point of all flows originating within this area. The forcemain from central Pumping station discharges directly to the Stanley Avenue treatment plant.

### 5.2.1 Ontario Street Trunk

The Ontario Street Trunk Sewer flows in a northeasterly direction parallel to the Niagara River. It accepts flows from pumping stations located at Bender Hill, Seneca Street, and Park Street, and terminates at the Central pumping station (CG&S, 1996). Overflows exist at the Bender Hill pumping station, Seneca pumping station, and Park Street pumping station, each of which discharges to the Niagara River through drop shafts. Most of the area serviced by this network of pumps and sewers has been separated by the City, with the exception of the downtown core and Epworth Circle/Eastwood Circle.

### 5.2.2 Valleyway Trunk/Muddy Run Extension

The valleyway trunk extends from Brookfield Avenue and terminates at the Central pumping station via a 380 mm diameter bottom orifice. Flows in the Valleyway trunk that exceeds the capacity of the orifice bypass the station, and enter the Muddy Run Extension directly. Flows in excess of the pumping capacity also flow back through the orifice into the Muddy Run Extension, which continues to an overflow chamber immediately upstream of the Muddy Run pumping station. A 300 mm diameter pipe in the chamber's invert diverts flow to the Muddy Run pumping station to be pumped back to the Central pumping station. An overflow pipe at the crown of the 300 mm pipe in the chamber leads to a drop shaft. Overflow from the Muddy Run pumping station also enters this drop shaft which ultimately discharges to the Niagara River. Along the Valleyway Trunk sewer, there are four CSOs: Garden Avenue/Brookfield Avenue, Bellevue Street, Houck Park, and Valleyway / Stanley Avenue, which all discharge to the Hydro Electric Power Commission (HEPC) canal.

### 5.2.3 Central Pumping Station

The central pumping Station is located on the north side of Park street near the intersection of Park street and Ontario Avenue. The Central Pump Station services approximately 30% of the Niagara Falls urban area. The contributing sewer shed is largely combined and covers areas West of and including the downtown core. Flows at the Central Pumping station are conveyed for approximately 2600 m via a 900 mm diameter forcemain and for 175 –via a 900 mm diameter gravity sewer to the Stanley Avenue WPCP (CH2MHILL, 2001). The Central Pumping station consists of three pumps. Firm capacity refers to the maximum flows that can be pumped if one of the three pumps is not in service i.e. the third pump is maintained for standby purposes. Capacity testing of the pumps indicated that the combined firm capacities for two pumps ranged from 700 to 871 L/s. With three pumps running together, the capacity increased marginally to 933 L/s. Although the station has capacity to pump up to 933 L/s, the size of the inlet orifice to the station from the Muddy Run trunk will limit the amount of flow it can receive. Flows in the Muddy Run Trunk that exceed the capacity of the orifice (i.e. approximately 600 L/s) will bypass the station, and enter the Muddy run trunk directly (CH2MHILL, 2001).

### 5.2.4 Muddy Run Pumping Station

The Muddy Run pumping station is located underground in the road allowance on River Road, just north of Buttrey Street. The station was constructed in 1963, and new pumps were installed in 1998. There are two constant speed dry pit pumps in the station: each with a capacity of approximately 22 L/s. The combined capacities of both pumps are measured at 36.3 L/s. The muddy Run pumping station receives flow by gravity from the residential area north of Buttrey Street via two separate sewers. By pass and overflows from the Central pumping station are conveyed to an orifice immediately upstream of the Muddy Run station via the Muddy Run Extension. A small portion of the flows in the Orifice are directed to the Muddy Run pumping station and pumped back to the Central pumping station through a 200 mm forcemain located within the Muddy Run trunk. The remainder of the flow will overflow through a drop shaft to the Niagara River. Overflow from the Muddy Run pumping station also enters this drop shaft and discharges to the Niagara River (CH2MHILL, 2001).

### 5.2.5 Muddy Run CSO

The Muddy Run CSO is one of the major overflows from the City's sanitary sewer system, contributing close to 60% of the total overflow from the city. The frequency and volume of combined sewer overflows at the Muddy Run CSO and at the other river road pumping stations are presented in Table 5.1. These frequencies and volumes were predicted using STORM model simulations based on an average rainfall year of 1976.

**Table 5.1 Overflow Frequency and Volume at Different Overflow Stations***

| Overflow Location | Frequency | Volume ($m^3$) |
|---|---|---|
| Bender Hill Pumping Station | 3 | 1,444 |
| Seneca St. Pumping station | 20 | 3,852 |
| Park St. Pumping Station | 24 | 6,696 |
| Garden/Brookfield | 3 | 338 |
| Bellevue St. | 12 | 1,998 |
| Valleyway/Houck | 12 | 14,501 |
| Valleyway/Stanley | | |
| Muddy Run CSO | | |
| Bypass and overflow from Central Pumping Station | 34 | 534,949 |
| Overflow from Muddy Run Pumping station service area | 34 | 42,890 |

* Source: Sewer System Analysis and CSO Abatement Study (CH2M Gore & Storrie Ltd, 1996)

As illustrated in Table 5.1, bypassing and overflowing at the Central Pumping Station has the largest impact on the overflows occurring at the Muddy Run CSO. Therefore, solving this overflow problem is the major concern of the City of Niagara Falls.

### 5.3 Description of the Pilot Study Site

The pilot study site is approximately 1500 square meters in area and is bound by Niagara Transit to the East and North and various commercial uses to the West and South. Figure 5.2 and figure 5.3 illustrate its position and size relative to the properties surrounding it.

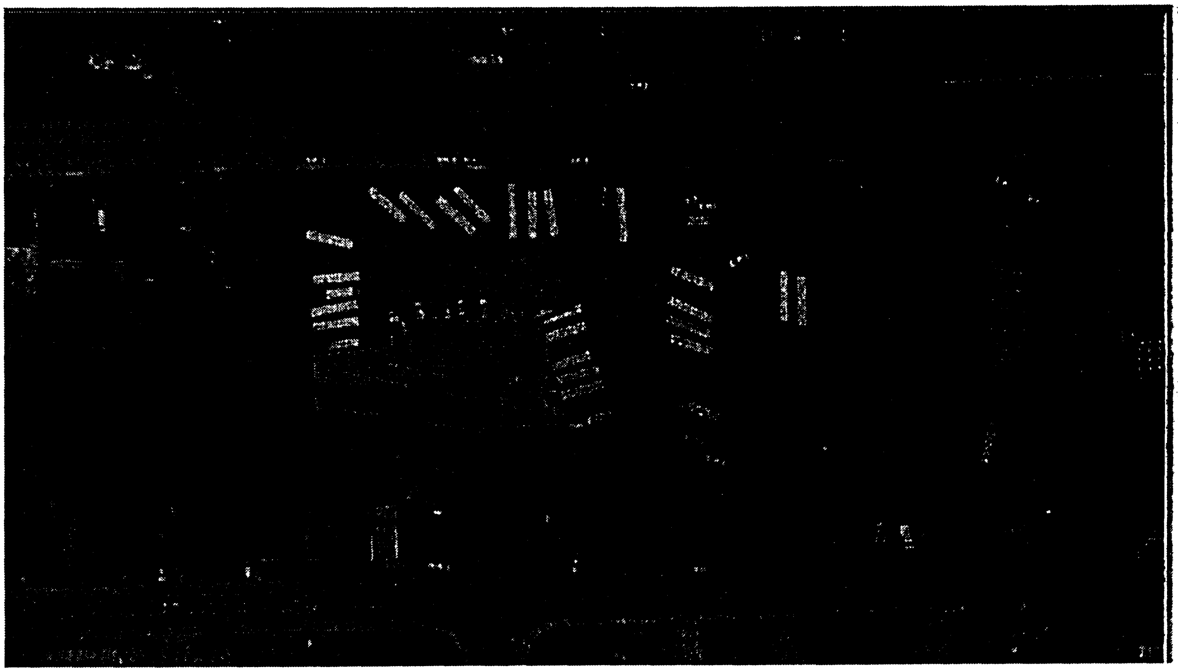


**Figure 5.2 Aerial View of the Pilot Testing Site** (Source: Li, 2004)

It is located at the site of the existing Central Pump Station, which is slated for removal in 2006/2007 when the full scale High Rate Treatment Facility and new Central Pump Station come on line. The existing Central Pump Station will be relocated to a vacant property approximately 400 meters to the north at the new High Rate Treatment Facility. The lay out and photographs of the testing site are shown in the following Figures.

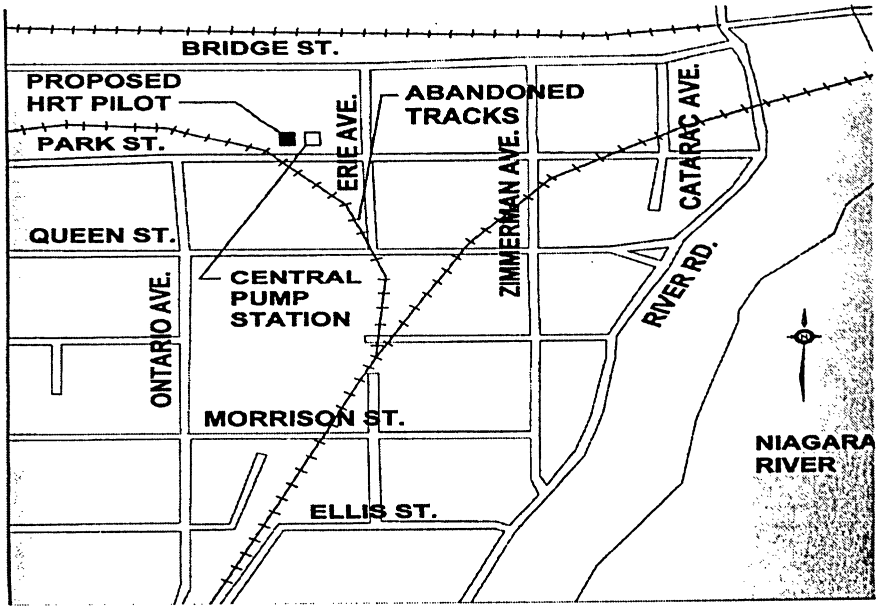



**Figure 5.3 Schematic View of the Pilot HRT Project Site** (Source: City of Niagara Falls High Rate Treatment Plan, 2004)

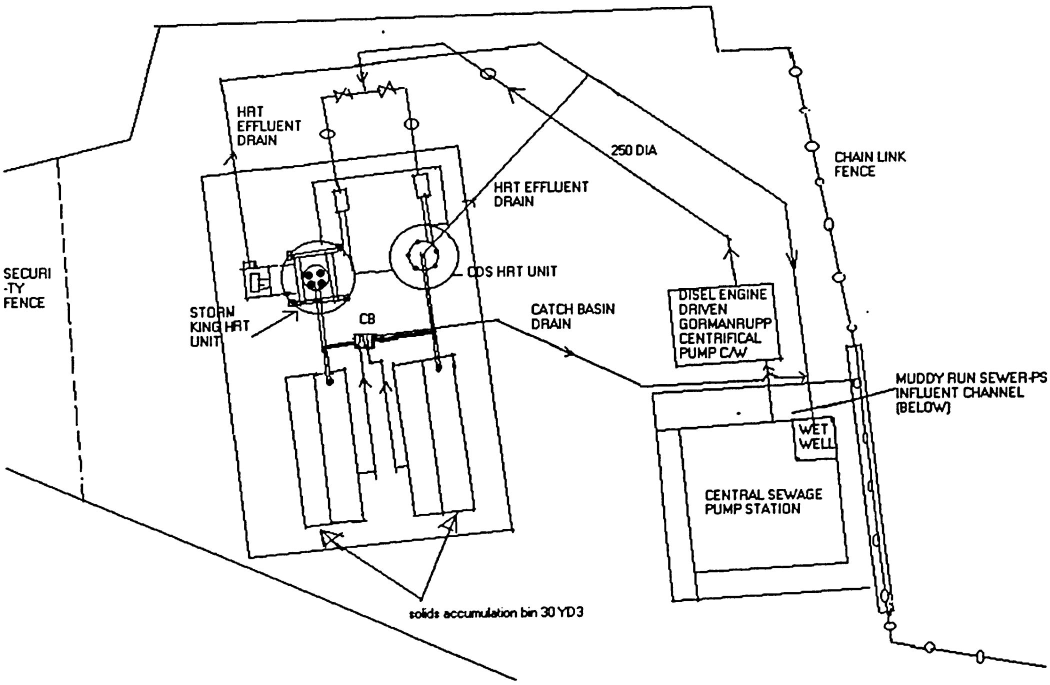



**Figure 5.4 Lay Out Plan of Test Site**

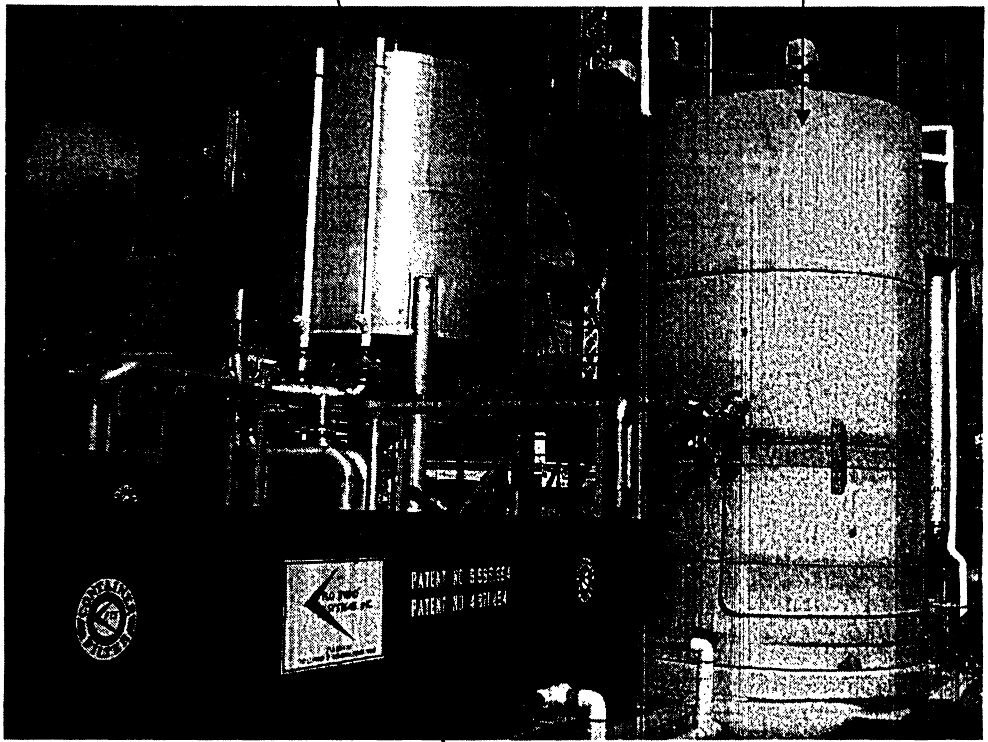



**Figure 5.5 Photographs of CDS and Storm King Unit at Test Site**

# CHAPTER SIX

## MONITORING

This chapter will discuss the approach and methodology of this study including field monitoring program. Each of these task components is summarized in the next few sections of this chapter.

### 6.1 Methodology

The performance evaluation of Storm King and CDS unit to be conducted by comparing both field performance as well as laboratory test result. The CDS system selected for this verification project is the Model PSWC40_30_8 as illustrated in Figures 6.1 and 6.2. The Storm King$^{TM}$ model selected for this verification project is the 3.4 m diameter unit as shown in Figures 6.3 and 6.4.

Influent and effluent composite samples of each HRT devices to be collected and analyzed for Total Suspended Solids (TSS), Volatile Suspended Solids (VSS), Settleable solids, Particle size distribution, BOD5 and COD (soluble and total), Heavy metals (such as zinc, copper, lead, aluminum, chromium), Nutrients, and TKN, and Bacteria such as total coliform and E. Coli by the laboratory services at the CCIW. As the settleability of the influent solids is the most important parameter in the design of HRT, the influent solids collected at the pilot testing site will be analyzed by NWRI using the US EPA Long Column, Bombart Column, Aston Column, and Elutriatiom Column. The underflow solids will be collected by RV Anderson Associates Ltd. and analyzed by the laboratory services at the CCIW. Atmospheric Environment Service of Environment Canada (AES) station identification number for the study area is 6135638. The rainfall parameters statistics for this station and catchment characteristics for this area is collected and analyzed for existing system using Analytical Probabilistic Model. Upon receipt the monitored data and laboratory test results for influent and effluent, the pollutant removal efficiency as well as pollutant concentration in CSO after provide treatment for each device to be determined.

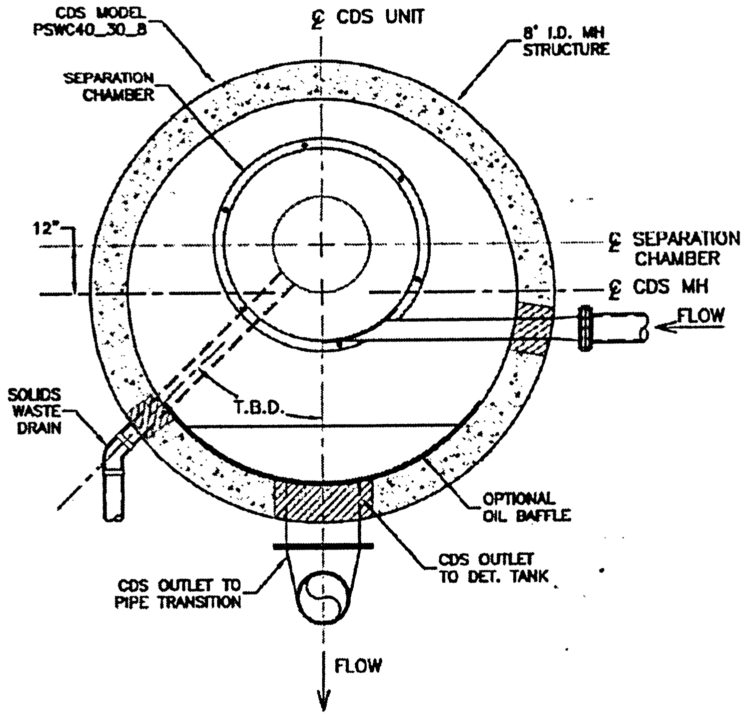



**Figure 6.1 Plan View of CDS Model PSWC40_30_8** (Source: Li, 2004)

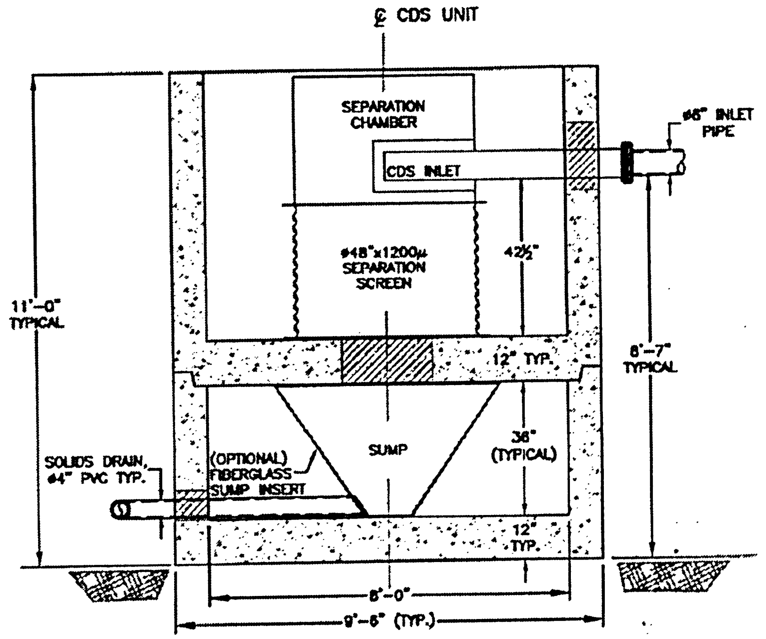



**Figure 6.2 Elevation View of a CDS Model PSWC40_30_8** (Source: Li, 2004)

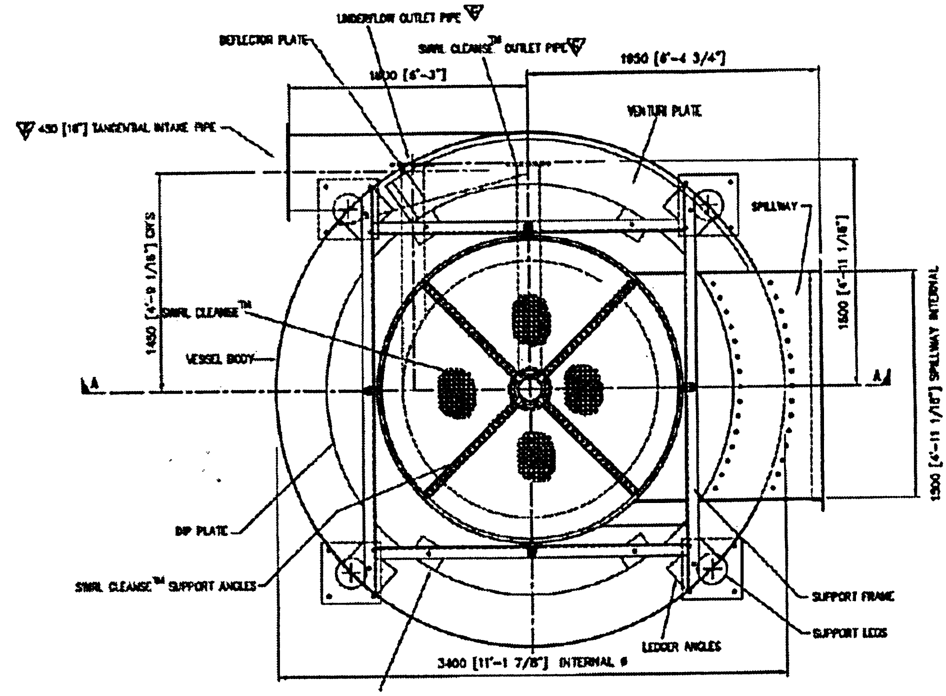


**Figure 6.3 Plan View of a 3.4 m Diameter Storm King Unit** (Source: Li, 2004)

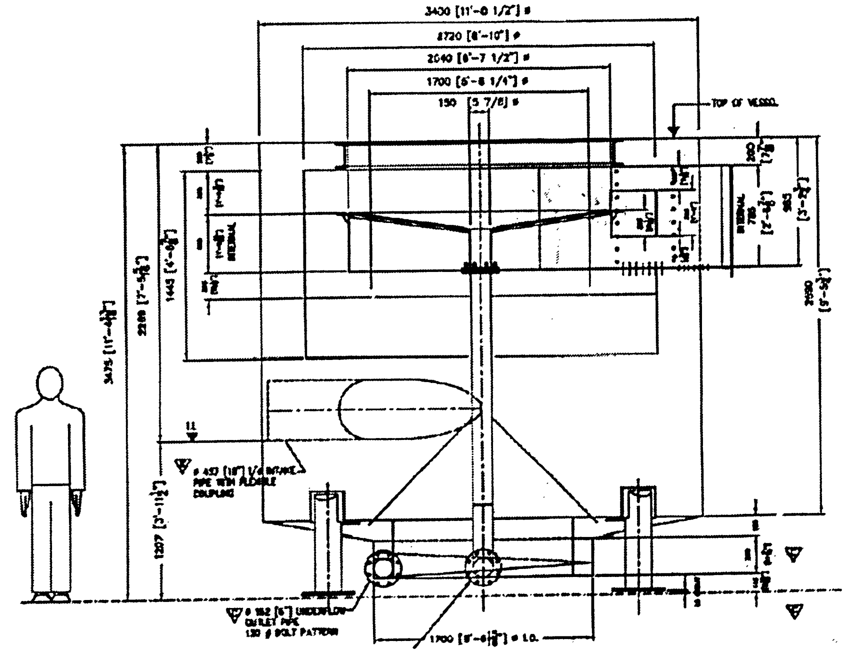


**Figure 6.4 Elevation View of a 3.4 m Diameter Storm King Unit** (Source: Li, 2004)

## 6.2 Field Operation

Figure 6.5 show the schematic diagram of the monitoring flow diagram & equipment of the system. A 10 inch diesel engine centrifugal pump, with a maximum capacity of 140 L/s, is installed to pumping the flow from the Muddy Run sewer when the wet well level rises. The flow will split into two 8 inch forcemains where equal flow will be attained by adjusting the control valves. Magmeters is used to measure the total pumped flow and the flow in each line. The flow is transported by each 8 inch forcemain to a sampling box where three 3/4 inch intake pipes is used to capture a total of 100 L of influent water samples. As the elevation of the sample box is about 10 ft high, there is enough head to allow water samples to be collected at the ground level. The intake pipes is fitted with time-controlled valves so time-weighted composite samples can be collected. Effluent water samples are taken at the outlet of each device using automatic wastewater samplers. The underflow from each device will be diverted to a sediment filter box as illustrated in Figure 6.6. When the filter box is filled up, extra underflow will bypass the box, merge with the treated effluent and return back to the Muddy Run sewer. After the sediment box has been drained, the remaining sediments will be weighted manually using a scale and 4 sediment samples will be collected for gradation and chemical analyses. A tipping bucket rain gauge is set up at the roof of the existing Central Pumping Station to measure rainfall.

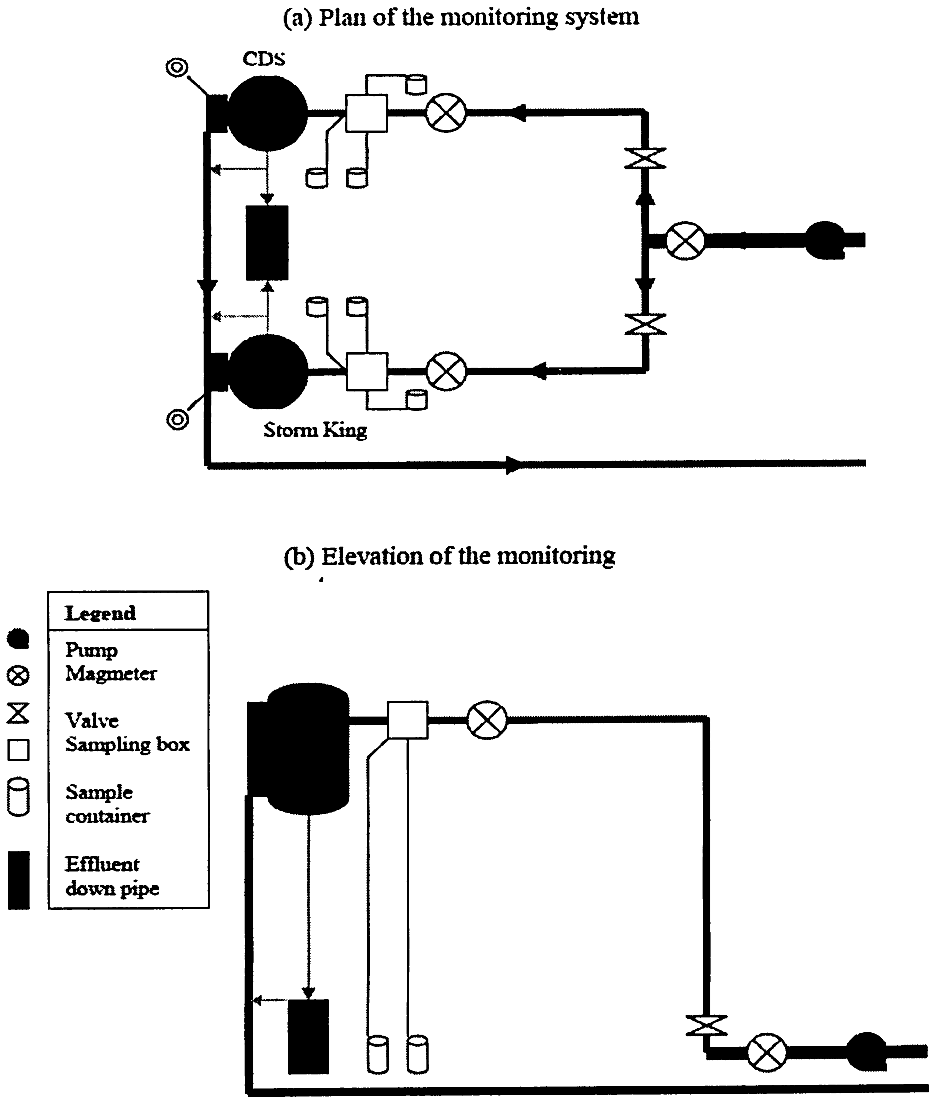
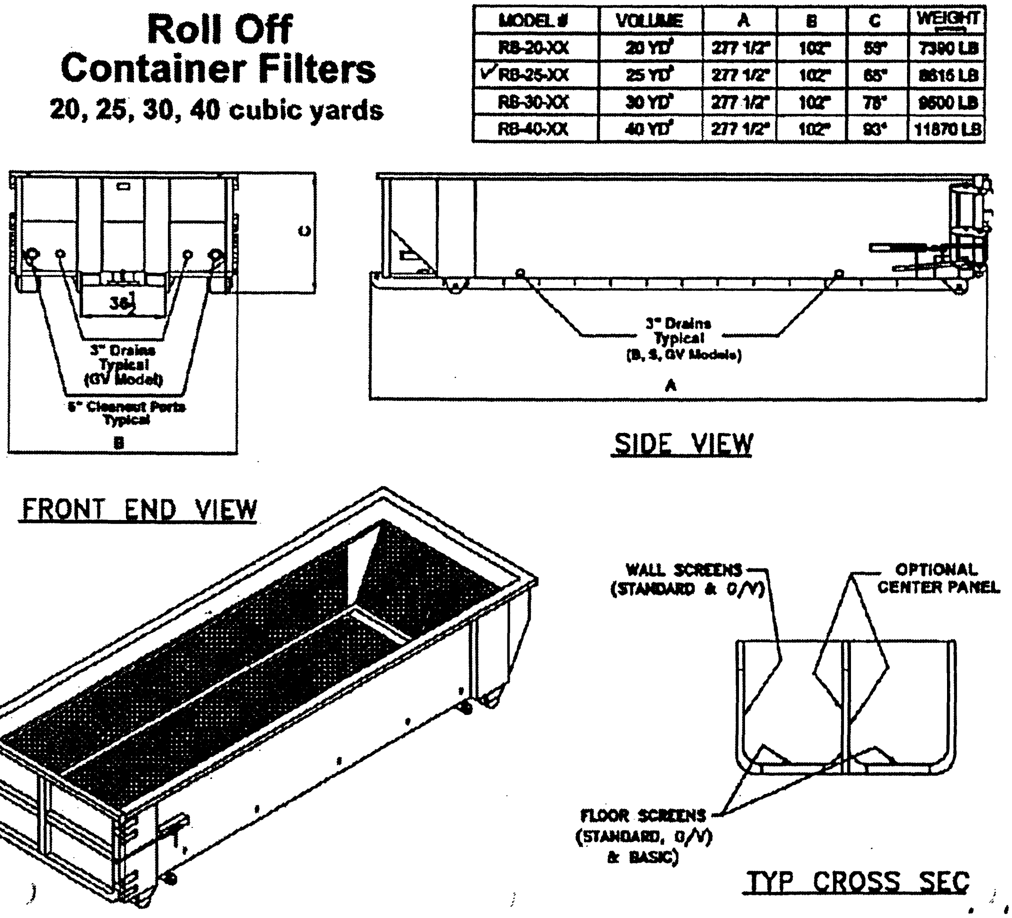



**Figure 6.5 Flow Diagram of the Monitoring System** (Source: Li, 2004)




| MODEL # | VOLUME | A | B | C | WEIGHT |
|---|---|---|---|---|---|
| RB-20-XX | 20 YD³ | 277 1/2" | 102" | 59" | 7390 LB |
| ✓RB-25-XX | 25 YD³ | 277 1/2" | 102" | 65" | 8616 LB |
| RB-30-XX | 30 YD³ | 277 1/2" | 102" | 78" | 9600 LB |
| RB-40-XX | 40 YD³ | 277 1/2" | 102" | 93" | 11870 LB |

**Figure 6.6 A Sediment Filter Box for Underflow of HRT** (Source: Li, 2004)

# CHAPTER SEVEN

## WATER POLLUTION IMPACT ANALYSIS FOR HRT

The Muddy Run CSO is one of the major overflows from the City's sanitary sewer system, contributing close to 60% of the total overflow from the city (CH2MHILL, 2001). As mentioned earlier, this CSO consists of the Muddy Run pumping station overflows and overflows from Central pumping station, which is ultimately discharged through a drop shaft near Muddy Run pumping station to the Niagara River. The main objective of the HRT facility is to control the CSO problem at this location. The HRT pilot study project tests the Storm King™ and CDS™ devices. The Storm King's manual claims that the TSS control performance varies within 50% to 80% while the CDS' manual claims that the TSS control varies within 74% to 84%. On the basis of their claims, the predicted overall TSS concentration and loading in CSO after treatment are calculated in this chapter.

To calculate the CSOs volume, the Analytical Probabilistic Model (APM) is used. The APM is first calibrated against the overflows simulated by the STORM model for future condition as described in the "City of Niagara Falls Sewer System Analysis and CSO Abatement Study" prepared by CH2M Gore & Storrie Limited (1996). The calibrated model is then used to calculate the overflow volumes that are presented in this chapter. In the following section, the CSO assessment conducted by CH2M Gore & Storrie Limited is briefly discussed.

### 7.1 CSO Assessment by CH2M Gore & Storrie Limited

CH2M Gore & Storrie Limited (1996) conducted an extensive study on the existing sewer system of the City of Niagara Falls and their service area as a part of the "City of Niagara Falls Sewer System Analysis and CSO Abatement Study." One of the objectives of this study was to evaluate the frequency and volume of combined sewer overflows and to assess the capacity of the collection system in conveying dry and wet weather flows under existing and future development conditions. Two computer models were applied to analyze the combined/sanitary sewer system and evaluate different control alternatives: U.S. Corps of Engineers Storage, Treatment, Overflow, Runoff Model (STORM, 1997) and U.S. EPA's Stormwater Management Model (SWMM). To evaluate inflow and infiltration (I/I) volumes STORM model was used to generate hydrographs from actual precipitation data recorded at Niagara Falls (the Stanley Avenue Climatological Station # 6135638). From these I/I estimates, STORM was then used to determine the combined sewer overflow quantities and frequencies discharged to the City's receiving waters. XP-SWMM EXTRAN, a more complex model, was used as well to investigate the effects of wet weather I/I flow in Niagara Falls main sanitary sewer trunks, at combined sewer overflow structures, as a result of pumping and backwater effects, and in-line storage within the system, and to assess the capacity of the collection system. XP-SWMM EXTRAN was also used to confirm the STORM model predictions.

The study area was divided into subcatchments as shown in Figure 7.1 (partial) that were described in terms of area, land use, weighted imperviousness and depression storage, infiltration rate, evaporation data, dry weather flow, and through flow capacity. The STORM model applied an hourly record of precipitation, which might extend over a number of years, to each subcatchment. The rainfall applied was transformed to wet weather I/I of the subcatchment by a volumetric runoff coefficient. The system's sewer capacity was represented by a treatment rate in the model for this wet weather I/I. Flows in excess of this sewer capacity might be stored for later recovery, or considered as surcharge or overflows from the system, depending on the magnitude and length of overflow. The study assumed that the overflow capacity at each pumping station is the maximum capacity determined by the Regional Municipality of Niagara in 1992 with actual drawdown tests in the field. The maximum capacity was defined as the maximum tested capacity when all pumps were running. At a combined sewer overflow, the capacity of the overflow or the capacity of the downstream pipe, whichever was smaller, was used as the overflow capacity.

STORM was modified to enable the linking of subcatchment areas by accepting the throughput hydrograph from an upstream subcatchment and adding it to the hydrograph generated downstream. To define the STORM CSO model, multiple sanitary sewersheds were combined into a single model node. These nodes were linked together by conduits of known diameters and slopes that have a maximum flow capacity. Where overflow structures exist, this capacity was limited to the maximum rate of flow out of the structure before an overflow occurs.

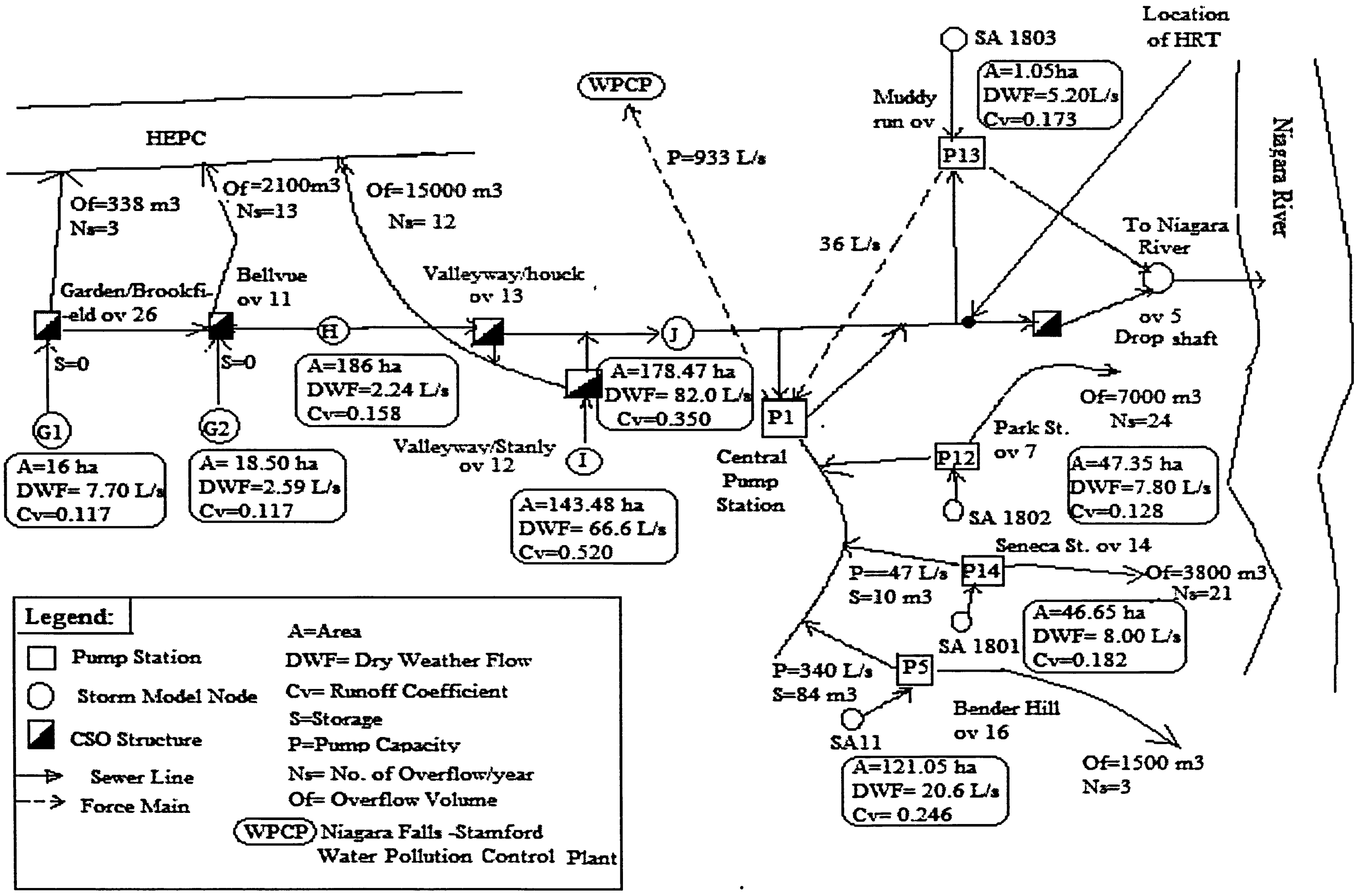



Figure 7.1 Schemetic Diagram of Sewer System with Overflow Locations at the Study Area

The STORM model was first calibrated to overflow occurrences in 1991 and 1992. The calibrated model was then used to analyze the performance of the existing CSOs and collection system and modified to evaluate the effects of future land development on the pattern and severity of CSOs. Final STORM model input parameters and outputs for future conditions are presented in Table 7.1.

**Table 7.1 STORM model input and output data ***

| Location | Input parameters | | | | | Outputs | |
|---|---|---|---|---|---|---|---|
| | Area (ha) | DWF (L/s) | Runoff Coefficient | Diversion Capacity (L/s) | Storage Volume ($m^3$) | Overflow volume ($m^3$) | No. of CSO events |
| SA11/ Bender Hill PS | 98.0 | 24.4 | 0.12 | 340 | 84 | 1501 | 3 |
| SA1801/Seneca St. PS | 46.65 | 9.61 | 0.09 | 47 | 10 | 3800 | 21 |
| SA1802/Park St. PS | 46.35 | 9.38 | 0.13 | 50 | 30 | 7000 | 24 |
| G1/Garden-Brookfield | 16 | 9.17 | 0.12 | 53 | 0 | 338 | 3 |
| G2/ Bellevue | 18.5 | 3.06 | 0.12 | 65 | 0 | 2100 | 13 |
| H | 186.68 | 26.6 | 0.16 | - | - | - | - |
| I | 143.48 | 79.10 | 0.52 | - | - | - | - |
| Valleyway-Houck/ Valleyway-Stanley | - | - | - | 496 | 0 | 15000 | 12 |
| SA1803/ Muddy Run Service area | 51.05 | 5.20 | 0.17 | 36 | 0 | 44366 | 34 |
| Bypass and overflow from Central Pumping station/Muddy Run Extension | - | - | - | 933 | - | 555279 | 34 |

* Source: CH2M Gore & Storrie Limited. 1996

## 7.2 Data Collection for Analytical Probabilistic Model

### 7.2.1 Rainfall Parameters Data

The rainfall parameters of the City of Niagara falls (AES station ID 6135638) are collected for the year of 1965-83 for March to November with IETD = 1 hr and summarized in Table 7.2 (Source: Adams, 2000 )

**Table 7.2 Rainfall Parameters of Niagara Falls Region**

| Parameter | Value |
|---|---|
| Rainfall Volume , v | 4.18 mm |
| Reciprocal of rainfall volume, $\zeta$ | 0.24 $mm^{-1}$ |
| Rainfall intensity, i | 1.37 mm/hr |
| Reciprocal of rainfall intensity, $\beta$ | 0.73 hr/mm |
| Rainfall duration, t | 2.68 hr |
| Reciprocal of rainfall duration, $\lambda$ | 0.37 $hr^{-1}$ |
| Inter event time, b | 46.3 hr |
| Reciprocal of interevent time, $\psi$ | 0.022 $hr^{-1}$ |
| Average annual number of event, $\theta$ | 136 |

### 7.2.2 Catchment Parameters Data

The catchment parameters required for the Analytical Probabilistic model are: catchment area, runoff coefficient, depression storage, dry weather flow, existing storage capacity and diversion capacity. The values of these parameters are taken from the previous study conducted by CH2M Gore & Storrie Limited as presented in Table 7.1.

## 7.3 Overflow Analysis Using Analytical Probabilistic Model

There are nine overflow locations in the study area as shown in Figure 7.1. They are: Bender Hill PS (OV #16), Seneca St. PS (OV # 14), Park St. PS (OV # 7), Garden/Brookfield (OV #26), Bellevue (OV #11), Valleyway/ Stanley (OV # 12), Valleyway/Houck (OV # 13), Muddy Run T (OV# 5b) and Muddy Run PS (OV # 5a). It is mentioned here that the Valleyway/Houck and Valleyway/ Sytanley overflow from a common shaft are discharged into HEPC canal. Similarly the Muddy Run PS overflow and the Muddy Run Extension (downstream of Central PS, OV # 5b) overflow are discharged into Niagara River from a common drop shaft. The overflow volume at each overflow locations and their contributing catchment area characteristics are presented in Table 7.1. At each overflow location, the analytical probabilistic model is calibrated with overflow number ($N_s$) by varying depression storage ($S_d$). Using these calibrated values for depression storage, the total runoff volume (R) and overflow volume ($P_u$) at each overflow locations are then calculated from the model. The outputs from the analytical probabilistic model are presented in Table 7.3.

**Table 7.3 Analytical Probabilistic Model's Output**

| Overflow Location | Overflow Volume ($m^3$) |
|---|---|
| Bender Hill PS (OV #16) | 1500 |
| Seneca St. PS (OV #14) | 3,800 |
| Park St.. PS (OV # 7) | 6,080 |
| Garden/Brookfield (OV # 26) | 260 |
| Bellevue (OV # 11) | 2,370 |
| Valleyway/Houck (OV #13) + Valleyway/Stanley (OV # 12) | 124,340 |
| Muddy Run PS (OV# 5a) | 12,400 |
| Muddy Run Extension (OV #5b) | 567,670 |

As shown in Table 7.4, the overflow volumes calculated from APM and that from STORM model (conducted by CG&S, 1996) are generally in good agreement, except at the Valleyway overflow and at the Muddy Run PS. The main purpose of the HRT facilities is to control the overflow quality for Muddy Run extension (bypass and overflow from Central Pumping Station). Since the overflow volume (567,670 $m^3$) at this location is reasonably matched with that (555,279 $m^3$) simulated by CG & S (1996), the overflow volume predicted by the APM is used in the subsequent sections.

**Table 7.4 Overflow Volumes from Analytical Probabilistic Model and STORM model**

| Overflow Location | Overflow Volume ($m^3$) from | |
|---|---|---|
| | Analytical Probabilistic Model | STORM Model |
| Bender Hill PS (OV #16) | 1500 | 1501 |
| Seneca St. PS (OV #14) | 3,800 | 3800 |
| Park St.PS (OV # 7) | 6,080 | 7000 |
| Garden/Brookfield (OV # 26) | 260 | 338 |
| Bellevue (OV # 11) | 2,370 | 2100 |
| Valleyway/Houck (OV #13) + Valleyway/Stanley (OV # 12) | 124,340 | 15000 |
| Muddy Run PS (OV# 5a) | 12,400 | 44366 |
| Muddy Run Extension (OV #5b) | 567,670 | 555279 |

### 7.4 Pollution Mass Load Reduction Analysis

As mentioned earlier, the main objective of the HRT facilities is to reduce the pollutant load of Muddy Run CSO before discharge it into the Niagara River. This CSO consists of the Muddy Run pumping station overflows (12,400 $m^3$/ year) and overflows from Central pumping station (567,670 $m^3$/ year). Therefore the total overflow volume at Muddy Run CSO is 580070 $m^3$/ year. The HRT facilities can reduce the pollutant load of CSOs from Central Pumping station but not from the Muddy Run pumping station. Therefore the overall pollutant concentration in CSO will be reduced according to the performance of the HRT devices. To evaluate the field performance of the HRT devices (Storm King and CDS unit), the pilot study is still on going. A large number of successfully monitored events (both for dry weather and wet weather condition) water quality data both for influent and effluent composite samples of each HRT devices are necessary to calculate the performance of the devices. But up to this date, only one sample for dry weather condition is collected and analyzed in NWRI laboratory. The sample was collected on September 23, 2005. The influent was pumped at the rate of 70 L/s for two hours and the flow in each unit was maintained at 35 L/s. Total 24 samples (1 L each) both for influent and effluent were collected at every 5 minutes and tested in NWRI laboratory. As per NWRI test report the TSS and VSS concentration in influent and effluent are calculated on the basis of time-weighted average. The summary of the test result is presented in Table 7.5.

**Table 7.5 Water Quality Data for Influent and Effluent of Each HRT Devices**

**(For dry weather condition)**

| Parameter | Influent Characteristic | | Effluent Characteristic | |
|---|---|---|---|---|
| | For CDS | For Storm King | For CDS | For Storm King |
| TSS (mg/L) | 143 | 143 | 88 | 89 |
| VSS (mg/L) | 108 | 116 | 74 | 69 |
| Total Coliform (cfu/100 ml) | 130,000,000 | 110,000,000 | 120,000,000 | 190,000,000 |
| E.coli (cfu/100 ml) | 10,000,000 | 20,000,000 | 10,000,000 | 40,000,000 |
| Heterotrophic Plate Count (cfu/ml) | 330,000,000 | 120,000,000 | 87,000,000 | 340,000,000 |

As the pollutant removal efficiency of each devices at the study site are not completed yet, therefore in this section, the predicted overall pollutant concentration (for TSS) after provide treatment in Muddy Run CSOs is calculated individually both for Storm King and CDS unit based upon their TSS removal efficiency claimed in their corresponding manuals. To calculate the overall TSS pollutant concentration, the following principle is used:

It is assume that every overflow will be treated by the HRT and then supplied to the Water Pollution Control Plant. But in practical situation, specially in wet weather condition, a large volume of overflow may be occurred, that exceeds the treatment capacity. In this case, the overflows that exceed the HRT capacity will be disposed directly to the Niagara river and thus reduce the overall mass load removal efficiency.

Let the total volume of overflow be $V$ m$^3$, pollutant concentration (e.g. TSS) before treatment be $C_i$ mg/L, the treated volume of overflow be $v$ m$^3$, and the efficiency of the devices be $\eta$%. Then the new pollutant concentration ($C_n$) in CSO is given by

$$C_n = \frac{(V-v)*C_i + v*C_i*(1-\eta)}{V} \qquad (7.1)$$

and the total pollution load after treatment $= V * C_n$ (7.2 )

As per CH2M Gore & Storrie Limited's (1996) study report, the mean TSS concentration in CSO at this study area is 100 mg/L (0.1 kg/m$^3$). On the basis of this data the TSS load in Muddy Run CSO before treatment is 58000 kg/yr $\approx$ 58 tons/yr.

**TSS After Treatment by the Storm King™ Unit**

As per Storm King™ product manual, the Total gross solid removal efficiency of this unit is 50% to 80%. Considering Efficiency, $\eta = 50\%$

The total volume of CSO, $V$= 580070 m$^3$, treated volume $v$ = 567,670 m$^3$ and TSS concentration before treatment $C_i$ = 100 mg/L. Therefore, from Equation 7.1, the new TSS concentration after treatment is 51.07 mg/L (or 0.051 kg/m$^3$), and from Equation 7.2, the annual TSS load after treatment is (580070 × 0.051 kg) = 29.58 tons.

Similarly the annual TSS load after treatment is calculated for 70% and 80% removal efficiency and result are presented in Table 7.6

**Table 7.6 TSS Load in CSO after Treatment by the Storm King**

| Removal Efficiency | TSS load (Tons/year) | |
|---|---|---|
| | Before Treatment | After Treatment |
| 50% | 58 | 29.58 |
| 70% | 58 | 18.27 |
| 80% | 58 | 12.60 |

**TSS After Treatment by the CDS™ Unit**

As per Combined Sewer Overflow Treatment Technologies Manual, the cumulative mass capture rates of CDS™ is between 73% and 84% for the 1200 μm screen. Considering Efficiency, $\eta = 73\%$

The total volume of CSO, $V$= 580070 $m^3$, treated volume $v$ = 567,670 $m^3$ and TSS concentration before treatment $C_i$ = 100 mg/L. Therefore, from Equation 7.1, the new TSS concentration after treatment is 28.56 mg/L (or 0.02856 kg/$m^3$), and from Equation 7.2, the annual TSS load after treatment is (580070 × 0.02856 kg) = 16.57 tons.

Similarly the annual TSS load after treatment is calculated for 80% and 84% removal efficiency and results are presented in Table 7.7

**Table7.7 TSS Load in CSO after Treatment by the CDS Unit**

| Removal Efficiency | TSS load (Tons/year) | |
|---|---|---|
| | Before Treatment | After Treatment |
| 73% | 58 | 16.57 |
| 80% | 58 | 12.60 |
| 84% | 58 | 10.32 |

# CHAPTER EIGHT

## CONCLUSIONS AND RECOMMENDATIONS

### 8.1 Concluding Remarks

Since CSO is considered as a major source of water quality impairment for the receiving waters, it is necessary to reduce the quantity and quality of CSO discharged. There are several approaches to control CSO. Ontario CSO guidelines recommend the development of pollution prevention and control plan, achievement of minimum CSO controls, and additional CSO controls for beach protection. The frequency of CSO events and the volume of wastewater discharged may be minimized by separation of storm and sanitary sewers and by the construction of new collector sewers, in conjunction with modification or enlargement of the major sewage treatment plants to accept greater flows. Wet-weather flow may also be stored within the existing sewer system where capacity exists, or stored in new tanks or tunnels, for subsequent treatment. However, these expensive options are not always feasible and cannot cope with all storms. The selection of a particular treatment technology depends on the site conditions, CSO characteristic and receiving water quality requirements. High- rate treatment facilities at overflow locations may be a practical, economical alternative (or addition) to the construction of new sewers and storage. Vortex separator technology is one of the preferred technologies for the high rate treatment facilities at overflow location. There are various vortex separator devices in different trade names. Storm King™ and CDS™ are two well known devices. These devices are currently used for CSO control at different locations in North America and Europe.

The City of Niagara Falls faces a major problem of CSO at Muddy Run extension resulting from the bypass and overflow from Central Pumping station. Approximately 580,000 $m^3$ CSO discharge annually from Muddy Run CSO location, which are contributed 58 tons TSS to the Niagara River. To mitigate the CSO problem at this location, the City of Niagara Falls conducted extensive studies for CSO control alternatives and decided to provide High-Rate Treatment (HRT) facility using vortex separator technology. To select the vortex separator devices for this HRT facility, a pilot study project is ongoing to evaluate the performance of two devices- Storm

King™ Vortex Separator and Continuous Deflective Separator (CDS™). CDS product manual claim that the TSS removal performance of CDS unit is varies from 73% to 84% and that for Storm King is 50% to 80%. As per their claimed, if the TSS removal efficiency of these devices is consider as 80%, then they can reduce yearly 45 tons TSS load from Muddy Run CSO. Up to this date, only one test sample for dry weather condition is collected and analyzed which is presented in this report. No wet weather condition's sample is collected yet. Without sufficient dry and wet weather condition's data, it is not a good representation of CSO characteristic as well as not possible to comment on the CSO control performance of these devices. Therefore, it is recommended that the further study and field monitoring data are required to test the full capabilities of the HRT devices.

Analytical Probabilistic model (APM) is used in this study to calculate the overflow volume at the central pumping station's contributing area. The results from APM show good agreement with that of STORM model conducted by CH2M Gore & Storrie Ltd in 1996 for the same area. As the APM is a simplified planning level model and can not handle a large volume of overflow, therefore it can be recommended to use the more dynamic model like XP-SWMM EXTRAN for comprehensive analysis.

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