SWAMP

Performance Assessment of a Swale/Perforated Pipe Stormwater Infiltration System - Toronto, Ontario

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Ministry of the Environment

Ontario

TORONTO

PERFORMANCE ASSESSMENT OF A SWALE AND PERFORATED PIPE STORMWATER INFILTRATION SYSTEM, TORONTO, ONTARIO

a report prepared by the

STORMWATER ASSESSMENT MONITORING
AND PERFORMANCE (SWAMP) PROGRAM

for

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NOTICE

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THE SWAMP PROGRAM

The Stormwater Assessment Monitoring and Performance (SWAMP) Program is an initiative of the Government of Canada's Great Lakes Sustainability Fund, the Ontario Ministry of Environment and Energy, the Toronto and Region Conservation Authority, and the Municipal Engineers Association. A number of individual municipalities and other owner/operator agencies have also participated in the SWAMP studies.

Over the past 15 years, the Great Lakes Basin has experienced rapid urban growth. Stormwater runoff associated with this growth is a major contributor to the degradation of water quality and the destruction of fish habitats. In response to these environmental concerns, a variety of stormwater management technologies have been developed to mitigate the impacts of urbanization on the natural environment. These technologies have been studied, designed and constructed on the basis of computer models and pilot-scale testing, but have not undergone extensive field-level evaluation in Ontario. The SWAMP Program was developed to address this need.

The SWAMP Program's objectives are:

- to monitor and evaluate the effectiveness of new or innovative stormwater management technologies; and
- to disseminate study results and recommendations within the stormwater management industry.

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Additional information concerning SWAMP and the sponsoring agencies is included in Appendix A.

ACKNOWLEDGEMENTS

This report was prepared for the Steering Committee of the Stormwater Assessment Monitoring and Performance (SWAMP) Program. The SWAMP Program Steering Committee is comprised of representatives from:

- the Government of Canada's Great Lakes Sustainability Fund
- the Ontario Ministry of Environment and Energy
- the Toronto and Region Conservation Authority
- the Municipal Engineers Association of Ontario

Funding support for this project was provided by the Great Lakes 2000 Cleanup Fund (superseded by the Great Lakes Sustainability Fund), the Ontario Ministry of Environment and Energy (OMOEE) and the City of Toronto (formerly the City of North York). The OMOEE also provided office facilities and logistic support for the SWAMP program. The Laboratory Services Branch of the OMOEE provided laboratory analyses. Administrative support to the SWAMP program was provided by the Toronto and Region Conservation Authority. The stormwater infiltration system evaluated in this report was conceived and designed by Raffi Bedrosyan and design staff from the City of Toronto. Staff at the City of Toronto also provided assistance with the hydrant test and drainage area calculations.

The following individuals provided technical advice and guidance:

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EXECUTIVE SUMMARY

In 1997, an innovative swale and perforated pipe infiltration stormwater system was constructed in a low density residential neighbourhood within the Wilket/Milne Creek subwatershed of the Don River. The system was intended to provide runoff quantity and quality control as part of the Wilket/Milne Creek Regeneration Plan and Don Watershed Management initiatives undertaken by the City of North York (now part of the City of Toronto) and the Toronto and Region Conservation Authority (TRCA). Use of this stormwater management approach was expected to provide significant improvements over the former ditched road network while avoiding the need to construct new storm sewer outfalls.

In 1998, a joint agreement was entered into by the City of North York, Ministry of Environment and Energy, the Government of Canada's Great Lakes 2000 Clean-up Fund (superceded by the Great Lakes Sustainability Fund) and TRCA to monitor the facility under the Stormwater Assessment Monitoring and Performance (SWAMP) program. The objectives of this monitoring study were to evaluate system performance in terms of runoff quality and quantity, identify benefits and limitations of the facility, and provide recommendations for improvements and further research needs.

Infiltration System Design

Figure 1 shows a simplified schematic of the infiltration system. The system consists of two components; a grassed swale (0.3 m deep x 3.0 m wide) and an underground infiltration trench (2 x 2 m in cross-section) located below the swale. The trench is lined with filter cloth and filled with granular 'A' gravel. The swale receives drainage from sidewalks, driveways and adjacent grassed areas. Runoff from the roadway is routed to catchbasins and subsequently directed to the infiltration trench via a 250 mm diameter lateral pipe. This lateral connects with a central 150 mm diameter filter cloth-wrapped perforated pipe laid within the trench aggregate at about 700 mm above the trench base.

At the downstream-most point in the trench, another 150 mm diameter pipe routes discharge water from the trench to a central storm sewer. A 250 mm diameter overflow relief pipe is connected to each catchbasin at 300 mm above the level of the infiltration lateral and drains into the central storm sewer. By design, the free water level within the trench must rise above 1.0 m from the base of the trench to engage the overflow relief pipe. Goss traps in the catchbasins, located at the lateral into the filtration trench and at the overflow lateral to the central storm sewer, reduce the amount of floating material that enters the pipes.

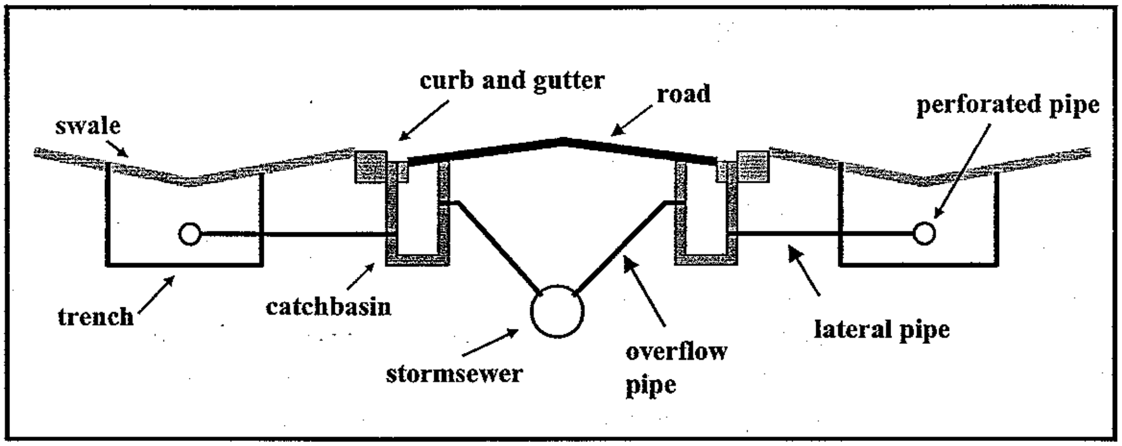

Figure 1: Simplified schematic of the infiltration system.

Study Approach

The performance assessment of the infiltration system was based on co-ordinated monitoring of rainfall, runoff and water quality. Pollutant concentrations and flow rates at the infiltration system inlet could not be directly monitored because of the multiplicity of overland flow and catchbasin inputs to the system. Therefore, water samples and flow measurements were taken from a nearby reference site with a conventional stormwater sewer system and similar land use. Performance of the infiltration system was assessed by comparing flow statistics and pollutant loads at the reference site with a similar set of measurements at the infiltration system outlet. Detailed impervious area estimates of roads, roofs and driveways provided the basis for comparing the two sites. Additional insights on the hydraulic capacity of the system were gained by conducting fire hydrant tests.

Study Results

Water quantity

A total of 21 small storms (< 5 mm rainfall), 12 medium sized storms (5 to 15 mm) and 8 large storms (>15 mm) were monitored from June to December in 1998 and 1999. Based on the reference site influent data set, flow reduction from the inlet to the outlet for storms with more than 5 mm of rain averaged 89%, ranging from a low of 77% to a high of 98%. The runoff coefficient, which is a measure of the proportion of catchment rainfall converted to stormwater runoff, was only 0.02 (or 2%) at the infiltration system outlet, compared to an estimate of 0.19 (or 19%) at the inlet.

Although the drainage area of the infiltration system site was more than five times larger than the reference site, peak flows at the infiltration system outlet were much smaller, averaging 7 and 42 L/s at the infiltration system and reference sites, respectively. Even during large storms, outlet peak flows were consistently less than 20 L/s. Most small storms (less than 5 mm) generated only negligible outflow, indicating that influent runoff was infiltrated to the surrounding soils.

A set of two 'hydrant tests' conducted on the downstream 100 m of the system indicated that the maximum inflow rate to each catchbasin without causing overflow was 11 L/s. This flow rate is roughly equivalent to a surface runoff rate within the study area of 554 m^3/hr, or a rainfall intensity of 16.0 mm/hr. However, a set of simple calculations based on the system geometry and data collected at the site indicates that the total capacity of the system to store, infiltrate and discharge runoff during a storm event would be approximately equivalent to 938 m^3 of surface runoff over the first hour of runoff, which in terms of rainfall, is roughly equivalent to an intensity of 28 mm/hr. Comparison of the hydraulic capacity of the system (554 m^3/hr) based on the maximum inflow rate, and the capacity of the system based on storage, infiltration and discharge (938 m^3/hr), suggests that system throughput is restricted to less than might be expected based on pipe and soil friction alone. Air entrapment within the pipes or gravel is suggested as a possible source of flow restriction.

Based on the difference between the volume of water pumped during the hydrant test and the volume of water exiting the system, together with the duration of the test, the unsaturated hydraulic conductivity of the soil surrounding the trench was crudely estimated at 1.4×10^{-5} m/s. This average infiltration rate corresponds to that of silty sand, and is just less than the 2×10^{-5} to 8×10^{-5} m/s estimate of average unsaturated hydraulic conductivity presented in the soils investigation report for the study area. Areas with soil infiltration rates less than 4.2×10^{-6} m/s are not considered suitable for perforated pipe infiltration systems.

Water quality

Water samples from 13 separate rainfall events were collected for this study between June 1998 and December 1999. Samples were analyzed for particle size and the major groups of pollutants found in stormwater runoff including heavy metals, nutrients, oil and grease, and total suspended and dissolved solids.

At the infiltration system outlet, average event mean concentrations (AEMCs) were greater than at the conventional system for 61% of the parameters analyzed, 64% of which were significantly higher at the 95% confidence level.

Outlet concentrations for total suspended solids (TSS) averaged 259 mg/l at the infiltration system outlet, compared to an average of only 29 mg/l at the conventional sewer reference site. The discrepancy in TSS concentrations between the reference site and infiltration system outlet was an unexpected finding.

By design, settling within the catchbasins was to have provided some pre-treatment of suspended solids, with further removal occurring within the infiltration trench before the water is discharged to the outflow pipe at the downstream end of the system. Possible explanations for the discrepancy in TSS concentrations between sites may include: (i) higher sediment loading rates at the infiltration system site, possibly due to frequent construction activity observed during the monitoring period; (ii) leakage of material through holes in the filter cloth wrapped around the outflow pipe, perhaps caused by rodents in the pipe; and (iii) bypass overflow into the storm sewer, which by design is flushed only during large storm events, and would therefore be subject to higher rates of sediment buildup from wind-blown dust and animals living in the pipe than the reference site. This comparison of concentrations should not be confused with the mass of suspended solids discharged from the two systems.

Other typical stormwater contaminants such as copper, lead, phosphorous and zinc had lower outlet concentrations relative to the reference site, although these still exceeded provincial guidelines for the protection of aquatic habitat in receivng waters. Only two parameters (mercury and oil/grease) had concentrations below the detection limit in one or more of the samples collected.

The particle size distributions from both the infiltration system and the reference site exhibited a large amount of variability. However, the average distrubutions suggest that the median particle size at the infiltration site (5.3 μm) was larger than that of the reference site (3.0 μm). As with TSS concentrations, this was an unexpected result, further suggesting that particulate material may have originated from overflow situations or an extraneous source.

Load-based removal efficiency calculations were based on two events for which composite samples were collected at both the reference site and infiltration system monitoring stations. The volumetric reduction in flow by the infiltration system for the two events was 91%. By comparison, overall load-based removal efficiencies were 91% for nutrients (average of nitrogen and phosphorus species), 51% for metals and, due to much higher concentrations at the infiltration system outlet (as explained above), only 24% for total suspended solids. Removal efficiencies above 80% were noted for mercury, all nitrogen species (TKN, ammonia, nitrates, nitrites), total phosphorus, phosphate, BOD, COD, titanium, cobalt, nickel, zinc and copper. Annual or seasonal removal efficiencies would probably be greater than cited above since 100% removal was achieved during several small events.

Conclusions and Recommendations

The results of this study indicate that the swale and perforated pipe infiltration system met most of its design objectives, including substantial reductions in total runoff and peak flow rates, removal of contaminants through infiltration and soil storage, and increased groundwater recharge. The relatively low TSS removal rate at the infiltration system outlet is a concern, but is likely due to a combination of construction activities during the study and the need for maintenance or minor modification to one or more elements of the system (*e.g.* goss traps, filter cloth, perforated pipe, sewer pipe), rather than an inherent defect in the system design.

The following recommendations are provided based on study findings:

System remediation and operation:

- The type of goss trap employed in the catch basins at this site should be examined to determine if they impose a restriction on system throughput.
- If the goss trap is found to restrict system throughput, the trap should be modified (*e.g.* add a breather to prevent air entrapment) and another hydrant test should be conducted to assess the full capacity of the filter bed to store, infiltrate and discharge runoff. Alternatively, the hydraulic capacity of the system could be assessed by pumping water directly into the perforated pipe.
- The invert of the system outlet pipe should be relocated well above the bottom elevation of the trench to provide additional net storage within the trench and increase infiltration rates. This configuration would further reduce outlet runoff.
- Individual catchbasins should be regularly inspected for goss trap clogging or excessive debris or sediment build-up.

Follow-up monitoring:

- After 10 years of system operation, soil and groundwater quality below infiltration trenches should be assessed against the pre-construction soil and water quality dataset to determine chemical impacts (if any) on groundwater resources.
- Hydraulic parameters documented in this study (*e.g.* inlet-outlet lag times, overflow thresholds, outlet runoff coefficients) should be used as a baseline against which follow-up monitoring can be compared. Changes in parmeters such as lag times or outlet runoff coefficients would provide a warning that specific components of the system may be in need of maintenance or repair.

Proposed future research:

- To provide a better basis for modeling system performance under different site conditions, continuous measurements of runoff and rainfall should be supplemented with information on water level changes in the filter beds and catchbasins and an additional flow measurement in the infiltration system overflow storm sewer upstream of the point at which the filter bed outflow pipe drains into the storm sewer. These additional measurements would permit independent assessment of the normal and overflow pathways through the facility and provide an improved understanding of system performance during rainfall events.
- Long-term effects may be examined and/or predicted by conducting tests on a model system. The model could be loaded more frequently than full-scale system permitting measurement of the fate of most pollutants.

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1.0 INTRODUCTION

Under natural conditions, most rainfall evaporates from the surface or infiltrates into the soil from where it is transpired by plants or recharges underground aquifers. Some of the groundwater is subsequently released slowly as continuous baseflow in creeks and streams[1]. With urban land development, new impermeable surfaces prevent water from infiltrating, and contaminated surface runoff is usually directed quickly through storm sewers to the nearest watercourse. The consequences can include a decline in stream baseflow, a rise in water temperature, increased pollutant loads, and channel instability caused by more frequent bankfull flows. Infiltration facilities help to mitigate these problems by mimicking natural hydrological processes existing prior to the installation of impervious surfaces and by filtering stormwater through the soil matrix[2].

The City of North York (now part of the City of Toronto) swale and perforated pipe infiltration system is an example of this innovative stormwater management practice. Installed in a low density residential neighbourhood within the Wilket/Milne Creek subwatershed of the Don River, the system consists of infiltration galleries with perforated pipes, located below roadside swales. The road includes a conventional curb and gutter, but with catchbasins that are connected to the infiltration galleries. The system was intended to provide runoff quantity and quality control as part of the Wilket/Milne Creek Regeneration Plan and Don Watershed Management initiatives undertaken by the City of North York and the Toronto and Region Conservation Authority (TRCA). Use of this stormwater management approach was expected to provide significant improvements over the former ditched road network and to avoid construction of new storm sewer outfalls.

In 1998, an agreement was entered into by the City of North York, the Ministry of Environment and Energy, the Government of Canada's Great Lakes 2000 Clean-up Fund (now the Great Lakes Sustainability Fund) and the TRCA to monitor the facility under the Stormwater Assessment Monitoring and Performance (SWAMP) program (see Appendix A). This report presents the results of this monitoring study, provides an evaluation of system performance and offers recommendations for modifications to the infiltration system.

1.1 Study Objectives

Since infiltration systems are intended to control downstream flooding and reduce pollutant loading to receiving waters, mainly by filtration and storage in the soil, the primary goal of the study was to evaluate the effectiveness of the facility in this regard. Specifically, the study aimed to:

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[1] See Appendix B for a glossary of terms

[2] See Appendix C for a short review of the literature on infiltration systems

- assess the capacity of the facility to reduce runoff and peak flows, and enhance the quality of catchment runoff;
- identify benefits and/or limitations of the facility; and
- provide recommendations for system improvements (if necessary) and further research needs.

It is hoped that this assessment will provide guidance to planners and stormwater management practitioners on the environmental benefit and design of similar treatment facilities.

2.0 STUDY SITE

2.1 Study Location

The infiltration system site is located in Toronto's Bridle Path area. The system was installed at Post Road from Hyde Park Circle up to and including Bridle Heath, Park Lane Circle, High Point Road and The Bridle Path from Post Road to Lawrence Avenue East (Figure 2.1). The 64 hectare stormwater infiltration system catchment area is characterized as low density residential. Most driveways are paved and lots are extensively landscaped. The roadways have a standard 20 m road allowance and incorporate a paved roadbed with urban cross-section of curbs and gutters. Road catchbasins are connected to adjacent twinned infiltration galleries 2 m from the curb. Sidewalks are generally single street side.

The location of the stormwater infiltration study was on High Point Road, from Post Road to Lawrence Avenue, covering a drainage area of 17.8 ha, 21% of which is impervious. Inlet volumes at this site could not be measured directly because of the diffuse nature of stormwater inputs to the system. Therefore, inflow volumes were estimated from flow measurements at a nearby reference site with a conventional stormwater sewer system and similar land use. This site was at Hyde Park Circle, directly north of Park Lane Circle, with a drainage area of 6.2 ha (Figure 2.1). Approximately 22% of the Hyde Park Circle drainage area is impervious.

2.2 Stormwater Catchment Area

2.2.1 Climate

The climate of the area is temperate with thermal highs and lows in all seasons moderated by a dominant lake effect from Lake Ontario about 10 km to the south of the study site. The mean January temperature is -6.5°C; the mean July temperature is 20°C. Annual liquid equivalent precipitation is 850 mm and annual snowfall is about 140 cm. Dominant winds are from the west and at an annual average speed of 17 km/h. In March, winds are mostly from the W, NW and N at an average velocity of 18.5 km/h, whereas in August winds are from the SW to N at an average velocity of 13.5 km/h. Annual global solar radiation is 115 Kcal/cm^2 and mean annual net radiation is 40 Kcal/cm^2. Mean annual lake evaporation is about 750 mm, whereas water balance derived evapotranspiration is about 625 mm (HAC, 1978).

2.2.2 Geology, Soils and Topography

Geologically, the area is dominated by flat-lying sedimentary carbonate rocks (limestones and dolomites). Depth to the bedrock in this area is about 70 to 100 m. This area is in the St. Lawrence Lowlands hydrogeologic region (OMOEE, 1997).

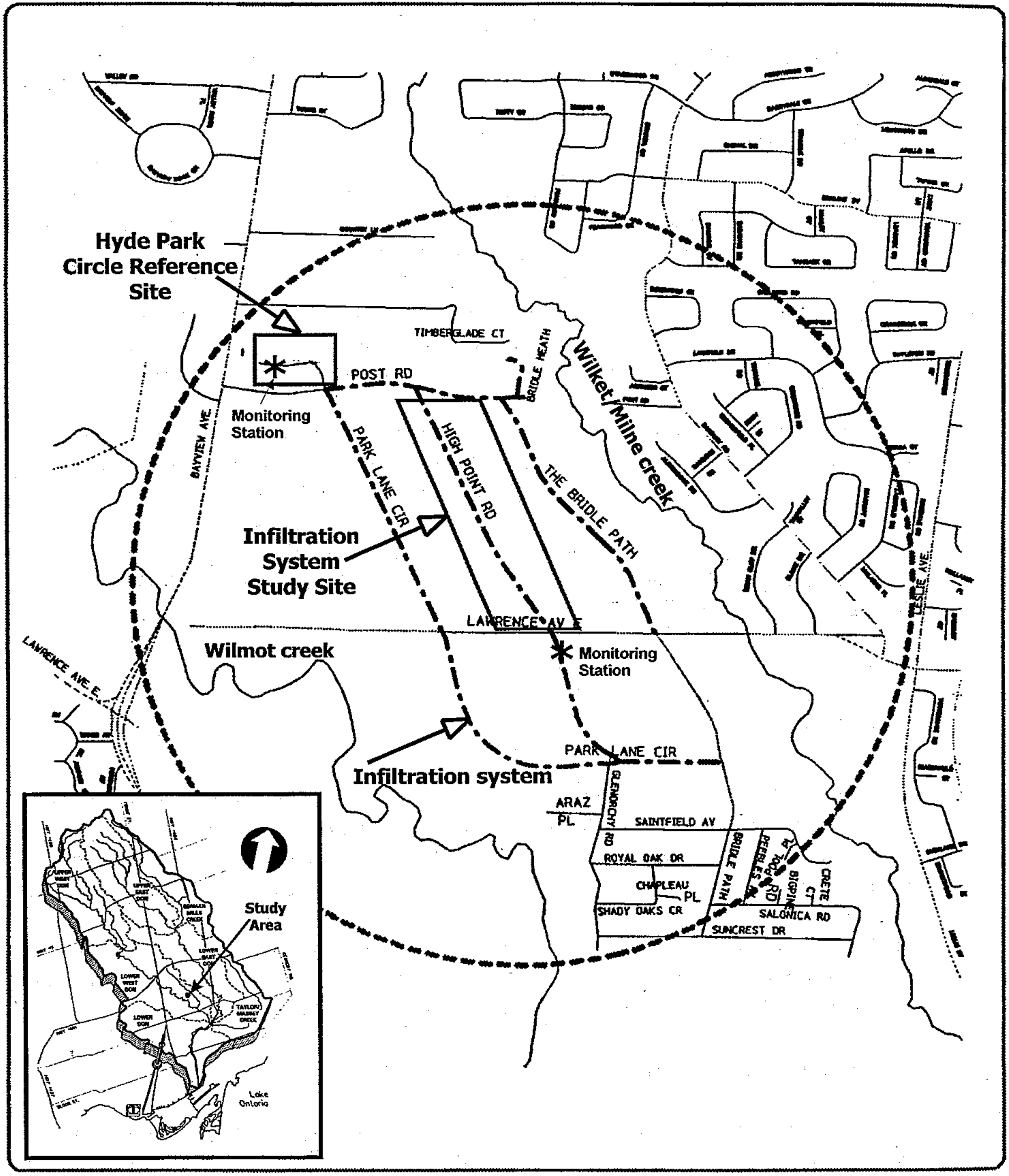

Figure 2.1: **Study area and location within the Don River watershed (Inset).**

A soils investigation conducted in the catchment prior to construction of the infiltration system indicated that soils consisted of surficial till and lake deposits of sand, silt and a very stiff clayey silt deposit. The residential lots are overlain by a veneer of topsoil 20 to 35 cm in depth and roadways are overlain by a compact fill veneer extending to 0.75 to greater than 2.0 m in depth. Along the 5 m borehole depth, the overburden generally transitions from smaller to larger grain size material in the range of silty sand and occasional clayey silt to coarser sand and gravel. Soils were considered to have relatively low permeability (CESI, 1997). The presence of coarser sand and silty sand seams of good to moderate permeability was observed in many boreholes.

Twenty four samples submitted for lab analysis during the baseline soils investigation met the Ministry of Environment and Energy guidelines as being uncontaminated and suitable for disposal in any industrial or commercial landfill site. The soils investigation also found no signs of contamination based on visual and olfactory tests. These results are useful as a baseline reference against which soil quality impacts of the infiltration system can be later evaluated.

2.2.3 Hydrology

2.2.3.1 Surface Water

The primary surface water body is the West Don River (Figure 2.1 inset), which drains to the Don River. Upstream of the study site, the West Don flows from its upper headwaters north of Kirby Road, in the City of Vaughan in the Oak Ridges Moraine complex. The lower West Don portion of the catchment is almost entirely urbanized, with the exception of the river valley and tributary corridors. About 60% of the subwatershed is in residential use, 20% in industrial/commercial use and 20% remains as open space. Stormwater runoff from the infiltration facility is routed to Wilket/Milne Creek, a tributary of the West Don River east of the site.

2.2.3.2 Groundwater

The soils investigation also monitored the 24 boreholes drilled in the infiltration system catchment area for groundwater. All boreholes were drilled to 5 m below the surface. Seven of the 24 boreholes penetrated the water table and four had water levels within 2 m of the ground surface. Three of these boreholes were located on High Point road, both north and south of Lawrence Ave and the fourth at the northern reach of Park Lane Circle. Examination of the borehole logs indicated that water levels were a result of perched water tables and were not representative of local or regional groundwater levels (CESI, 1997).

In conjunction with the general soils investigation, a field infiltration test was performed to assess the recharge capabilities of the area soils. This test was performed on Park Lane Circle, immediately south of Lawrence Avenue. At this silty sand dominated borehole, the average hydraulic conductivity was calculated to be between 2×10^{-3} and 8×10^{-3} cm/s.

Based on the soils investigation, the area was considered suitable for the implementation of an infiltration–type stormwater system. This determination was based on the presence of numerous medium to coarse sand seams believed to have high hydraulic conductivity (CESI, 1997). Soils surrounding these seams range from clay to silty sand and have infiltration capacities ranging from poor to moderate.

2.3 Infiltration System Design

The criteria used to guide the design of the facility include the following:

- meet Ministry of Environment and Energy design guidelines for stormwater quality and quantity control (OMOEE, 1994a);
- fit within the space limitations imposed by the existing storm sewer and roadside ditch network;
- remain compatible with adjacent land uses (e.g. nature trails and riparian park) and major utility services;
- maintain or reduce stormwater runoff volumes relative to the former stormwater drainage system
- continue to function during the winter; and
- be cost-effective.

The infiltration system (Figure 2.2) incorporates two primary and complementary components, a grassed swale (0.3 m deep x 3.0 m wide) and an underground infiltration trench[3] (2 x 2 m in cross section) located below the swale. The trench is lined with filter cloth and filled with granular 'A' gravel. The swale receives right-of-way boulevard and property lot overland drainage. Storm drains connected directly to the perforated pipe are located along the swale to help prevent ponding of water on the ground surface. Runoff from the roadway is routed to catchbasins from which stormwater is drained to the infiltration trench via a 250 mm diameter lateral pipe. The lateral is reduced to connect with a central 150 mm diameter filter cloth-wrapped perforated pipe laid within the trench aggregate at about 700 mm above the trench base.

At the downstream end of the trench, another 150 mm diameter pipe routes discharge water from the trench to the storm sewer. A 250 mm diameter overflow relief pipe is connected to each catchbasin at 300 mm above the level of the infiltration lateral and drains into the storm sewer. The free water level within the trench must rise 1.0 m above the base of the trench to engage the overflow relief pipe. Goss traps (29 x 14 cm opening) are incorporated in the catchbasins. Goss traps are standard features of most catchbains. They are syphon-like devices that permit only sub-surface flow into the discharge pipes. Hence, the majority of floating material is retained in the catchbasins.

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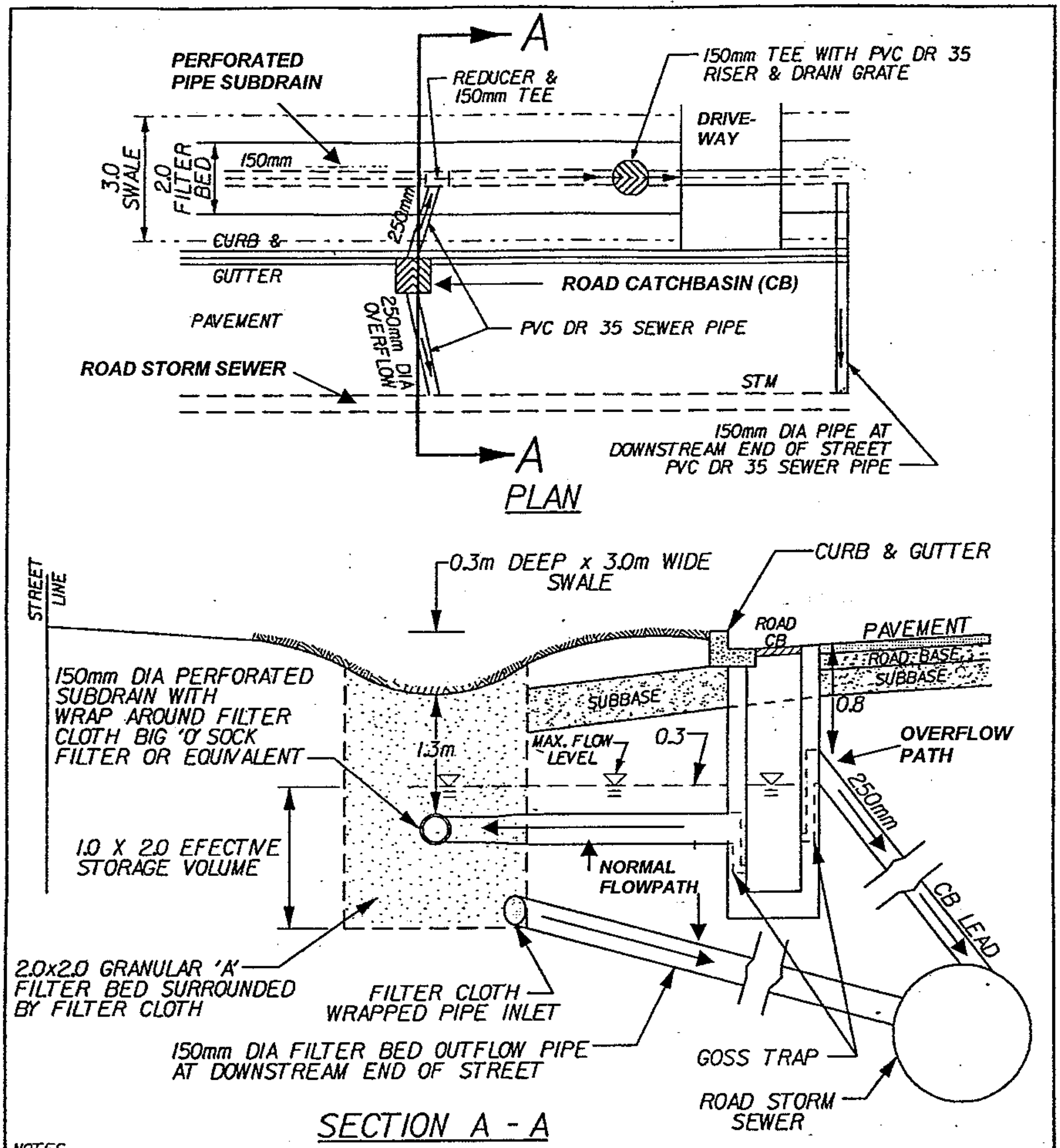
[3] In this report, the words trench, gallery and filter bed are used interchangeably.



NOTES

1. SWALE RECEIVES R.O.W. BLVD AND PRIVATE PROPERTY DRAINAGE
2. FILTER BED RECEIVES ROAD DRAINAGE VIA CATCHBASIN AND INFILTRATION FROM SWALE
3. ROAD STORM SEWER RECEIVES OUTFLOW FROM FILTER BED AND OVERFLOW DRAINAGE FROM ROAD CB

DIMENSIONS IN m EXCEPT AS NOTED

Figure 2.2: Infiltration system trench and road section

3.0 STUDY APPROACH

The performance assessment of the infiltration system was based on co-ordinated monitoring of rainfall, runoff and water quality. Pollutant concentrations and flow rates at the infiltration system inlet could not be directly monitored because of the multiplicity of overland flow and catchbasin inputs to the system. Hence, a reference system at Hyde Park Circle was used as a surrogate for the inlet. Performance of the infiltration system was assessed by comparing runoff statistics and pollutant loads at the reference site with a similar set of measurements at the infiltration system outlet. Detailed impervious area estimates of roads, roofs and driveways provided the basis for comparing the two sites. Since there was no continuous baseflow at either site, results were based on data collected during rainfall events. Additional insights into the hydraulic capacity of the system were gained by conducting two fire hydrant tests.

3.1 Instrumentation

Table 3.1 summarizes the types of equipment installed at each monitoring station. The location of the monitoring stations is shown in Figure 2.1.

Table 3.1: Location and Description of Monitoring Stations

Station	Description	Quantity
Reference site (450 mm storm sewer)	Automatic Sampler (time-weighted composite samples) Flow Logger	1 1
Infil. System Outlet (530mm outlet pipe)	Automatic Sampler (time-weighted composite samples) Flow Logger	1 1
Park Lane Public School	Continuous tipping bucket rain gauge	1

3.1.1 Rainfall

Rainfall was recorded continuously with a tipping bucket rain gauge at Park Lane Public School (Park Lane circle and Glenorchy Road), approximately 1 km south of the infiltration system catchment area. A Lakewood Systems™ datalogger connected to the rain gauge was programmed to record the exact time of each tip. Data were retrieved at regular bi-weekly intervals during the study period.

3.1.2 Reference Site (Hyde Park Circle)

Both flow and water quality data were collected at the Hyde Park Circle reference site. Flow was recorded at five minute intervals by an area-velocity flow meter immediately downstream of the confluence of the catchment's sewer network. Rainfall data, the area of the catchment and runoff volumes were used to calculate the runoff coefficient. Water quality samples were collected using an ISCO™ 3700 automated sampler. The sampler was set to commence sampling when runoff depth reached 10 mm in the storm sewer and to continue collection on a 15 minute sampling interval.

3.1.3 Infiltration System (High Point Road)

The infiltration system study site (High Point Road from Post Road to Lawrence Avenue) was monitored downstream of where the infiltration trench outlet drains into the central storm sewer. Flow at this point represents a combination of stormwater from the filter bed and catchbasin overflow pathway. As at the reference location, both an area-velocity flow meter and automated sampler were installed. Average flow was recorded at five minute intervals. Water quality samples were collected using an ISCO™ 3700 automated sampler. The flow meter was programmed to initialize the sampler and continue collection on a 15 minute sampling interval as long as flow depth remained above 10 mm.

3.1.4 Sampling

Composite samples were submitted to the Ministry of Environment and Energy Laboratory in Toronto and analyzed following principles outlined in Standard Methods (Eaton *et al.*, 1995) for metals, nutrients (P and N), bacteria, general chemistry and particle size distributions. A summary of analytical procedures for the major groups of pollutants analyzed in this study is provided in Appendix D. Since composite samples were weighted by time rather than flow, all samples represent only an approximate measure of the Event Mean Concentration (EMC), defined as the average concentration of a contaminant in storm runoff entering or leaving the infiltration system over the duration of a runoff event.

3.2 Mathematical and Statistical Methods

3.2.1 Inlet runoff volume and drainage area calculations

The total volume of stormwater entering the infiltration system at High Point Road (V_{HPR}) during rain events was estimated based on runoff volumes measured at the Hyde Park Circle reference site (V_{HPC}) such that:

$$V_{HPR} = 4.75 \times V_{HPC} \qquad \textit{equation 1}$$

where 4.75 represents the High Point Road (32,518 m^2) to Hyde Park Circle (6,843 m^2) impervious surface drainage area ratio. These impervious surface drainage area estimates include only the portion of the impervious drainage area that was considered to contribute to flow at each of the two monitoring points, rather than the entire impervious area within each catchment. The impervious area estimates were based on detailed area calculations of roads, buildings and driveways provided by the City of Toronto and differences in roof drainage characteristics between the two sites.

Table 3.2 provides a breakdown of the drainage and impervious area calculations for the infiltration system and reference study sites. Impervious drainage area calculations were based on the following assumptions:

- 100% of roof water from Hyde Park Circle drained to storm sewers (because downspouts were directly connected to road sewers);
- 65% of roof water from High Point Road drained to the infiltration system (because downspouts were directed to the surface); and
- all rain that fell on driveways, roads and sidewalks at both sites drained to storm or perforated pipe sewer networks.

Table 3.2: Impervious drainage area calculations for the infiltration system at High Point Road and the reference site at Hyde Park Circle.

	High Point Road (study site)	Hyde Park Circle^{++} (reference site)
Roads (m^2)	10,683	1,761
Driveways (m^2)	14,162	2,200
Buildings (m^2)	7,673*	2,882
Impervious drainage area connected to storm sewers/infil. system (m^2)	32,518	6,843
Impervious drainage area (m^2)	36,649	6,843
Total drainage area (m^2)	178,200	31,479
Percent imperviousness (%)	20.6^{+}	21.7

*7,673 m^2 represents 65% of total roof area (11,805 m^2) within the catchment. Since downspouts are directed to the surface, the remainder is assumed to have infiltrated on vegetated areas and bare soil.

\+ Based on total impervious area (36,649 m^2) rather than impervious drainage area contributing to storm runoff. The latter omits infiltrated roof drainage (*i.e.* water that does not drain to the infiltration system).

++ Area estimates for Hyde Park Circle include only that portion of the catchment that drains to the flow monitoring point.

3.2.2 Water quality

Water quality statistics were generated using a program developed specifically for the analysis of stormwater chemistry (Maunder *et al.*, 1995). The program employs Probability Distribution Estimation

(PDE) techniques (*e.g.* maximum likelihood estimation) to generate mean sample concentrations, standard deviations and 95% confidence intervals for data sets containing left-censored data (*i.e.* data at or below the analytical detection limit). These techniques determine values for left-censored data based on the log-normal probability distribution of the non-censored data. In cases where PDE techniques could not be applied, left-censored data were assigned values equal to half the analytical detection limit (Maunder *et al.*, 1995).

The primary statistic used for the analysis of water quality results was the Event Mean Concentration (EMC). In this study, EMC values were obtained from a single non-flow-proportioned composite sample at each location and each event. The maximum duration of each composite sample was 6 hours. Using contaminant loads from the Hyde Park Circle reference site as a substitute for inflow to the facility, the load-based removal efficiency (*LE*) of the infiltration system for each individual event was calculated as:

$$LE_{event} = \frac{4.75 \times V_{HPC} \times EMC_{HPC} - V_{out} \times EMC_{out}}{4.75 \times V_{HPC} \times EMC_{HPC}} \qquad \textit{equation 2}$$

where:

$4.75 \times V_{HPC}$ simulates the infiltration system influent volume (V_{in})
V_{out} = infiltration system effluent volume
EMC_{HPC} = reference site (Hyde Park Circle) event mean concentration
EMC_{out} = infiltration system event mean concentration

Similarly, load-based removal efficiency for multiple events was calculated as:

$$LE_{multiple} = \frac{\sum_{j=1}^{m}\left[4.75 \times V_{HPC_j} \times EMC_{HPC_j} - V_{out_j} \times EMC_{out_j}\right]}{\sum_{j=1}^{m}\left[4.75 \times V_{HPC_j} \times EMC_{HPC_j}\right]} \times 100\% \qquad \textit{equation 3}$$

where:

m = number of storm events

4.0 RESULTS AND DISCUSSION

4.1 Water Quantity

A summary of hydrologic statistics for the High Point Road infiltration system and Hyde Park Circle reference site is presented in Table 4.1. The summary represents 21 small storms (< 5 mm rainfall), 12 medium sized storms (5 to 15 mm) and 8 large storms (>15 mm) occurring from June to September in 1998 and 1999. Flow data collected at both monitoring stations between October and December 1998 were not reliable. Hydrographs for four representative rain events at the Hyde Park Circle reference and infiltration system sites are compared in Figure 4.1. Additional hydrologic insights gained from hydrant tests conducted at the facility in August 1998 are discussed in Section 4.1.4.

4.1.1 Inflow volumes

If it were assumed that most runoff is generated from impervious areas, then *equation 1* (Section 3.2.1) would be expected to predict flow volumes entering the infiltration system reasonably well. Figure 4.2 compares runoff volumes estimated from *equation 1* with runoff volumes calculated from rainfall data and High Point Road impervious area estimates (32,518 m^2 x rainfall). The two runoff calculations are strongly correlated ($r = 0.98$). Average runoff volumes ($n = 41$) were 268 and 256 m^3 for the impervious area ratio and impervious area rainfall calculations, respectively.

4.1.2 Runoff coefficients

The average runoff coefficient at the Hyde Park Circle reference site was 0.22, ranging from a low of 0.14 to a high of 0.29. This average coefficient compares with the High Point Road coefficient of 0.19 (based on impervious areas ratio) and a range from 0.12 to 0.25. These relatively low runoff coefficients reflect the low density, well-vegetated nature of the catchments. Due to substantial infiltration and storage losses during storm events, runoff coefficients at the outlet of the infiltration system were dramatically lower, averaging only 0.02 over the two monitoring seasons.

4.1.3 Rainfall-runoff analysis

Total rainfall from June to September in 1998 (204 mm) and 1999 (218 mm) were both less than the 30-year (1951-1980) normals (277 mm) recorded at the Toronto-Bloor meteorological station during the same months (Beak and Aquafor Beech, 1997). For storm events monitored during the study period, rainfall depths were well correlated with effluent volumes ($r = 0.92$), and reasonably well correlated with peak ($r = 0.84$) and mean ($r = 0.84$) flow rates, indicating that other factors affecting rainfall-runoff generation (*e.g.* antecedent moisture conditions, groundwater levels) tend to be of lesser importance during runoff events. Plots of these and other rainfall-runoff relationships at both sites are presented in Appendix E.

Table 4.1: Summary of hydrologic statistics at the Hyde Park Circle reference site and the High Point Road infiltration system for the 1998 and 1999 monitoring seasons.

	Rain				Hyde Park Circle reference site (3.1 ha.)						High Point Road infiltration system (17.8 ha.)								
Storm date	Depth (mm)	Duration (hrs)	Mean intensity (mm/hr)	Peak 5 min. intensity (mm/hr)	Flow duration (min)	Flow volume (m^3)*	Peak flow (l/s)	Mean flow (l/s)	Lag time (min)	Runoff coefficient	Inflow volume (m^3)+	Inlet runoff coefficient++	Outflow duration (min)	Outflow volume (m^3)	Peak outflow (l/s)	Mean outflow (l/s)	Lag time (min)	Outlet runoff coefficient	Flow reduction (%)
2/6/98 16:30	1.8	1.2	1.6	5.4	250	11.5	4.5	0.8	55	0.20	54.6	0.17			no flow				100
7/6/98 13:20	1.4	1.0	1.4	11.1	85	9.3	3.1	1.8	15	0.21	44.1	0.18			no flow				100
10/6/98 13:30	2.5	2.0	1.3	2.9		*17.2*					81.9				no flow				100
11/6/98 23:10	18.6	8.1	2.3	14.4	600	172.0	40.0	4.8	20	0.29	817.0	0.25	365	76.2	8.7	3.4	255	0.023	90.7
16/6/98 9:15	1.8	2.3	0.8	2.8	145	14.2	4.5	1.6	15	0.24	67.3	0.21			no flow				100
16/6/98 17:00	1.4	0.9	1.5	2.8	90	6.7	1.8	1.2	20	0.15	31.6	0.13			no flow				100
17/6/98 13:55	1.6	1.8	0.9	5.5	145	10.1	5.5	1.2	20	0.20	47.9	0.17			no flow				100
23/6/98 9:15	3.7	2.0	1.8	8.3		*25.1*					119.1				no flow				100
26/6/98 2:02	10.6	4.8	2.2	28.8		*72.1*					342.3		255	43.8	9.1	2.8	30	0.023	87.2
30/6/98 6:55	11.0	1.3	8.8	22.3		*75.2*					357.2		180	57.7	0.9	0.5	25	0.029	83.8
4/7/98 5:50	2.1	2.6	0.8	2.8		*14.1*					67.0				no flow				100
4/7/98 15:34	1.2	0.3	3.5	5.5		*7.8*					37.2				no flow				100
7/7/98 2:15	8.5	4.3	2.0	11.1		*58.0*					275.4		130	5.4	0.9	0.7	200	0.004	98.1
7/7/98 7:45	9.2	10.7	0.9	19.4		*62.7*					297.7		265	28.0	4.4	1.7	215	0.017	90.6
8/7/98 21:25	11.0	0.8	14.7	33.2		*75.2*					357.2		275	32.9	6.2	2.0	35	0.017	90.8
16/7/98 3:40	1.2	0.3	3.5	5.5		*7.8*					37.2				no flow				100
16/7/98 16:15	20.9	3.3	6.3	44.3		*142.6*					677.3		350	81.6	11.1	3.8	15	0.022	88.0
19/7/98 17:25	1.8	4.7	0.4	13.9		*12.5*					59.5				no flow				100
27/7/98 13:15	1.4	0.3	4.2	11.1		*9.4*					44.7				no flow				100
6/8/98 2:45	5.5	11.4	0.5	2.8		*37.6*					178.6				no flow				100
6/8/98 17:35	5.5	3.8	1.5	5.5		*37.6*					178.6				no flow				100
7/8/98 3:30	9.0	7.1	1.3	19.4		*61.1*					290.3		430	66.2	6.9	2.5	210	0.041	77.2
9/8/98 0:00	19.3	0.9	21.1	58.2		*131.6*					625.2		400	108.9	19.6	4.5	30	0.032	82.6
6/9/98 22:45	1.8	2.8	0.7	11.1		*12.5*					59.5				no flow				100
14/9/98 23:50	4.1	9.0	0.5	11.1		*28.2*					134.0				no flow				100
15/9/98 19:10	6.4	1.3	4.8	24.9		*43.9*					208.4		305	25.2	4.9	1.4	25	0.022	87.9
27/9/98 6:20	2.1	0.8	2.8	5.5		*14.1*					67.0				no flow				100
25/6/99 2:00	26.5	5.1	5.2	33.3	515	221.8	91.8	7.2	5	0.27	1053.6	0.22	510	100.5	13.6	3.3	35	0.021	90.5
27/6/99 10:45	8.5	4.1	2.1	33.3	385	72.6	58.7	3.1	10	0.27	344.9	0.23	245	9.7	4.0	0.7	20	0.006	97.2
3/7/99 16:10	2.5	1.0	2.5	13.9	130	20.6	9.9	2.6	15	0.26	97.9	0.22			no flow				100
17/7/99 16:50	3.7	1.6	2.3	36.0		*25.1*					119.1		20	1.1	2.0	0.9	20	0.002	99.1
31/7/99 10:45	17.5	6.8	2.6	30.5	660	146.4	68.6	3.7	10	0.27	695.5	0.22	145	20.2	3.6	1.6	25	0.007	97.1
4/8/99 4:45	23.0	13.1	1.8	39.2	965	181.1	26.3	3.1	10	0.25	860.4	0.21	770	66.5	5.0	1.4	70	0.016	92.3
6/8/99 9:05	1.6	0.8	2.1	5.5		*11.0*					52.1		15	0.6	1.2	0.6	50	0.002	98.9
7/8/99 19:45	2.3	2.4	1.0	5.5	195	10.3	3.3	0.9	35	0.14	49.0	0.12	10	0.1	0.3	0.2	135	0.000	99.8
7/8/99 23:55	3.2	6.0	0.5	8.3	420	28.9	4.4	1.1	15	0.29	137.3	0.24	30	0.7	1.0	0.4	105	0.001	99.5
10/8/99 9:30	4.6	9.1	0.5	9.3	620	22.2	2.8	0.6	50	0.15	105.2	0.13	50	2.5	2.5	0.8	240	0.003	97.6
20/8/99 8:45	7.4	11.5	0.6	11.1	540	50.4	20.4	1.6	25	0.22	239.5	0.18	450	53.9	5.6	2.0	30	0.041	77.5
13/9/99 11:15	8.5	3.8	2.2	8.3	750	48.5	3.9	1.1	5	0.18	230.4	0.15	240	10.6	1.9	0.7	40	0.007	95.4
29/9/99 9:50	13.3	4.4	3.0	2.5	400	70.1	27.7	2.9	10	0.17	333.1	0.14	245	12.9	5.3	0.9	80	0.005	96.1
29/9/99 17:55	35.0	6.0	5.8	27.7		*238.4*					1132.5		900	170.4	13.0	3.1	5	0.027	85.0
1998 & 1999 average																			
All storms	7.9	4.0	3.0	15.9	406	56.5	22.2	2.3	19.7	0.22	268.5	0.19	286	42.4	5.7	1.7	82	0.016	95.2
Storms with rain > 5mm	13.8	5.6	4.5	23.5	601.9	120.4	42.2	3.4	11.9	0.24	474.8	0.20	359	53.9	6.9	2.0	75	0.020	89.3

notes: Data were not available from August 23 to September 6, 1998, September 1 to September 8, 1999, and from Oct 1 to Dec 5, 1999. Outlet averages only include storms with measureable flow.

* Italisized runoff values were calculated as rainfall depth (m) x catchment area (31,479m^2) x the average Hyde Park Circle runoff coefficient (0.22). Actual runoff data during these times were either not available or judged to be not sufficiently reliable.

\+ Inflow volumes were calculated as Vin x 4.75 where Vin is the runoff volume at Hyde Park Circle and 4.75 is a coefficient equal to the ratio of the High Point Road (32,518m^2)to Hyde Park Circle (6843m^2) impervious surface drainage areas (see section 3.2.1).

++ Runoff coefficients of rainfall-runoff that were based on calculated Hyde Park Circle runoff volumes were not included. All of these were 0.18.

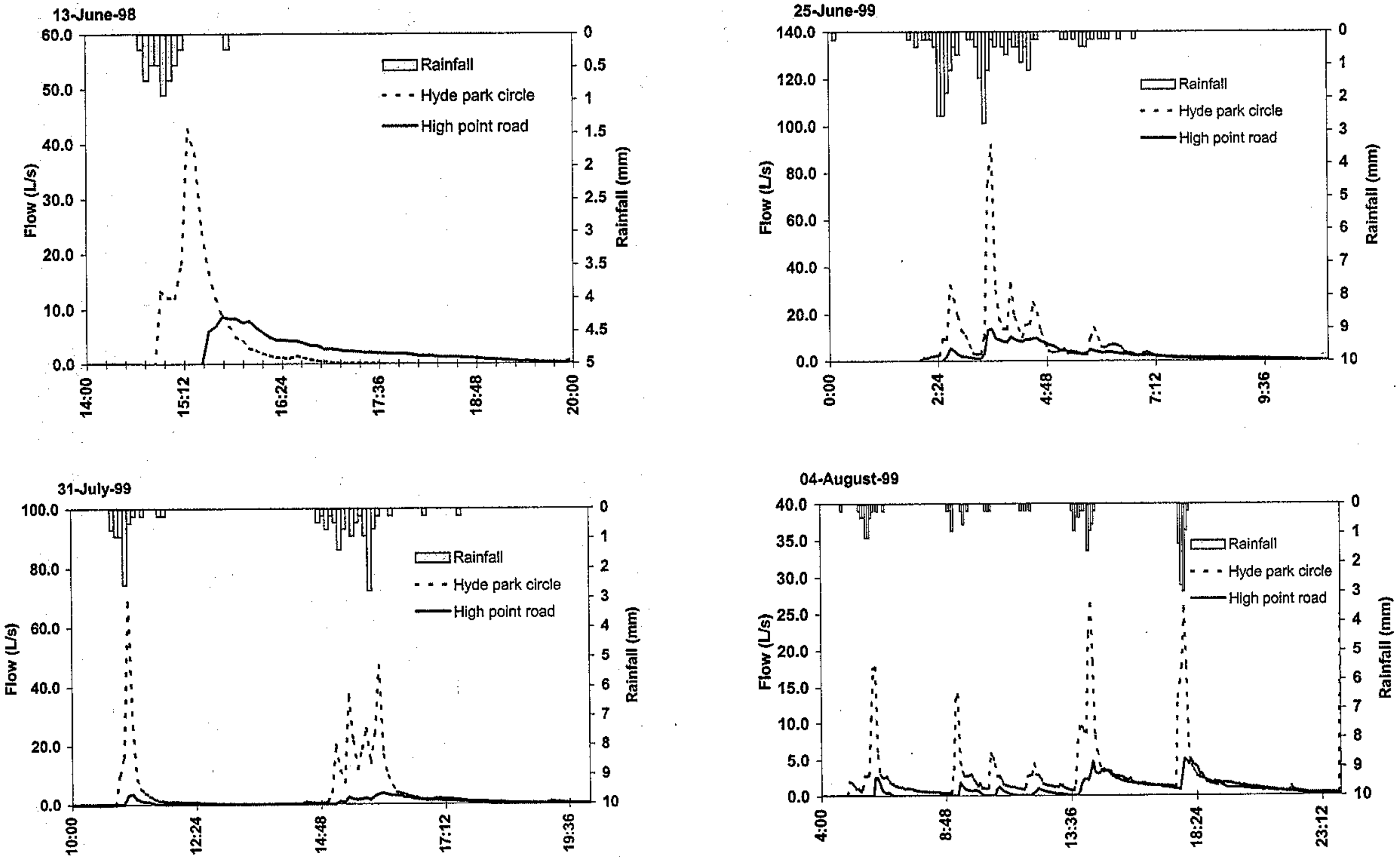


Figure 4.1: Hydrographs and hyetographs at the Hyde Park Circle reference site and the High Point Road infiltration system outlet for selected events. The drainage basins are 3.1 and 17.8 ha., respectively. Note differences in horizontal and vertical scales.

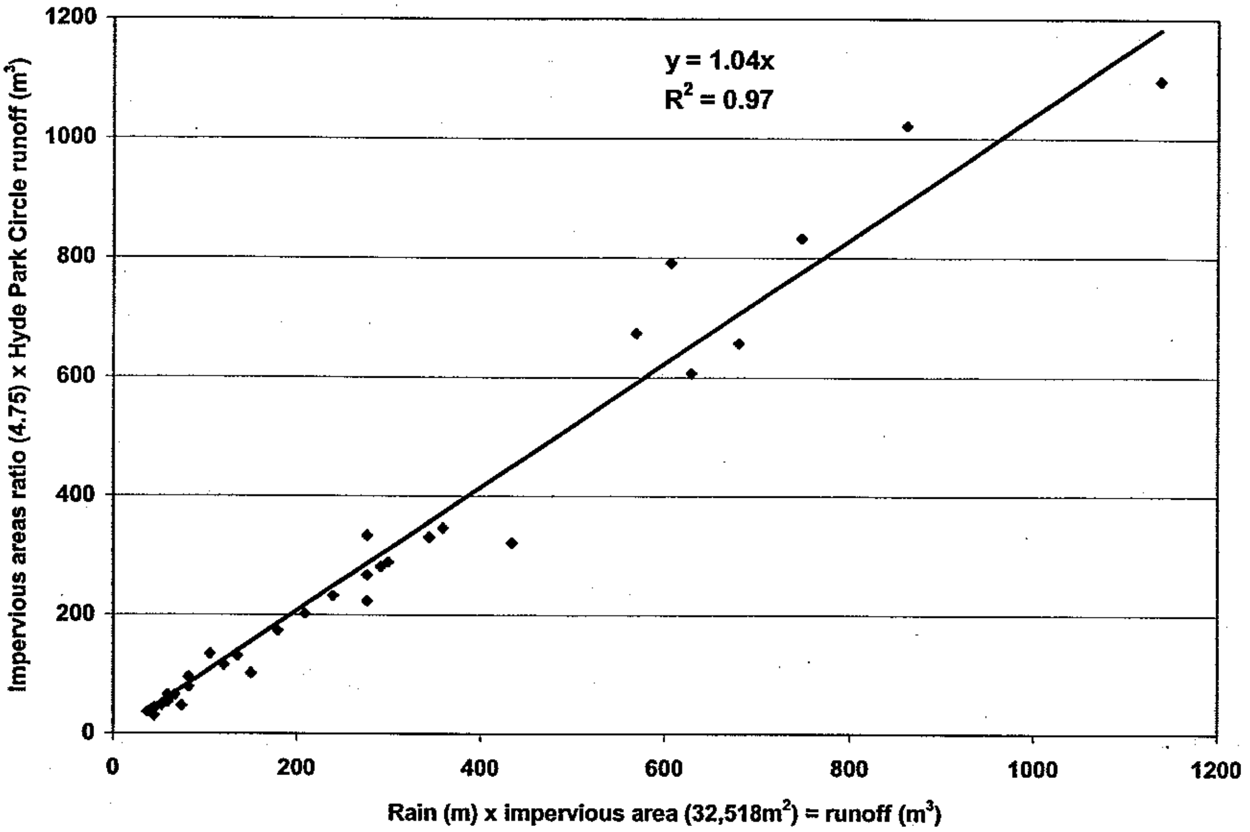


Figure 4.2: Relationship between High Point Road influent runoff as calculated from: (i) the High Point Road (infiltration system) to Hyde Park Circle (reference site) impervious area ratio (eq. 1) (*y*-axis) and (ii) rainfall data and the High Point Road impervious area (*x*-axis).

Among the 41 storms listed in Table 4.1, only 18 produced effluent volumes greater than 3 cubic meters and peak flow rates were consistently less than 20 L/s. By comparison, runoff volumes at the much smaller Hyde Park Circle reference site catchment were on average 2.8 times greater than at High Point Road (for storms > 5 mm), and the mean peak flow rate was 31 L/s. Based on the surrogate influent data set for storms with more than 5 mm of rain, runoff reduction from the inlet to outlet of the infiltration system averaged 89%, ranging from a low of 77% to a high of 98%.

Several small storms that generated significant flow in the conventional reference site storm sewer produced negligible flow at the infiltration system outlet. The largest event for which no outflow was observed had 5.5 mm of rain, or an estimated 173 cubic meters of inflow (August 6, 1998). Assuming soils within the trench have a porosity of 0.35 and specific retention of 0.1 (Fetter, 1994), this runoff volume represents 23% of the total filter bed and catchbasin storage volume below the perforated pipes. Hence, runoff begins after approximately one quarter of the trench below the perforated pipe is saturated, which is to be expected since the outflow pipe is connected at the lowest possible elevation within the infiltration gallery's downstream end (Figure 2.2).

The importance of soil antecedent moisture conditions to outflow generation is shown by differences in runoff response to the two back-to-back events on July 7, 1998 (Table 4.1). Although the inflow volumes were similar during the two events, the later event generated outlet discharges over 5 times greater than the event just a few hours before. The difference in response between the two flows may be attributed to reduced storage availability and slower infiltration into freshly wetted soils (Marshall *et al.*, 1996). The back-to-back storms on September 29 showed a similar pattern, with a 5 times increase in the runoff coefficient from storm one to storm two. However, in this case, storm two was much larger, and may have caused significant overflow from the catchbasins.

The time lag between the start of rainfall and outflow initiation at the infiltration system was primarily controlled by the storage capacity and infiltration rate of filter bed media. Therefore, it was not surprising that infiltration system lag times were consistently longer than observed at the Hyde Park Circle reference site (Table 4.1 and Figure 4.1). The average infiltration system time lag was skewed by a few storms that started with low intensity drizzle, but generally exceeded 100 minutes for events with less than 10 mm of rain, and was less than 40 minutes for larger storms. The shorter lag times during large storms may be attributed to short circuiting through the overflow relief pipe which, as will be discussed, appears to be controlled by the hydraulic capacity of the system.

4.1.4 Fire Hydrant Test

On August 28, 1998, two tests were performed to assess the hydraulic capacity and conductivity of the infiltration system. The tests involved pumping water from a fire hydrant into a catchbasin located 100 m from the infiltration system outlet at Lawrence Avenue. The tests, using an in-line flow indicator, were conducted with the assistance of the North York branch of the Toronto Fire Department. The first test commenced at 9:04 and concluded at 10:52. The second pumping test started at 12:07 and concluded at 13:15. The rationale behind conducting two tests in series was that the first test would flood the infiltration gallery and fill the soil pore spaces so that the system would approach saturated conditions, subsequently permitting an approximate measurement of the hydraulic conductivity of surrounding soils during the second test.

Figure 4.3 shows the inlet and outlet hydrographs for both tests. Test 1 pumped 72.2 m^3 of water into a catchbasin from where it flowed through the lateral pipe into the infiltration gallery. After an initial variable rate of up to 13.8 L/s, the inflow rate was stabilized at 11 L/s. Observation of the catchbasin water level indicated overflow to the road storm sewer. Evidence of this was also noted from 10 to 11 am, when outflow rates measured at an average of 12 L/s were greater than inflow rates. Total outflow from Test 1 was 66.4 m^3, indicating that approximately 5.8 m^3 or 8% of the inflow infiltrated into the surrounding soil. Note that the infiltration gallery outlet pipe is connected at the lowest elevation point in the gallery's downstream end. The pipe will receive flow from the trench only when the rate of flow into the gallery exceed the rate of infiltration and the soil pore (or matric) pressure surrounding the intake points is positive, which only occurs under fully saturated conditions. The outlet flow rate showed some variability but averaged 7.4 L/s over the duration of outflow. The lag between the initial input and the first flow output was 20 minutes, while the lag from the stoppage of inflow to the virtual stoppage of outflow was 61 minutes.

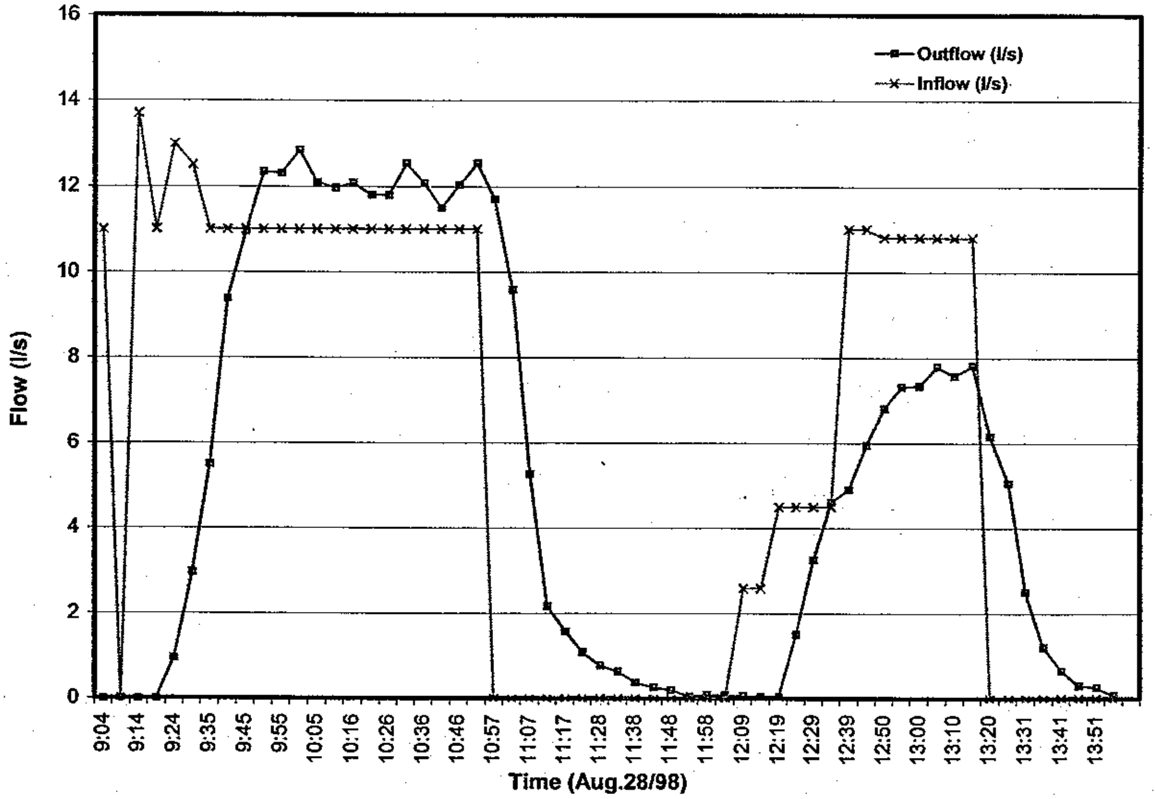


Figure 4.3: Inlet and outlet hydrant test hydrographs

Inflow from Test 2 commenced 10 minutes after the outflow from Test 1 had ceased, and continued for 68 minutes. Initial lag time from flow input to output was 17 minutes and from stoppage of inflow to virtual stoppage of outflow was 42 minutes. The start lag time was shorter (by 3 minutes) than during test one probably because, after the first flush, soils more nearly approached saturated conditions.

During Test 2, a total of 33.2 m^3 was pumped into the system at an average rate of 8.2 L/s and 24.4 m^3 was discharged via the outlet pipe at an average rate of 4.9 L/s. This equates to water loss of 26.5%, mostly to infiltration, since the available storage within the trench would have been less than in Test 1, and observed water levels in the catchbasins were consistently below that required for overflow. Inflow rates during the second test were explicitly held below the hydraulic capacity of the system (estimated at 11 L/s) to prevent overflow from the catchbasin to the storm sewer. The lower percent flow reduction during the first test (8% of inflow volume) may be attributed to overflow caused by higher inflow rates, and perhaps to other factors affecting the conductive portion of the cross sectional area available for infiltration, such as air entrapment or temporary clogging of particles along the geotextile lining the trench base and side-walls.

A crude estimate of the unsaturated hydraulic conductivity[4] of the soil surrounding the trench during Test 2 can be made if it is assumed that:

(i) the entire volume of water loss during Test 2 was routed to surrounding pore spaces (i.e. retention losses within the trench are negligible);
(ii) the wetted perimeter through which water infiltrates is constant during the test and extends up to the base of the perforated pipe;
(iii) capillary suction draws water into the horizontal plane almost as well as the vertical, permitting integration of both the base and side-walls of the trench into the calculation; and
(iv) infiltration is limited to the downstream 100 m of the system's gallery (*i.e.* water did not back up within the perforated pipe).

Based on these assumptions, the calculation of unsaturated hydraulic conductivity (Q_u) can be expressed as:

$$Q_u = \{V / [A \times p]\}/t \qquad \textit{equation 4}$$

where:

Q_u = average unsaturated flux rate through the surrounding media (m/s)
V = total volume routed to pore spaces during the test (m^3)
A = area of the wetted perimeter of the submerged zone (m^2)
p = system pore space fraction (estimated at 0.35)
t = time duration of the test (s)

Therefore, for Test 2:

$$Q_u = \{(8.8/[340(0.35)]\}/5400$$
$$= 1.4 \times 10^{-5} \text{ m/s}$$

This average infiltration rate corresponds to that of silty sand and is just less than the 2 x 10^{-5} to 8 x 10^{-5} m/s estimate of average permeability presented in the soils investigation report for the study area (CESI, 1997). Equation 4 predicts that, over the 90 minute duration of the test, the surrounding soils were

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[4] The trench and surrounding soils after the first test would have drained and are therefore considered to be 'unsaturated'

penetrated an average of 7.6 cm. Since the infiltration rate is negatively correlated with the initial soil moisture content (Marshall *et. al.*, 1996), and the soils prior to the test were freshly wetted from the previous test, the estimated infiltration rate probably approaches the steady-state saturated rate of flow through the soil.

Readers should note that an infiltration system was installed in this area, not because local soils had high infiltration capacity, but because the local soils contained many sand lenses thought to have greater ability to transport water. The infiltration gallery was designed as a 2 m deep trench because the trench cut through several lenses at this depth, providing hydraulic pathways for significant water transportation. With this in mind, the determination of an average saturated hydraulic conductivity rate is of limited importance. While the average infiltration rate over the entire surface may be relatively good, the actual rate to finer soil layers may be less as the sand lenses are conducting water at a much higher than average rate.

Additional observations and insights gained from the two hydrant tests are as follows:

- The maximum inflow rate to the catchbasin, without causing overflow to the storm sewer, was 11 L/s. This threshold would be exceeded in the entire system (14 catchbasins) for runoff volumes greater than 554 m^3/hr, or in terms of rainfall, at intensities greater than 16.0 mm/hr. During the study period, 14 events had 5 min peak rainfall inensities greater than 16 mm/h, one of which also had an average rainfall intensity greater than 16 mm/hr. Thus, based on the maximum inflow rate of the one catchbasin tested, 1/3 of the events monitored would cause some bypass and 2% of events would overwhelm the system.

- The hydrant test provided a basis for estimating the total capacity of the system to store, infiltrate and discharge runoff during a storm event. Total storage of the trench, catchbasins and perforated pipe below the point at which the overflow relief pipe connects to the catchbasin is approximately 791 m^3. Outflow during a storm will depend on storm intensity, but based on the maximum outflow observed during the second test, roughly 40 m^3 exits every hour. Water loss to infiltration into surrounding soils ($K = 1.4 \times 10^{-5}$ m/s) is estimated at 107 m^3/h. Adding these components yields a total system capacity of 938 m^3 of inflow during the first hour (equivalent to a rainfall intensity of 28 mm/hr). Total throughput is 147 m^3/hr, therefore the total system capacity over 2 hours is 1085 m^3 (storage = 791 m^3 + throughput = 2 x 147 m^3).

- The discrepancy between total estimated system capacity (938 m^3/hr) and the hydraulic capacity (554 m^3/hr) determined from the maximum observed flow rate (11 L/s) during the hydrant test, suggests that, during intense rainfall events, a throughput 'bottleneck' forms, causing water to back-up in the catchbasins and overflow into the storm sewer. In theory, the hydraulic capacity of the infiltration system is determined by all headlosses, given that the driving head is limited to the bypass pipe elevation. In a simple liquid system, the head losses would result from pipe wall friction, bends, gravel porosity, etc. However, in this case, a two-phase (air/water) system may prevail. The pipe and gravel network may be able to 'breath' through the riser pipes and other routes to some extent. However, if air pockets are trapped in the goss traps (which were found to be constructed without breather pipes), lateral pipes or gravel, they would restrict throughput to less than might be expected based on pipe and soil friction alone.

- Only 6.6% of the full length of the infiltration gallery was affected by the hydrant tests. Based on a proportionate reduction in the size of the High Point Road drainage area (17.8 ha. x 0.07) and the average High Point Road inlet runoff coefficient (0.19), equivalent rainfall depths for Test 1 (V_{in} = 72 m^3) and Test 2 (V_{in} = 33 m^3) were approximately 34.1 and 15.7 mm, respectively. Accounting for the duration of each test, these total depths translate into rainfall intensities of 18.9 and 13.8 mm/hr.

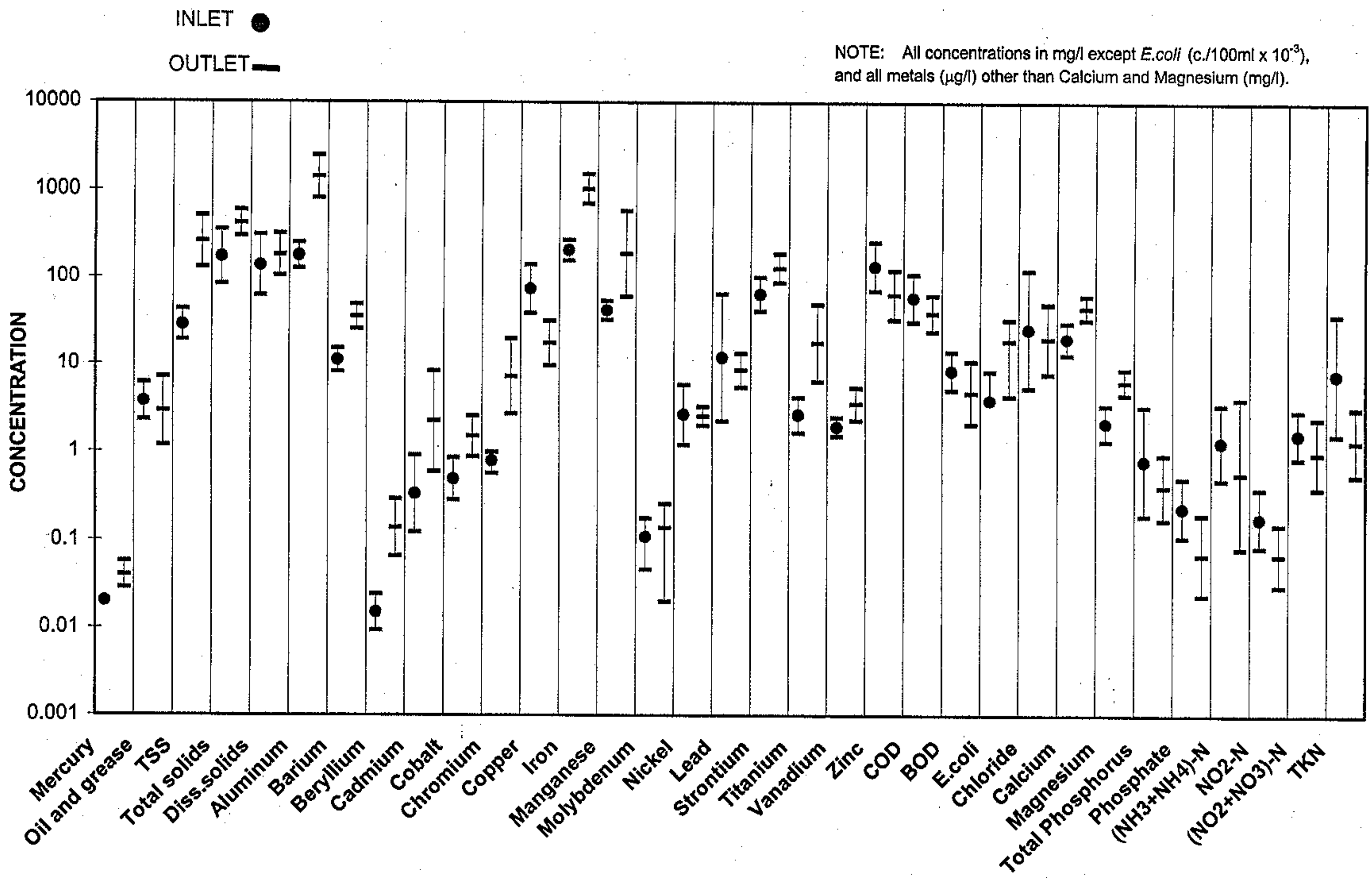
- There was a large discrepancy between the flow reduction (89%) estimated for the infiltration system site (Table 4.1), and the flow reduction (26%) observed during the hydrant test. While relatively rapid inflow rates during the hydrant test may account for part of this discrepancy, it also seems logical to suggest that a much greater proportion of water infiltrates at locations more distant from the outflow pipe, possibly due to high conductivity sand seams upstream. Water entering the system at more remote inlets would also have increased residence time within the filter bed, providing greater opportunity for infiltration to the surrounding soil.

4.2 Water Quality

Water samples from 13 separate rainfall events were collected for this study between June 1998 and December 1999. Among these, 9 were collected from the reference site and 7 were collected from the infiltration system site. The samples were analyzed for particle size and the major groups of pollutants found in stormwater runoff, including heavy metals, nutrients, oil and grease, and total suspended and dissolved solids (Appendix D summarizes laboratory analytical procedures).

4.2.1 Pollutant Concentrations

A summary of water quality results at the reference and infiltration system sites is presented in Table 4.2 and Figure 4.4. Results for individual events are provided in Appendix F. At the infiltration system outlet, average event mean concentrations (AEMC's) were greater than at the conventional system for 61% of parameters analyzed. Among these parameters, 64% were significantly higher at the 95% confidence level. Outlet concentrations for total suspended solids (TSS) were especially high, averaging 259 mg/l at the infiltration system, compared to an average of only 29 mg/l at the reference site. Other typical stormwater contaminants such as copper, lead, phosphorous and zinc had lower outlet concentrations relative to the reference site, although these still exceeded provincial guidelines (OMOEE, 1994b). Only two parameters (mercury and oil/grease) had concentrations below the detection limit in one or more of the samples collected.



NOTE: All concentrations in mg/l except *E.coli* (c./100ml x 10^{-3}), and all metals (µg/l) other than Calcium and Magnesium (mg/l).

Figure 4.4: Average event mean concentrations and 95% confidence limits at the Hyde Park Circle reference site (representing the inlet) and the High Point Road infiltration system (outlet). For actual concentrations, see Table 4.2

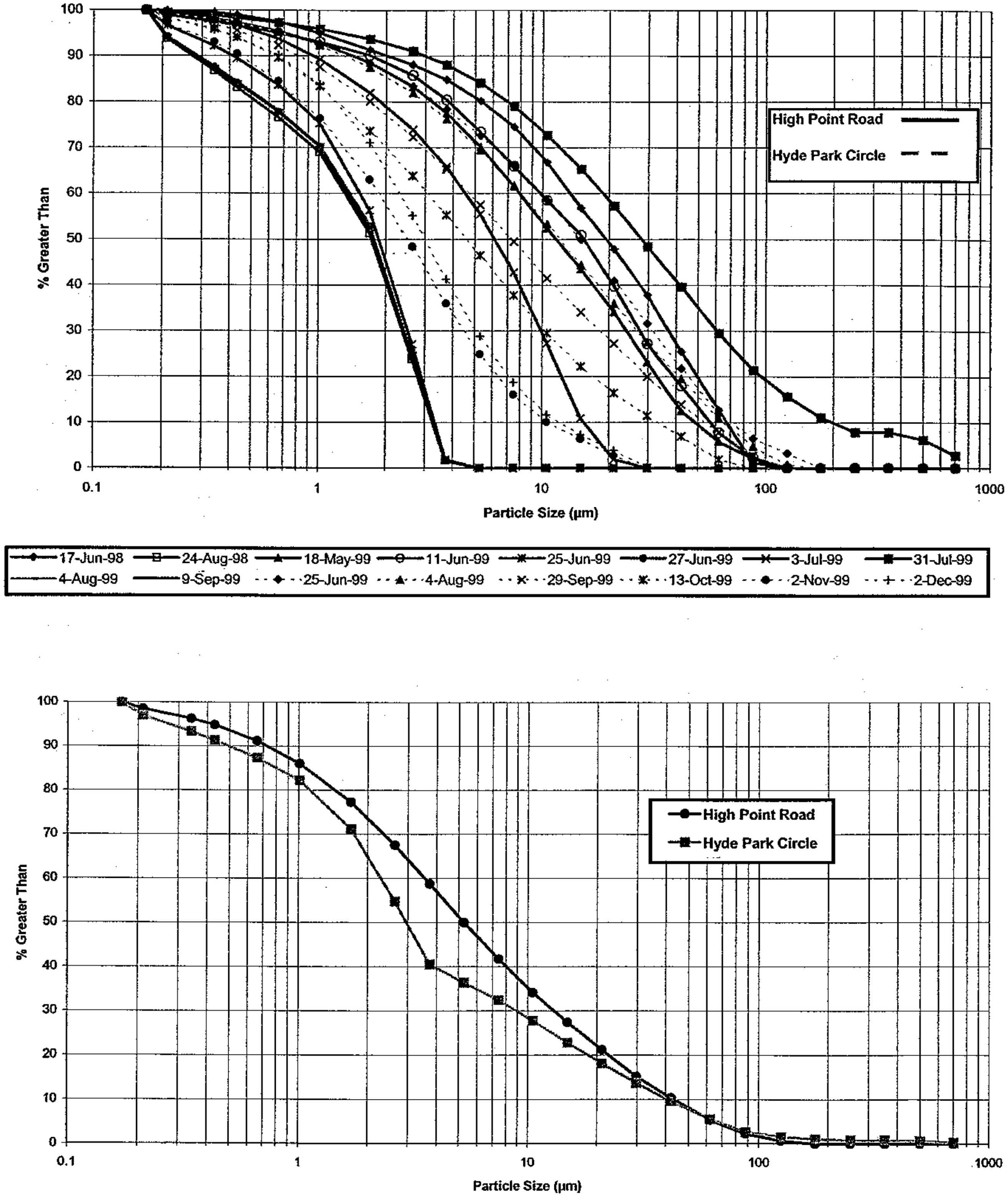


Figure 4.5: Individual event and average particle size distributions at the Hyde Park Circle reference site and the High Point Road infiltration system outlet for the 1998 and 1999 monitoring periods.

Table 4.3: Load-based removal efficiencies for two rain events at the High Point Road infiltration system. Inlet masses are based on a surrogate influent data set from Hyde Park Circle.

PARAMETERS	**25-Jun-99**			**4-Aug-99**			**Overall**
	Mass (g)		**Rem. Eff.**	**Mass (g)**		**Rem. Eff**	**Rem. Eff.**
	Inlet	**Outlet**	**%**	**Inlet**	**Outlet**	**%**	**(%)**
Mercury	0.02	0.00	80.9	0.02	0.00	84.5	82.5
TSS	15804	19095	-20.8	21510	9111	57.6	24.4
Total solids	56894	31155	45.2	61949	19684	68.2	57.2
Dissolved solids	42144	12060	71.4	39578	10507	73.4	72.4
Oil and grease	2634	603	77.1	n/a	n/a	n/a	77.1
Aluminum	232.8	203.0	12.8	113.6	34.6	69.5	31.4
Barium	7.60	3.58	52.9	6.07	1.62	73.4	62.0
Beryllium	0.01	0.01	-27.8	0.01	0.01	1.0	-14.8
Calcium	11.17	4.90	56.1	8.95	2.41	73.0	63.6
Cadmium	0.32	0.77	-144.2	0.26	0.05	78.9	-43.9
Cobalt	0.53	0.13	74.4	0.43	0.03	92.3	82.5
Chromium	0.53	1.32	-149.9	0.43	0.14	68.0	-52.0
Copper	60.37	2.94	95.1	54.64	1.28	97.7	96.3
Iron	312.9	160.8	48.6	154.0	31.7	79.4	58.8
Magnesium	1.06	0.79	26.1	1.01	0.26	73.8	49.3
Manganese	42.78	27.44	35.9	22.71	6.06	73.3	48.9
Nickel	2.63	0.25	90.5	2.15	0.17	92.3	91.3
Lead	4.44	1.51	66.0	4.16	0.68	83.5	74.5
Strontium	27.50	11.86	56.9	25.81	6.64	74.3	65.3
Titanium	5.27	0.50	90.5	4.30	0.33	92.3	91.3
Vanadium	1.05	0.46	56.2	0.86	0.14	83.4	68.4
Zinc	138.02	13.97	89.9	113.57	3.17	97.2	93.2
COD	21072	4824	77.1	38718	2660	93.1	87.5
BOD	4425	683	84.6	2925.36	279.30	90.5	86.9
Chloride	2318	1427	38.4	n/a	2048.20	n/a	38.4
$(NH_3 + NH_4)$-N	347.69	0.20	99.9	354.48	1.60	99.5	99.7
NO_2-N	33.72	2.01	94.0	61.09	5.52	91.0	92.1
(NO_2+NO_3)-N	400	106	73.6	1007	60	94.1	88.2
Phosphate	95.88	2.46	97.4	51.19	3.26	93.6	96.1
Total Phosphorus	202.29	33.37	83.5	3.44	n/a	n/a	83.5
TKN	1117	169	84.9	17	16	7.2	83.7
Flow volume (m^3)	**1053.6**	**100.5**	**90.8**	**860.4**	**66.5**	**92.6**	**91.3**

5.0 SUMMARY AND RECOMMENDATIONS

5.1 Summary

Overall, the infiltration system performed well in terms of groundwater recharge, peak flow attenuation and runoff reduction. The main study findings were as follows:

1. The infiltration system significantly reduces runoff discharge to the downstream stormwater system. Excluding storms with less than 5 mm of rain, the average runoff reduction for storms monitored during the study period was 89%. This reduction is reflected in average runoff coefficients at the infiltration system inlet and outlet, which were 0.19 and 0.02, respectively.
2. Although the infiltration system drainage area was much larger than the reference site, the average outlet peak flow for storms greater than 5 mm was only one-sixth that of the reference site. Even during large storms, outlet peak flows were consistently less than 20 L/s.
3. Most events with less than 5.5 mm of rain produced negligible runoff due to storage and infiltration of stormwater influent.
4. Event lag times from the start of rain to the initial discharge of runoff at the infiltration system outlet were often less than 40 minutes for storms greater than 10 mm, but typically exceeded 100 minutes for smaller events.
5. The hydraulic capacity of the system, rather than the total capacity of the system to store, infiltrate and discharge runoff, appeared to be an important factor controlling the incidence of overflow. Based on the hydrant test, the maximum out-flow rate from the catchbasin was estimated to be 11 L/s, which translates into an inflow volume of approximately 554 m^3/hr for the demonstration facility, and a rainfall intensity of 16.0 mm/hr.
6. The unsaturated hydraulic conductivity of the silty sand soil surrounding the trench was estimated to be 1.4×10^{-5} m/s.
7. Mean contaminant concentrations at the infiltration system outlet exceeded PWQOs for several constituents, including cadmium, copper, lead, zinc and phosphorus. Mean concentrations of suspended solids and bacterial parameters were also high relative to the reference site.
8. Despite apparent increases in the concentration of several pollutants from inlet to outlet (using a surrogate influent data set), load-based removal efficiencies for two large events were greater than 70% for several key parameters, with the exception of TSS, which had a removal efficiency of only 24%. Reduction in flow volume, rather than concentration, was the primary factor controlling pollutant removal rates.
9. The average median particle size of effluent samples from six relatively large events was marginally greater than the average of samples from the reference site, suggesting that, for the events monitored, little filtration of larger particle sizes occurred. Overflow situations or an extraneous source of solids are proposed as possible explanations for this unexpected result.

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APPENDIX A

Historical Context of the SWAMP Program

HISTORICAL CONTEXT OF THE SWAMP PROGRAM

Over the past 15 years, the Great Lakes Basin has experienced rapid urban growth. Stormwater runoff associated with this growth has been identified as a major contributor to the degradation of water quality and the destruction of fish habitats. In response to these concerns, a variety of stormwater management programs have been developed in the Great Lakes basin.

A number of complementary programs have been established at the international, national, provincial and municipal levels to protect the Great Lakes ecosystem. The SWAMP program and the study that is the subject of this report are parts of the overall effort.

International Joint Commission

The International Joint Commission (IJC) prevents and resolves disputes between the United States of America and Canada under the Boundary Waters Treaty of 1909. The IJC pursues the common good of both countries as an independent and objective advisor of the two governments.

In particular, the IJC rules upon applications for approval of projects affecting boundary or transboundary waters and may regulate the operation of these projects; it assists the two countries in the protection of the transboundary environment. Among the responsibilities of the IJC is the implementation of the Great Lakes Water Quality Agreement.

Great Lakes Water Quality Agreement

The first Great Lakes Water Quality Agreement (GLWQA) between Canada and the United States was signed in 1972 in recognition of the urgent need to improve environmental conditions in the Great Lakes. The focus of the agreement was to improve water quality through pollution control programs. Objectives included the reduction of nuisance conditions and control of toxic substances. Specific numerical targets were included for the reduction of phosphorus loadings.

The Great Lakes Water Quality Agreement was amended in 1978 to include the objective of controlling persistent toxic substances. The new agreement also incorporated the ecosystem approach to environmental management.

In 1987, the Canadian and U.S. governments signed a protocol that identified local Areas of Concern (AOC's) where beneficial uses of the ecosystem had been significantly degraded. Remedial Action Plans (RAP's) were to be prepared by various levels of government for the AOC's. The plans would contain strategies to clean up problem areas in the Great Lakes region. In addition, the 1987 protocol included annexes addressing specific subjects such as non-point contaminant sources and contaminated sediments.

Ontario Ministry of Environment and Energy

The Ontario Ministry of Environment and Energy (OMOEE) manages a number of programs that contribute to the protection and clean-up of the Great Lakes basin. The Provincial Water Protection Fund assists municipalities to address water and sewage treatment problems and to undertake related studies. The Ontario Great Lakes Renewal Foundation, established in 1998, provides seed money to support local projects which include habitat restoration and stormwater management. The OMOEE works in partnership with federal and state agencies and municipal governments to achieve numerous environmental goals; the Great Lakes Remedial Action Plans have been a prominent example of such work.

Toronto and Region Conservation Authority

The Toronto and Region Conservation Authority (TRCA) is one of 38 conservation authorities in Ontario that develop and implement programs for the management of water and natural resources on a watershed basis. Conservation authorities are created and given their mandate under the Conservation Authorities Act and involve a partnership of the municipalilties within a watershed and the Province of Ontario. The TRCA jurisdiction includes nine watersheds in the Toronto Region.

The TRCA and the Waterfront Regeneration Trust are the local coordinating agencies for the Toronto and Region Remedial Action Plan. The two agencies help the provincial and federal governments fulfill their obligations under the Great Lakes Water Quality Agreement and Canada-Ontario Agreement. The TRCA's general RAP role is to focus implementation activities on an individual watershed basis and provide technical expertise to its implementation partners. Stormwater management and the remediation of combined sewer overflows are integral to the restoration of the Toronto and Region Area of Concern.

SWAMP

In 1995, the Storm Water Assessment Monitoring and Performance Program (SWAMP) was created as a cooperative initiative of agencies interested in monitoring and evaluating the performance of various stormwater management technologies. The SWAMP program acts as a vehicle whereby federal, provincial, municipal and other interested agencies can pool their resources in support of shared research interests.

The objective of SWAMP is to collect data and report on the performance of stormwater treatment facilities. SWAMP is supported by the Great Lakes Sustainability Fund, the Ontario Ministry of Environment and Energy, the Toronto and Region Conservation Authority, the Municipal Engineers Association, a number of individual municipalities in Great Lakes Areas of Concern, and other owner/operator agencies.

A variety of stormwater management technologies have been developed to mitigate the impacts of

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APPENDIX B

Glossary

Glossary

adsorption: The adherence of a gas, liquid, or dissolved chemical to the surface of a solid (IWA, 2000).

baseflow: Sustained or dry-weather flow. In most streams baseflow is composed largely of groundwater runoff (James and James, 2000).

Best Management Practice (BMP): A device, practice, or method for removing, reducing, retarding, or preventing targeted stormwater runoff constituents, pollutants, and contaminants from reaching receiving waters (ASCE, 1999).

biochemical oxygen demand (BOD): The quantity of oxygen consumed during the biochemical oxidation of organic matter over a specified period of time, at a specified temperature, and under specified conditions. It is not related to the oxygen requirements in chemical combustion, being determined entirely by the availability of the material as a biological food and by the amount of oxygen utilized by the microorganisms during oxidation. See also COD. The following terms modify biochemical oxygen demand: first-stage (q.v.), immediate (q.v.), second-stage, standard, ultimate (James and James, 2000).

bypass: An arrangement of pipes, conduits, gates, and valves by which the flow may be passed around a hydraulic structure appurtenance, or treatment process; a controlled diversion (James and James, 2000).

capillary fringe: The lower subdivision of the unsaturated zone that overlies the saturated zone and in which the pressure of water in the interstices is lower than atmospheric (Parker, 1989).

catchment: That area determined by topographic features within which falling rain will contribute to runoff to a particular point under consideration. The area tributary to a lake, stream, sewer or drain. See also drainage area, drainage basin, river basin, catchment area, watershed (James and James, 2000).

chemical oxygen demand (COD): A measure of the oxygen equivalent of that portion of organic matter in water that is susceptible to oxidation by a strong chemical oxidant (also see biochemical oxygen demand) (adapted from IWA, 2000).

event mean concentration (EMC): The arithmetic mean concentration of an urban pollutant measured during a storm runoff event. The EMC is calculated by flow-weighting either grab samples or consecutive composite concentrations collected over the course of an entire storm event. (James and James, 2000).

geotextile: A woven or nonwoven fabric manufactured from synthetic fibers or yarns that is designed to serve as a continuous membrane between soil and aggregate in a variety of earth structures (Parker, 1989).

groundwater table: The upper surface of groundwater, or the surface below which the pores of rock or soil are saturated (James and James, 2000).

hydraulic conductivity: The rate of water flow through a cross section under a unit hydraulic gradient (Parker, 1989).

hydrograph: A graph showing, for a given point on a stream or conduit, the discharge, stage, velocity, available power, or other property of water with respect to time (James and James, 2000).

runoff: That part of the precipitation which runs off the surface of a drainage area and reaches a stream or other body of water or a drain or sewer (James and James, 2000).

runoff coefficient: The ratio of the depth of runoff from the drainage basin to the depth of rainfall (James and James, 2000)

sand seam: A stratum or bed of sand (Parker, 1989).

saturated zone: A subsurface zone in which water fills the interstices and is under pressure greater than atmospheric pressure (Parker, 1989).

specific retention: The ratio of the volume of water the rock or sediment will retain against the pull of gravity to the total volume of the rock or sediment (Fetter, 1994).

specific yield: The ratio of the volume of water a rock or soil will yield by gravity drainage to the volume of the rock or soil. Gravity drainage may take many months to occur (Fetter, 1994).

till: Unsorted and unstratified drift consisting of a heterogeneous mixture of clay, sand, gravel and boulders which is deposited by and underneath a glacier (Parker, 1989).

transpiration: The transport of water vapour from the soil to the atmosphere through actively growing plants (IWA, 2000).

unsaturated zone: A subsurface zone containing water below atmospheric pressure and air or gases at atmospheric pressure (Parker, 1989).

watercourse: A natural or artificial channel for passage of water (James and James, 2000).

watershed: A topographically defined area drained by a river or a stream or a system of connecting rivers and streams such that all outflow is discharged through a single outlet (James and James, 2000).

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APPENDIX C

Literature Review

LITERATURE REVIEW

Introduction

In urban environments, stormwater is generally routed back to the atmosphere by evaporation, into the underlying soil by infiltration, or into sewer networks, which convey it to treatment facilities or receiving waters. Infiltrated water may enter sewer pipes directly through perforated collection pipes, or in areas where the water table rises above the pipe horizon, as seepage through pipe joints. Infiltration of stormwater into the soil is a preferred method of dealing with stormwater because of its positive hydraulic and chemical effects on percolating water.

Hydraulically, rainfall that infiltrates the soil surface must pass through soil media, which, in most cases, significantly reduces its transport velocity. Conventional urban stormwater management without infiltration or detention practices results in urban stream peaks 2 to 5 times greater than predevelopment conditions (Leopold, 1968). In areas where infiltrated water recharges water bodies or is routed to storm sewers downstream via seepage, the effect is that of a much reduced runoff peak in the receiving water body or pipe system. This phenomenon, known as the "peak shaving effect", is of great importance in urban areas where short duration runoff peaks cause erosion or flooding damage downstream (GCSA, 1987). A study of 78 Philadelphia area streams reported a 2 to 4 fold increase in channel-width following urbanization (Hammer, 1972). In Washington State, Scott (1982) found urbanized streambed scour of 18 cm to be twice that of comparable rural reference streams. In the same study, sediment transport was three times greater than the rural reference streams (Richey 1982).

Infiltration of stormwater can also significantly improve water quality. Stormwater runoff typically contains high concentrations of heavy metals, nutrients, organic compounds and solids (USEPA, 1983). Percolation of stormwater through soil immobilizes suspended solids by filtration. Nutrients infiltrating through upper soil horizons may be fixed by plant roots and metabolized by soil biota. Most heavy metals and Polycyclic Aromatic Hydrocarbons (PAH) have a strong affinity for soil solids and are typically immobilized by adsorption within the upper 5 to 15 cm of soil (Nightingale, 1978; Dierkes and Geiger, 1999). Soluble and more mobile compounds, such as nitrate-N, chloride, some pesticides and enteroviruses, have a higher potential for groundwater contamination, especially if the infiltration system lacks a mechanism for pretreatment (*e.g.* grass buffer strip) (Pitt et al., 1994).

In Long Island, New York, "recharge basins" have been used since the 1930's to infiltrate urban stormwater and avoid costly storm sewer construction costs (Seaburn and Aronson, 1974). In a study of the basins, Ku and Simmons (1986) found that pre-development groundwater levels had been maintained and groundwater aquifers had not been measurably polluted either chemically or microbiologically by the infiltrating stormwater. Nitrate pollution of the aquifer from septic tank systems was found to be diluted by fresh water from some basins. These results are similar to those from other studies of infiltration basins in Fresno, California (Nightingale, 1987) and Perth, Australia (Appleyard, 1993), where pollutants in the upper soil horizons is still a concern that will eventually need to be addressed. In a study of highway runoff impacts on roadside soils in Germany, Dierkes and Grieger (1999) noted a strong relationship between the age of roadside soils and soil concentrations of PAHs and several heavy metals, but even after more than 20 years, leaching of pollutants to the groundwater was limited. In this study, the organic content and pH of soils were identified as important variables influencing the buffering capacity of soils.

Design Features of Infiltration Systems

Most infiltration systems fall into two groups, those where infiltrated water percolates to the groundwater table and those where infiltrated water is eventually drained away by drainage pipes. Most infiltration systems of the former type are swales or depressions intended to collect local overland flow and detain it until it seeps into underlying soils (Figure C1,a). Systems with drainage pipes also often use surface swales or depressions to collect runoff, but also employ an underground infiltration gallery designed to enhance flow through the system and increase the infiltration surface area (Figure C1,b). Drainage from the infiltration gallery is intended as an overflow safety precaution to avoid system surcharging and failure.

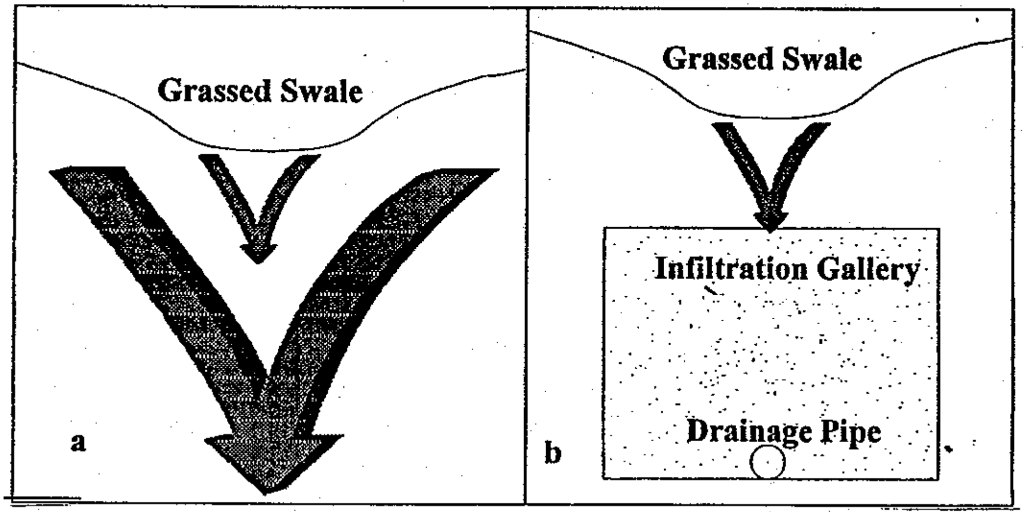

Figure 1.1. Diagrams of infiltration systems. (a) is a swale soakaway system. (b) is known as a Mulden-Rigole system, incorporating a surface swale and subsurface infiltration trench

Once percolating water enters the infiltration gallery it begins to fill the void spaces of trench aggregate. The aggregate is often comprised of coarse sand or fine gravel to promote rapid movement of water through the trench. Ferguson and Debo (1990) recommend clean, open- graded crushed stone such as No. 4 or No. 5 stone (4.2 – 4.8 mm coarse sand). Water infiltrates into the surrounding soils from the trench

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SWAMP

APPENDIX D

Laboratory Analytical Procedures

Table D1: OMOEE Laboratory Analytical Procedures Employed in this Study

Method Number	Product Number	Constituent	Procedure	Comments
E3016A	CL3016	Chloride	Colourimetry following two-stage reaction with mercuric thiocyanate and ferric iron	interferences from bromide, iodide, sulphide, cyanide, thiosulphate
E3060B	HG3060	Mercury	Cold vapour flameless atomic adsorption spectrophotometry (CV-FAAS) at 253.7 nm following acid digestion and reduction with stannus chloride solution	method is suitable for "clean" waters
E3170A	COD3170	Chemical Oxygen Demand	Samples mixed with an acidified Potassium Dichromate Solution to suppress chloride interference. Sluphuric acid containing silver sulphate is added and mixture is digested in a mechanical convection oven for 3 h at 149 C±1 C. Analysis by colourimetric measurement of trivalent chromium.	
E3182A	BOD3182	Biochemical Oxygen Demand	Sample is diluted such that 50% of dissolved oxygen is depleted after 5 days incubation. Dissolved oxygen is determined after preparation and again after incubation. BOD expressed as the amount of dissolved oxygen in mg utilized by 1 litre of sample during a 5-day incubation period at 20 C.	
E3188B	TSD3188	Total, Suspended and Dissolved solids	Suspended solids are determined as the material removed from suspension by a 1.5 to 2.0 µm glass fibre filter, after drying at 103° ±2° C. Dissolved solids is the material that remains in solution after suspended solids are filtered out, as determined by evaporating to dryness at 103° ±2° C. Total solids is the sum of suspended and dissolved solids.	
E3218A	CONDPH 3218	Conductivity, pH	Automated system using electrodes in a constant temperature bath of 25 C for conductivity and a calibrated potentiometric system for pH.	Supernatant or filtrate is analyzed. Gran alkalinity by special request only
E3311A	TURB3311	Turbidity	Measurement of light scattering at 90° ±30° by nephelometry calibrated to Formazin turbidity standards	

APPENDIX E

Rainfall-runoff Relationships

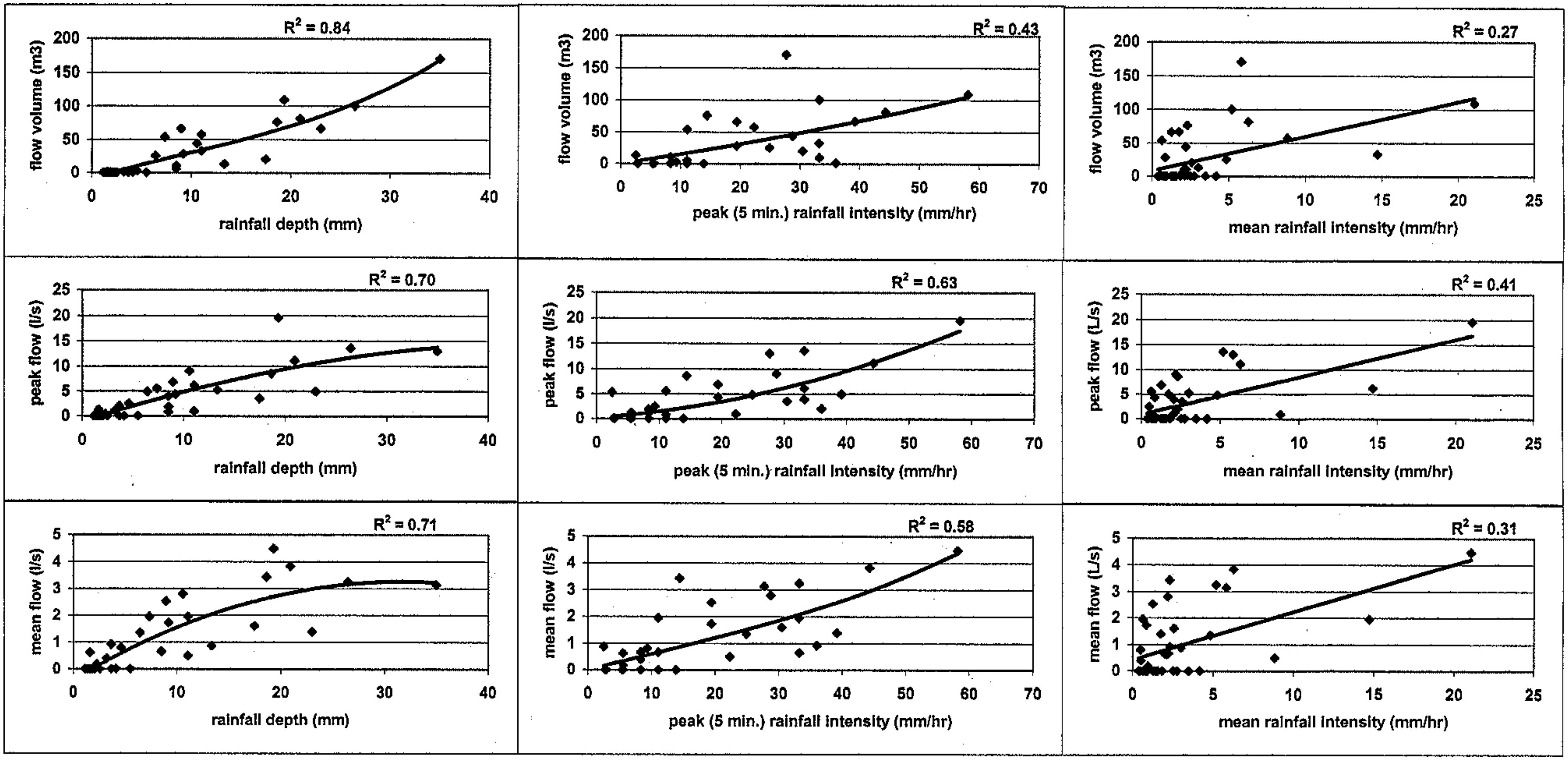

Figure E1: Relationships between rainfall and infiltration sytem outflow statistics

APPENDIX F

Water Quality Data for Individual Rain Events

Table F1: Sample concentrations for individual events at the Hyde Park Circle reference site and High Point Road infiltration system outlet over the 1998 and 1999 monitoring seasons.

Parameter	Units	Hyde Park Circle Reference Site									High Point Road Infiltration System Outlet						
		June 17/98	August 24/98	May 18/99	June 11/99	June 25/99	June 27/99	July 3/99	July 31/99	August 4/99	May 18/99	June 25/99	August 4/99	Sept 29/99	Oct 13/99	Nov 2/99	Dec 2/99
BOD	mg/L	23.4	5.4	10.2		4.2	5.8	8.2	8.0	3.4		6.8	4.2	3.4	3.6	7.0	0.8
COD	mg/L	16	38	88		20	35	160	60	45		48	40	32	28	67	18
pH	none	7.8	7.1	6.6		7.3	7.0	7.3	7.1	7.1		7.9	7.7	8.0	8.1	7.4	8.2
Conductivity	µS/cm	1200	110	143		60	96	108	123	71		185	243	117	140	195	468
Turbidity	FTU	6.0	5.4	15.0		6.2	4.3	14.9	2.0	5.6		84.4	40.9	136.0	452.0	312.0	47.5
TSS	mg/L	27.0	12.0	41.0		15.0	19.0	44.5	41.0	25.0	404.0	190.0	137.0	211.0	442.0	241.0	49.5
Diss. Solids	mg/L	846	72	92		40	62	66	80	46	378	120	158	76	92	128	304
Total solids	mg/L	874	84	134		54	82	110	120	72	782	310	296	288	532	368	354
Chloride	mg/L	263.0	4.0	6.4		2.2	2.2	3.2	3.8			14.2	30.8	5.4	6.8	8.0	48.8
P (PO4)	mg/L	1.15	0.08	0.22		0.09	0.10	0.22	0.14	0.06	0.01	0.02	0.05	0.10	0.20	0.01	0.04
P (TP)	mg/L	1.30	0.16	0.49		0.19	0.23	0.27	0.37	0.00		0.33		0.39	0.33	0.74	0.11
N (NH3 + NH	mg/L	5.66	0.29	1.91		0.33	0.73	0.91	0.15	0.41	1.40	0.00	0.90	0.02	0.04	0.14	0.04
N (NO2)	mg/L	0.62	0.20	0.09		0.03	0.06	0.14	0.21	0.07	0.19	0.02	0.08	0.03	0.07	0.05	0.02
N (NO2 +NO3	mg/L	2.68	1.25	0.49		0.38	1.51	1.43	3.20	1.17	1.02	1.05	0.90	0.52	0.72	0.07	1.19
N (TKN)	mg/L	14.60	1.36	4.54		1.06	2.04	1.90	1.02	0.02		0.33	0.24	1.20	1.38	2.12	0.52
Solv. extract.	mg/L	0.5	3.5	6.0		2.5	2.5		5.5					1.0	3.0	2.0	0.5
E. coli	c./100ml	9	1400											10300	13000	34000	
Cu	µg/L	10.7	43.1	71.7	18.5	57.3	96.7	103.0	142.0	63.5	27.8	29.3	19.2	16.9	9.8	15.4	4.7
Ni	µg/L	17.8	2.2	1.5	0.8	0.9	2.1	0.5	1.7	0.9	1.9	4.1	2.5	7.8	2.8	2.6	2.1
Zn	µg/L	13.8	81.5	114.0	55.2	131.0	202.0	154.0	191.0	132.0	73.5	139.0	47.7	78.2	69.6	44.9	14.1
Cd	µg/L	0.3	0.7	0.0	0.5	0.4	0.2		0.1	0.0	3.8	7.7	0.8	1.1	0.7	0.3	0.3
Co	µg/L	0.1	0.2	0.5	0.7	0.2	0.3	0.7	0.9	0.5	1.4	1.3	0.0	4.6	2.0	1.4	0.5
Cr	µg/L	0.6	0.8	0.8	0.7	0.7	0.5	0.9	1.4	0.4	26.4	13.1	2.1	2.5	3.0	2.5	1.4
Mn	µg/L	34.3	25.7	1.2	61.8	40.6	36.5	44.9	56.3	26.4	8.4	7.8	91.1	128.0	244.0	224.0	46.1
Mo	µg/L	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Sr	µg/L	169.0	55.7	64.2	103.0	26.1	40.7	49.3	51.5	30.0	244.0	118.0	99.9	84.8	122.0	90.8	165.0
Ti	µg/L	4.2	0.7	1.1	2.7	2.8	1.6	3.1	3.9	2.8	14.9	3.6	5.8	4.6	53.5	27.6	7.4
Ca	mg/L	62.4	14.7	16.0	25.2	10.6	12.3	15.0	19.4	10.4	69.2	48.8	36.3	27.7	54.6	33.1	51.5
Mg	mg/L	5.7	1.4	1.2	4.0	1.0	1.3	1.7	2.2	1.2	8.4	7.8	4.0	3.8	7.3	4.3	8.7
flow volumes (m^3)		10.1	n/a	*77.6*	n/a	221.8	72.6	20.6	146.4	181.1	*368.6*	100.5	66.5	170.4	*629.0*	*1552.3*	n/a
rainfall depths (mm)		1.6	3.2	*10.9**	n/a	26.4	8.5	2.5	17.5	23.0	*10.9**	26.4	23.0	35.0	18.6	*45.9**	n/a

* data from a rain guage located in the City of Markham (approx. 25km northeast of the study site)