Evaluation of Underground Stormwater Infiltration Systems

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Evaluation of Underground Stormwater Infiltration Systems

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NOTICE

Pictures of infiltration chambers on the front cover are courtesy of Cultec Inc and StormTech.

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THE SUSTAINABLE TECHNOLOGIES EVALUATION PROGRAM

The Sustainable Technologies Evaluation Program (STEP) is a multi-agency program, led by the Toronto and Region Conservation Authority (TRCA). The program helps to provide the data and analytical tools necessary to support broader implementation of sustainable technologies and practices within a Canadian context. The main program objectives are to:

- monitor and evaluate clean water, air and energy technologies;
- assess barriers and opportunities for implementing technologies;
- develop supporting tools, guidelines and policies; and
- promote broader use of effective technologies through research, education and advocacy.

Technologies evaluated under STEP are not limited to physical products or devices; they may also include preventative measures, alternative urban site designs, and other innovative practices that help create more sustainable and liveable communities.

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EXECUTIVE SUMMARY

Background

Stormwater infiltration practices are an integral part of progressive approaches to stormwater management. These practices help to reduce the volume of urban runoff discharged to watercourses thereby minimizing flood risk and preventing alterations to the stream flow regime and channel form. They help to maintain groundwater levels and sustain stream flows during dry periods. They also reduce pollutant loading to receiving watercourses by reducing runoff volume and retaining or breaking down pollutants in the engineered structures and underlying native soil. They can be designed for application at the ground surface (e.g., permeable pavement, bioretention, infiltration basins) or below ground (e.g., infiltration chambers, trenches, soakaways, and pervious pipe exfiltration systems). An advantage of underground infiltration technologies is that they can be located below parking lots, roads, parkland or other landscaped areas. In densely developed urban areas, where the value of land is very high, this often makes them preferable to surface practices.

Despite their advantages stormwater management system designers are often reluctant to recommend the application of infiltration practices on fine textured soils due to their limited permeability and concerns over the size of facilities that would be required to meet regulatory requirements. Since most of the designated urban growth areas in the Greater Toronto Area and other urban centres in southern Ontario are located on fine-textured glacial till soil, there is considerable interest in how effective infiltration practices are in such contexts. This study helps to address this knowledge gap by evaluating the effectiveness of three underground stormwater infiltration systems constructed over fine-textured glacial till soils draining roof runoff from industrial/commercial developments located in the Greater Toronto Area.

Study Sites

The three underground stormwater infiltration systems evaluated in this study are:

- An infiltration chamber system servicing the Elgin Mills Crossing commercial development in Richmond Hill, Ontario;
- An infiltration trench system servicing the Mayfield Industrial Park in Bolton, Ontario; and
- An infiltration chamber system servicing the Bramport commercial centre in Brampton, Ontario.

The table below provides an overview of design characteristics of each system.

The Elgin Mills Crossing infiltration chamber system is installed under the parking lot of a shopping centre development in Richmond Hill, Ontario. The chamber system receives roof runoff from two large commercial buildings (combined roof area of 25,449 m^2). The StormTech SC-740® stormwater infiltration chamber system was installed to maintain average annual infiltration volume over the site to pre-development levels, thereby helping to maintain aquifer water levels and groundwater discharge to the Rouge River tributary adjacent to the site, and minimize the increase in runoff caused by the development. Native subsoil at the site is sandy silt glacial till, which underlies the entire site. The low permeability sandy silt till deposits overlie a lens-shaped deposit composed mainly of more permeable silty fine sand at shallow depths. The chambers were strategically located near the river valley over an area where the gravel base could intersect with the silty fine sand lens to help ensure that pre-development groundwater discharge rates and volumes to the Rouge River would be maintained. The infiltration chamber system and contributing storm sewers were designed to temporarily store 955 m^3 of runoff which is roughly equivalent to a 41 mm precipitation event over the combined roof drainage area, assuming 10% loss of precipitation to evaporation. The seasonally high water table in the vicinity of the chamber system is estimated to be between 2 to 3 m below the ground surface in April and May, raising the possibility that groundwater levels may reach elevations above the base of the chamber system during the spring, which would affect drainage times.

Facility Name and Type	Drainage Area (m^2)	Storage Capacity (m^3)	Infiltration Volume Target (m^3/y)	Native subsoil texture	Assumed Subsoil Hydraulic Conductivity (cm/s)
Elgin Mills Crossing infiltration chamber system	25,449	955	14,494	Sandy silt glacial till	1×10^{-6}
Mayfield Industrial Park infiltration trench #1	14,962	126	6,015	Clayey silt glacial till	1×10^{-6}
Mayfield Industrial Park infiltration trench #2	20,101	126	8,081	Clayey silt glacial till	1×10^{-6}
Mayfield Industrial Park infiltration trench #3	23,268	126	9,380	Clayey silt glacial till	1×10^{-6}
Mayfield Industrial Park infiltration trench #4	14,420	184	6,456	Clayey silt glacial till	1×10^{-6}
Bramport infiltration chamber system	33,500	1,192	0	Sandy silty clay glacial till	1×10^{-6}

The Mayfield Industrial Park infiltration trench system is installed below parking and landscaped areas in a commercial/industrial area in Bolton, Ontario. The system is composed of four trenches (referred to as Mayfield Trenches 1, 2, 3 and 4) that receive runoff from the roofs of two large buildings in the industrial park. The system was designed to maintain average annual infiltration volume over the lots at pre-development levels and reduce post-development runoff volume to the Humber River tributary to which drainage from the site is directed. Native subsoil at the site is low permeability clayey silt glacial till which underlies the entire site. Mayfield Trenches 1, 2 and 3 receive runoff from roof drainage areas ranging from 14,962 m^2 to 23,268 m^2. Each infiltration trench, control manhole and contributing storm sewer pipe provides temporary storage of 126 m^3 of roof runoff. Based on this storage volume and the size of each roof drainage area, it can be estimated that Mayfield Trenches 1, 2 and 3 should be able to capture roof runoff from storm events up to 9.4, 7.0 and 6.0 mm in depth respectively, assuming 10% loss of water to evaporation. Mayfield Trench 4 receives runoff from a 14,420 m^2 roof area and provides temporary storage of 184 m^3 of runoff. Based on this storage volume and the roof drainage area, it can be estimated that Mayfield Trench 4 should be able to capture roof runoff from storm events up to 14 mm in depth, assuming 10% loss of water to evaporation. Seasonally high water table elevations in the vicinity of the trench system are estimated to be between 14 to 19 metres below the base of the trench system, so groundwater levels should not affect trench drainage times.

The Bramport infiltration chamber system is installed below a parking lot in a shopping centre development in Brampton, Ontario. The CULTEC Recharger V8HD® subsurface stormwater chamber system receives roof, road and parking lot runoff from a 33,500 m^2 drainage area via two storm sewer inlets and a catchbasin inlet directly above it and ultimately drains to a warm water tributary of the Humber River. The system was designed to provide temporary storage and controlled release of runoff from storms up to the 100 year return period event with the opportunity for infiltration. However, the water storage capacity of the system (1,192 m^3) was sized assuming no infiltration would be achieved, because the characteristics of the native subsoil and underlying geology encountered during geotechnical investigations suggested low levels of permeability. Native subsoil at the site is low permeability sandy silty clay glacial till which underlies the entire site. The chamber system covers an area of 1,520 m^2, including an 833 m^2 area below the western half of the chamber system where the underlying gravel bed is 0.3 m deeper than the bed below the remainder of the system and has no outlet. This gravel bed storage area provides 100 m^3 of water storage in addition to what was required to meet flood control requirements. It was included in the design of the system to act like a permanent pool to provide better retention of sediment. It also allows monitoring of water levels to evaluate whether or not the additional gravel bed storage area drains between storm events and to estimate the post-development infiltration rate of the native subsoil if drainage does occur. Seasonally high water table elevations in the vicinity of the infiltration chamber system are estimated to be between 2.5 and 4.6 metres below ground surface and that perched groundwater likely occurs in places at shallow depths during wet seasons, which could periodically affect drainage times.

Monitoring Locations and Parameters

Monitoring at the Elgin Mills Crossing infiltration chamber system site was initiated in September 2008 and continued to the end of July 2011. Rainfall was measured with a three season tipping bucket rain gauge located less than one kilometre south of the site. Winter precipitation data (daily totals) were taken from the Buttonville Airport meteorological station located approximately 5 km southeast of the site. An area velocity sensor was installed in a storm sewer pipe that conveys flows from the chamber system control manhole to downstream storm sewers to provide information on the volume of outflow from the system for each storm event and cumulatively over the whole monitoring period. Continuous water level measurements in the inlet side of the control manhole at 5 minute intervals using calibrated pressure transducers provided the basis for calculating drainage times and infiltration rates. A pressure transducer was also installed to collect continuous water level measurements in a well located 20 metres from the chamber system and installed to an elevation that corresponds to a depth of one metre below the base of the system. Well water level data was collected over a June 3, 2009 to December 13, 2010 monitoring period and provided the basis for determining if water table elevation reaches the base of the chamber system.

Monitoring at the Mayfield Industrial Park infiltration trench system site was initiated in July 2009 and continued to the end of June 2011. Rainfall was measured with a three season tipping bucket rain gauge located within 5 km of the site. Winter precipitation data (daily totals) were taken from the Lester B. Pearson International Airport meteorological station located approximately 20 km southwest of the site. Continuous water level measurements in the inlet sides of the control manholes at 5 minute intervals using calibrated pressure transducers provided the basis for calculating drainage times and infiltration rates. Pressure transducers were also installed on the outlet sides of the control manholes for a portion of the monitoring period to determine if the control manholes were leaking.

Monitoring at the Bramport infiltration chamber system site was initiated in June 2009 and continued to the end of July 2011. Rainfall was measured with a three season tipping bucket rain gauge located within 5 km of the site. Winter precipitation data (daily totals) were taken from the Lester B. Pearson International Airport meteorological station located approximately 12 km southeast of the site. Calibrated pressure transducers were installed in observation port wells that extend to the depth of the deepest portion of the gravel base in which the chambers were installed to estimate post-development infiltration rates in the undrained 0.3 m deep gravel bed area under the western half of the chamber system. An area velocity sensor was installed in the pipe connecting the control manhole to the chamber system to provide an indication of when flow from the chamber system into the control manhole had ceased, confirming that subsequent water level declines in the observation port well reflect losses to infiltration only.

Study Findings

At all the sites examined in this study, post-development infiltration rate of the underlying native subsoil was lower than expected resulting in slower than expected drainage of stored runoff and in some cases more frequent occurrence of overflow than anticipated. In the case of the Bramport system, it was observed that the portion of the gravel bed below the chamber system that has no outlet does not drain between storm events. In both the Elgin Mills Crossing and Mayfield Industrial Park infiltration systems it was observed that infiltration rates decreased exponentially as water levels (i.e. hydraulic head) in the systems declined. Observed infiltration rates for the Elgin Mills Crossing and Mayfield Industrial Park infiltration systems did not exhibit significant seasonal variation. This is due to the fact that the base of each system is approximately 2.5 to 3.5 metres below ground surface and therefore is well insulated from surface temperature fluctuations throughout the year. The infiltration systems examined required longer than 72 hours to fully drain once filled to capacity and the control manholes contained standing water for much of the year, which raises the question of whether or not they provide mosquito breeding habitat.

Although these systems did not achieve their drainage time design objectives, two of the three stormwater infiltration systems evaluated provided substantial reductions in runoff volume from their roof drainage areas. In the case of the Elgin Mills Crossing infiltration chamber system, even though the control manhole was observed to be leaking a small amount of water around the weir plate and the system drained slower than expected, monitoring indicates that during a normal precipitation year, the system reduces runoff from the roof drainage area in the order of 90% and infiltrates the volume of water necessary to match what would have infiltrated over the lot area prior to development. In the case of the Mayfield Industrial Park infiltration trench system, it was possible to conclude that the system is not achieving the design objective of fully compensating for the loss of infiltration caused by development of the site. In a normal precipitation year it is estimated that Mayfield Trenches 1 and 3 reduce runoff from their respective roof drainage areas in the order of 16% and Mayfield Trench 4 reduces roof runoff by approximately 36%. It was estimated through modeling that water storage capacity of the trenches would need to be between 1.9 and 4.5 times greater than the trenches currently provide in order to infiltrate the targeted volume of roof runoff that would mitigate the loss of infiltration on an average annual basis. If the

Mayfield trenches had been sized as such, it is estimated that they would reduce runoff from their respective roof drainage areas in the order of 53 to 61%. While the Mayfield Industrial Park infiltration trenches are draining more slowly than expected, results of this evaluation suggest that if they had been designed based on better knowledge of the permeability of the native subsoil, they could fully compensate for the loss of infiltration caused by the development through infiltration of roof runoff alone. This finding is significant considering that infiltration practices are widely considered to have limited effectiveness on fine-textured soils.

Deficiencies in the function of control manholes designed to retain water in the infiltration systems were observed in 3 of the 5 facilities monitored, which suggests that improvements to control manhole design, material specifications, or construction and inspection practices are warranted. Visual inspection of the stormwater infiltration systems at a time when the facilities were filled to a level above the elevation of the outflow sewer pipe invert would have revealed that some of the control manholes were leaking.

Sediment accumulation at the bottom of control manholes was also observed in 3 of 5 facilities monitored. While sediment accumulation was not found to be impacting the hydrologic performance of the facilities, it was causing issues with function of the control manholes for evaluating drainage times.

While results from the Elgin Mills Crossing and Mayfield Industrial Park systems support the implementation of stormwater infiltration systems on fine textured soils, monitoring results from the Bramport infiltration chamber system suggest that little or no runoff reduction benefit is being achieved by designing the system to provide the opportunity for infiltration, as was anticipated at the time of its design. It is possible that in the Bramport case, the infiltration chamber system has raised the seasonally high water table or that a perched water table has been created that does not dissipate or drains so slowly that the inter-event periods during this monitoring study were not long enough to observe substantial change in water levels. While results from this study could not confirm whether or not a perched water table has been created, they highlight the importance of careful consideration of the underlying stratigraphy and predevelopment water table elevation when deciding on suitability of the site for stormwater infiltration practices.

Recommendations

1. It is strongly recommended that stormwater infiltration facilities be thoroughly inspected by the construction project manager, system designer or ultimate owner/manager of the infrastructure prior to assumption (i.e. acceptance). Inspection procedures should include continuous water level monitoring over several storm events or a synthetic runoff test to determine if the system is functioning as designed. Contracts that include construction of such stormwater infrastructure should include conditions whereby any defects or deficiencies revealed through final inspection and testing can be corrected prior to assumption.
2. Leakage of water from the inlet side to the outlet side of the control manholes was observed in 2 of the 5 facilities indicating that sealing of the joints between components used to construct the manholes needs to be improved or given more attention during construction.

3. In the control manholes, including an outlet with a valve through the weir plate that can be operated from the outlet side would better facilitate inspection and maintenance by allowing the system to be drained via gravity.
4. Incorporating sumps in the control manholes would help prevent clogging of the bottom perforated pipes from sediment accumulation.
5. Since infiltration rates observed when the systems were full or nearly full were approximately 2.5 times higher than when the systems were half full or less, stormwater infiltration practices to be located on fine-textured soil should be designed to maintain hydraulic head in the water storage reservoirs for longer than the typical target of 48 to 72 hours. This would help maximize the drainage rate and thereby, the volume of water infiltrated on an annual basis. On low permeability, clayey silt soils like those occurring at the Mayfield Industrial Park site this means designing infiltration systems that never fully drain between storm events.
6. A practice that would improve the runoff reduction effectiveness of underground stormwater infiltration systems located on low permeability soils while providing an additional benefit of conserving potable water is to install a submersible pump in the inlet side of the control manhole to draw on the stored water for uses not requiring potable water (e.g. landscape irrigation, vehicle washing), like a rainwater harvesting cistern.
7. In future designs of stormwater infiltration systems located on low permeability soils, it is recommended that the type of covers used on control manholes should contain no holes at all, or screens to help ensure mosquitoes cannot enter them.
8. In locations with fine textured subsoil and seasonally high water table or bedrock surface at least 3.5 metres below ground surface, design criteria for stormwater management systems servicing industrial, commercial and institutional developments, where roof area represents 50% or greater of the lot area, should include maintaining average annual pre-development infiltration volume over the lot through infiltration of roof runoff, where feasible.

Topics For Future Research

1. *Medium to long term evolution of hydrologic performance*. This study examined hydrologic performance of underground stormwater infiltration practices located on fine textured soils within a short time period following their construction (less than 5 years post-construction). Little information is currently available in published reports regarding their performance over longer periods of service. Monitoring drainage times of stormwater infiltration practices located on fine-textured soil over the medium term (10 years post-construction) to long term (20 years post construction) to examine how hydrologic performance evolves over time is of interest to better understand the useful lifespan of such facilities and possibly to evaluate what maintenance or rehabilitative procedures could be undertaken to restore their effectiveness, other than complete reconstruction.
2. *Presence of mosquito larvae*. While grab samples from control manholes of the underground infiltration practices examined in this study indicated no presence of mosquito larvae, considering that such practices may contain standing water for much of the year, additional sampling of a larger number of facilities is of interest to evaluate whether or not they provide mosquito breeding habitat.

3. *Routine operation and maintenance*: Monitoring sediment accumulation in pretreatment structures and control manholes of underground stormwater infiltration practices is of interest to determine accumulation rates, evaluate quality and disposal options and estimate cost of inspection and maintenance over the lifespan of the facilities.

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1.0 INTRODUCTION

1.1 Background

Conventional urban development alters the natural hydrologic cycle, or water balance, through the creation of hard surfaces (e.g., roofs and pavement) that prevent precipitation from infiltrating into the soil. In particular, urbanization increases the portion of annual precipitation that becomes surface runoff and decreases the portions that infiltrate and evaporate. This increase in runoff results in higher stream flow rates and volumes during wet weather and causes the stream channels to adjust their form through erosion of the bed and bank materials to accommodate the changed flow regime. Decreased infiltration can result in lower groundwater levels in shallow aquifers and reduced input of cool, clean groundwater to streams. Such changes, combined with new sources of water pollution and changes to sediment inputs that typically accompany urbanization, can significantly degrade aquatic ecosystems and impair human uses of downstream water resources.

While traditional stormwater management facilities such as detention ponds and constructed wetlands have helped to reduce peak flows and improve water quality to some extent, they do not address alterations to the water balance brought about by urbanization (Aquafor Beech, 2006). Progressive management approaches sometimes referred to as low impact development, sustainable urban drainage systems or water sensitive urban design, attempt to maintain the pre-development water balance through site planning and engineering techniques aimed at infiltrating, filtering, evaporating and detaining runoff, as well as preventing pollution (CVC & TRCA, 2010). Stormwater infiltration practices are an integral part of these progressive management approaches. These practices help to reduce the volume of runoff discharged to watercourses thereby minimizing flood risk and preventing alterations to the stream flow regime and channel form. They help to maintain groundwater levels and sustain stream flows during dry periods. They also reduce pollutant loading to receiving watercourses from contaminated runoff by retaining or breaking down pollutants in the engineered structures and underlying native soil.

Stormwater infiltration practices can be designed for application at the ground surface (e.g., permeable pavement, bioretention, infiltration basins) or below the ground surface (e.g., infiltration chambers, trenches, soakaways, and pervious pipe exfiltration systems). An advantage of underground infiltration technologies is that they can be located below parking lots, roads, parkland or other landscaped areas. In densely developed urban areas, where the value of land is very high, this makes them preferable to surface technologies.

Suitability of a given site for application of stormwater infiltration practices depends on a number of factors. Infiltration practices should not be located on contaminated soils, nor on steep slopes. In order to provide ample opportunity for infiltrating runoff to interact with the soil and thereby attenuate contaminants, the seasonally high water table should be at least one metre (m) below the base of the infiltration facility (OMOE, 2003; CVC & TRCA, 2010). In Ontario, guidelines recommend that the infiltration rate of the native soil should be at least 15 millimetres per hour (mm/hr) (OMOE, 2003) otherwise a subdrain is required. Stormwater manuals from other jurisdictions in Canada and the northeastern U.S. suggest a much lower minimum infiltration rate (*e.g.*, PDEP, 2006) or do not recommend infiltration rate as a criterion for site suitability (*e.g.*, BC MWLAP, 2002; MPCA, 2005).

1.2 Infiltration Chambers

Infiltration chambers are a type of stormwater infiltration practice that includes a range of proprietary manufactured modular structures installed below the ground surface, typically under parking or landscaped areas, which temporarily store stormwater runoff allowing it to infiltrate into a surrounding gravel bed and the underlying native soil. Structures usually have open bottoms and perforated side walls and can be installed individually or in series in trench or bed configurations and arranged to best mimic the pre-development infiltration conditions (e.g., StormTech, 2009). They can be designed to withstand loads from heavy vehicles by adapting the depth of fill material over the chambers. They are well suited to commercial, industrial or institutional lots and densely developed areas where lands available for surface best management practices are limited. In addition to the storage available in the chambers, the gravel bed in which they are installed and the pipes that direct runoff to them provide additional water storage capacity. Chamber systems provide greater water storage per unit of volume than soakaways and infiltration trenches, which are typically filled with coarse granular material that provides between 30 to 40% void space. Infiltration chambers are well suited to treatment of relatively clean stormwater, such as roof runoff. However, when pretreatment is provided through hydrodynamic separators (i.e., oil and grit separators), in-line filters or grassed swales they may also be applied successfully to treat runoff from parking lots and low traffic roads, which typically generate higher loads of sediment and other pollutants than roofs.

There are few published field monitoring studies evaluating the runoff reduction performance of infiltration chambers, trenches and soakaways, particularly in cold climates and on fine-textured soil with low infiltration rates, like the glacial till soils that occur over much of southern Ontario. This is not surprising considering that many designers of stormwater management systems do not recommend the application of infiltration practices on fine textured soils due to their limited permeability and the size of facilities that would be required to meet regulatory requirements. However, there have been several studies of perforated pipe systems installed in Ontario as part of the stormwater conveyance system, which reduced runoff in a manner similar to infiltration chambers, trenches and soakaways.

In studies examining two residential neighborhoods with perforated pipe systems constructed in 1993 in Nepean, Ontario, runoff volumes have been observed to be between 94% and 70% less than a similar conventional pipe system, with little reduction in performance over the 20 year period of monitoring (Paul Wisner and Associates., 1994; J.F. Sabourin and Associates, 1999; J.F. Sabourin and Associates, 2008). In 1998, peak flows were 90% less than those observed for the conventional system and runoff volumes were 94% and 70% of the conventional system flows (J.F. Sabourin and Associates, 1999). In 2006, peak flows were between 47% and 86% less than those from the conventional system and runoff volumes were 86% and 73% of the conventional system flows (J.F. Sabourin and Associates, 2008).

Performances of two perforated pipe systems in Etobicoke and North York, Ontario that receive roof and road runoff from low density residential areas were also examined with regard to effects on runoff quantity and quality (SWAMP, 2002). Soils at the Etobicoke exfiltration system were clay to clayey-silt till over silty sand (infiltration rate between 0.004 to 36 mm/hr). Soils at the North York system were silty sand (infiltration rate between 72 to 288 mm/hr). The systems were found to be effective in exfiltrating most of the runoff directed into the perforated pipes, exceeding their design criteria. The Etobicoke and North York exfiltration systems were observed to exfiltrate 95% and 89% of all runoff from storms greater than 5 millimetres (mm), respectively over the two years of monitoring (SWAMP, 2005; SWAMP, 2002). High exfiltration rates to soils under the Etobicoke system were attributed to the presence of local sand lenses or fissures in the native clay soil matrix (SWAMP, 2005). The North York system had lower exfiltration due to the presence of a small pipe draining the trench at the downstream end of the system.

While many design professionals and regulators recommend that stormwater infiltration practices should not be implemented on fine-textured soils of low permeability, recent evaluations of bioretention and permeable pavement installations indicate that infiltration of substantial volumes of runoff can be achieved in some contexts. In a North Carolina case study, preliminary results from monitoring the performance of newly installed rain gardens in a residential community indicates that they can be effective infiltration practices, even on soils with high clay content (Estes, 2009). The rain gardens were located on sandy clay soil (approximately 37% clay, 27% silt and 24% sand) where pre-development infiltration rates ranged from 29 to 38 mm/h, with an average rate of 33 mm/h, and were designed to retain and infiltrate the two-year design storm (a 79 mm event). After 4.5 months of monitoring, including 37 storm events of up to 38 mm in depth, the average infiltration rate through the facility was 7 mm/h, with the rate increasing to 25 mm/h in the underlying native soil, once water levels were past the bottom of the installed soil mixture and geotextile filter fabric (Estes, 2009). In the same study, a porous concrete pavement installation on sandy clay soils (approximately 45% clay, 25% silt and 30% sand) was observed to achieve an average infiltration rate of 2 mm/h over a two month monitoring period. Observed infiltration rates reached a maximum of 7 mm/h during periods of maximum head (i.e., maximum height of water stored in the gravel reservoir) and declined as reservoir water levels declined (Estes, 2009).

Widespread adoption of stormwater infiltration practices has been lacking in cold climate regions largely due to concerns about poor winter performance relating to frozen filter media. The question of seasonal effects on hydrologic performance of various stormwater practices prompted investigations to be undertaken by the University of New Hampshire Stormwater Center. The seasonal functionality of six different best management practices installed in parallel, including an underground infiltration chamber system were examined. With regard to seasonal impacts on hydraulic efficiency, as indicated by peak flow reduction and lag time, it was found that the underground infiltration system showed the least seasonal variability (Roseen et al., 2009). This was attributed to its location almost two metres below ground surface, which is below the local maximum frost penetration depth (Roseen et al., 2009).

Considering the fact that much of the planned and potential urban growth areas around the Greater Toronto Area are located on glacial till soils, field monitoring evaluations of the effectiveness of three underground stormwater infiltration systems installed on such fine-textured soils to reduce runoff volume were undertaken by the Sustainable Technologies Evaluation Program (STEP).

1.3 Study Objectives

This study evaluates the hydrologic performance of three underground stormwater infiltration systems in relation to their original design objectives and provides recommendations on the conditions under which this technology can be an effective means of maintaining pre-development infiltration volume and reducing runoff volume from development sites. This final report presents field monitoring results from systems located in Richmond Hill, Bolton and Brampton, Ontario collected between September 2008 and August 2011.

Parallel monitoring of the three systems allows for direct comparison of performance under varying soil and geological contexts. The Brampton and Bolton sites represent contexts in which stormwater infiltration technologies are often not recommended in other jurisdictions due to the low permeability of the native soil and underlying geologic deposits. Therefore, these evaluations provide evidence regarding whether or not this technology can be effective in such contexts.

2.0 ELGIN MILLS CROSSING INFILTRATION CHAMBER SYSTEM

2.1 Site Description

The Elgin Mills Crossing infiltration chamber system is installed under the parking lot of a shopping centre development in Richmond Hill, Ontario. The site is bordered by Leslie Street to the west, Elgin Mills Road to the south, and a tributary of the Rouge River to the north and east (Figure 2.1), which ultimately receives flow from the stormwater management system. The chamber system receives roof runoff from two large commercial buildings (Buildings A and B) with a combined roof drainage area of 25,449 m^2. The StormTech SC-740® stormwater infiltration chamber system is in a rectangular configuration and located in the northeastern portion of the development site. Outflow from the chamber system and runoff from paved surfaces and landscaped areas associated with the development is conveyed by storm sewers to a stormwater detention pond. The pond provides enhanced water quality and post-to-pre-development peak flow control for the 100 year storm event. The pond drains to a tributary to the main branch of the Rouge River, which ultimately flows to the north shore of Lake Ontario.

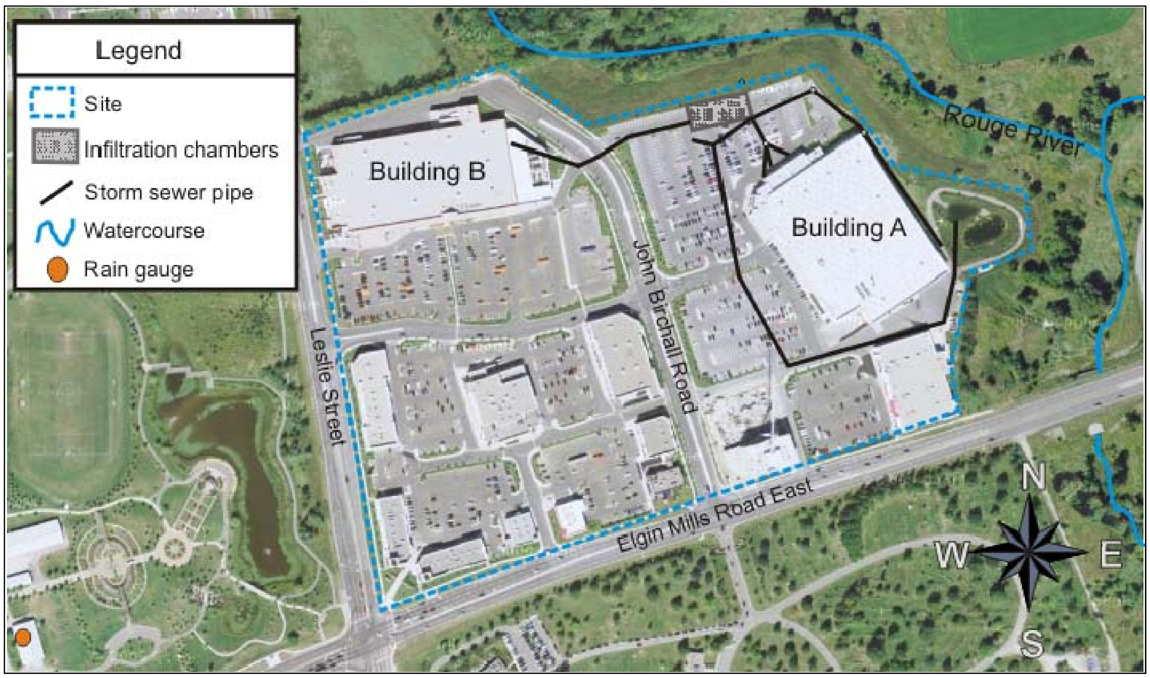


Figure 2.1: Elgin Mills Crossing infiltration chamber system site

Prior to development of the site, groundwater in underlying aquifers was recharged from infiltrating precipitation on the site. Detailed hydrogeological investigations show that the groundwater flows northeast to discharge into the adjacent Rouge River tributary, which supports cold water aquatic habitat.

An infiltration chamber system was installed to maintain the post-development volume of infiltration that occurs over the site to pre-development levels, thereby helping to maintain aquifer water levels and groundwater discharge to the Rouge River tributary, and minimize the increase in runoff caused by the development. This was the preferred option because it does not require dedication of land for surface stormwater management practices and is well suited to the geology of the site. Based on water budget analyses by Dillon Consulting (2006), during an average year when total depth of precipitation is 893 mm, the volume of runoff from the developed site that would need to be captured and infiltrated to maintain the annual infiltration volume to pre-development conditions is estimated to be 14,494 m^3.

The geology of the site is characterized by sandy silt glacial till deposits, which underlie the entire site. Based on grain-size analyses of soil samples taken from 7 boreholes at various locations across the site the till consists of about 40 to 45% fine to coarse sand, 35 to 45% silt and 20 to 25% clay (Dillon Consulting, 2006). Using the Hazen correlation (Freeze and Cherry, 1979) the hydraulic conductivity (K) of the sandy silt till deposits was estimated to be 1×10^{-6} centimetres per second (cm/s) (Dillon Consulting, 2006), which represents an infiltration rate of 12 mm/h based on their approximate relationship (OMMAH, 1997). The low permeability sandy silt till deposits overlie a lens-shaped deposit composed mainly of more permeable silty fine sand at shallow depths. The sand lens averages about 2.5 m in thickness and is very heterogeneous in its composition with regard to grain-size distribution and hydraulic conductivity. Based on constant-head tests in 5 boreholes, the hydraulic conductivity of the sand lens is estimated to be 1×10^{-3} cm/s (Dillon Consulting, 2006), which represents an infiltration rate of approximately 75 mm/h (OMMAH, 1997). Estimates of hydraulic conductivity from a borehole closest to the location of the infiltration chamber system, based on constant-head test and grain-size analysis, support this estimate (9×10^{-4} cm/s and 4×10^{-4} cm/s, respectively).

The chambers were strategically located near the river valley over an area where the gravel base could intersect with the sand lens to help ensure that pre-development groundwater discharge rates and volumes to the Rouge River would be maintained (Dillon Consulting, 2006).

2.2 Infiltration System Design

As shown in Figure 2.2, the Elgin Mills Crossing infiltration chamber system receives runoff from the roofs of two large retail buildings. The roof areas of Building A and B are 14,217 and 11,232 m^2, respectively, comprising a total roof drainage area of 25,449 m^2. The chamber system, including the gravel bed in which it is embedded, covers an area of 1,292 m^2 and stores up to 840 m^3 of stormwater (StormTech, 2006a). An additional 115 m^3 of storage is available in the contributing storm sewers. The two inlets are on the south and west sides of the chamber system. Outflows from the system are controlled by a weir plate installed in the control manhole at the south inlet to the system (Figure 2.2). The top of the weir plate is set at the same elevation as the top of the chamber system (231.55 metres above seal level (masl)). When the chambers are full, flows overtop the weir plate and outlet to the conventional sewer system (Figure 2.3), which drains to the stormwater pond. Accumulated runoff is temporarily stored in the chamber system until it infiltrates into the native soil.

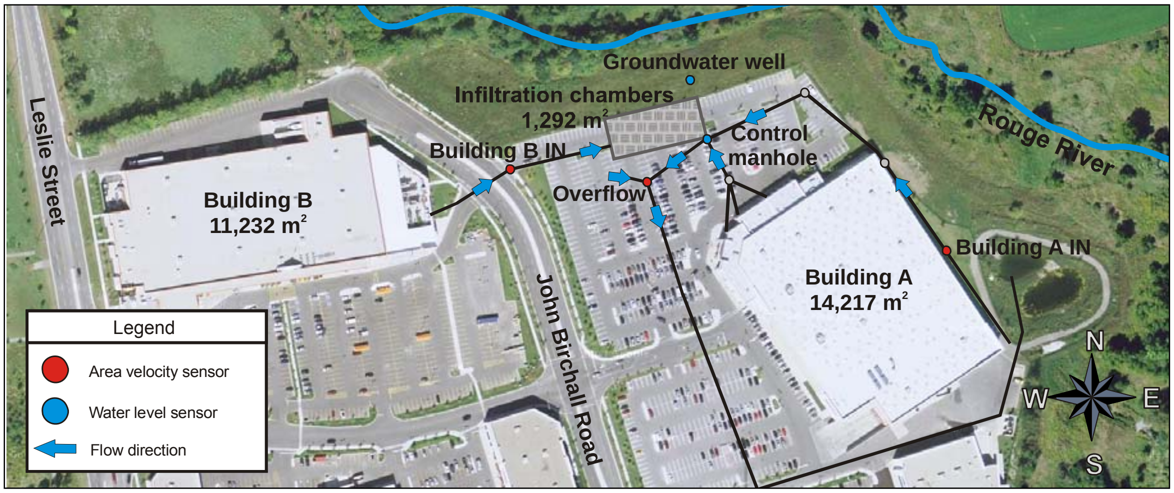


Figure 2.2: Location and configuration of Elgin Mills Crossing infiltration chamber system

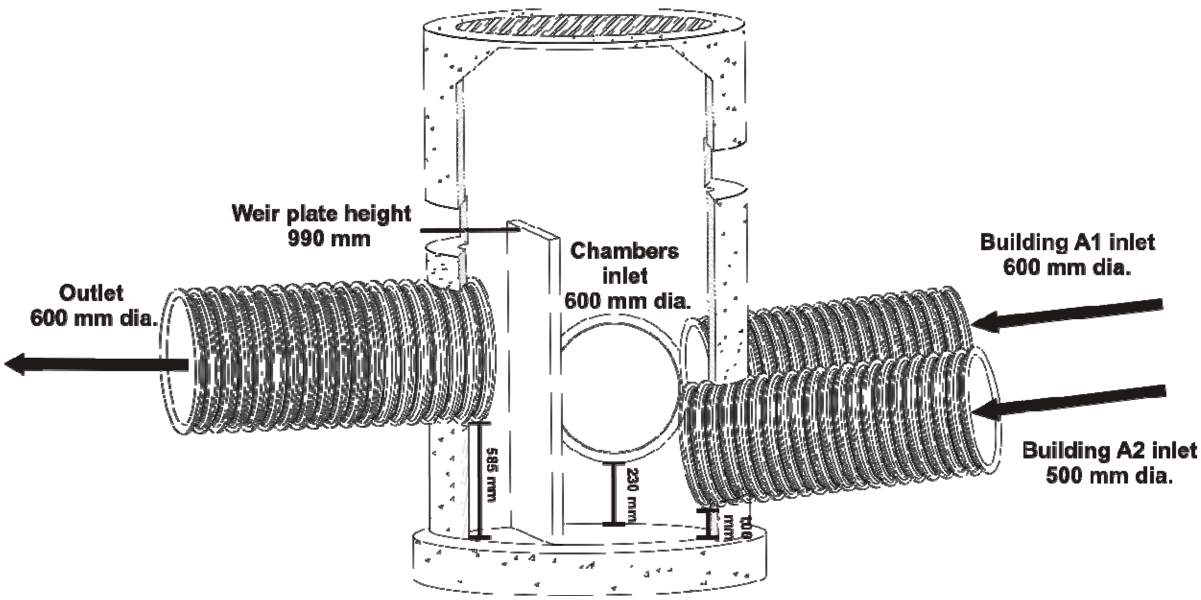


Figure 2.3: Elgin Mills Crossing infiltration chamber system control manhole design (Adapted from StormTech, 2006b; not to scale)

The chambers are embedded in a gravel bed filled with 20 to 50 mm diameter clear crushed stone with an assumed void space ratio of 40%, to provide storage and structural support to the chamber system and overlying pavement. A layer of well-graded soil and aggregate mixture is placed between the top of the gravel bed and parking lot pavement bedding. The gravel bed is lined with a non-woven geotextile fabric (ADS 601 or equivalent fabric that meets AASHTO M288 Class 2 specifications; AASHTO, 2002) to provide separation between the clear stone material in the gravel bed and the underlying native soil and overlying soil and aggregate mixture (Figure 2.4).

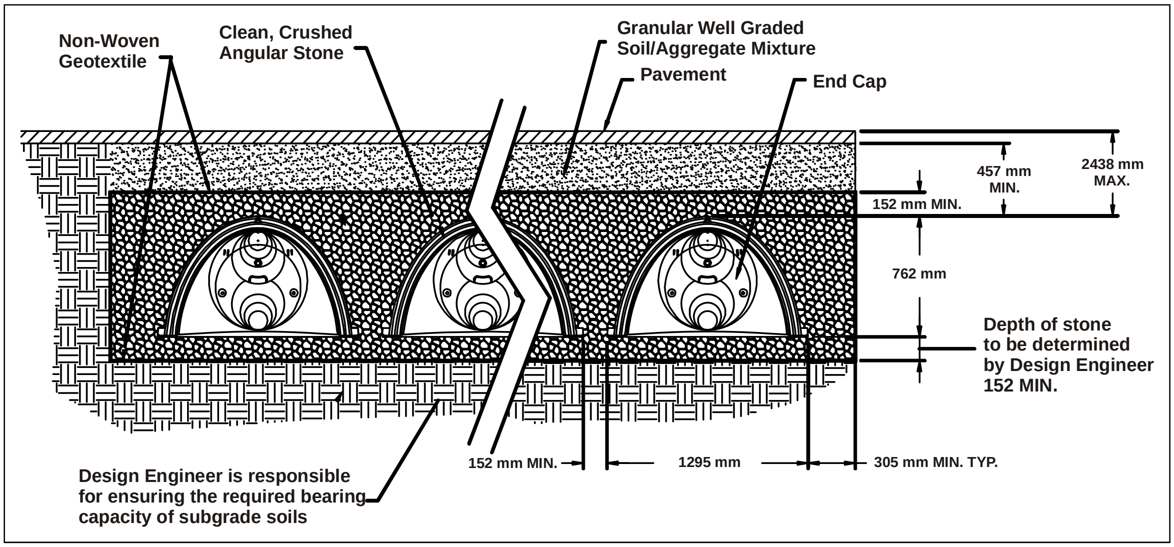


Figure 2.4: Cross section detail of Elgin Mills Crossing infiltration chamber system (Adapted from StormTech, 2006c)

The infiltration chamber system and contributing storm sewers were designed to temporarily store 840 and 115 m^3 of stormwater, respectively, which combined, provide 955 m^3 of storage (Table 2.1). This is roughly equivalent to a 41 mm precipitation event over the roof drainage area, assuming 10% loss of precipitation to evaporation. Assuming the base of the chamber system penetrated the fine sand lens and that the estimated pre-development infiltration rate of 75 mm/h was maintained in post-development conditions, drainage of stored runoff from an event of this size would occur over roughly 9 hours. If the base of the chamber system was installed in the lower permeability sandy silt till and the estimated pre-development infiltration rate of 12 mm/h was maintained, drainage would occur over roughly 62 hours (approximately two and a half days). Considering that greater than 99% of precipitation events in this region are less than 40 mm in depth and the average intervening period between storm events is approximately 3 days (City of Toronto, 2006), most of the annual precipitation that runs off the two roofs can be expected to infiltrate if infiltration rates in the order of 12 mm/h or greater are achieved.

The obvert of inlet pipes to the chamber system are approximately 1.4 m below ground surface which is the limit of the local maximum frost penetration depth (MTO, 2005) and therefore, should not be subject to freezing during cold winter temperatures.

The base of the chamber system was designed to be installed at an elevation of 230.5 masl so that it intersects with the underlying higher permeability sand lens deposits, which continue to the northeast and are known to outcrop along the channel of the adjacent Rouge River tributary which is a groundwater discharge zone. The elevation of the top of the sand lens in the vicinity of where the chamber system is located is predicted to be between 230 and 231 masl, based on interpretation of available borehole data (Dillon Consulting, 2006).

Table 2.1: Elgin Mills Crossing infiltration chamber system and drainage area specifications

Impervious drainage area	25,449 m^2
Runoff source area type	Roof only
Location of chamber system	Under parking lot for Building A
Chamber system footprint area	1292 m^2
Chamber system storage volume [1]	840 m^3
Contributing storm sewer storage volume [2]	115 m^3
Combined storage volume	955 m^3
Combined storage volume per hectare of impervious drainage area	375.3 m^3/ha.
Ratio of impervious drainage area to chamber system footprint area	20:1
Hydraulic conductivity of sandy silt till [3]	$1x10^{-6}$ cm/s
Approximate infiltration rate of sandy silt till [4]	12 mm/h
Hydraulic conductivity of fine sand lens [5]	$1x10^{-3}$ cm/s
Approximate infiltration rate of fine sand lens [4]	75 mm/h
Gravel bed fill material	Clear 20-50 mm diameter, crushed stone (assumed 40% void space)

Notes:
1. StormTech, 2006a.
2. Estimated from Site Servicing Plan Drawing G2 (A.M. Candaras, 2007b) based on a high water elevation of 231.55 masl.
3. Based on the geometric mean of values estimated from grain-size analyses of soil samples from 7 boreholes within the development site (Dillon Consulting, 2006).
4. Based on approximate relationship between hydraulic conductivity and infiltration rate (OMMAH, 1997)
5. Based on the geometric mean of values from constant-head tests in 5 boreholes screened in the sand lens within the development site and confirmed by the mean value of estimates from grain-size analyses of soil samples from the same boreholes (Dillon Consulting, 2006).

The seasonally high water table in the vicinity of the chamber system is estimated to be between 2 to 3 m below the ground surface in April and May (Dillon Consulting, 2006). A high water table elevation of 230.6 masl was observed in early May 2006 in a monitoring well located a short distance down-gradient from the chamber system prior to construction. By July 2006, groundwater levels in the well had receded to 230.0 masl (Dillon Consulting, 2006). This suggests that groundwater levels may routinely be within one metre of the gravel bed base of the chamber system and may reach elevations above the base during the spring. It is recommended that stormwater infiltration systems should be designed and installed to maintain a minimum of one metre separation between the base elevation of the system and the seasonally high water table elevation. This separation distance is intended to ensure that any contaminants reaching the native soil underlying the system do not directly interact with the water table which could increase their mobility and leach them further into the soil profile.

2.3 Monitoring Locations and Equipment

Monitoring at the Elgin Mills Crossing site was initiated in September 2008 and continued to the end of July 2011. Rainfall (5 minute interval precipitation totals) was measured with a three season 8 inch diameter tipping bucket rain gauge located at Richmond Green (1300 Elgin Mills Road, Richmond Hill), less than one kilometre (km) south of the site (Figure 2.1). A second gauge located at a York Region Public Works yard approximately 5 km northwest of the site served as a back-up in case the primary gauge malfunctioned. Winter precipitation data (daily precipitation totals) were taken from the Buttonville Airport meteorological station located approximately 5 km southeast of the site.

Area velocity sensors were installed in the three storm sewer inlet pipes directing water from the roofs of Building A and B into the chamber system, as well as in the pipe that conveys outflows from the chamber system control manhole to downstream storm sewers (Figure 2.5). The flow modules provided continuous data on water level, flow and velocity at 5 minute intervals. However, backflows in the inlet pipes created conditions unsuitable for flow measurement; hence inflow volumes were determined from rainfall depth and roof drainage areas using a calibrated roof runoff model developed through a previous STEP study (TRCA, 2010). The outflow sensor was not subject to backflow conditions and provided reliable flow measurements throughout the monitoring period. The sensor in the outflow storm sewer pipe remained in service since September 10, 2008, with the exception of the period from August 20 to 27, 2009, during which times it was out of service for maintenance.

Continuous water level measurements using calibrated pressure transducers provided the basis for calculating infiltration rates and drainage times after rain events. Water levels were measured on the inlet side of the control manhole with the sensor set at a minimum depth that corresponds with the bottom of the manhole.

A pressure transducer was also installed to collect continuous water level measurements in a well located 20 m from the chamber system and installed to 229.50 masl which corresponds to a depth of one metre below the base of the system. Well water level data was collected over a June 3, 2009 to December 13, 2010 monitoring period and provided the basis for determining if water table elevation reaches the base of the chamber system.

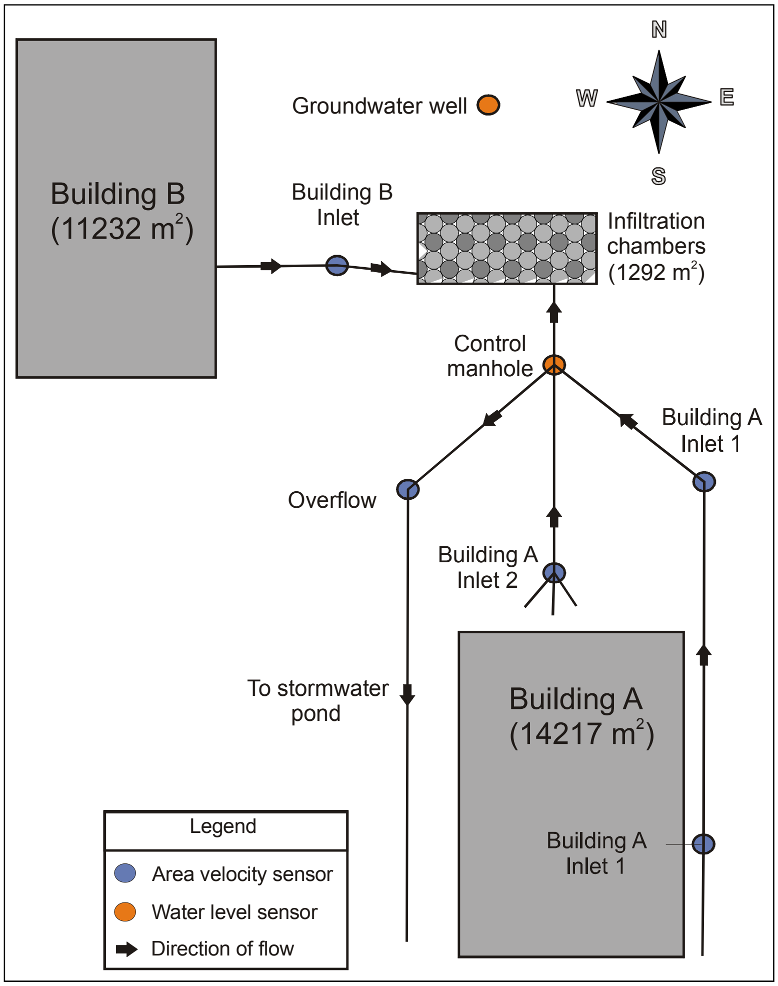


Figure 2.5: Simplified schematic of monitoring locations (not to scale)

2.4 Results

2.4.1 Precipitation

Figure 2.6 illustrates monthly totals for precipitation over the September 2008 to December 2011 monitoring period from the Richmond Green and Buttonville Airport rain gauges and 30 year normals (monthly averages from 1971 to 2000) from the nearest rain gauge with a sufficient period of record ("Richmond Hill" climate station; Environment Canada, 2010). While annual total precipitation over the 2008 to 2011 period remained within 10% of the long term average, monthly and seasonal totals often deviated significantly from 30 year climate normal values. Total precipitation depth during the spring (April to June) of 2009 was 24% higher than normal owing to much wetter weather during April and May. This was followed by a much drier than normal fall (October to December) 2009 season when precipitation in November was 56% less than the long term average. Overall in 2009, total precipitation depth at the Elgin Mills Crossing site was 899 mm which is very close to the long term average of 893 mm for the region (Environment Canada, 2010). The dry fall of 2009 was followed by a very dry winter of 2010 when precipitation was 41% less than normal. Drier than normal weather continued during April and May of 2010 but was more than made up for in June when a total of 206 mm of precipitation fell, breaking monthly maximum rainfall records in the region. Overall in 2010, total precipitation depth at the site was 840 mm, representing a drier than normal year. Spring of 2011 was also one of the wettest on record with much greater precipitation than normal occurring during May and June. This wet spring was followed by a very dry month of July 2011 when precipitation was 53% less than normal.

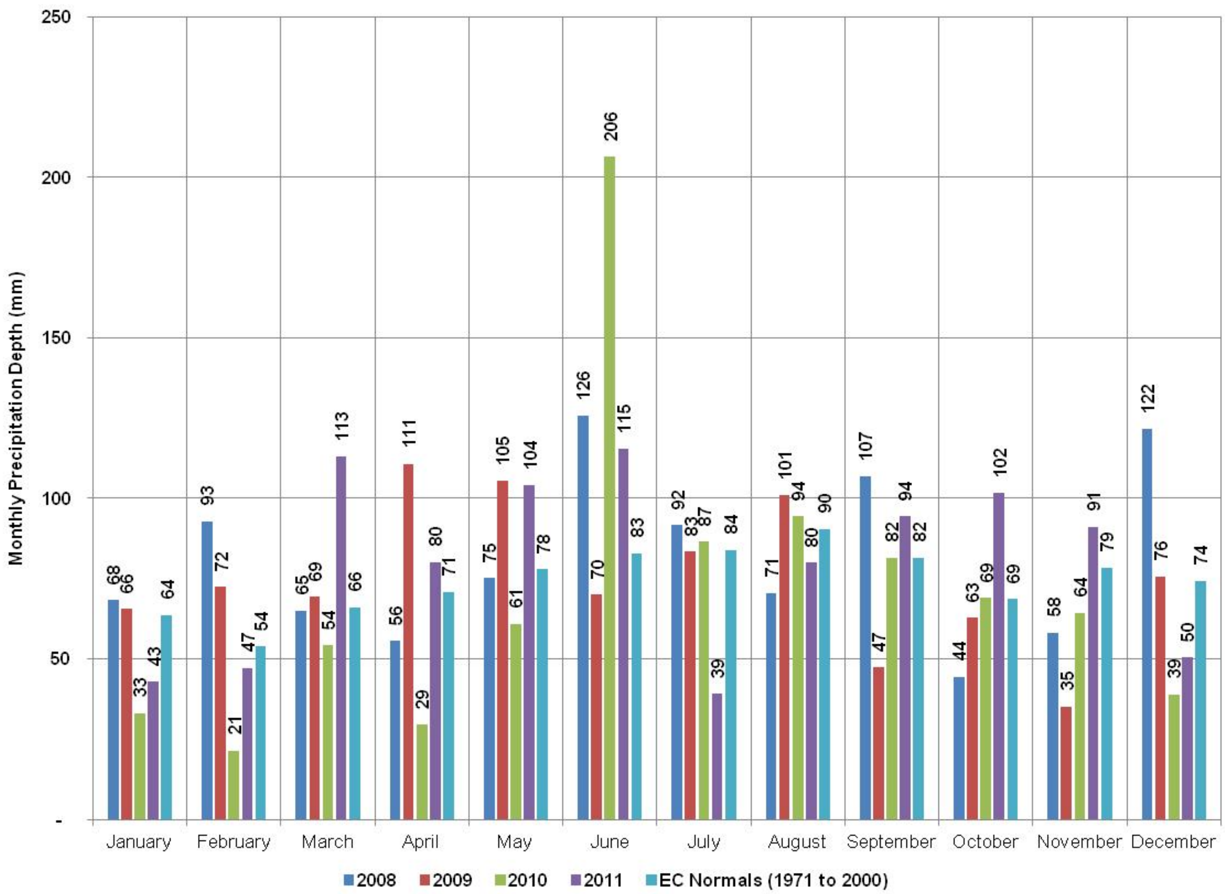


Figure 2.6: Monthly precipitation totals – Elgin Mills Crossing site.
Note: Precipitation data for the Elgin Mills Crossing site was obtained from tipping bucket rain gauges located at Buttonville Airport (5 km from the site) during winter months (daily precipitation totals) and Richmond Green (less than 1 km from the site) during spring, summer and fall months (5 minute interval precipitation totals).

Table 2.2 provides a breakdown of the number of storm events according to precipitation depth ranges (i.e. bins) that occurred over the September 2008 to July 2011 monitoring period and the number of events that caused outflow from the infiltration chamber system, as indicated by water level in the control manhole or flow being detected by the area velocity sensor installed in the outflow pipe. Over the combined monitoring period a total of 391 storm events[1] occurred. The largest storm event in terms of maximum depth of precipitation was 54.8 mm, occurring on July 23, 2010. The maximum intensity storms occurred on August 9, 2010 when precipitation was recorded at a rate of 26.0 mm/h between 4:00 PM and 5:00 PM and on June 22, 2011 when precipitation was recorded at a rate of 10.0 mm/5 minutes.

[1] Individual storm events were defined as periods of precipitation totaling a minimum of 0.4 mm depth and separated by a minimum antecedent dry period of 3 hours.

Over the monitoring period only 8 instances occurred where total precipitation depth over a 48 hour period exceeded the designed storage capacity of the infiltration chamber system (41 mm precipitation over the 25,449 m^2 roof drainage area). However, based on water levels in the control manhole, the chamber system was filled to capacity and flowing over the weir plate during 15 events over the monitoring period. Flows were also consistently detected in the outflow pipe when water levels in the control manhole were below the top of the weir plate. As described in Table 2.2 and Figure 2.7, flow was detected in the outflow pipe during the full range of events that occurred and that all events greater than 22 mm depth produced flow in the outflow pipe. Analysis of storm event hyetographs, described in the next section, showed that the system was infiltrating slower than the expected rate (12 to 75 mm/h) and that the control manhole was somehow leaking water from the inlet side of the weir to the outlet side. The higher than expected frequency of overflow events and flow being detected in the outflow pipe (i.e. outflow events) can be attributed to these two factors.

Table 2.2: Summary of storm events captured during the September 13, 2008 to July 31, 2011 monitoring period

Precipitation Depth Range (mm)	Total # of Events	Total # of Outflow Events	% of Events Causing Outflow
0.4 - 2.6	170	26	15
2.8 - 5.0	67	8	12
5.2 - 7.6	50	12	24
7.8 - 10.0	24	3	13
10.2 - 12.4	21	9	43
12.6 - 14.8	8	2	25
15.0 - 17.2	14	6	43
17.4 - 19.6	9	5	56
19.8 - 22.0	8	4	50
22.2 - 24.4	2	2	100
24.6 - 26.8	4	4	100
27.0 - 29.2	3	3	100
>29.2	11	11	100
Total	**391**	**95**	**24**

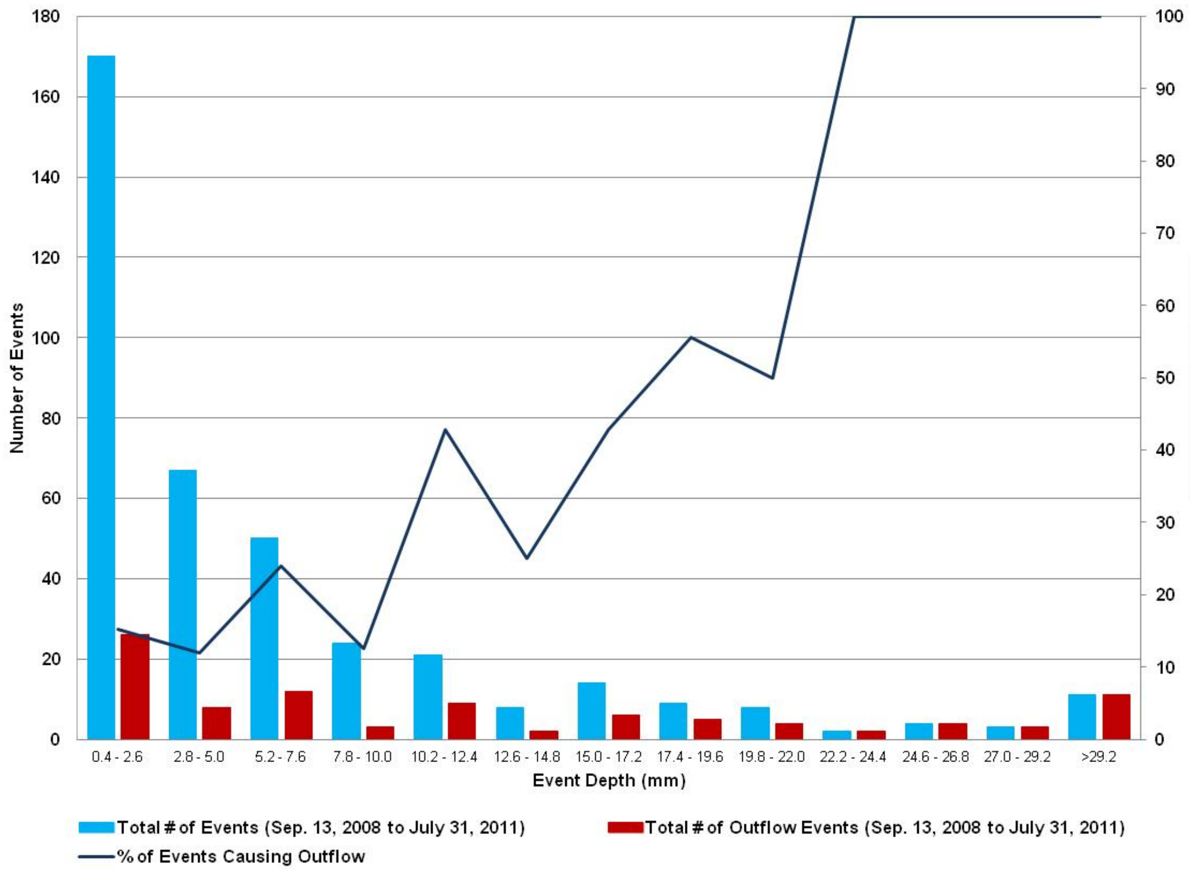


Figure 2.7: Elgin Mills Crossing infiltration chamber system frequency of outflow events by precipitation event depth bin

2.4.2 Infiltration

Performance of the Elgin Mills Crossing infiltration chamber system was evaluated with regard to the volume of water infiltrated, post-construction infiltration rate of the underlying native soil and water storage capacity. Precipitation and flow monitoring data were used to estimate the total volume of stormwater infiltrated over the monitoring period to evaluate the degree to which the design objective of maintaining pre-development infiltration volume is being achieved. Infiltration rates were estimated for a number of drainage periods and compared to estimates of pre-development native soil infiltration rates that were used as design criteria for sizing the facility.

It is assumed that the flow of water from the infiltration chamber system into the underlying native soil can be predicted by Darcy's Law (Equation 1; Darcy, 1856). Darcy's Law predicts that as the volume of water in the chamber system and contributing storm sewers decreases and exerts less pressure at the interface between the gravel bed of the chamber system and the underlying native soil that the rate of flow into the native soil will decrease. Therefore, it can be expected that infiltration rate will be highest when the chamber system and contributing storm sewers are full, as this will be when the pressure difference is greatest. It can also be expected that infiltration rate will decrease as water levels decline in the system, reaching a minimum when they are close to the bottom of the gravel bed.

Equation 1: $Q = - (\kappa * A/\mu) * (P_b - P_a)/L$

Where,

Q = Total discharge (m^3/s)
κ = Permeability of the medium (m^2)
A = Cross-sectional area (m^2)
μ = Dynamic viscosity ($kg/m \cdot s$)
$P_b - P_a$ = Pressure drop ($kg/m \cdot s^2$)
L = Length the pressure drop is taking place over (m)

For the Elgin Mills Crossing infiltration chamber system, estimates of infiltration rates achieved were generated based on the volume of runoff infiltrated over the period of time from the beginning of a storm event to the time when the water level in the system had returned to the pre-event level. Estimates represent average infiltration rates over the entire drainage period, acknowledging that actual rates are higher during periods when water levels in the chamber system are at their highest and lowest when water levels are near the base. For drainage periods where antecedent dry periods following the end of individual storm events were not long enough for water levels in the chamber system to return to pre-event levels, cumulative precipitation over the period during which drainage to pre-event levels occurred was used to calculate the total inflow volume over the drainage period.

Because backflow conditions occurred in locations where area-velocity flow meters were installed in the inlet pipes, it was not possible to accurately measure runoff volumes flowing into the chamber system over the monitoring period. Instead, inflow volumes were estimated on an event-by-event basis based on precipitation depth and assumptions regarding typical water yield from the roof drainage areas arrived at through development of a calibrated roof runoff model as part of a previous evaluation study examining the effectiveness of rainwater harvesting systems in the Greater Toronto Area (TRCA, 2010). Area-velocity flow meters installed in the outflow storm sewer pipe did not experience backflow conditions and produced reliable measurements of flow from the chamber system over the monitoring period. Therefore, infiltration volumes were calculated by subtracting measured outflow volume values from estimated inflow volume values on an event-by-event basis.

Table 2.3 provides a summary of inflow, outflow and infiltrated volume results, as well as overall runoff reduction ratio estimates for various periods over the course of the evaluation. Results are summarized for four different monitoring periods corresponding to: i) The period when the control manhole was leaking and allowing some water to flow around the weir plate from the inlet side to the outlet side; ii) The period following an attempt to reduce leakage around the weir plate; iii) The whole monitoring period; and iv) The period encompassing all of 2009, during which total precipitation depth was nearest to the long term average for the region. Absolute runoff reduction ratios calculated for these monitoring periods range from 0.83 to 0.96, indicating that despite some leakage in the control manhole, the chamber system was successfully capturing and infiltrating the majority of runoff from the roof drainage area. Following the attempt to reduce leakage in the control manhole the runoff reduction ratio increased suggesting that leakage was substantially reduced. The total volume of runoff that was infiltrated during the January 1 to December 31, 2009 monitoring period was estimated to be 14,941 m^3. This excludes flows during a week long period in August, during which time no measurements of outflow volume were available and approximately 47 mm of rainfall occurred. It is also based on the monitoring period prior to the attempt to reduce leakage in the control manhole, thereby representing a conservative estimate of total annual infiltration volume. Despite being a conservative estimate, it indicates that the objective of maintaining post-development infiltration volume to pre-development levels is being achieved, as the reduction in infiltration that would occur over the development site without mitigation, during an average precipitation year was estimated to be 14,494 m^3 (Dillon Consulting, 2006).

Table 2.3: Inflow, outflow and infiltrated volume summary for the Elgin Mills Crossing chamber system.

Parameter	Monitoring Period			
	Sept. 13, 2008 to July 14, 2010[1]	July 15, 2010 to July 31, 2011[2]	Sept. 13, 2008 to July 31, 2011[3]	Jan. 1, 2009 to Dec. 31, 2009[1]
Total Precipitation Depth (mm)[4]	1,421.3	903.4	2,324.7	800.7
Total Inflow Volume (m^3)[5]	32,958.3	21,261.9	54,220.2	17,953.2
Total Outflow Volume (m^3)	4,598.6	896.9	5,495.5	3,012.1
Total Infiltrated Volume (m^3)[6]	28,359.7	20,365	48,724.7	14,941.1
Runoff Reduction Ratio[7]	0.86	0.96	0.90	0.83

Notes:
1. During this period the control manhole was leaking water around the weir plate. Totals exclude precipitation and flows from August 20 to 27, 2009, during which time no measurements of outflow volume were available as the area velocity sensor installed in the outflow pipe was out of service.
2. This period follows efforts on July 15, 2010 to better seal the control manhole and reduce leakage around the weir plate. Runoff reduction effectiveness has been evaluated for this period separately from the previous period when leakage was occurring at a greater rate.
3. These results represent absolute runoff volume reduction achieved over the entire monitoring period, including periods when the control manhole was leaking water around the weir plate and when leakage was greatly diminished. Totals exclude precipitation and flows from August 20 to 27, 2009, during which time no measurements of outflow volume were available as the area velocity sensor installed in the outflow pipe was out of service.
4. Precipitation Depth data for the periods of September to October 2008, April to November 2009, April to November 2010 and April to July 2011 was from the Richmond Green rain gauge; Precipitation during November 2008 was from the York Works Yard rain gauge; Precipitation data for the periods of December 2008, January to March 2009, December 2009, January to March 2010, December 2010 and January to March 2011 was from Buttonville Airport meteorological station, and only available as daily totals.
5. Total Inflow Volume was estimated based on precipitation depth and assumptions regarding water yield from the roof drainage areas. Assumed 100% for rainfall events greater than 9 mm depth; 70% for 9 to 6.1 mm; 60% for 6 to 4.1 mm; 40% for 4 to 2.1 mm; and 30% for 2 mm or less (TRCA, 2010). Total Inflow Volumes include snowmelt from the roof during winter months. Snowmelt was estimated using a roof runoff model calibrated at a nearby site (TRCA, 2010).
6. Total Infiltrated Volume was calculated by subtracting Total Outflow Volume from Total Inflow Volume.
7. Runoff Reduction Ratio is calculated as the ratio of Total Infiltrated Volume to Total Inflow Volume, over the monitoring period.

As indicated previously, flow was often detected in the outflow storm sewer pipe during periods when the water level in the control manhole was below the top of the weir plate, suggesting that water was leaking from the inlet side of the control manhole to the outlet side around the weir plate. Evidence of this is provided by hyetographs plotted for selected storm events and drainage periods (Figure 2.8). Water level measurements in the inlet side of the control manhole are shown as the solid red line. Outflow occurred when water levels in the control manhole were above the invert of the outflow storm sewer pipe (585 mm). In 2009, water levels were also monitored on the outlet side of the control manhole, which confirmed that either the weir plate or manhole structure was leaking.

A scatter plot of storm event precipitation depth versus outflow provides further evidence of the performance of the chamber system over the September 13, 2008 to December 31, 2011 monitoring period (Figure 2.9). In this figure, blue diamonds represent individual storm event data. Outflow volume measured by the area-velocity flow sensor has been divided by the roof drainage area and converted into mm. The “1:1 Ratio” line represents the performance that would be expected if no volume reduction or addition occurred in the chamber system. As shown in Figure 2.9, two points are above the line, representing events where measured outflow was greater than theoretical inflow. These events occurred during winter and are due to snow that had accumulated on the roofs being converted to snowmelt at a time when the system was filled to capacity and overflowing over the weir plate. The red line represents the percentage of total precipitation depth over the monitoring period that occurred as events less than or equal (LTE) to the event precipitation depth. Had the chamber system been performing as designed (i.e. capable of capturing and infiltrating events up to 41 mm depth) it would have been able to fully capture and infiltrate most events (approximately 98%) over the period with little or no outflow. However, the scatter plot shows that only events up to about 11 mm precipitation depth were fully captured and infiltrated on a consistent basis. Events of 11 mm depth or less accounted for 40% of the total precipitation that occurred during the monitoring period. The scatter plot also shows that although some outflow was routinely observed for events greater than 11 mm in depth, the chamber system routinely produced very little outflow in relation to inflow, indicating favorable performance in terms of runoff volume reduction.

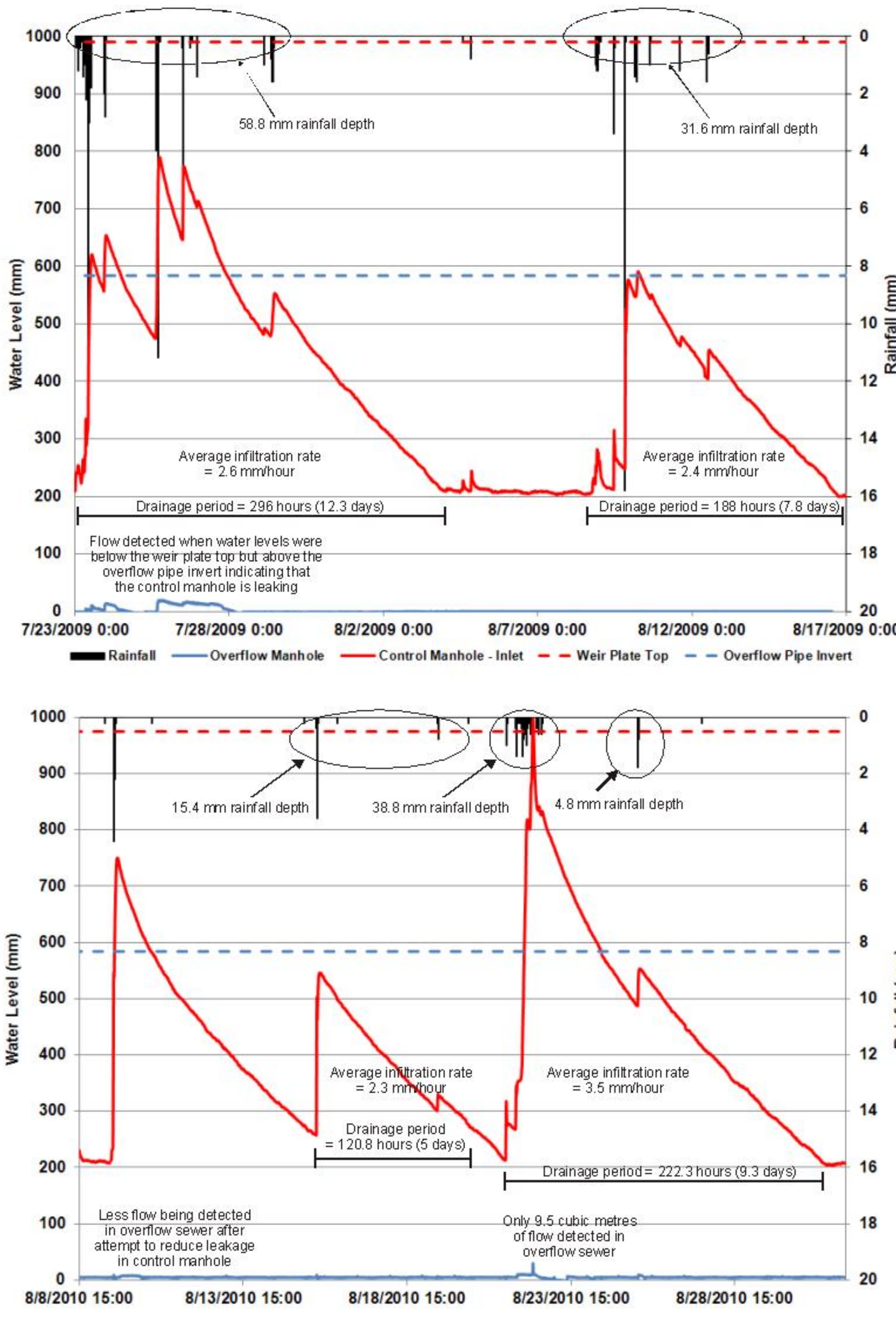


Figure 2.8: Water levels in Elgin Mills Crossing chamber system for selected drainage periods

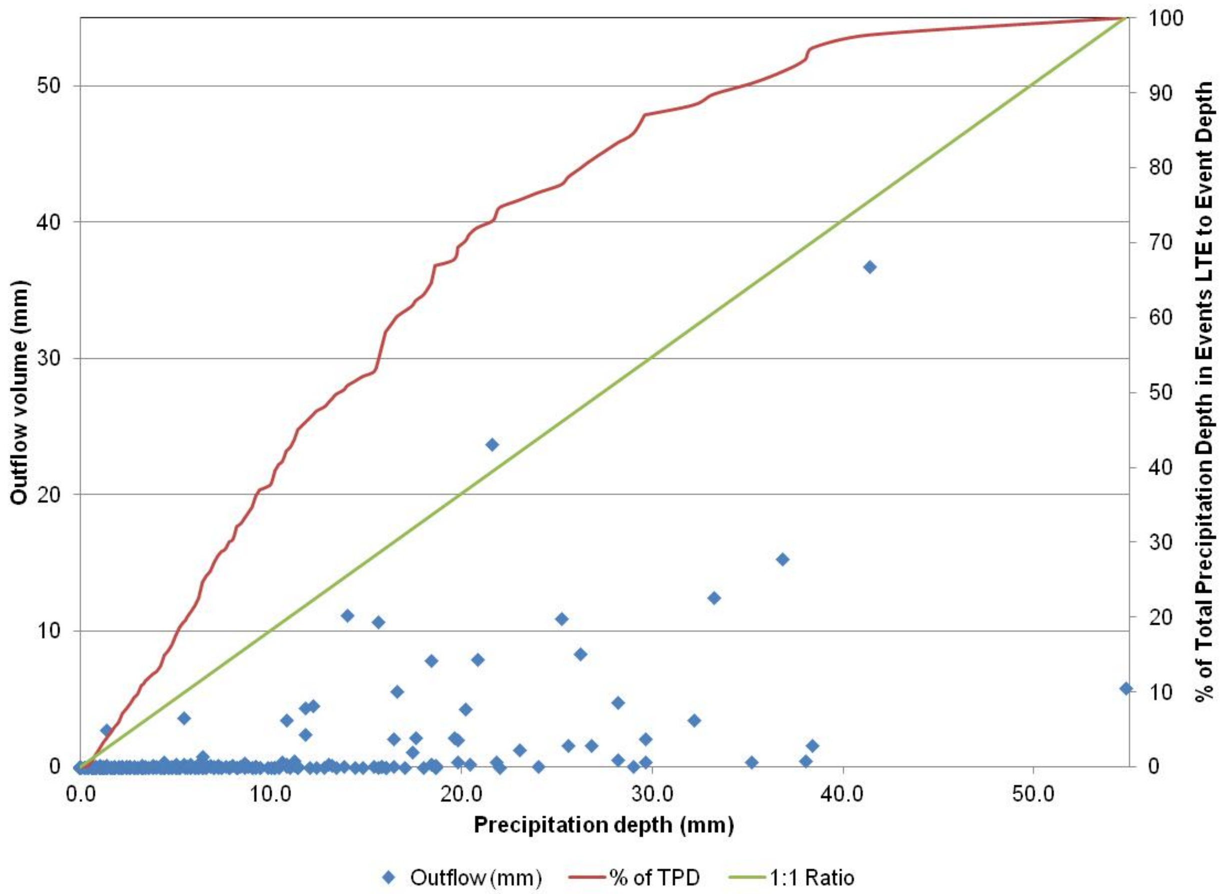


Figure 2.9: Scatter plot of storm event outflow versus precipitation depth for the Elgin Mills Crossing infiltration chamber system

To evaluate whether or not the chamber system was draining at the rate assumed in its design (12 to 75 mm/h), average infiltration rate over selected drainage periods were calculated using Equation 2, which assumes infiltration is occurring primarily by vertical flow of water from the base of the system:

Equation 2: $i = (V_i - V_o)/A_f * 1/t * 1000$

Where,

i = Average infiltration rate over the drainage period (mm/h)
V_i = Inflow Volume (m^3) during the drainage period
V_o = Outflow Volume (m^3) during the drainage period
A_f = Footprint surface area of chamber system (m^2) = 1,292 m^2
t = Drainage period duration (h)

Table 2.4 summarizes average infiltration rates of the chamber system observed over 60 drainage periods during which all necessary information was available. As anticipated, infiltration rates varied according to water level in the chamber system. When the system was full to capacity and exerting the greatest amount of pressure on the underlying soil, infiltration was observed to occur at rates as high as

13.5 mm/h. During periods when the system was about half full infiltration rates ranged between 2.5 and 5.0 mm/h. As the system approached complete drainage (i.e. nearly empty) rates ranged between 1.0 and 2.5 mm/h. Based on selected drainage periods it is estimated that the average infiltration rate of the chamber system over a full drainage cycle (i.e., from full to empty) is in the range of 3.0 to 3.5 mm/h. An example of such a drainage period began on August 21, 2010 and is shown in Figure 2.8. During the drainage period from August 21 to 31, 2010, a total of 43.6 mm of rainfall occurred which completely filled the system and required approximately 9 days for it to fully drain. The range of infiltration rate values observed over the monitoring period (1.0 to 13.5 mm/h) clearly shows that the system is draining at a much slower rate than what was assumed to be the capacity of the underlying soil when designing the system (12 to 75 mm/h). This is causing the system to require much longer periods to achieve complete drainage than the estimated 62 hours (i.e., 2.6 days) based on a full drainage cycle, an assumed infiltration rate of 12 mm/h and infiltration area of 1292 m^2(i.e. the facility footprint surface area).

Table 2.4: Average infiltration rates of the Elgin Mills Crossing chamber system over selected periods

Drainage period Start Date	Starting/Ending Water Level (mm)	Maximum Water Level (mm)	Cumulative Precipitation Depth (mm)	Drainage period Duration (h)	Infiltration Rate (mm/h)
11/14/08	269	738	28.2	172.25	3.2
3/29/09	212	1079	82.0	506.2	1.6
4/20/09	215	1036	103.8	708.7	2.0
4/30/09	424	765	18.4	100.83	2.1
5/9/09	380	1036	35.0	138.17	3.0
6/20/09	272	554	18.6	125.0	2.9
6/28/09	209	510	31.2	214.50	2.0
7/23/09	209	790	58.8	296.0	2.6
7/25/09	478	790	23.8	89.33	2.0
7/29/09	479	553	4.4	24.58	2.1
8/8/09	208	591	31.6	188.0	2.4
8/28/09	210	604	18.6	136.5	2.7
9/28/09	213	818	75.2	428.0	2.5
10/2/09	416	562	11.8	70.17	2.8
10/9/09	362	637	16.4	104.5	2.7
10/23/09	215	346	6.6	60.92	1.5
10/28/09	200	252	4.2	21.5	2.3
10/31/09	201	308	6.2	47.0	1.8
11/25/09	208	320	12.0	73.5	1.4
12/2/09	208	504	18.6	134.25	2.4
1/24/10	213	528	20.2	155.25	2.9
3/13/10	210	731	48.2	487.67	1.2
3/22/10	245	449	19.2	141.6	1.7
3/28/10	241	400	10.2	108.7	1.2
4/6/10	214	605	28.4	232.92	2.0
5/7/10	213	779	44.8	305.5	2.4
5/13/10	376	572	11.0	78.0	2.8
6/2/10	210	1106	201.2	826.25	4.2
6/3/10	741	1030	10.8	15.75	13.5
6/5/10	548	1018	20.2	66.0	6.0
6/24/10	505	1106	63.6	171.92	6.5
6/27/10	614	1050	19.6	50.42	7.7
7/9/10	213	355	10.2	72.08	2.8
7/19/10	211	521	15.8	86.92	3.6
7/23/10	211	1138	59.4	222.67	4.6
8/9/10	213	998	88.0	519.17	3.1
8/15/10	257	546	15.4	120.83	2.3
8/21/10	233	998	43.6	222.23	3.5
9/2/10	204	466	18.8	126.83	2.5

Drainage period Start Date	Starting/Ending Water Level (mm)	Maximum Water Level (mm)	Cumulative Precipitation Depth (mm)	Drainage period Duration (h)	Infiltration Rate (mm/h)
9/16/10	206	527	31.8	251.08	2.1
9/27/10	206	553	18.0	160.08	1.8
10/5/10	213	559	20.4	148.5	2.7
10/13/10	209	553	21.8	157.49	2.7
10/20/10	204	282	5.0	45.08	1.3
10/23/10	203	447	20.4	179.92	1.4
11/16/10	222	845	76.0	572.42	2.1
11/25/10	312	495	8.8	88.17	1.4
3/5/11	361	1046	69.6	544.5	4.5
3/9/11	539	1046	35.6	233.5	7.4
3/21/11	461	598	6.6	70.5	1.7
4/3/11	214	449	13.2	156.67	1.0
4/25/11	444	596	18.4	129.42	1.4
5/3/11	465	578	6.8	53.42	1.8
5/14/11	210	931	128.6	879.08	2.5
5/25/11	511	894	24.6	141.5	2.5
6/11/11	393	689	15.8	104.92	3.0
6/23/11	648	1112	44.2	77.92	6.6
6/24/11	777	1061	17.2	141.68	7.9
7/25/11	228	481	23.6	162.17	2.2
7/26/11	366	481	7.0	45.42	2.1
				MINIMUM:	**1.0**
				MAXIMUM:	**13.5**

2.4.3 Storage Volume

To verify that the chamber system provides the targeted water storage capacity of 955 m^3 changes in water level in the control manhole during storm events of similar size to the designed capacity of the system (41 mm over the roof drainage area) were examined. While very few individual storm events of this size occurred during the monitoring period, the cumulative 38.8 mm of precipitation that occurred on August 21 and 22, 2012 began at a time when the chamber system was nearly fully drained. As shown in Figure 2.8, at the onset of the rainfall period the water level in the control manhole indicated that the system was drained to roughly the bottom elevation of the chambers themselves, with water remaining only in the 152 mm deep granular bed below the chambers. Over the next 26.5 hours a total of 38.8 mm of precipitation fell, generating approximately 932.5 m^3 of runoff inflowing to the system from the roof drainage area. Water levels in the control manhole show that this size storm event filled the system to capacity and generated a brief period of flow over the weir plate. Only 9.5 m^3 of flow was detected in the outflow sewer. Assuming the granular material making up the base of the chamber system provides 40% void space, it can be predicted that it provides an additional 77.5 m^3 of storage that would be capable of capturing runoff from an additional 3 mm of rainfall over the roof drainage area. It can then be predicted that if the granular base had been fully drained at the onset of the storm event, that the system would have been capable of successfully capturing an event in the order of 41 mm in total depth.

2.4.4 Water Table Elevation

To determine if the recommended one metre of separation between the base of the chamber system (230.5 masl) and the water table is maintained, groundwater levels in a monitoring well located down-gradient from the chamber system and installed to 229.5 masl were examined during the 2009 and 2010 monitoring seasons. Water levels in the well during 2010 in relation to the elevation of the chamber system base are shown in Figure 2.10. During the majority of 2010, water table elevations were less than one metre from the base of the chamber system with the maximum groundwater elevation reaching 230.02 masl on March 18, 2010 (less than 0.5 m from chamber system base). Considering that the winter and spring months of 2010 were much drier than normal and that during wetter winter/spring seasons the water table elevation is likely to rise even higher, these results indicate that the recommended one metre separation distance between the water table and chamber system base has not been achieved. Water table elevation did not rise above the base of the chamber system during the period of June 2, 2009 to December 14, 2010, but based on pre-construction monitoring data that observed a maximum water table elevation of 230.6 masl in a nearby well (Dillon Consulting, 2006) it is likely that this could occur. During such periods, it can be expected that the system will stop draining until groundwater levels recede below the base of the system, or drain at a slower rate than those observed during this study. In this region a typical pattern of water table fluctuation involves elevations reaching a maximum during early spring and minimum during mid-summer to early fall with levels rising again throughout the late fall and winter months. Water table fluctuations observed in 2010 deviate from this typical pattern of gradual rise and decline, suggesting that infiltration of stormwater by the chamber system is influencing water table elevations in the local area. In the much wetter than normal month of June 2010, during which 206 mm of rainfall occurred in the study area, water table elevations rose to a maximum of 229.9 masl but remained below the maximum observed in March and greater than 0.5 m below the base of the chamber system.

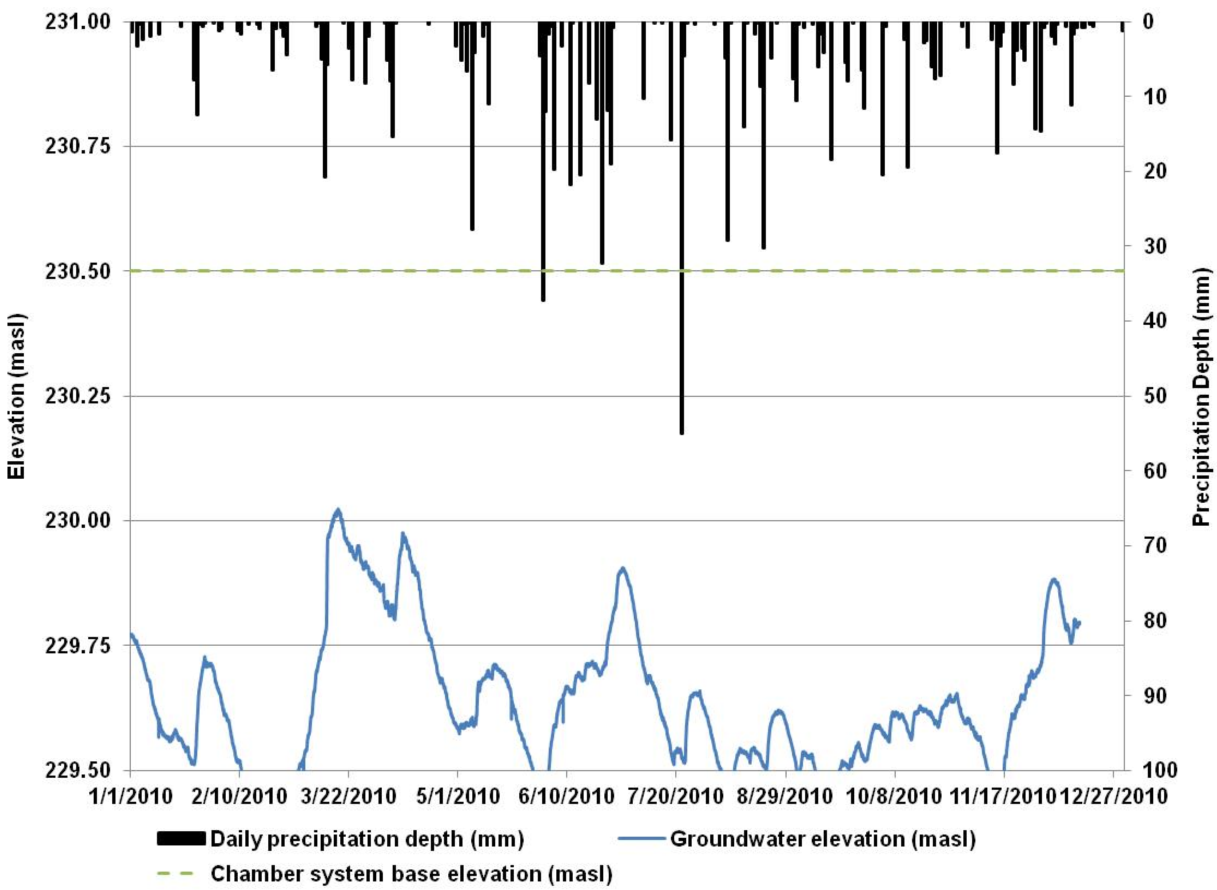


Figure 2.10: Water table elevations, January to December 2010.

2.5 Discussion

Field monitoring results indicate that the control manhole was leaking and allowing some water to pass from the inlet side of the weir plate to the outlet side which resulted in flow being detected in the outflow sewer pipe more frequently than expected. Efforts to improve seals in the control manhole midway through the study by applying additional concrete between the weir plate and manhole structure were only partly successful in reducing leakage. It is strongly recommended that such facilities be thoroughly inspected by the construction project manager, system designer or ultimate owner/manager of the infrastructure prior to assumption. Inspection procedures should include continuous water level monitoring over several storm events or a synthetic runoff test to determine if the system is functioning as designed. Contracts that include construction of such stormwater infrastructure should include conditions whereby any defects or deficiencies revealed through final inspection can be corrected prior to assumption. Visual inspection of the Elgin Mills Crossing chamber system at a time when the facility was filled to a level above the elevation of the outflow sewer pipe invert would have revealed that leakage around the weir plate was occurring. An improvement to the design of the control manhole that would better facilitate inspection and maintenance activities is including an outlet pipe with a valve at the bottom of the weir plate that can be operated from the outlet side that would allow the system to be drained via gravity without the use of a pump.

Monitoring also indicated that the post-development infiltration rate of the soils underlying the chamber system is lower than expected resulting in slower than expected drainage of stored stormwater and more frequent occurrence of overflow than expected. The lower infiltration rate may be a result of soil compaction during construction or from the weight of the overlying chamber system and stored stormwater. It is also likely that the base of the chamber system was installed in the fine textured sandy silt glacial till, rather than the fine sand lens. Four geotechnical boreholes drilled within or close to the location of the infiltration bed prior to construction show the top of the fine sand lens at an elevation only slightly above the elevation of the base of the chamber (Dillon Consulting, 2006). Depending on how the bed was excavated and graded, part or all of it likely drains to the less permeable sandy silt till layer.

Although the system did not achieve its drainage time design objective, the chamber system provides substantial reduction in runoff volume from the roof drainage area. The system infiltrated 86% of roof runoff over the September 13, 2008 to July 14, 2010 monitoring period, prior to an attempt to reduce leakage in the control manhole, and 96% of runoff over the July 15, 2010 to July 31, 2011 monitoring period, after additional sealing of the control manhole.

In the original water balance calculations for the site, the infiltration chamber system was estimated to fully compensate for the loss of infiltration over the site (14,494 m^3 in a normal precipitation year) that would be caused by the development without mitigation. These estimations were conservative in nature and did not account for runoff storage in the contributing storm sewers. Even though the system is leaking a small amount of water around the weir plate and draining slower than expected, monitoring indicates that during a normal precipitation year, the chambers would infiltrate a volume of water greater than the post-development recharge target for the site. Total precipitation depth during the 2009 monitoring period (899 mm) was closest to the 30 year climate normal value (893 mm) during which it is estimated that over 14,941 m^3 of runoff was infiltrated by the Elgin Mills Crossing chamber system. As this estimate does not include 47 mm of precipitation that occurred in 2009 during a period when the flow sensor in the outflow storm sewer was out of service, it can be concluded that the system is currently capable of exceeding the annual infiltration volume target it was designed to meet.

Water levels in boreholes and standpipes that were monitored prior to construction suggest that the water table could intersect with the base of the chamber system during peak groundwater levels in the spring (Dillon Consulting, 2006). Monitoring of groundwater levels in a well adjacent to the chamber system during the June 3, 2009 to December 13, 2010 monitoring period indicated that while water table elevation did not reach the base of the chamber system during this time period, it was often within one metre of the base. Design guidance recommends that stormwater infiltration practices should be designed to maintain a one metre separation between the base elevation of the practice and the seasonally high water table (CVC & TRCA, 2010). This guideline is intended to prevent direct interaction of groundwater with any contaminants trapped in the soils below the infiltration practice and thereby prevent them from being re-mobilized and leached further into the soil profile. As the Elgin Mills Crossing chamber system receives only roof runoff, which is relatively clean stormwater in comparison to road or parking lot runoff, the fact that groundwater levels are routinely within one metre of the base of the system is not a major concern. During periods of seasonally high water table elevation (typically early spring) it is possible that groundwater levels could reach the base of the chamber system, causing drainage to stop for the period of time it takes for levels to recede below the base again. During such periods drainage may also occur at slower rates than the minimum rate observed in this study.

In summation, despite minor leakage, slower than expected drainage time and the potential for water table elevation to interact with the base of the practice, the Elgin Mills Crossing infiltration chamber system was found to be capable of infiltrating the volume of runoff it was meant to on an annual basis. The favorable performance is due to the substantial water storage capacity of the system which is 375 m^3/ha. of impervious area draining to the facility, including the capacity of the chambers, gravel bed surrounding the chambers, and the contributing storm sewers when the chambers and gravel bed are filled to capacity.

3.0 MAYFIELD INDUSTRIAL PARK INFILTRATION TRENCH SYSTEM

3.1 Site Description

The Mayfield Industrial Park infiltration trench system is located off Pillsworth Road in the community of Bolton, Ontario, just west of Highway 50 and north of Mayfield Road (Figure 3.1). The trenches receive runoff from the roofs of two commercial buildings in the industrial park. The system is composed of four rectangular infiltration trenches, hereafter referred to as Mayfield Trenches 1, 2, 3 and 4 (Figure 3.1). Mayfield Trenches 1, 2 and 3 are installed under a truck loading/parking area along the south side of 86 Pillsworth Road. Mayfield Trench 4 is installed under a landscaped boulevard in the northwest corner of 99 Pillsworth Road. Outflow from the infiltration trench system and runoff from paved surfaces and landscaped areas is captured by catchbasins and conveyed by storm sewers to a stormwater detention pond. The pond is sized to provide enhanced water quality control. The pond drains to Rainbow Creek, a tributary to the Humber River, which ultimately flows to the north shore of Lake Ontario. This branch of Rainbow Creek supports small riverine warm water aquatic habitat. The Humber River Watershed Plan, *Pathways to a Healthy Humber* (TRCA, 2008) identified the existing and future urban portions of Rainbow Creek subwatershed as being badly in need of stormwater management practices that reduce the volume of runoff being directed to it in order to help manage major increases in stream flow and channel erosion that were predicted to occur as planned development in the catchment proceeds.

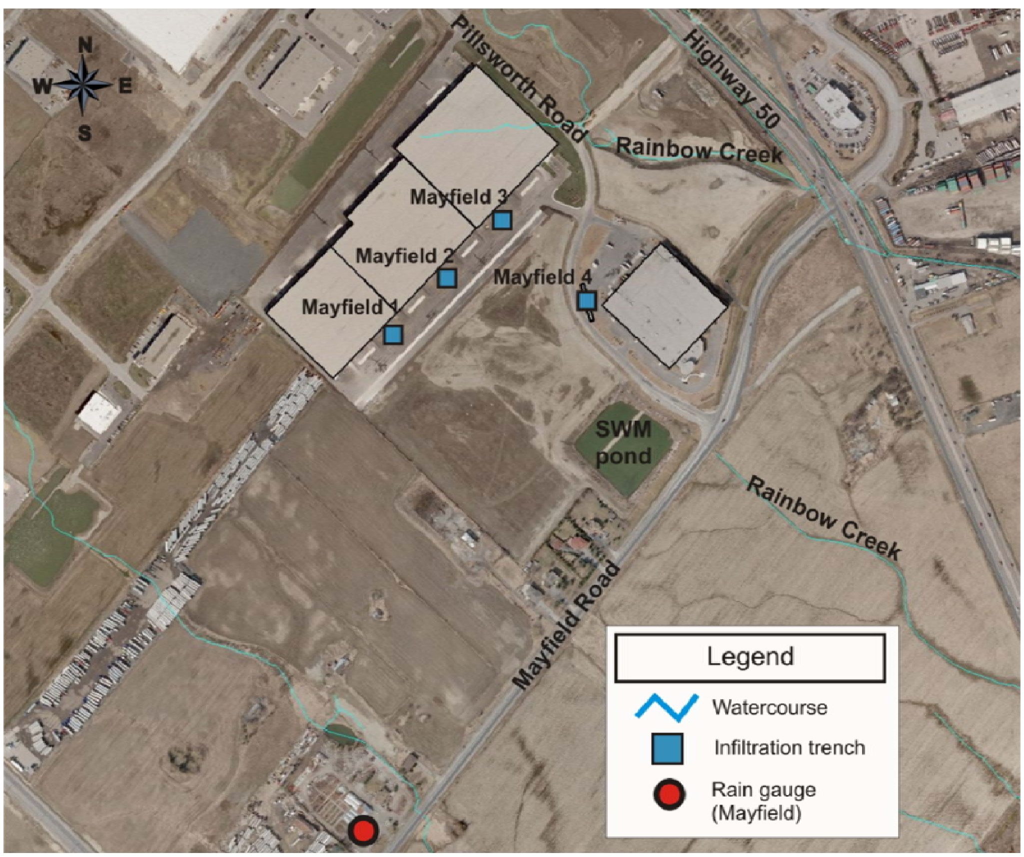


Figure 3.1: Mayfield Industrial Park infiltration trench system site

Surficial soils and underlying geology of the Mayfield Industrial Park site are composed of low permeability clayey silt glacial till down to the top of the underlying Georgian Bay Formation bedrock which is at a depth of 30 m or more below ground surface (OMOE, 2006). Based on grain-size analyses of surficial subsoil samples taken from three locations across the site the till consists of approximately 57% silt and 42% clay with a trace of sand. Some discontinuous sand and gravel layers have been encountered within the clayey silt till deposits in water well records in the local area surrounding the site at varying depths ranging between 10 and 63 m (Shaheen and Peaker, 2006). These beds comprise a shallow aquifer system, providing groundwater to a number of wells in the local area. The shallow Georgian Bay Formation bedrock is weathered and fractured shale and considered to be a productive aquifer, contributing significantly to private water supplies in the area (Shaheen and Peaker, 2006). Groundwater movement in the vicinity of the site is interpreted to be predominantly in a northeasterly direction (Shaheen and Peaker, 2006) towards a first order tributary of Rainbow Creek (Figure 3.1).

The overall design objective for the infiltration trench system was to maintain average annual infiltration volume over the development site at pre-development levels and reduce post-development runoff volume in order to minimize impacts on water levels in receiving aquifers and the hydrology of the receiving watercourse.

3.2 Infiltration System Design

Available information from water wells (OMOE, 2006) and boreholes in the vicinity of the trench system indicate that the water table, as determined by the potentiometric surface interpreted from observations of static groundwater levels, ranges in elevation from 210 to 215 masl (Shaheen and Peaker, 2006). These elevations are between 14 to 19 m below the base of the trench system, confirming that seasonally high groundwater levels will not interfere with its performance and that hydraulic gradient at the base should remain positive throughout the year.

Neither pre-development hydraulic conductivity tests, nor grain-size analyses of native soils in the locations of the infiltration trenches were available to help characterize the permeability of the clayey silt till when the system was designed. However, based on the approximate relationship between Unified Soil Classification soil type and permeability it can be estimated that hydraulic conductivity of the clayey silt till deposits is in the order of 1×10^{-6} cm/s, which represents an infiltration rate of approximately 12 mm/h (OMMAH, 1997).

The top of the trenches are a minimum of 1.5 m below ground surface which is below the local maximum frost penetration depth of 1.4 m (MTO, 2005) and should not be subject to freezing during cold winter temperatures.

<u>Mayfield Trenches 1, 2 and 3</u>
The Mayfield Industrial Park Infiltration Trenches 1, 2 and 3 receive runoff from a 58,331 m^2 roof of an industrial building located at 86 Pillsworth Road in Bolton (Figure 3.2). The 11.66 hectare (ha.) lot contains approximately 90% impervious cover. The trench system was designed to provide a total water storage capacity of 28.8 cubic metre per hectare (m^3/ha.) of lot area or a total trench volume of 72 m^3/ha. of lot area (assuming the gravel filling the trench provides 40% void space). The roof drainage areas contributing runoff to each trench varies, ranging from 14,961.9 m^2 to 23,268.3 m^2 (Table 3.1). Each

infiltration trench, control manhole and contributing storm sewer pipe provides temporary storage of 126 m^3 of roof runoff. Based on this storage volume and the size of each roof drainage area, it can be estimated that Mayfield Trenches 1, 2 and 3 should be able to capture roof runoff from storm events up to 9.4, 7.0 and 6.0 mm in depth respectively, assuming 10% loss of water to evaporation (Table 3.1). Each of the three infiltration trenches covers an area of 150 m^2, with a combined footprint of 450 m^2. They are two (2) m in depth, lined with geotextile filter fabric (Terrafix 270R®) and filled with 20 to 50 mm diameter clear crushed stone with an assumed void space ratio of 40%. Assuming a native soil infiltration rate of 12 mm/h, infiltration of stormwater in the trench system when filled to capacity would occur over roughly 70 hours (approximately 3 days).

Table 3.1: Mayfield Industrial Park infiltration trench system drainage areas and specifications

	Mayfield 1	**Mayfield 2**	**Mayfield 3**	**Mayfield 4**
Roof drainage area	14,961.9 m^2	20,100.8 m^2	23,268.3 m^2	14,420.1 m^2
Runoff source area type	Roof only	Roof only	Roof only	Roof only
Trench footprint area	150 m^2	150 m^2	150 m^2	225 m^2
Trench depth	2 m	2 m	2 m	2 m
Water storage volume of infiltration trench [1]	120 m^3	120 m^3	120 m^3	180 m^3
Water storage volume of contributing storm sewer and control manhole [2]	6 m^3	6 m^3	6 m^3	4 m^3
Combined water storage volume	126 m^3	126 m^3	126 m^3	184 m^3
Combined water storage volume in drainage area millimetres [3]	9.4 mm	7.0 mm	6.0 mm	14.2 mm
Combined water storage volume per hectare impervious drainage area	84.21 m^3/ha.	62.68 m^3/ha.	54.15 m^3/ha.	127.6 m^3/ha.
Ratio of impervious drainage area to trench system footprint area	100:1	134:1	155:1	64:1
Hydraulic conductivity of clayey silt till [4]	1 x 10^{-6} cm/s	1 x 10^{-6} cm/s	1 x 10^{-6} cm/s	1 x 10^{-6} cm/s
Approximate infiltration rate of clayey silt till [5]	12 mm/h	12 mm/h	12 mm/h	12 mm/h
Trench fill material	Clear 20-50 mm diameter crushed stone	Clear 20-50 mm diameter crushed stone	Clear 20-50 mm diameter crushed stone	Clear 20-50 mm diameter crushed stone

Notes:

1. Assuming 40% void space in the trench.
2. Estimated based on Site Servicing, Grading and Stormwater Management Plan Drawings (A.M. Candaras, 2007a; A.M. Candaras, 2006.
3. Assuming 10% loss to evaporation.
4. Based on approximate relationship between Unified Soil Classification System soil type and permeability (OMMAH, 1997).
5. Based on approximate relationship between hydraulic conductivity and infiltration rate (OMMAH, 1997).

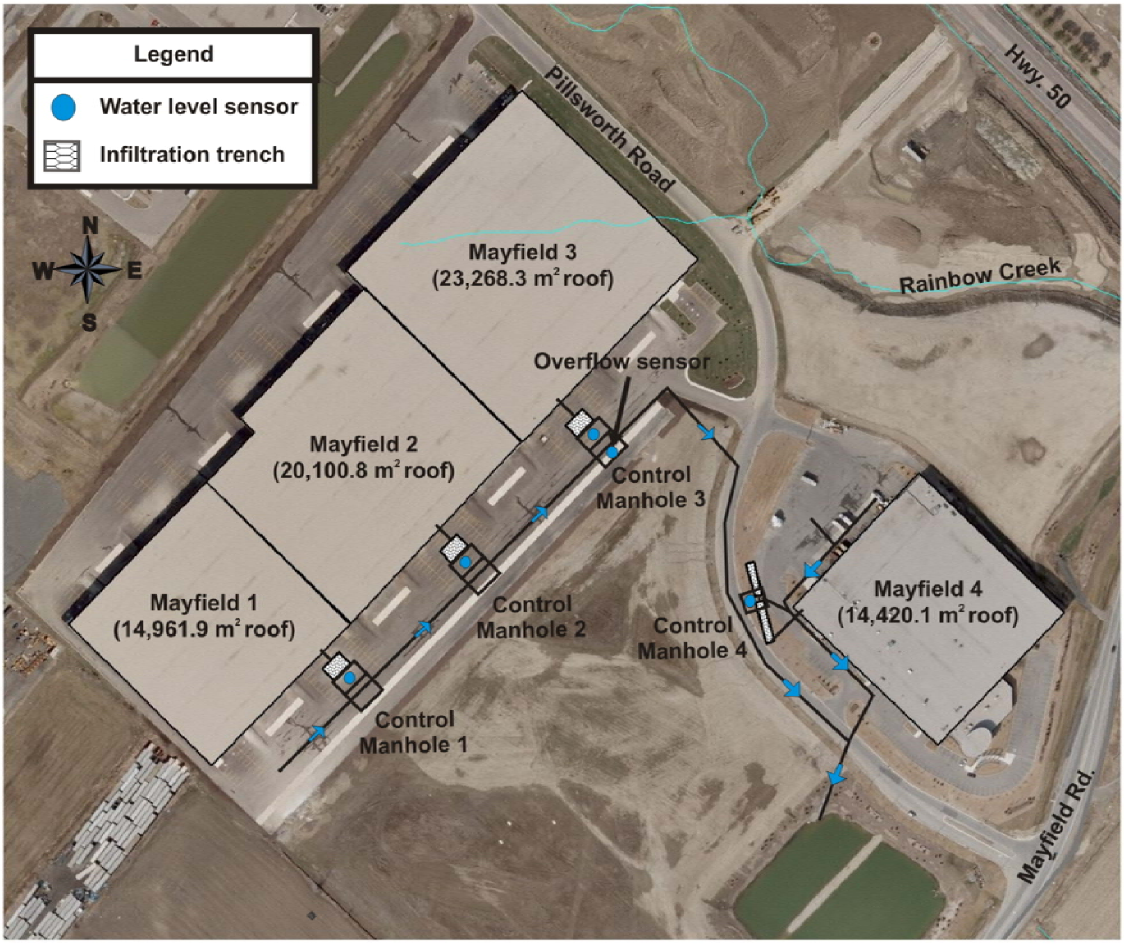


Figure 3.2: Mayfield Industrial Park infiltration trench system, plan view

Roof runoff is released into each trench at a height of 1.4 m above the base by a 300 mm diameter perforated pipe wrapped with geotextile filter fabric (Figure 3.3). A 200 mm diameter perforated pipe is installed along the bottom of each trench and acts as an underdrain connected to the inlet side of the control manhole. Each control manhole contains a weir plate with a top elevation that corresponds to approximately 0.9 m above the top of the trench (Figure 3.4) which serves to back water up into the trench and the trench inlet pipe, providing additional water storage capacity. When the trench inlet pipes, trenches and inlet sides of the control manholes are full, inflowing runoff overtops the weir plates and flows into the outlet storm sewer, which drains to the stormwater detention pond. During and after precipitation events, accumulated runoff is temporarily stored in the trench inlet pipes, trenches and control manholes until it infiltrates into the native soil.

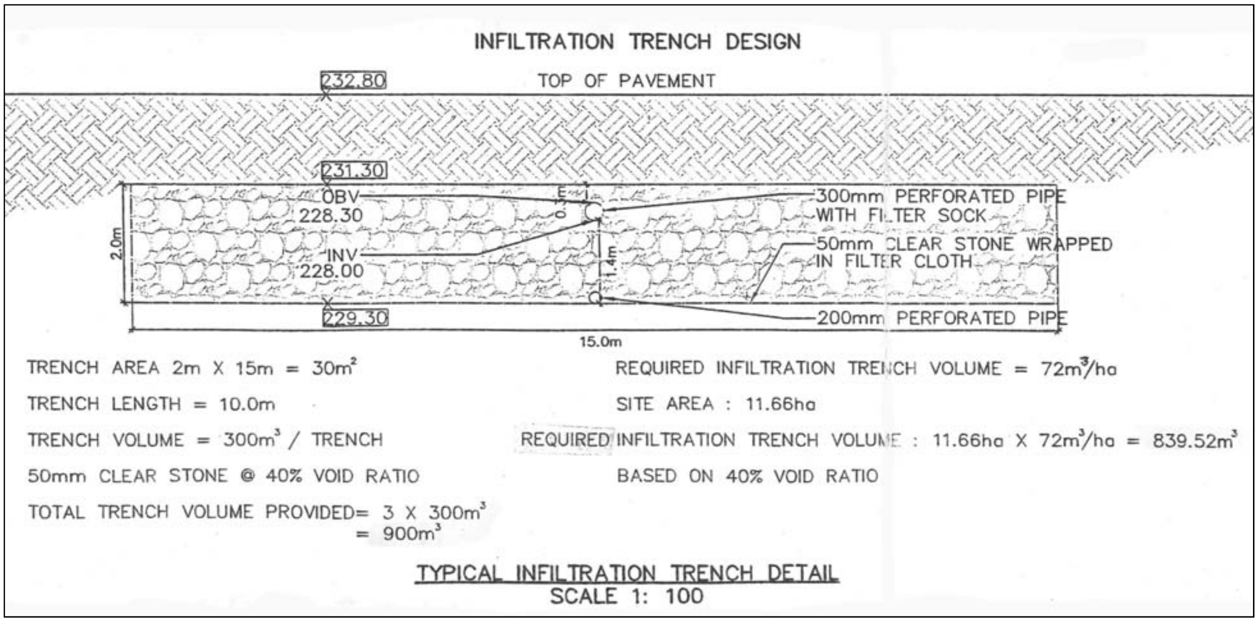


Figure 3.3: Mayfield Infiltration Trenches 1, 2 and 3 trench cross-section (A.M. Candaras, 2007a)

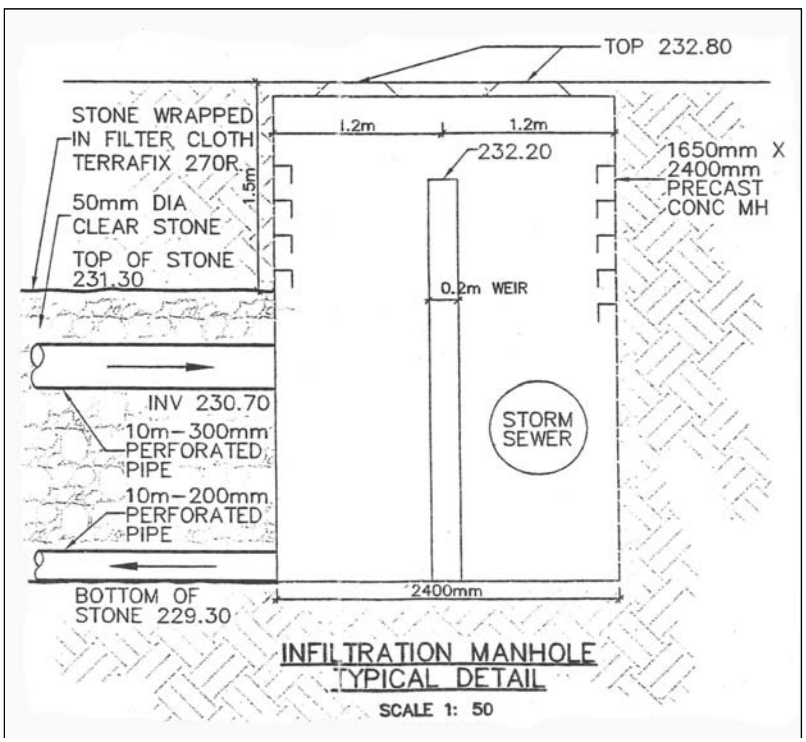


Figure 3.4: Mayfield Infiltration Trenches 1, 2 and 3 control manhole cross-section (A.M. Candaras, 2007a)

<u>Mayfield Trench 4</u>

The Mayfield Infiltration Trench 4 receives runoff from a 14,420.1 m^2 roof of an industrial building located at 99 Pillsworth Road in Bolton (Figure 3.2) and is composed of two sections of trench connected by a control manhole. The 3.21 ha. lot contains approximately 85% impervious cover. The trench system was designed to provide a total water storage capacity of 56 m^3/ha. of lot area or a total trench volume of

140.4 m^3/ha. of lot area (assuming the gravel filling the trench provides 40% void space). The trench is 2 m in depth and 5 m in width with the first section being 25 m in length and the second being 20 m in length. Roof runoff first enters a manhole containing a Goss trap outlet structure prior to entering the first section of trench, which provides sedimentation pretreatment for coarse sediment and floating debris. The infiltration trench covers an area of 225 m^2, is lined with geotextile filter fabric (Terrafix 270R®) and filled with clear, 20 to 50 mm diameter crushed stone with an assumed void space ratio of 40%. The combined water storage capacity of Mayfield Trench 4, including the two sections of trench, control manhole and trench inlet pipe is 184 m^3, which is approximately what would be produced by a 14.2 mm precipitation event over the roof drainage area assuming 10% loss of water to evaporation.

Flow from the pretreatment manhole is released into the first section of trench at a height of 1.4 m above the base by a 300 mm diameter perforated pipe wrapped with geotextile filter fabric (Figure 3.5). The first and second sections of the trench are connected to the control manhole by 300 mm diameter perforated pipes wrapped with geotextile filter fabric. Unlike Mayfield Trenches 1, 2 and 3, there is no perforated pipe installed along the bottom of each section of Mayfield Trench 4 that connects to the inlet side of the control manhole. Therefore water levels in the control manhole are indicative of water levels in the trench only when they are above the invert of the 300 mm diameter pipes connecting the trenches to the control manhole. The control manhole contains a weir plate with a top elevation that corresponds to approximately 0.6 m above the top of the trench (Figure 3.6) which serves to temporarily store additional roof runoff in the storm sewer pipe leading from the building to the pretreatment manhole. When the infiltration trench system is full, inflowing runoff overtops the weir plate and flows into the outlet storm sewer, which drains to a stormwater detention pond. During and after precipitation events, accumulated runoff is temporarily stored in the trench, control manhole and trench inlet pipe until it infiltrates into the native soil.

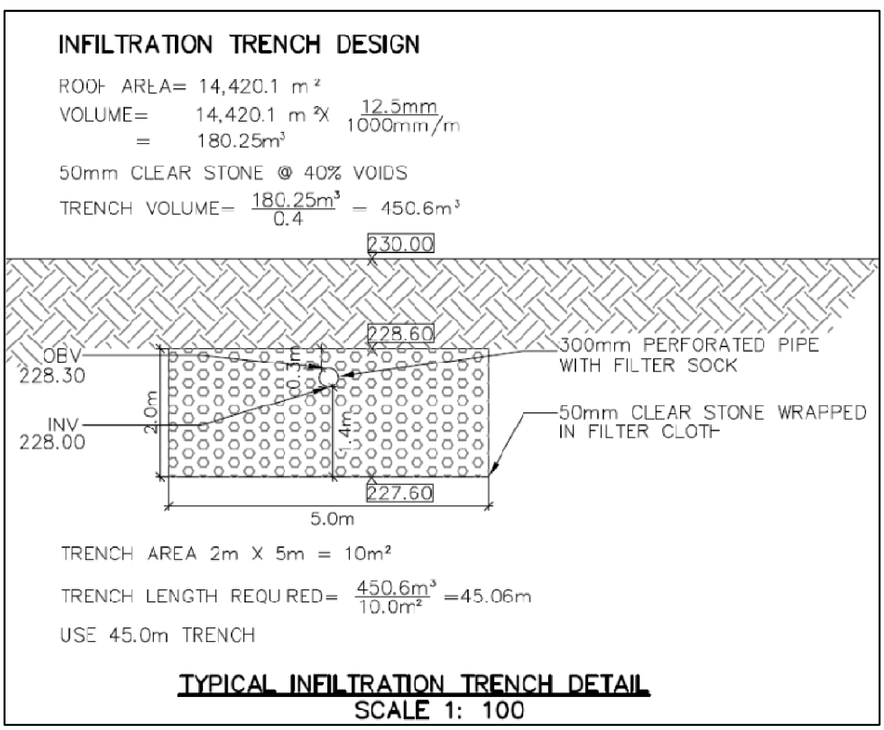


Figure 3.5: Mayfield Infiltration Trench 4 cross-section (A.M. Candaras, 2006)

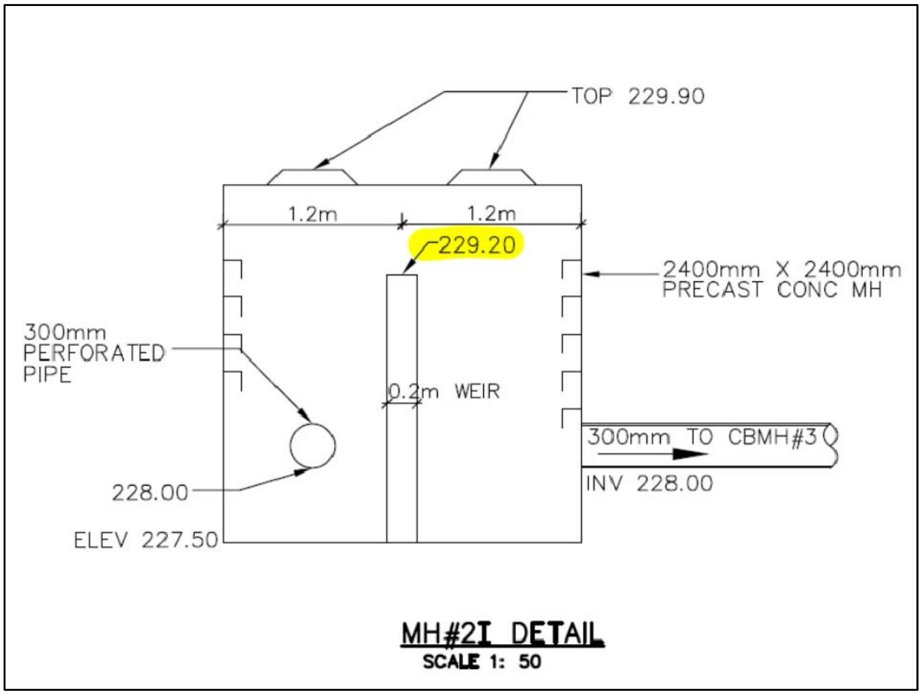


Figure 3.6: Mayfield Infiltration Trench 4 control manhole cross-section (A.M. Candaras, 2006)

3.3 Monitoring Parameters, Locations and Equipment

Monitoring at the Mayfield Industrial Park site was initiated in July 2009 and continued to the end of June 2011. Monitoring parameters, locations and equipment were selected to provide the information needed to estimate actual infiltration rates and drainage times being achieved by each trench, and under what conditions each trench overflows.

Rainfall during July 2009 and May to November 2010 monitoring periods was measured with a three season 8 inch diameter tipping bucket rain gauge located approximately 1 km south of the site (Figure 3.1). A second gauge (referred to as the Laidlaw rain gauge) located on the northeast corner of Mayfield Road and Airport Road, approximately 8 km west of the site, served as a back-up gauge for periods when the primary gauge was out of service. Rainfall during the period from August to November 2009 was measured with the Laidlaw rain gauge. Precipitation data (daily precipitation totals) during the months of December 2009, January to April 2010 and December 2010 were taken from the Lester B. Pearson International Airport meteorological station located approximately 20 km southwest of the site. Precipitation data during the months of January to June 2011 were taken from a four season tipping bucket rain gauge located at King Road and Albion-Vaughan Road, approximately 5 km north of the site.

Calibrated pressure transducers were installed on the inlet side of each of the four control manholes which indicate water levels in their respective trenches and provide continuous measurements at 5 minute intervals (Figure 3.2). Pressure transducers were also installed on the outlet side of the control manholes for a portion of the monitoring period to determine if the control manholes were allowing water to leak around the weir plates. Pressure transducers installed in the control manholes for Mayfield Trenches 1, 2 and 4 were set at the bottom of each manhole. Due to accumulated sediment in the inlet side of the Mayfield Trench 3 control manhole the pressure transducer had to be set at a height of 457 mm above the bottom of the manhole.

3.4 Results

3.4.1 Precipitation

Figure 3.7 illustrates monthly totals for precipitation over the 2009 to 2011 period from the rain gauges and 30 year climate normals (monthly averages from 1971 to 2000) from the nearest site with a sufficient period of record (“Sandhill” climate station; Environment Canada, 2010).

While annual total precipitation over the 2009 to 2011 period remained within 12% of the long term average, monthly and seasonal totals often deviated significantly from 30 year climate normal values. Total precipitation depth during the summer (July to September) of 2009 was only slighty higher than normal with a much wetter than normal August balanced by a drier than normal September. This was followed by a drier than normal fall (October to December) 2009 season when precipitation in November was 62% less than the long term average. Overall in 2009, total precipitation depth at the site was 922 mm, only 4% greater than the 30 year average of 887 mm. The dry fall of 2009 was followed by a very dry winter (January to March) of 2010 when precipitation was 37% less than normal. Drier than normal weather continued during April and May of 2010 but was more than made up for in June when a total of 204 mm of precipitation fell, breaking monthly maximum rainfall records in the region. Overall in 2010, total precipitation depth at the site was 776 mm, representing 12% less precipitation than a normal year. Spring (April to June) of 2011 was wetter than normal with a particularly wet month of May.

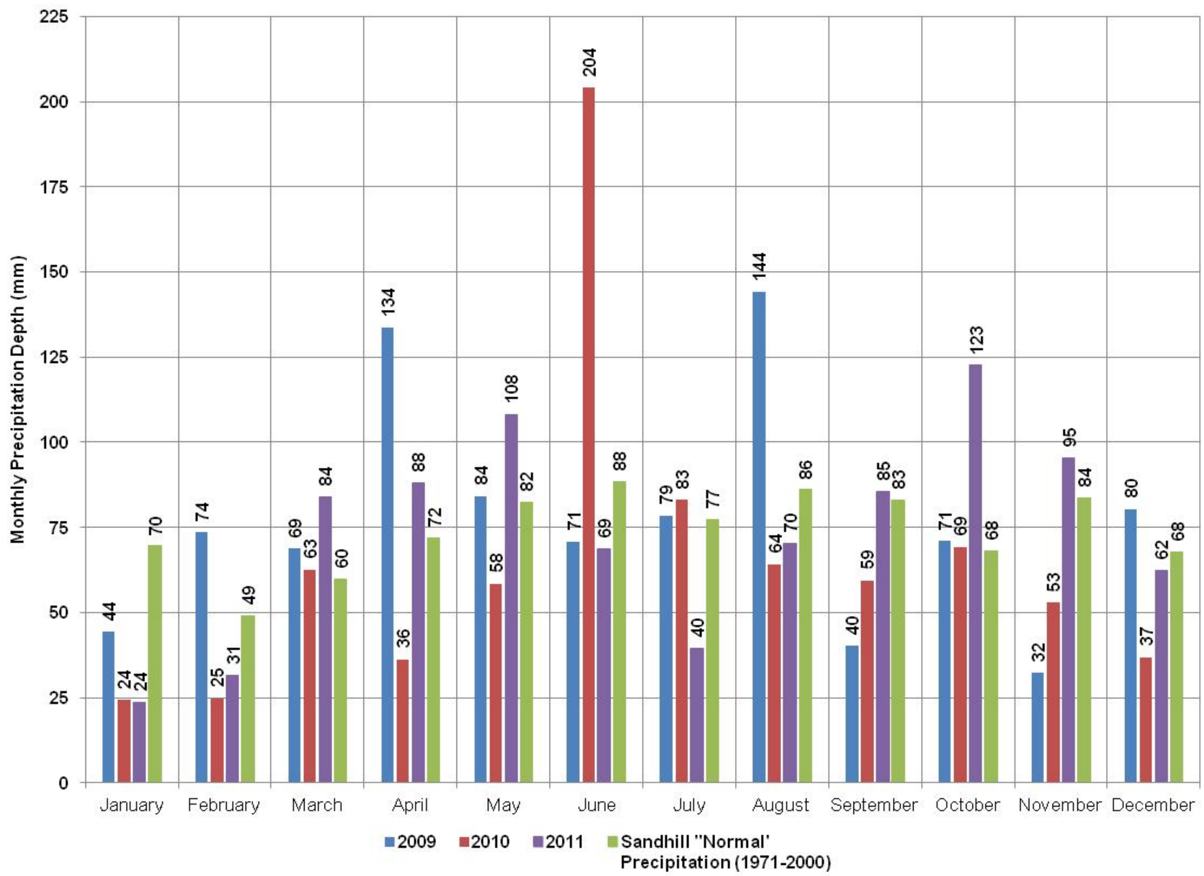


Figure 3.7: Monthly precipitation totals – Mayfield Industrial Park site

Tables 3.2 provides a breakdown of the number of storm events according to precipitation depth ranges (i.e. bins) that occurred over the July 10, 2009 to June 10, 2011 monitoring period and the number of events that caused each infiltration trench to overflow, as indicated by water level in the control manhole. Results for Mayfield Trench 2 are not provided because it was determined early in the study that the control manhole was rapidly leaking water from the inlet side of the weir plate to the outlet side (see Section 3.4.2). Over the monitoring period a total of 229 storm events[2] occurred. The largest storm event in terms of maximum depth of precipitation was 49.6 mm, occurring on July 23, 2010. The July 23, 2010 storm event was also the most intense in terms of hourly rainfall depth, when precipitation was recorded at a rate of 22.4 mm/h between 4:10 AM and 5:05 AM. On August 4, 2009, precipitation was recorded at a rate of 11.0 mm/5 minutes between 1:05 and 1:10 PM representing the most intense 5 minute rainfall period.

Examination of storm event depths that caused each trench to overflow and the frequency of overflow events provide insight into what size storm events can be fully captured and infiltrated and what size events consistently produce overflow. Based on the roof drainage area, water storage capacity of the trenches, a native soil infiltration rate of 12 mm/h and assuming that 10% of incoming precipitation is lost to evaporation, it can be estimated using Equation 3 that Mayfield Trench 1 should be capable of fully

[2] Individual storm events were defined as periods of precipitation totaling a minimum of 0.4 mm depth and separated by a minimum antecedent dry period of 3 hours.

capturing and infiltrating events up to 9.4 mm in depth with no overflow, if the antecedent dry period between events is at least 70 hours.

Equation 3: $d = V_c / A * 1000 / 0.9$

Where,
d = Precipitation depth (mm)
V_c = Combined storage volume of trench, control manhole and sewer pipe (m^3)
A = Drainage area (m^2)

However, based on observed frequencies of events causing overflow over the monitoring period (Table 3.2 and Figure 3.8), Mayfield Trench 1 only fully captured and infiltrated storm events up to about 3.0 mm in depth on a consistent basis. Conversely, most storm events greater than 12.8 mm in depth caused the trench to overflow. Table 3.2 and Figure 3.8 show that 17% of storm events between 6.0 and 7.2 mm in depth caused overflow (2 of 12 events) indicating that events of this size were fully captured on a fairly consistent basis, acknowledging that antecedent dry periods between events will occasionally be less than 70 hours. However, the percentage of events causing overflow significantly increases for events greater than 7.2 mm in depth (Figure 3.8). Indeed, overflow was observed for 71% of events between 8.8 and 10.0 mm in depth that occurred during the monitoring period (5 of 7 events) suggesting that Mayfield Trench 1 overflows more often than expected and that the underlying native soils are likely infiltrating at a rate less than the assumed rate of 12 mm/h.

Table 3.2: Summary of storm events captured during the June 10, 2009 to July 10, 2011 monitoring period and frequency of overflow for Mayfield Trenches 1, 3 and 4

Precipitation depth bin (mm)	Storm events	Overflow events Mayfield 1		Overflow events Mayfield 3		Overflow events Mayfield 4	
	#	#	%	#	%	#	%
0.4 to 1.6	76	1	1	8	11	4	5
1.8 to 3.0	28	0	0	6	21	2	7
3.2 to 4.4	27	2	7	10	37	3	11
4.6 to 5.8	17	2	12	12	71	3	18
6.0 to 7.2	12	2	17	9	75	2	17
7.4 to 8.6	9	4	44	8	89	3	33
8.8 to 10.0	7	5	71	6	86	1	14
10.2 to 11.4	8	5	63	8	100	3	38
11.6 to 12.8	4	2	50	2	50	1	25
13.0 to 14.2	3	3	100	3	100	1	33
14.4 to 15.6	2	2	100	2	100	1	50
15.8 to 17.0	5	5	100	5	100	5	100
≥ 17.2	31	26	84	30	97	26	84
Total	**229**	**59**	**26**	**109**	**48**	**55**	**24**

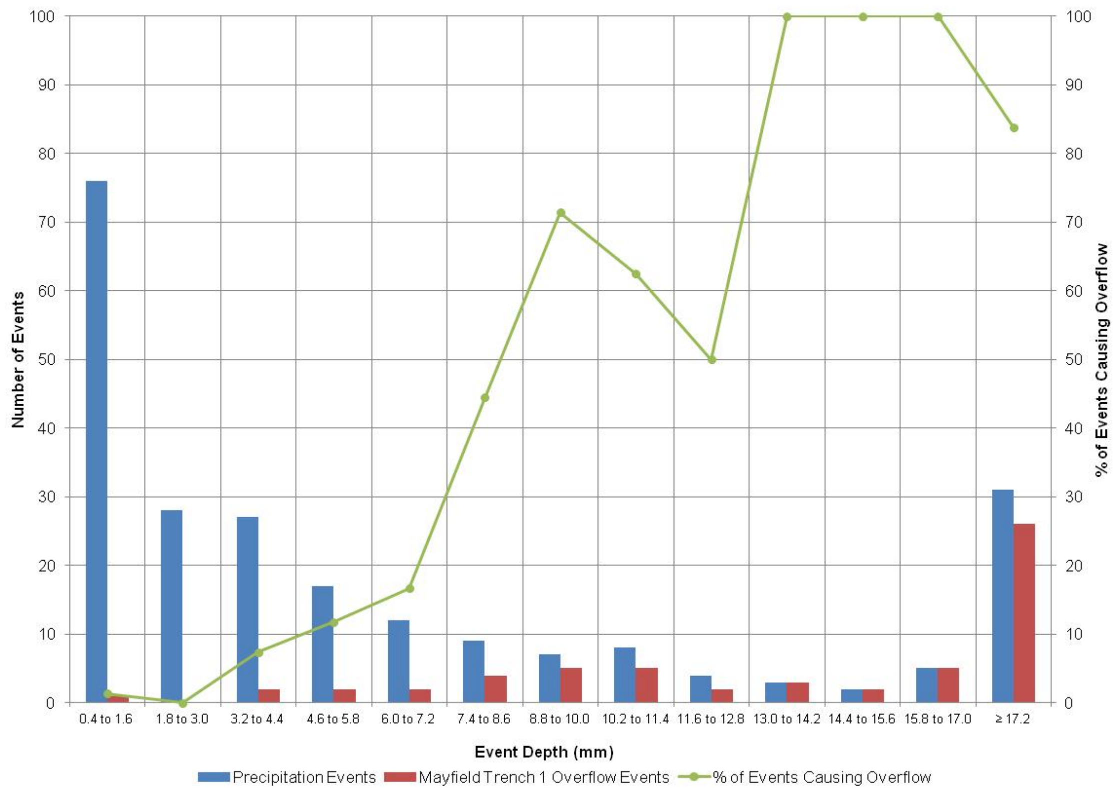


Figure 3.8: Mayfield Trench 1 frequency of overflow events by precipitation event depth bin

Based on similar assumptions, Mayfield Trench 3 should be capable of capturing runoff from events up to 6.0 mm in depth with no overflow, if the antecedent dry period between events is at least 70 hours. Table 3.2 and Figure 3.9 show that Mayfield Trench 3 was observed to overflow during all event depth ranges with 71% of events between 4.6 to 5.8 mm in depth (12 of 17 events) causing overflow, indicating that it is overflowing more often than expected as well.

Based on similar assumptions, Mayfield Trench 4 should be capable of capturing runoff from events up to 14.2 mm in depth with no overflow, if the antecedent dry period between events is at least 70 hours. Examination of the frequency of events causing overflow in Mayfield Trench 4 (Table 3.2 and Figure 3.10) show that overflow was observed during all event depth ranges with 33% of events between 13.0 and 14.2 mm in depth (1 of 3 events) causing overflow. However, these results do not provide strong evidence regarding whether or not the trench was overflowing more often than expected.

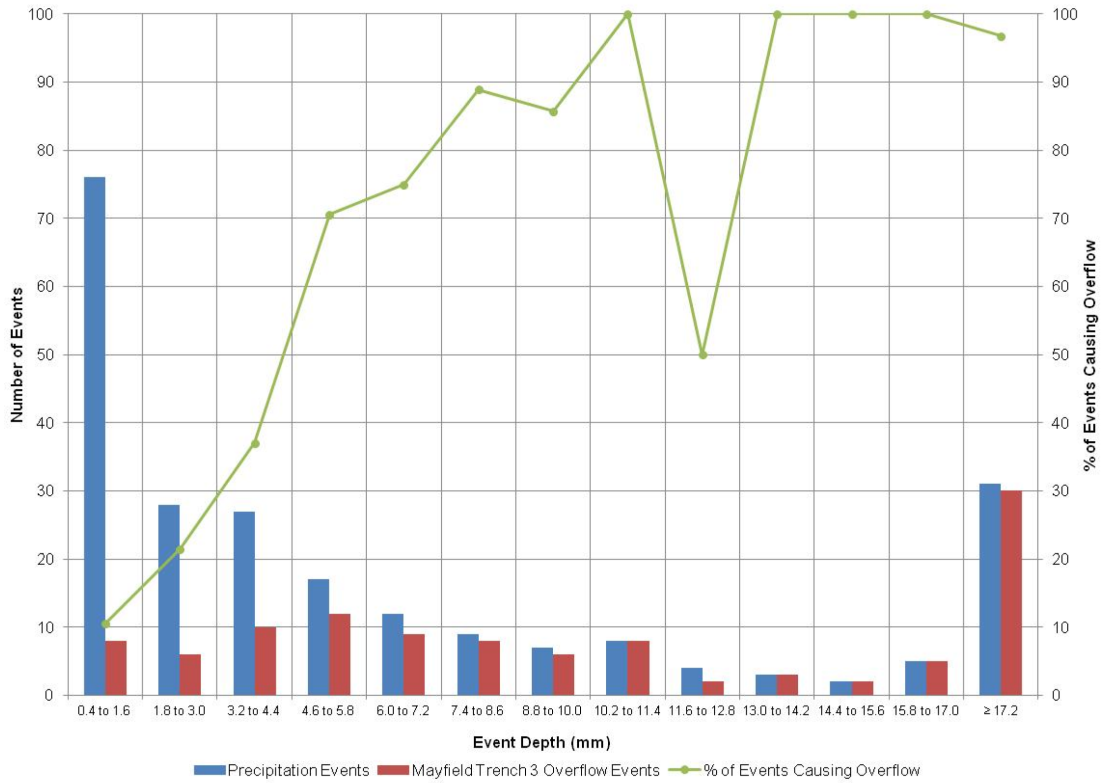


Figure 3.9: Mayfield Trench 3 frequency of overflow events by precipitation event depth bin

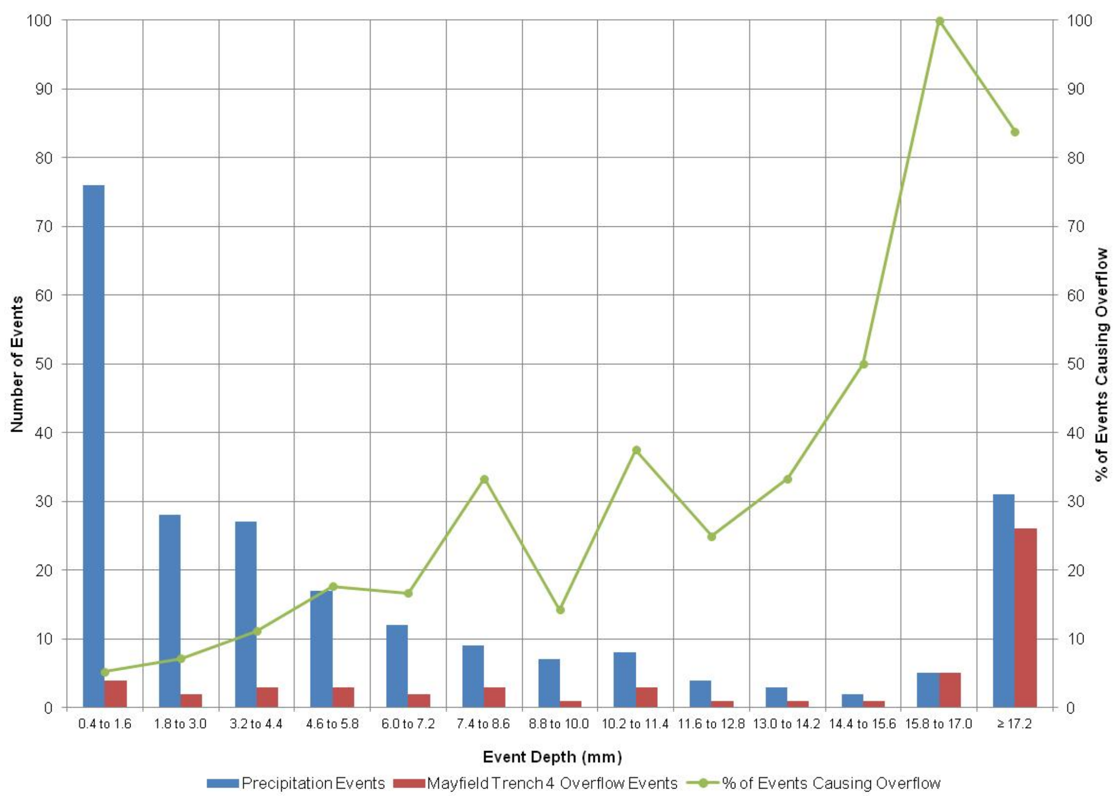


Figure 3.10: Mayfield Trench 4 frequency of overflow events by precipitation event depth bin

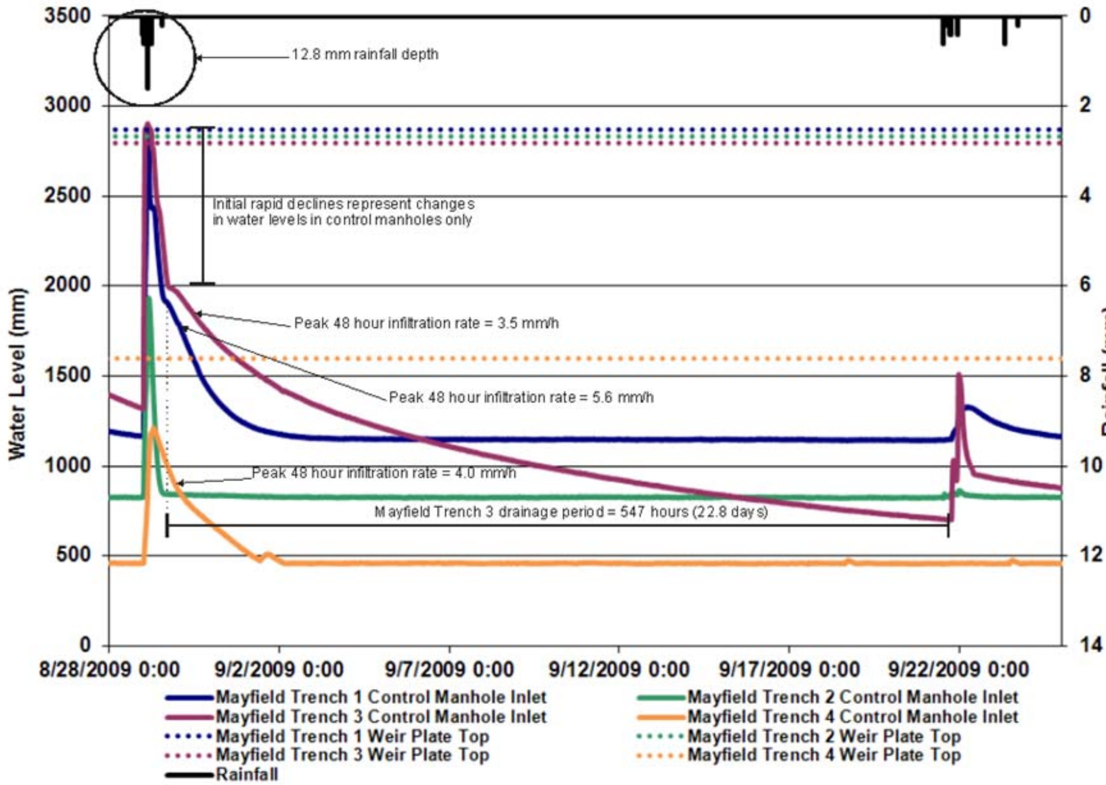
3.4.2 Infiltration

Performance of the Mayfield Industrial Park infiltration trench system was evaluated with regard to the post-construction infiltration rate of the underlying native soil. Infiltration rates of each trench were examined for a number of drainage periods and compared to the estimate of pre-development native soil infiltration rate that was used to size the facility (12 mm/h).

It is assumed that the flow of water from the infiltration trenches into the underlying native soil can be predicted by Darcy's Law (Equation 1; Darcy, 1856). Darcy's Law predicts that as the volume of water in the trench decreases and exerts less pressure at the interface between the trench bottom and the underlying native soil that the rate of flow into the native soil will decrease. Therefore, it can be expected that infiltration rates will be highest when the trench and contributing storm sewer is full, as this will be when the pressure difference is greatest. It can also be expected that infiltration rates will decrease as water level in the trench declines, reaching a minimum when it is close to the base of the trench.

Due to the configuration of the control manholes and the outlet storm sewer pipes, which also receive runoff from upstream paved areas it was not feasible to accurately measure volume of outflow from each trench in the system, limiting the approaches available to measure infiltration rates for each trench. However, the rectangular box design of each trench and control manhole configuration enables the volume of water in the trench at any time to be estimated using water level measurements from the inlet side of the control manhole, assuming the gravel in the trench provides a certain amount of void space. It is assumed that the open graded gravel used to fill the trenches provides 40% void space for water storage (Freeze and Cherry, 1979). Using water level measurements from the inlet sides of the control manholes, infiltration rates were estimated over selected portions of the overall drainage period for each trench. Drainage periods used to estimate post-development infiltration rates were restricted to periods when water levels in the control manhole were at the elevation that corresponds to the top of the trench (i.e. trenches are filled to capacity) and no flows were being registered in the outlet side of the downstream control manhole (i.e., control manhole was not leaking water), to ensure that changes in water levels only reflected drainage by infiltration into the underlying native soil.

Figure 3.11 shows hyetographs and control manhole water levels for selected storm events and drainage periods for Mayfield Trenches 1, 2, 3 and 4. In control manholes 1, 3 and 4 water levels decline rapidly when at levels between the top of the weir plate and the height which corresponds to the elevation of the top of the trench (1950 mm for Trench 1, 2000 mm for Trench 3, 1060 mm for Trench 4). During these periods, rate of water level change represents change in volume within the control manhole only. Once water level reaches the height which corresponds to the elevation of the top of the trench, rate of decline decreases as from that point on, they represent decreases in volume in both the control manhole and trench. As predicted by Darcy's Law, infiltration rate is highest when the trench is full and exerting maximum pressure at the base, and decreases as water level in the trench declines.



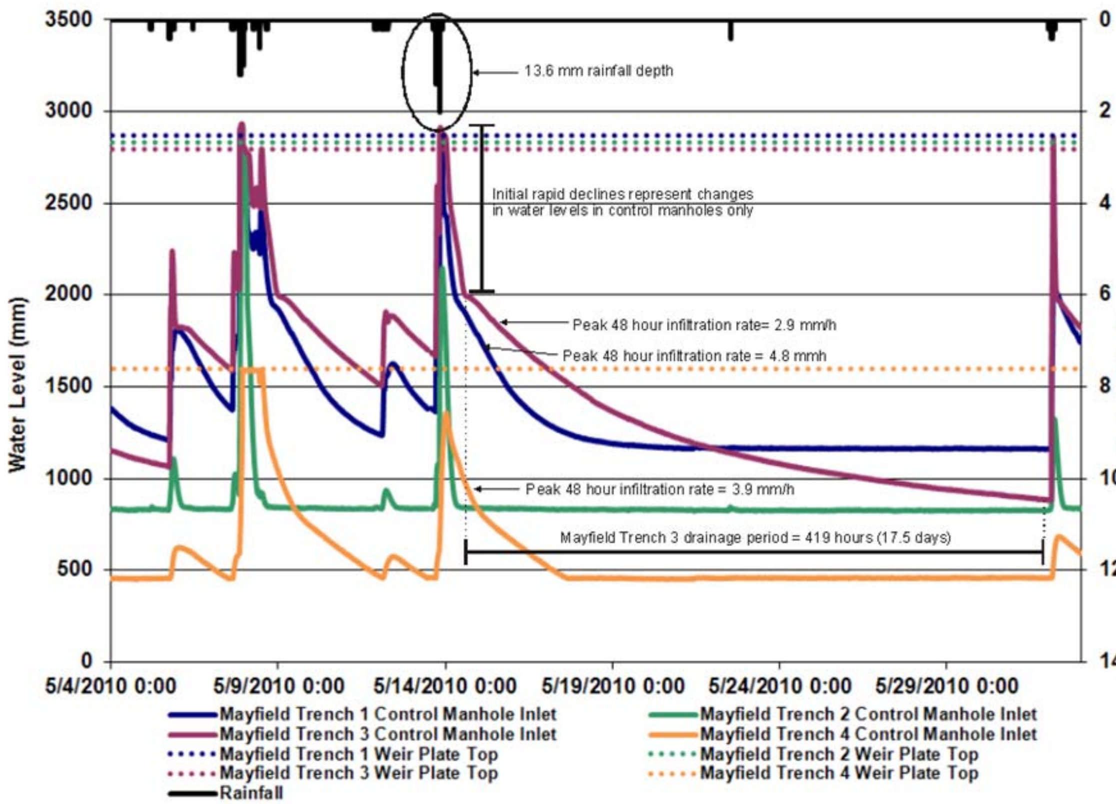


Figure 3.11: Water levels in Mayfield Trenches 1, 2, 3 and 4 for selected drainage periods.

The rapid and continuous rate of decline of water levels in Mayfield Trench 2 control manhole indicate that water was not being stored in the control manhole for very long (Figure 3.11). It is believed that water rapidly leaks around the weir plate in the control manhole and into the outlet storm sewer. Following a storm event, water levels consistently decline rapidly to a minimum height of approximately 830 mm, which corresponds to the elevation of the invert of the storm sewer pipe on the outlet side of the control manhole. The fact that water levels did not decline below this point over the monitoring period indicates that the bottom perforated pipe connecting Mayfield Trench 2 to the inlet side of the control manhole is clogged, which was confirmed through inspection. As runoff from the roof area enters the trench before reaching the control manhole, the trench is likely capable of being filled to the elevation of the invert of the top perforated pipe before it begins to flow into the control manhole, around the weir plate and into the outlet storm sewer pipe. If this is the case, the trench only provides about 70% of the water storage capacity it was designed to hold. Unfortunately, due to leakage of water around the weir plate, water level measurements from the Mayfield Trench 2 control manhole cannot be used to determine if the trench is infiltrating nor at what rate infiltration is occurring.

Over the 2009 to 2011 monitoring period, water levels in Mayfield Trench 1 control manhole consistently reached a minimum height of 1150 mm above the bottom of the manhole (Figure 3.11) which corresponds to the elevation of the invert of the top perforated pipe (228 masl.) connecting the trench to the control manhole (Figure 3.3). It was also observed that the bottom perforated pipe connecting the trench to the manhole was clogged with sediment, preventing water levels in the control manhole to be indicative of water levels in the trench once they reach the invert of the top perforated pipe. This limited the portion of the overall drainage period that infiltration rates could be calculated for Mayfield Trench 1 to those from when the trench is completely full to when it is about 70% full.

Over the 2009 to 2011 monitoring period water level measurements in Mayfield Trench 3 control manhole indicated that the trench never fully drained between storm events. The minimum water level observed in Mayfield Trench 3 was 700 mm above the bottom of the control manhole, following a dry period between storm events of 22.8 days, at which time the trench was still 35% full of water (Figure 3.11). From this evidence it can be concluded that the drainage period target of 70 hours to fully drain the trench is not being achieved. Furthermore, the full storage capacity of Mayfield Trench 3 was never available at the onset of all storm events during the monitoring period. To evaluate whether or not Mayfield Trench 3 is capable of infiltrating the targeted volume of roof runoff necessary to maintain pre-development infiltration volume over the site, modeling work was undertaken to predict the volume of water infiltrated during a simulated average year of precipitation, as described in Section 3.6.

Over the 2009 to 2011 monitoring period water level measurements in Mayfield Trench 4 control manhole consistently reached a minimum height of 460 mm above the bottom of the manhole (Figure 3.11) which corresponds to the elevation of the invert of the perforated pipe (228 masl) connecting the trench to the control manhole (Figure 3.3). As the design of Mayfield Trench 4 did not include a perforated pipe along the bottom of the trench that connects it to the inlet side, water levels in the control manhole are only indicative of levels in the trench when they are above the invert of the perforated pipe. As in the case of Mayfield Trench 1, this limited the portion of the overall drainage period that infiltration rates could be calculated for Mayfield Trench 4 to when the trench is completely full to when it is about 70% full.

Estimates of infiltration rates over selected drainage periods during the 2009 to 2011 monitoring period were calculated for Mayfield Trenches 1, 3 and 4 using Equation 4.

Equation 4: $i = ((\Delta V_t * p) + \Delta V_m)/A_f * 1/t * 1000$

Where,

i = infiltration rate over selected drainage period (mm/h)
ΔV_t = Change in volume in trench (m^3) = $\Delta h * A_f$
p = Porosity of gravel in trench (assumed to be 0.4)
Δh = Change in control manhole water level (m)
A_f = Footprint surface area of trench (m^2)
ΔV_m = Change in volume in control manhole (m^3) = $\Delta h * A_m$
A_m = Footprint surface area of control manhole (m^2)
t = Drainage period duration (h)

In order to compare the performance of each trench, estimates of post-development infiltration rates were made based on drainage periods that began with the trench being full or nearly full following a storm event, with no flow being registered in the outlet side of the downstream control manhole, and ended 48 hours later. This infiltration rate estimate, hereafter referred to as the "peak 48 hour infiltration rate", represents the maximum infiltration rate each trench is capable of achieving over a 48 hour period. A 48 hour period was chosen as the drainage period for comparison purposes as it is the recommended drainage time for design of infiltration practices in Ontario (OMOE, 2003, CVC & TRCA, 2010).

Table 3.3 summarizes estimates of peak 48 hour infiltration rates for post-development conditions for Mayfield Trench 1, 3, and 4. Examination of peak 48 hour infiltration rates indicate that the infiltration rate of the clayey silt till native soil underlying the trenches is considerably less than the 12 mm/h assumed in the design of the trench system. Over the monitoring period, peak 48 hour infiltration rate ranged between 2.5 and 6.4 mm/h with lowest rates observed for Mayfield Trench 3 and highest rates observed for Mayfield Trench 1. The slower than expected infiltration rates explain why the trenches were observed to overflow more frequently than expected. It is worth noting that no consistently downward trend was observed in infiltration rates over the 2 year period of monitoring, despite the high ratios of impervious drainage area to facility footprint area (64:1 to 155:1). This is likely due to the fact that the trenches only receive roof runoff which typically contains a very low sediment load, which limits the potential for clogging of the trench bottom. In comparison, an experimental infiltration trench installed on the campus of Villanova University in Pennsylvania, that receives runoff from a heavily used parking lot with a similar impervious drainage area to facility footprint area ratio (160:1) and no pretreatment to remove sediments, infiltration rates were observed to decrease exponentially during the first and second years after installation (Emerson et al., 2010).

Table 3.3: Peak 48 hour infiltration rates for Mayfield Trenches 1, 3 and 4 over the monitoring period and seasonal comparison

Trench	Season	Infiltration Rate (mm/h)				
		Mean	Minimum	Maximum	Standard Deviation	Number of Observations
Mayfield Trench 1	All	5.1	3.6	6.4	0.8	51
	Winter	5.5	4.5	5.9	0.5	8
	Spring	5.5	4.5	6.4	0.6	16
	Summer	4.4	3.6	5.6	0.8	11
	Fall	5.0	3.8	5.6	0.6	16
Mayfield Trench 3	All	3.1	2.5	3.8	0.3	52
	Winter	3.0	2.6	3.5	0.3	9
	Spring	3.0	2.7	3.4	0.2	16
	Summer	3.0	2.5	3.5	0.3	10
	Fall	3.2	2.6	3.8	0.4	17
Mayfield Trench 4	All	3.8	3.3	4.1	0.2	40
	Winter	3.7	3.3	3.9	0.2	6
	Spring	3.7	3.5	3.9	0.1	13
	Summer	3.9	3.6	4.1	0.2	9
	Fall	3.8	3.6	4.0	0.1	12

To provide additional information about the properties of the native subsoil in the area, soil core samples were collected at a depth of 30 centimetres (cm) below grade in an undeveloped portion of the subdivision area adjacent to 86 Pillsworth Road and submitted to the Ontario Ministry of the Environment analytical laboratory for grain size distribution analyses. Measurements of field saturated hydraulic conductivity of the subsoil at this depth were also taken at the soil core locations using a Guelph permeameter. Based on the mean values obtained from three samples (0.3% sand; 57.2% silt; 42.5% clay), the grain size analyses indicated that the predominant texture of the subsoil near the surface is clayey silt. This is consistent with the results obtained through samples from boreholes drilled in the subdivision area as part of preliminary geotechnical investigations for development planning and design (Shaheen and Peaker, 2004). The mean value for field saturated hydraulic conductivity obtained from measurements of the clayey silt subsoil at 30 cm below grade was 3.4 mm/h, which is within the range of infiltration rate values observed for Mayfield Trenches 1, 3 and 4 (2.5 to 6.4 mm/h). These results provide further evidence that the infiltration rate of the native subsoil in the area is less than the assumed 12 mm/h (1 x 10^{-6} cm/s).

Figure 3.12 illustrates how infiltration rates change as water levels in Mayfield Trench 3 decline. Only about 20% of the water storage capacity of the trench (approximately 25.2 m^3 of runoff) drains within the first 48 hours after a storm event with an infiltration rate between 2.5 and 3.8 mm/h being achieved. Infiltration rate decreases to between 1.5 to 2.5 mm/h when water levels are between 1600 to 1200 mm in depth (from 80% to 60% of the water storage capacity). Once water levels in the trench are below 1200 mm depth (less than 60% water storage capacity) infiltration rates in the range of 1.0 to 1.5 mm/h were observed.

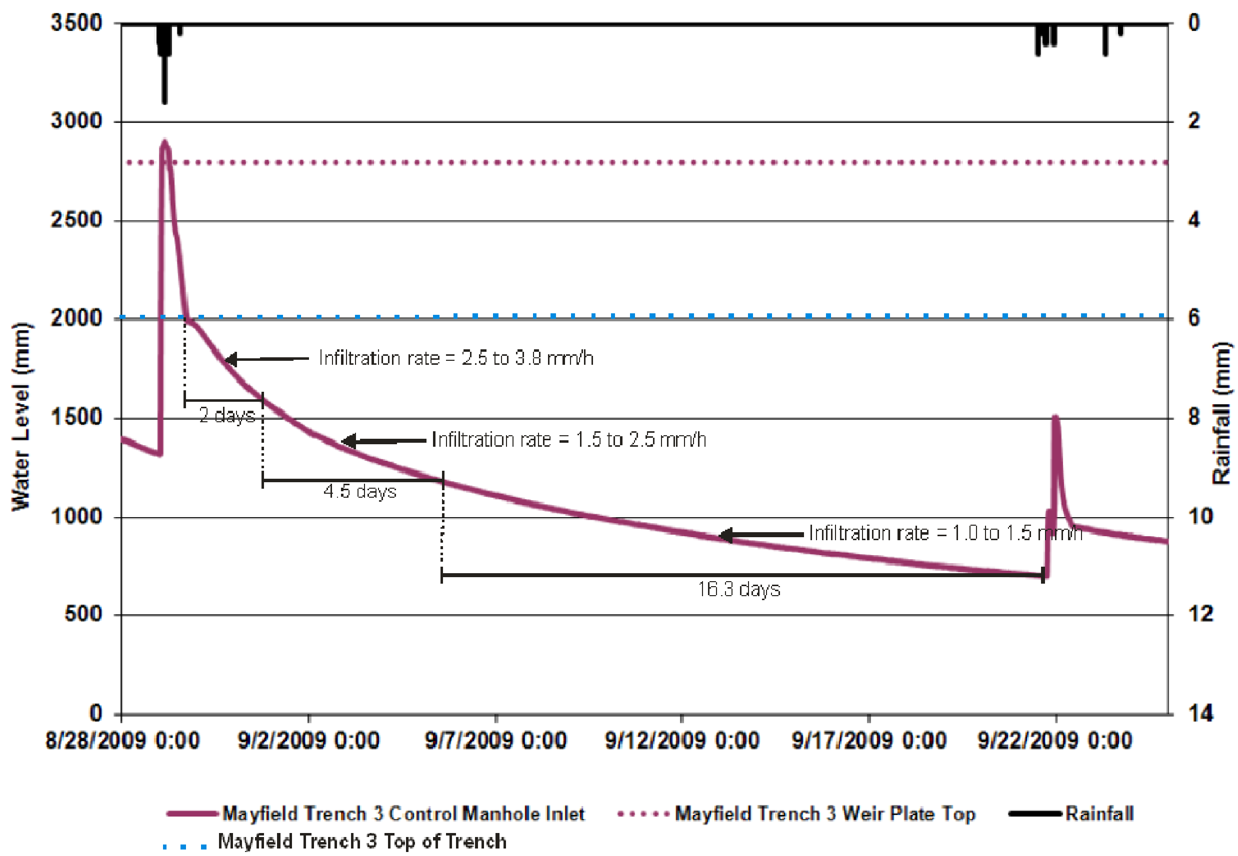


Figure 3.12: Changes in Mayfield Trench 3 infiltration rates over a selected drainage period

To investigate if infiltration rates vary considerably between seasons, peak 48 hour infiltration rates of Mayfield Trenches 1, 3 and 4 were compared for winter, spring, summer and fall drainage periods. The results of seasonal comparisons, summarized in Table 3.3, indicate that drainage performance of the trenches does not vary substantially throughout the year. This is contrary to findings by Emerson et al. (2010) in their Villanova University campus study of the hydraulic evolution of an infiltration trench. They observed strong seasonal variations in drainage periods and infiltration rates with higher rates during warm summer temperatures and lower rates during winter. This cyclical pattern of variation was attributed to the temperature dependency of the viscosity of water and its direct impact on hydraulic conductivity (Emerson et al., 2010). It is possible that strong seasonal patterns of variation in infiltration rates were not observed in this study due to the depth at which the Mayfield Industrial Park infiltration trenches are installed (1.5 m below ground), which likely insulates the trenches well from temperature fluctuations at ground surface.

3.5 Annual Infiltration Volume

Performance of the Mayfield Industrial Park infiltration trench system was also evaluated with regard to the overall design objective of maintaining average annual infiltration volume over the development site at pre-development levels. Due to the configuration of roof drainage pipes, catchbasins and storm sewers servicing the development sites it was not possible to accurately measure the volume of roof runoff that flowed into each trench and out of each control manhole, which limited the means available to estimate annual infiltration volume to modeling methods. The modeling method chosen involved using an event based roof runoff model to predict the volume of water being delivered to an individual trench on a daily basis and using observed water levels from Mayfield Trench 3 during the longest duration drainage event observed over the monitoring period to predict the volume of water infiltrated on a daily basis. The Mayfield Trench 3 drainage event used is illustrated in Figure 3.12.

To predict the volume of runoff infiltrated by Mayfield Trench 3 over an average year of precipitation, a simulated daily precipitation data set was created using historical daily values from the Orangeville meteorological station from months between 1990 and 2000 where monthly totals were within 10% of 30 year climate normal monthly values (Table 3.4). The resulting data set produced a total precipitation depth for the simulated average precipitation year of 910.7 mm which is within 3% of the 30 year climate normal value of 891.7 mm (Environment Canada, 2011). Precipitation data from the Orangeville meteorological station was used for this analysis because it was the source of data used to develop the water balance model used to estimate the average annual infiltration volume over the development site which informed the sizing of the trenches (Shaheen & Peaker Limited, 2007). This simulated precipitation data set, representing an average year of precipitation on a daily time step, was input into the roof runoff model to predict the volume of water input to the trench.

Table 3.4: Description of the simulated "average precipitation year" data set

Month	Year	Total Precipitation (mm)[1]	30 Year Climate Normal Total Precipitation (mm)[2]	% Difference
January	1996	66.8	65.2	+2.5
February	2000	55.7	50.9	+9.4
March	1996	62.8	65.8	-4.6
April	2000	67.0	69.9	-4.1
May	1999	85.0	79.3	+7.2
June	1997	79.7	83.9	-5.0
July	1990	80.4	75.3	+6.8
August	1998	96.4	95.6	+1.0
September	1990	90.9	83.7	+8.6
October	1993	72.3	71.0	+1.8
November	1990	83.1	81.8	+1.6
December	1992	70.6	69.3	+1.9
Annual	**n/a**	**910.7**	**891.7**	**+2.1**

Notes:

1. Total precipitation includes both rain and snowfall. Values are monthly sums of daily precipitation totals from the Orangeville meteorological station for the year indicated (Environment Canada, 2011).
2. Climate normal monthly total precipitation values are based on average monthly values observed between 1971 and 2000 at the Orangeville meteorological station (Environment Canada, 2011)

The roof runoff model used was developed and calibrated as part of a previous study evaluating the performance of three rainwater harvesting systems in the Greater Toronto Area that receive runoff from the roofs of commercial buildings (TRCA, 2010). The spreadsheet model uses measured precipitation depth (both rain and snow), mean daily temperature and depth of snow on the ground as input data. The model assumes different percentages of rainfall loss due to temperature, wind and rainfall totals. These losses account for rainfall that evaporates directly from the roof surface or is not captured by the roof drainage system and spills off the roof surface. A simplified snowmelt equation was used to estimate the timing of rooftop snowmelt and resulting water yield. The model assumptions regarding rainfall loss were calibrated using measured precipitation depth and observed changes in cistern water levels over a one year period (TRCA, 2010). This roof runoff model was used to estimate daily water volume input to Mayfield Trench 3 over the simulated average year of precipitation.

Daily water volume input to the trench was translated into a daily increase in water level based on the dimensions of the trench and assuming the granular material used to fill the trench provides 40% void space. It was assumed that the trench was full (i.e. trench water level = 2000 mm) at the beginning of the modeling period (i.e. January 1, 12:00 AM), and that the decrease in water level due to drainage between the start and end of each day (24 hour period) can be predicted from water level measurements from Mayfield Trench 3 over the August 30, 2009 to September 21, 2009 drainage event. For each daily time step the trench water level at the end of each day was predicted using Equation 5. When the value yielded by Equation 5 was greater than 2000 mm (i.e. the depth of the trench), the maximum value of 2000 mm was used.

Equation 5: $H_e = H_s - \Delta H + (V_i/A_{f)}*(1/p)*(1/1000))$

Where,
H_e = Water level (height) at end of the day (mm), maximum value = 2000.
H_s = Water level (height) at start of the day (mm)
ΔH = Change in water level (height) due to drainage over a 24 hour period beginning at H_s, as predicted by Mayfield Trench 3 drainage event (mm)
V_i = Daily volume of water input to the trench (m^3)
A_f = Footprint area of the trench (m^2)
p = Porosity of gravel in trench (assumed to be 0.4)

To verify that the roof runoff/trench drainage model predicts trench water levels reasonably well, precipitation and temperature data measured over the 2010 calendar year were input to the model and the predicted trench water levels were compared to measured water levels from the Mayfield Trench 3 control manhole over the same time period. The results of this comparison (Figure 3.13) show that the model predicts water levels in the trench reasonably well during spring, summer and fall seasons. Predicted water levels during winter months deviated significantly from observed water levels during periods of snow accumulation and snowmelt. It is believed that the simplified snowmelt equation being used to predict water input to the trench oversimplifies the process and predicts flow to the trench over a longer period of time than what actually occurred. It is also believed that actual snowfall at the site and measured snowfall at the gauge site (Lester B. Pearson meteorological gauge), located 20 km away, differed significantly on occasions. However, once the predominant form of precipitation switched to rainfall in the spring (around March 15, 2010), the model predicts trench water levels with good accuracy (Figure 3.13).

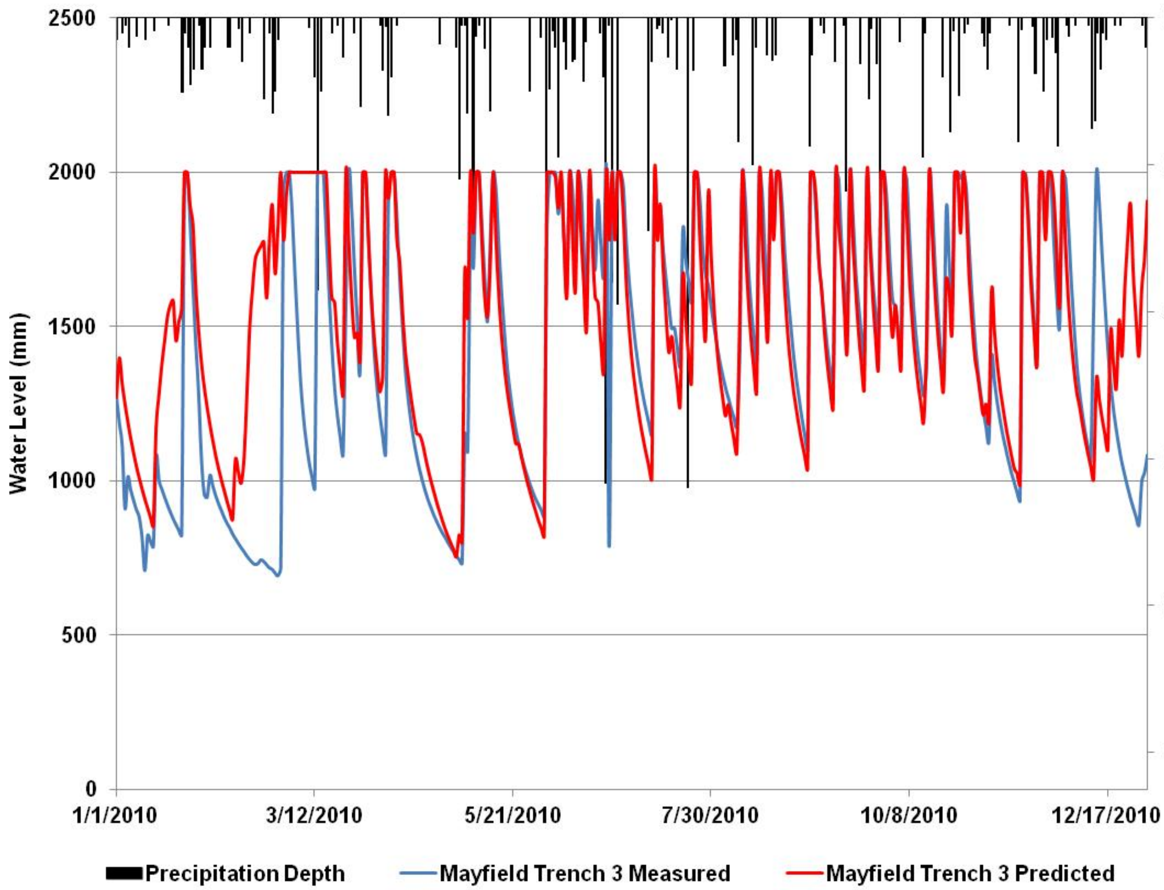


Figure 3.13: Comparison of model predicted Mayfield Trench 3 water levels with measured water levels for the 2010 monitoring period

The volume of water infiltrated over each 24 hour period was calculated based on the predicted change in trench water level due to drainage. The total volume of water infiltrated over the simulated average year of precipitation was calculated by summing daily infiltration volume for the year.

As part of the subdivision planning process to design the stormwater management system servicing the developments in which the Mayfield Trenches are located, a Thornthwaite monthly water balance model was developed to estimate pre-development and post-development conditions (Shaheen and Peaker, 2007). Based on the monthly water balance model it was predicted that development of the entire 29.85 ha. subdivision would result in a net deficit of 60,030 m^3 of infiltration in an average year of precipitation, without stormwater infiltration practices to mitigate the impact of increased impervious cover (Shaheen and Peaker, 2007). Assuming each lot in the subdivision contains roughly the same proportion of impervious cover this translates to an average annual infiltration deficit of 2,011.1 m^3/ha. (201 mm precipitation depth per year over the subdivision area). Since the lot in which Mayfield Trench 3 is located is 11.66 ha., this translates to an estimated annual infiltration deficit of 23,449 m^3 for the lot (2,011.1 * 11.66). Infiltration of roof runoff was intended to mitigate this annual infiltration deficit. Since Mayfield Trench 3 captures 40% of the total roof area on the lot in which is it situated, the portion of the annual infiltration deficit for the lot that the trench would need to infiltrate can be estimated as 9,380 m^3/year (23,449 * 0.4).

The model was used to predict the total volume of roof runoff infiltrated by Mayfield Trench 3 over the simulated average precipitation year. Five scenarios of differing trench sizes were also modelled, keeping trench depth constant, in an effort to identify an optimum trench water storage capacity that would achieve the targeted annual infiltration volume of 9,380 m^3. A description of the Mayfield Trench 3 modelling scenarios and results are summarized in Table 3.5.

Table 3.5: Mayfield Trench 3 modeling scenarios and predicted annual infiltration volumes

Scenario	Trench Depth (m)	Trench Footprint Area (m^2)	Ratio of impervious drainage area to trench footprint area	Trench water storage capacity per hectare drainage area (m^3/ha.)	Annual Infiltration Volume (m^3)	% Inflow
1	2	150	155:1	51.6	2,914	16
2	2	235	99:1	80.8	4,168	23
3	2	465	50:1	159.9	7,103	40
4	2	1165	20:1	400.5	13,314	75
5	2	680	34:1	233.8	9,381	53

Outputs of the model indicate that Mayfield Trench 3 would infiltrate 2,914 m^3 of roof runoff (16% of inflow) during the simulated average precipitation year, falling well short of the target of 9,380 m^3 (Scenario 1). By increasing the trench footprint area to 680 m^2 (4.5 times greater than existing) a total of 9,381 m^3 of roof runoff would be infiltrated (53% of inflow), which would just meet the target (Scenario 5). The water storage capacity of a trench this size would be 544 m^3 or 233.8 m^3/ha. of impervious drainage area (544 ÷ 2.3263). Assuming that Mayfield Trenches 1 and 2 drain at similar rates to Mayfield Trench 3 the water storage capacities needed to meet their portions of the infiltration volume target for the lot would be 350 m^3 (233.8 * 1.49619) and 470 m^3 (233.8 * 2.01008) respectively. This translates to a total required water storage capacity of the trench system of 1364 m^3 (544 + 350 + 470) and a sizing criterion of 117 m^3 water storage capacity per hectare of lot area (1364 ÷ 11.66). If the 2 m deep trench had been sized with a ratio of impervious drainage area to trench footprint area of 20:1, which is recommended in the Low Impact Development Planning and Design Guide (CVC & TRCA, 2010), it is predicted that the trench would infiltrate 75% of inflow from the roof drainage area and exceed the annual infiltration volume target by 3,934 m^3 (1.4 times the targeted volume).

Based on the same average annual infiltration deficit per hectare for the subdivision and a lot size of 3.21 ha., the volume of roof runoff that Mayfield Trench 4 needs to infiltrate to mitigate the loss of infiltration due to development of the lot can be estimated as 6,456 m^3/year (2,011.1 * 3.21). By applying the same modeling approach and assuming that Mayfield Trench 4 drains at a rate similar to Mayfield Trench 3, it can be predicted that Mayfield Trench 4 would infiltrate 3,981 m^3 (36% of inflow) in an average precipitation year. If the 2 m depth of Mayfield Trench 4 was maintained and the footprint area increased from 225 m^2 to 420 m^2 (1.9 times greater than existing), the model predicts it would infiltrate 6,709 m^3 (61% of inflow) in an average precipitation year, thereby exceeding the target by just 253 m^3. An infiltration trench of this size (336 m^3 water storage capacity) would provide a water storage capacity of 105 m^3/ha. of lot area.

3.6 Discussion

As was the case with the Elgin Mills Crossing infiltration chamber system (Section 2.0), field monitoring results indicate problems with the function of the control manholes in the Mayfield Industrial Park infiltration trench system. The control manhole of Mayfield Trench 2 was found to be leaking and allowing water to rapidly pass from the inlet side of the weir plate to the outlet side. The bottom perforated pipe connecting Mayfield Trench 2 to the control manhole was also found to be clogged with sediment. These deficiencies limit the water storage capacity of the trench to 70% of the designed volume. It also prevents water levels in the control manhole from being indicative of water levels in the trench. While the control manhole continues to leak and the bottom perforated pipe remains clogged, there is no means of evaluating the infiltration performance of the trench. The fact that the bottom perforated pipe in Mayfield Trench 2 is clogged is actually preserving a portion of the water storage capacity of the trench as it allows the trench to be filled to the level of the invert of the top perforated pipe before flows reach the control manhole, leak to the outlet side and flow into the outlet storm sewer pipe.

To repair Mayfield Trench 2, water in the control manhole should be pumped into the outlet storm sewer pipe and additional sealing of the weir plate and sides of the manhole structure should be undertaken to stop leakage of water from the inlet side to the outlet side. The inlet side of the manhole should then be filled with water to determine if leakage has been addressed. If leakage has been remedied, steps should be taken to unclog the bottom perforated pipe. If leakage cannot be remedied, it is recommended that steps be taken to permanently seal off the bottom perforated pipe from the control manhole to preserve the existing function of the infiltration trench over the lifespan of the facility, albeit at a reduced water storage capacity.

In Mayfield Trench 1 the bottom perforated pipe connecting it to the control manhole was also found to be clogged with sediment, which prevents water levels in the manhole from being indicative of water levels in the trench when below the invert of the top perforated pipe. While this does not affect the water storage capacity of the trench it limits the utility of the control manhole as a device for monitoring drainage times over the lifespan of the trench. Steps should be undertaken to remove sediment accumulated at the bottom of the inlet side of the control manhole and unclog the bottom perforated pipe, which could done with a vacuum truck typically used to clean storm sewer catchbasins. A design feature that would help prolong periods between sediment removal maintenance activities is provision of a sump in the control manhole that would allow sediment to accumulate without clogging the bottom perforated pipe.

Sediment accumulation was also noted in the inlet side of the Mayfield Trench 3 control manhole, and in the inlet manhole of Mayfield Trench 4, which features a Goss trap structure designed to prevent coarse sediment and debris from entering the trench. Sediment removal maintenance should also be undertaken in these locations and periodic inspections should be done to determine the frequency at which such maintenance activities will be needed in the future.

As noted in the Discussion section for the Elgin Mills Crossing infiltration chambers evaluation (Section 2.5), it is strongly recommended that stormwater infiltration facilities be thoroughly inspected by the construction project manager, system designer or ultimate owner/manager of the infrastructure prior to assumption. Inspection procedures should include continuous water level monitoring over several storm events or a synthetic runoff test to determine if the system is functioning as designed. Contracts that include construction of such stormwater infrastructure should include conditions whereby any defects or deficiencies revealed through final inspection can be corrected prior to assumption. Visual inspection of Mayfield Trench 2, during a storm event when the other trenches on the property were filled to a level above the elevation of the outflow sewer pipe invert would have revealed that leakage around the weir plate was occurring.

Monitoring also indicated that the post-development infiltration rate of the soils underlying the infiltration trench system is lower than expected, resulting in slower than expected drainage of stored stormwater and more frequent occurrence of overflow. It is worth noting that the method used to estimate the pre-development infiltration rate of the native subsoil was based on general relationships between soil texture and hydraulic conductivity (OMMAH, 1997), rather than field measurements. Tests to estimate the permeability of the subsoil at the approximate depth to which the trenches extend should have been undertaken prior to detailed design. Guelph permeameter measurements of the field saturated hydraulic conductivity of surficial subsoil in an undeveloped lot adjacent to 86 Pillsworth Road, indicated a value of 3.4 mm/h for the clayey silt glacial till soil, which agrees well with the range of values observed through monitoring of drainage times of Mayfield Trenches 1, 3 and 4 (2.5 to 6.4 mm/h).

Observed peak 48 hour infiltration rates for all trenches monitored were very similar and did not exhibit significant seasonal variation. This is consistent with observations from underground stormwater infiltration facilities by other researchers (Roseen et al., 2009). Of particular note is the fact that infiltration rates do not substantially decrease during winter. This is owing to the fact that the base of each trench is approximately 3.5 m below ground surface and therefore is well insulated from surface temperature fluctuations throughout the year.

Since the trenches do not fully drain between storm events, the control manholes contain standing water year-round, which raises the question of whether or not they provide mosquito breeding habitat. To investigate this, water samples were collected from the control manholes during the mosquito breeding season, in accordance with the TRCA West Nile Virus monitoring protocol, and analyzed to determine if they contained mosquito larvae. No larvae were observed suggesting that the presence of standing water in control manholes does not provide mosquito breeding habitat. This is likely due to the fact that the manhole covers only have small openings on the surface. In future designs of stormwater infiltration systems located on low permeability soils, it is recommended that the type of covers used on control manholes should contain no holes at all, or screens to help ensure mosquitoes cannot enter them.

Future phases of development of the subdivision in which the study area is located should consider including stormwater infiltration facilities that are designed with the knowledge gained from this evaluation regarding permeability of the native subsoil. If additional infiltration trenches will be implemented, they should continue to be 2 m deep, which will maintain 1 m of hydraulic head or more in them the vast majority of time between storm events. As indicated by observed infiltration rates from Mayfield Trench 3, designing to maintain hydraulic head in the infiltration trench helps to maximize the drainage rate and thereby, the volume of water infiltrated on an annual basis. Infiltration rates for Mayfield Trench 3 when the trench was full or nearly full (i.e. 1.6 to 2 m of hydraulic head available) were observed to be 2.5 times higher than when the trench was half full or less (Figure 3.12). Future infiltration trenches should be sized assuming a peak 48 hour infiltration rate of 3 mm/h for the native clayey silt subsoil and that a portion of the water storage capacity of the trench will never fully drain between storm events. As part of geotechnical investigations in support of future development phases, measurements of the permeability of the subsoil at the approximate depth that the facilities will be installed are also recommended to confirm or refine this assumption.

Modelling of the volume of stormwater infiltrated by Mayfield Trench 3 during an average precipitation year indicates that the Mayfield Industrial Park infiltration trench system is not meeting the design objective of mitigating the annual infiltration volume deficit caused by development of the lots. While modeling was only undertaken for Mayfield Trenches 3 and 4, based on the similarity of peak 48 hour infiltration rates observed for Mayfield Trench 1, and the fact that Mayfield Trench 2 provides only 70% of its designed water storage capacity, it can be concluded that all trenches in the system are not meeting this objective. As described in the previous section, it is estimated that Mayfield Trenches 3 and 4 would need to have been built with footprint areas 4.5 and 1.9 times larger than their existing footprints, while maintaining their depth at 2 m in order to meet their respective targets for average annual infiltration volume. This translates to a facility sizing criterion of at least 117 m^3 of water storage capacity per hectare of lot area, which could be used as the basis for design of stormwater infiltration practices in future phases of development of the subdivision if impervious cover and ratios of lot area to roof area are similar to the lots examined in this study.

While the Mayfield Industrial Park infiltration trenches are draining more slowly than expected and not fully compensating for the loss of infiltration from development of their respective lots, results of this evaluation suggest that if they had been designed based on better knowledge of the permeability of the native subsoil, they could fully compensate for the loss of infiltration caused by the development through infiltration of roof runoff alone. This finding is significant considering that stormwater infiltration practices are widely considered to have limited effectiveness on fine-textured soils.

A practice that would improve the effectiveness of the Mayfield Industrial Park infiltration trenches to reduce runoff volume from their respective lots, while providing an additional benefit of conserving potable water is to use submersible pumps installed in the inlet sides of the control manholes to draw on the stored water for uses not requiring potable water (e.g. landscape irrigation, vehicle washing), like a rainwater harvesting cistern. This would help to reduce trench drainage times, the frequency at which the trenches overflow and runoff volume from the lot.

In summation, despite deficiencies encountered with the function of control manholes, it was possible to observe that drainage of the Mayfield Industrial Park infiltration trench system is occurring at a slower rate than expected and that infiltration rate decreases exponentially as water levels in the trench decline. Slower than expected infiltration rates are causing drainage times to be much greater than the targeted 48 to 72 hours and causing the trenches to overflow much more frequently than expected. Over the monitoring period of this study it is likely that all of the trenches never fully drained between storm events, so their water storage capacity was never fully available at the onset of all storms. It was also possible to conclude that the trench system is not achieving the design objective of fully compensating for the loss of infiltration caused by development of the site. It was estimated through modeling that water storage capacity of the trenches would need to be between 1.9 and 4.5 times greater than the trenches currently provide in order to infiltrate the targeted volume of roof runoff that would mitigate the loss of infiltration on an average annual basis. While the infiltration trenches are not achieving their performance objectives as currently designed, it is estimated that if 2 m deep trenches sized to provide at least 117 m^3 of water storage capacity per hectare of lot area were implemented in future phases of the industrial/commercial subdivision development, that the average annual infiltration volume target could be achieved through infiltration of roof runoff alone on lots with similar levels of impervious cover (85 to 90%) and where the ratio of lot area to roof area is approximately 2:1.

4.0 BRAMPORT INFILTRATION CHAMBER SYSTEM

4.1 Site Description

The Bramport infiltration chamber system is located in a shopping centre development at the southwest corner of Airport Road and Bovaird Drive (also known as Castlemore Road), in the City of Brampton, Ontario (Figure 4.1). The chamber system receives runoff from a 33,500 m^2 drainage area composed of the roofs of seven commercial buildings and associated parking lots and access roads. The CULTEC Recharger V8HD® subsurface stormwater chamber system is installed under a parking lot and receives flow from two storm sewer inlets and a catchbasin inlet directly above it and ultimately drains to a warm water tributary of the Humber River. The system is designed to provide temporary storage and controlled release of runoff from storms up to the 100 year return period event and the opportunity for infiltration. However the water storage capacity of the system was sized assuming no infiltration would be achieved, as the characteristics of the native soils and underlying geology encountered during geotechnical investigations suggested low levels of permeability (Soil Engineers, 2005; Weslake Incorporated, 2006).

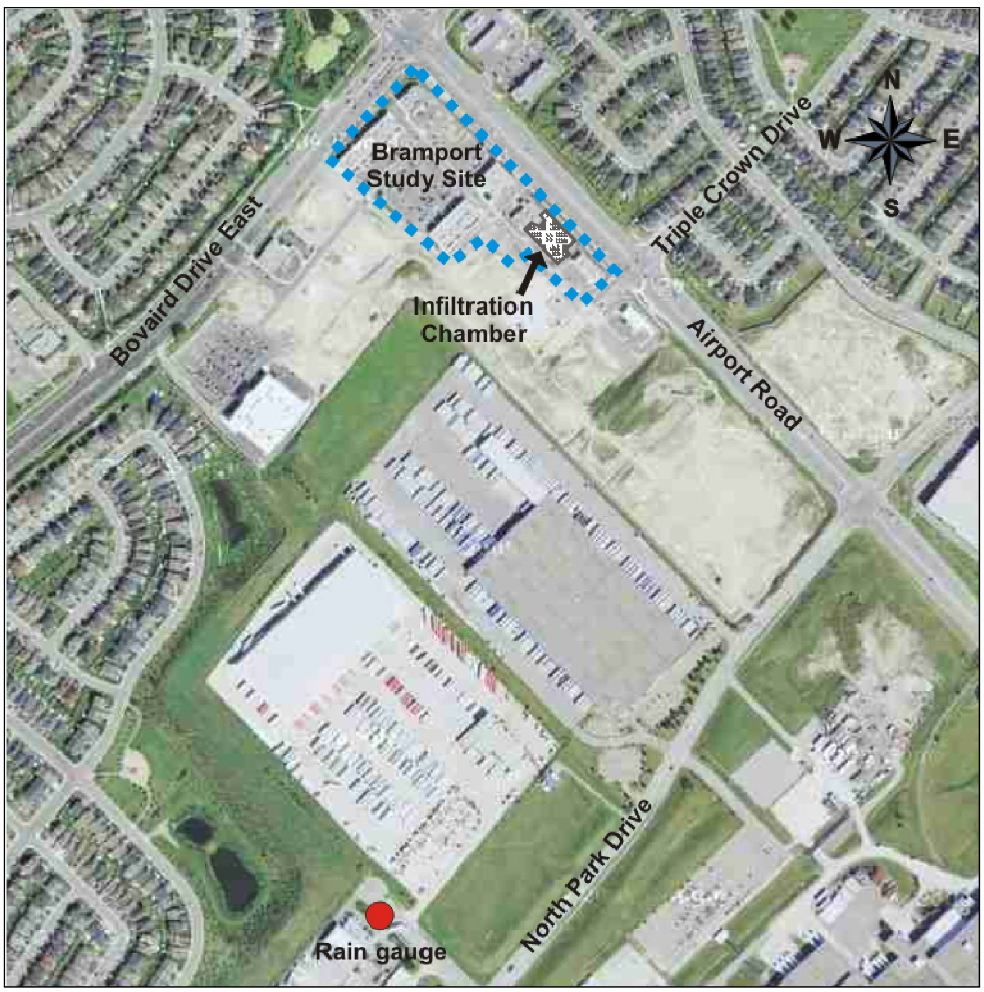


Figure 4.1: Bramport infiltration chamber system site

Surficial soils and underlying geology of the Bramport site are composed of low permeability sandy silty clay glacial till containing occasional cobbles and boulders, from ground surface to the top of the underlying Georgian Bay Formation bedrock (Soil Engineers, 2005). The sandy silty clay till deposit is cohesive material with low plasticity. Lenses or discontinuous layers of very dense sandy silt till were found embedded in or below the firm to hard sandy silty clay till at various depths and locations at the site. Bedrock has been mapped in the region at depths of 25 to 63 m below ground surface (Soil Engineers, 2005). The shallow Georgian Bay Formation bedrock is weathered and fractured shale and considered to be a productive aquifer, contributing significantly to private water supplies in rural areas to the north (Shaheen and Peaker, 2006). Based on soil samples taken from boreholes completed to 5 m depth within the development site it is estimated that the seasonally high water table occurs at depths between 2.5 and 4.6 m below ground surface in the vicinity of the infiltration chamber system, although the water table was not encountered in any boreholes at the time of drilling (Soil Engineers, 2005). Slight seepage during borehole drilling at various depths was observed and interpreted to indicate that perched groundwater likely occurs in places at shallow depths during wet seasons (Soil Engineers, 2005).

The stormwater management system for the Bramport development site, which includes six infiltration chamber systems, was designed to maintain pre-development peak flow rates and provide enhanced water quality control without the need for a stormwater detention pond. Temporary storage and gradual release of runoff is achieved through a combination of outflow controls in the storm sewer system, on roof drains and parking lot catchbasins. Orifice flow restricting plates, mounted on the outlets of each manhole at which underground infiltration chamber systems are located, cause stormwater to back up into the chambers and contributing storm sewers. Outflow controls on roof drains and parking lot catchbasins also cause stormwater to temporarily pond on these surfaces during large or very intense precipitation events (i.e., rooftop and parking lot storage). Oil and grit separator units installed in-line in the storm sewer system provide the required level of stormwater quality control. The storm sewer system drains to the lower branch of the West Humber River, a tributary to the Humber River, which ultimately flows to the north shore of Lake Ontario. This branch of the West Humber River supports small to intermediate riverine warm water aquatic habitat.

4.2 System Design

The Bramport infiltration chamber system receives roof, road and parking lot runoff from a 33,500 m^2 drainage area composed of commercial and retail developments (Table 4.1; Figure 4.2). Pretreatment of runoff prior to it being released into the chambers is provided by in-line oil and grit separator units upstream of the manholes controlling flow into and out of the chambers. The chamber system covers an area of 1520 m^2. The chambers are 0.86 m in height and are embedded in a 1.16 m deep gravel bed (Figure 4.3) filled with 20 to 50 mm diameter clear crushed stone that provides additional stormwater storage capacity. The gravel bed excavation is lined with CULTEC No. 410 geotextile fabric to provide separation between aggregate bed material and the underlying native soil and overlying soil and aggregate mixture. Assuming a void space ratio of 40% in the surrounding gravel bed, the chamber system provides a total of 1,192 m^3 of storage volume. This includes an 833.3 m^2 (17.54 m x 47.51 m) area below the western half of the chamber system where the underlying gravel bed is 0.3 m deeper than the bedding below the remainder of the system and has no outlet (i.e., located below the invert of the inlet/outlet pipe from the control manhole). This gravel bed storage area provides 100 m^3 of water storage, in addition to what was required to meet flood control requirements. It was included in the design of the system to act like a permanent pool to provide better retention of sediment. It also allows monitoring of water levels to evaluate whether or not infiltration is being achieved and to estimate the post-development infiltration rate of the native soils if infiltration does occur.

Table 4.1: Bramport infiltration chamber system and drainage area specifications

Impervious drainage area	33,500 m^2
Runoff source area type	Roof, access road and parking lot
Location of chambers	Under parking lot
Chamber system footprint area (total)	1520 m^2
Ratio of impervious drainage area to chamber system footprint area	22:1
Chamber system storage volume (total)	1192 m^3
Contributing storm sewer storage volume	Not applicable
Ratio of impervious drainage area to chamber system footprint area	22:1
Additional gravel bed footprint area	833.3 m^2
Additional gravel bed storage volume	100 m^3
Hydraulic conductivity of sandy silty clay till [1]	1 x 10^{-6} cm/s
Approximate infiltration rate of sandy silty clay till [2]	12 mm/h
Gravel bed fill material	Clear 20-50 mm diameter crushed stone

Notes:
1. Estimated based geometric mean value obtained from grain size analyses of soil samples from 6 boreholes in the vicinity of the chamber system (Soil Engineers, 2005)
2. Based on approximate relationship between hydraulic conductivity and infiltration rate (OMMAH, 1997)

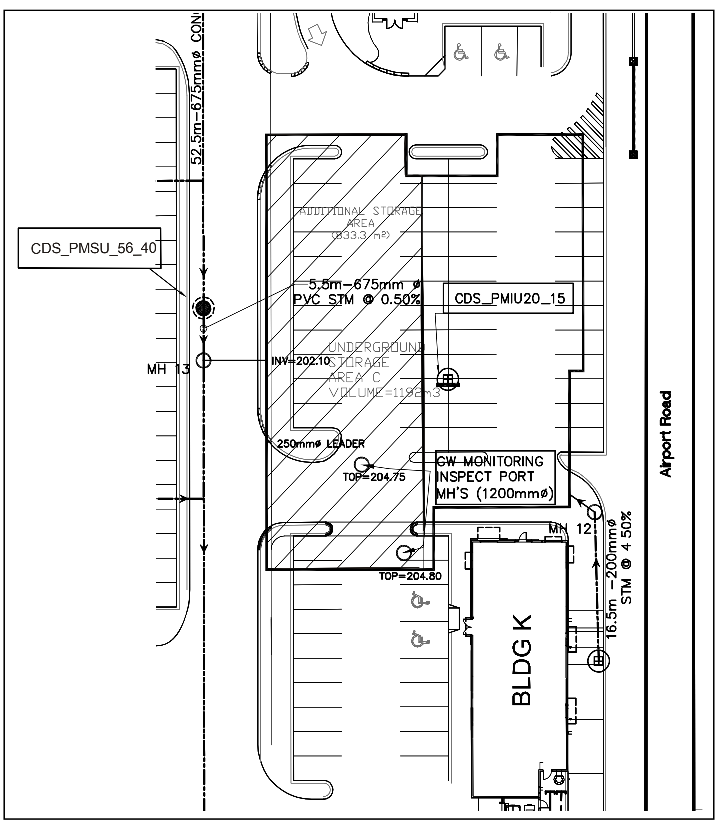


Figure 4.2: Bramport infiltration chamber system plan view (Adapted from Counterpoint Engineering, 2007)

At the main inlet on the west side of the chamber system, inflow of stormwater is controlled by a manhole (MH 13) where outlet of flows from the incoming 675 mm diameter storm sewer pipe are restricted by a 230 mm diameter orifice plate mounted on the outlet side of the manhole. When flow into the manhole exceeds the maximum flow rate of the orifice plate, stormwater backs up and flows into the chambers through a 600 mm diameter pipe. The invert of the pipe is set at an elevation that allows the majority of water temporarily stored in the chamber system to drain out the orifice plate, except for the 100 m^3 of water stored in the additional gravel bed area below the western half of the chamber system (Figure 4.3). At the inlet directly over the chamber system, inflow of stormwater is controlled by a catchbasin equipped with an orifice flow restricting plate on the outlet side that flows into an oil and grit separator unit and then into the chamber system. At the inlet on the east side of the system, stormwater collected by a single catchbasin with no outflow control flows to a manhole containing a sump and then into the chamber system.

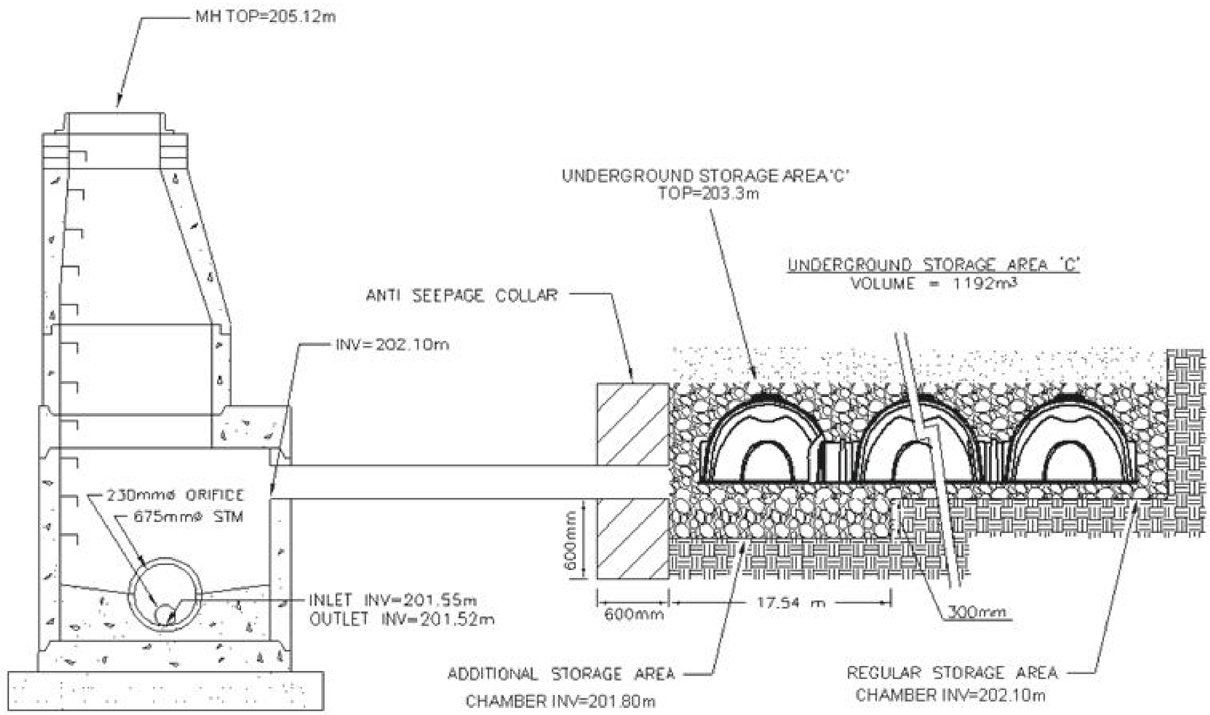


Figure 4.3: Bramport infiltration chamber and control manhole cross-section (Adapted from Counterpoint Engineering, 2007)

The top of the chamber system gravel bed is a minimum of 1.5 m below ground surface which is below the local maximum frost penetration depth of 1.4 m (MTO, 2005) and should not be subject to freezing during cold winter temperatures. The base of the additional gravel bed area below the western half of the chamber system is approximately one metre above the interpreted seasonally high water table elevation which suggests that seasonally high groundwater levels will not interfere with its performance and that hydraulic gradient at the base should remain positive throughout the year.

Boreholes completed as part of geotechnical investigations in support of the development design indicate that overburden native soils are sandy silty clay till to a depth of at least 5 m below ground surface in the vicinity of the chamber system. Mean values for grain size distribution obtained from six borehole samples indicate that the native subsoil in the development area is 9% gravel, 28% sand, 28% silt and

35% clay (Soil Engineers, 2005). Based on these grain size analyses results, the estimated hydraulic conductivity of the native soil is in the order of $1x10^{-6}$ cm/s (Soil Engineers, 2005) which represents an infiltration rate of approximately 12 mm/h (OMMAH, 1997). Assuming a post construction native soil infiltration rate of 12 mm/h is achieved, infiltration of 100 m^3 of stormwater in the 833 m^2 additional gravel bed area would occur over roughly 10 hours (100 m^3/833 m^2*1000 mm/m/12 mm/h).

Included in the design of the chamber system are two observation port wells located in the portion of the chamber system that includes the additional gravel storage area. The observation port wells are 1200 mm diameter concrete manholes that are open on the bottom and extend to the bottom of the additional gravel storage area below the chamber system. The manholes are connected to the chamber system by 600 mm diameter pipes and intended to provide a means of monitoring water levels in the chamber system and surrounding gravel bed. A groundwater monitoring well was intended to be located in a landscaped area to the east of the chamber system that extends 5 m below ground surface (to the elevation of the seasonally high water table). The well was called for by the system designer but was never constructed. The groundwater monitoring well was intended to enable examination of the effect of the chamber system on the local water table.

4.3 Monitoring Locations and Equipment

Monitoring parameters, locations and equipment for the Bramport site were selected to provide the information needed to estimate the post-development native soil infiltration rate below the chamber system and drainage times being achieved in the additional gravel bed below the western half of the chamber system. They also provide a means of determining what size storm events cause runoff to be temporarily stored in the chamber system. Also of interest was estimating total runoff volume flowing into and out of the chamber system on the basis of individual storm events and over each monitoring season, which could be used to estimate the total volume of runoff infiltrated by the chamber system. However, due to the various configurations of the three inlets to the chamber system and the orifice plates mounted on the outlet sides of the control manhole and catchbasin inlet, flow metering at all the locations necessary to estimate total volume of flow into and out of the chamber system was not possible.

Rainfall was measured with a three season 8 inch diameter tipping bucket rain gauge located approximately 1 km south of the site during the period from June 1 to August 31, 2009 and May 2010 (Figure 4.1). A second gauge located at the Laidlaw Transit property at 12117 Airport Road, approximately 5.3 km northwest of the site, served as a back-up gauge during periods when the primary gauge was out of service. Precipitation data (daily totals) during winter months (January to March and December) are taken from the Lester B. Pearson International Airport meteorological station located approximately 12 km southeast of the site.

Calibrated pressure transducers were installed in the observation port wells (Figure 4.4) and set to the bottom of the manholes. The pressure transducers provide continuous water level measurements at 5 minute intervals. Due to the design of the chamber system, water level data from the observation port wells could only be used to estimate post-development infiltration rates in the 833.3 m^2 additional gravel bed area under the western half of the chamber system, which is essentially a 0.3 m deep infiltration trench. To do so, data from the observation port wells could only be used when water levels were at or just below the elevation of the top of the additional gravel bed area (i.e., at or below the invert of the pipe connecting the chamber system to the control manhole).

An area velocity sensor was installed in the pipe connecting the control manhole (MH 13) to the chamber system (Figure 4.4). This sensor measures flow volume and rate into and out of the chamber system, and provides an indication of when flow from the chamber system into the control manhole had ceased, confirming that subsequent water level declines in the observation port well reflect losses to infiltration only.

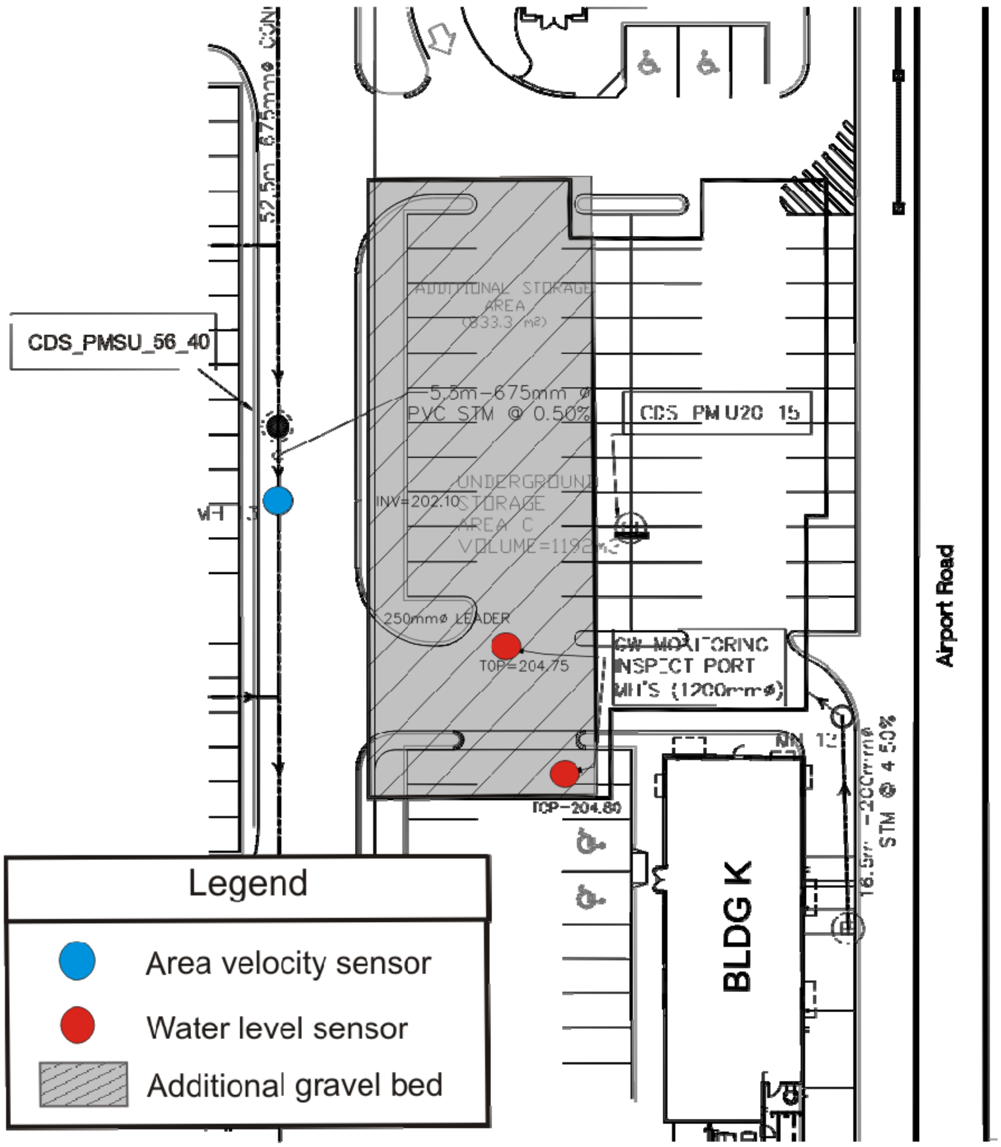


Figure 4.4: Schematic of monitoring locations at Bramport study site (Adapted from Counterpoint Engineering, 2007)

4.4 Results

4.4.1 Precipitation

Figure 4.5 illustrates monthly totals for precipitation over the 2009 to 2011 period from the Bramport, Laidlaw Transit and Pearson Airport rain gauges and 30 year normals (monthly averages from 1971 to 2000) from the nearest rain gauge with a sufficient period of record ("Sandhill" climate station; Environment Canada, 2010). During the June 2009 to July 2011 period when monitoring at the Bramport site was underway, some particularly noteworthy periods occurred when precipitation deviated substantially from 30 year climate normals. Precipitation during the months of September 2009 and July 2011 was in the order of half the normal depths. The longest dry periods observed between storm events occurred during these months. By far the wettest month during the monitoring period was June 2010 when 164 mm of precipitation occurred (86% more than normal), breaking monthly rainfall records in many areas in the region.

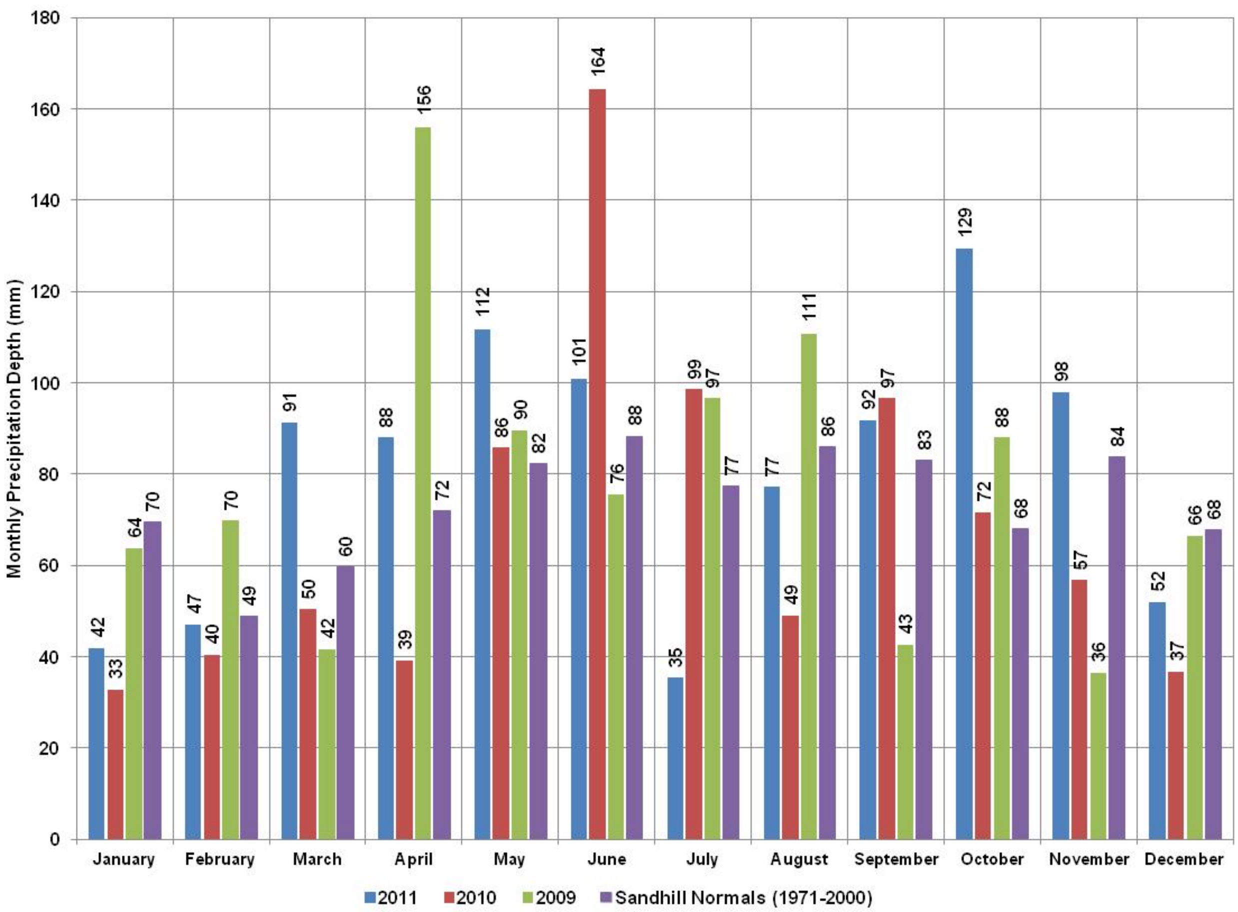


Figure 4.5: Monthly precipitation totals – Bramport site

A total of 181 storm events [1] occurred over the June 2009 to July 2011 monitoring period. The largest storm event in terms of precipitation depth was 54.6 mm, occurring on August 20, 2009 over a period of 3.4 hours. The maximum intensity storm was also the August 20, 2009 event, when precipitation was recorded at a maximum rate of 46.4 mm/h between 5:45 and 6:45 PM and 9.2 mm/5 min. The August 20, 2009 storm event very nearly exceeded the 50 year return period, 1 hour duration storm event for the area (46.8 mm), based on historical rainfall data from Lester B. Pearson Airport (Environment Canada, 2005). Unfortunately, during this large and intense storm event, sensors in the observation port wells malfunctioned and no data was captured regarding water levels in the chamber system.

Over the monitoring period, precipitation was intense enough to cause water to be temporarily stored in the chambers on only 23 occasions. Based on analysis of precipitation depth and intensity during these 23 storm events it was determined that it takes a storm event of at least 7.2 mm depth and a maximum intensity of at least 1.2 mm/5 min. over the chamber system drainage area to cause flows to begin to back up in the control manhole and be temporarily stored in the chamber system.

4.4.2 Infiltration

Performance of the Bramport infiltration chamber system was evaluated with regard to assumptions used in facility design regarding infiltration rate of the native subsoil. Post-development infiltration rates were examined for a number of inter-event drainage periods between June 2009 and July 2011.

The inclusion of observation port wells in the design allows infiltration rates to be estimated over selected portions of the overall drainage period. To determine if infiltration was occurring drainage periods were examined that begin with the additional gravel bed (i.e. gravel reservoir) being full and no flow being registered in the pipe connecting the chamber system to the control manhole so that changes in water levels in the observation port well only reflected losses to infiltration. Drainage periods end with the onset of the next precipitation event.

Figure 4.6 shows hyetographs and water levels for selected storm events and drainage periods obtained from the observation port wells. During the 2009 monitoring period, water level monitoring was conducted in Observation Well 1 only with results indicating that the undrained portion of the gravel bed below the chamber system does not drain between storm events. The top hyetograph shown in Figure 4.6 illustrates that over a 22.6 day period with no rainfall during the summer of 2009, water levels did not decrease substantially below the elevation of the top of the undrained gravel reservoir. This was the longest inter-event period observed over the June 2009 to July 2011 monitoring period.

To help confirm these findings, water level monitoring in both Observation Well 1 and 2 was initiated in 2010 and continued in 2011. Comparison of data from the two wells in 2010 showed that water level readings did not agree, with Observation Well 2 exhibiting longer drainage times when levels were above the top of the gravel reservoir but below the invert of the pipes connecting the well to the chamber system. On May 31, 2011, accumulated sediment on the bottoms of the wells and the geotextile liners was removed to determine if clogging was the cause of the slower drainage being observed in Observation Well 2. Indeed, once the geotextile was removed from Observation Well 2, water level readings from both wells became consistent. This troubleshooting confirmed that water level readings from both wells indicate that the undrained gravel reservoir below the chamber system does not drain between storm events.

4.5 Discussion

These results suggest that very little or no infiltration is occurring from the undrained portion of the gravel bed in which the chamber system is installed. It is possible that the chamber system has created a perched water table in the area that does not dissipate, or drains so slowly that the inter-event periods observed during this monitoring study were not long enough to observe substantial change in water levels. If some infiltration is occurring over the whole gravel bed in which the chamber system is installed it is possible that some of the infiltrated water gradually flows into the undrained portion through lateral groundwater flow. Such shallow groundwater flows could be contributing to the observation that water levels in the undrained portion of the gravel bed do not substantially decrease between storm events.

Without a nearby groundwater monitoring well installed to the depth of the water table, it is not possible to determine if a perched water table exists. Seepage observed during borehole drilling completed as part of soil investigations during planning of the development area support the possibility that stratigraphy in the area could produce perched groundwater at shallow depths (Soil Engineers, 2005). Therefore, it is recommended that a borehole be drilled and a groundwater monitoring well be installed in a suitable area in close proximity to the chamber system to determine if a perched water table has indeed been created. To confirm that the observation port wells are functioning as designed, drive point piezometers could also be installed in the manhole bottoms to confirm that water level readings in the wells agree with water levels in piezometers installed to a depth below the gravel bed of the chamber system. In the absence of this information, the results of this evaluation are inconclusive regarding whether or not any runoff volume reduction benefits are being achieved by the chamber system.

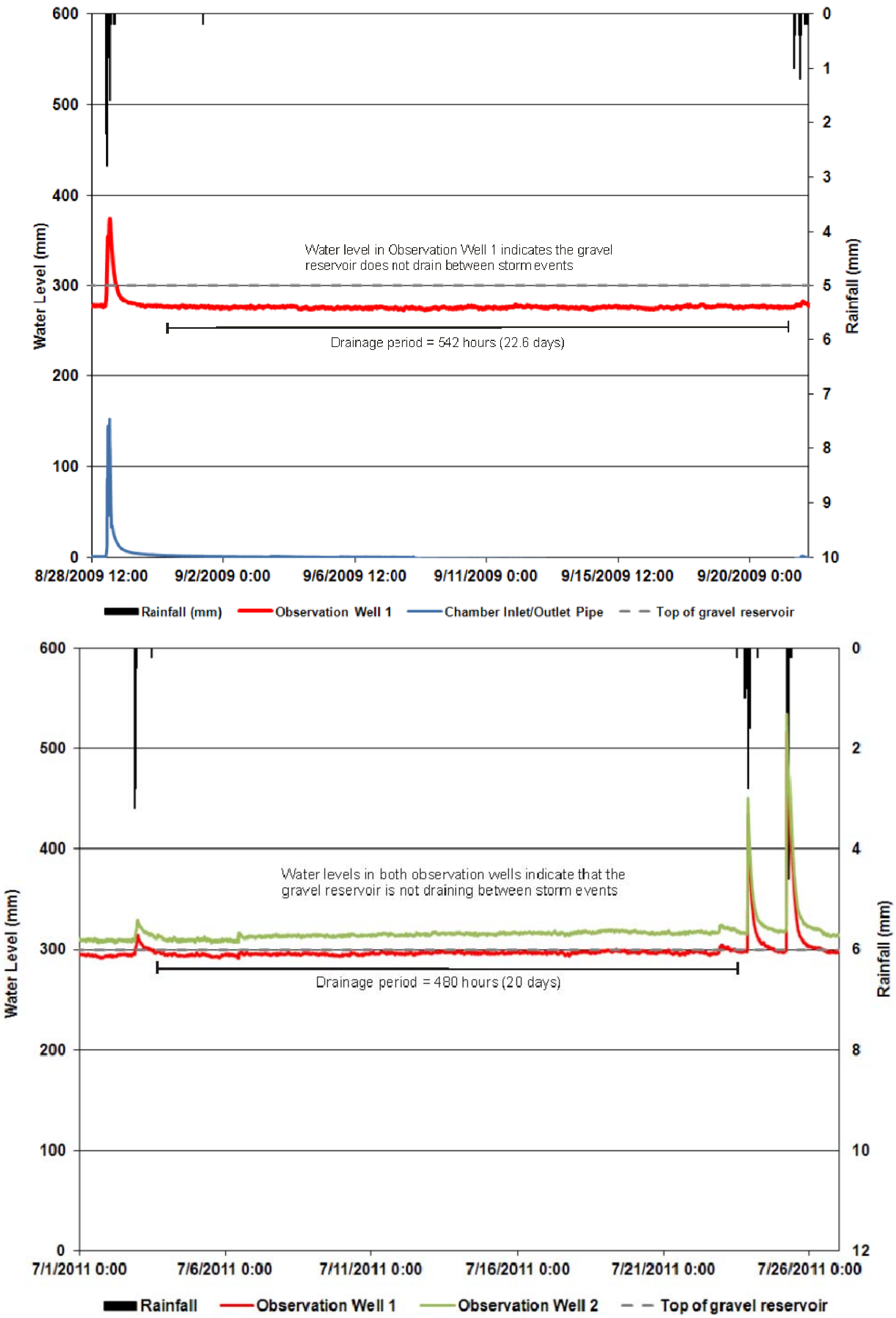


Figure 4.6: Water levels in Bramport infiltration chamber system for selected drainage periods

5.0 CONCLUSIONS AND RECOMMENDATIONS

In the underground stormwater infiltration systems examined in this study, deficiencies in the function of control manholes designed to retain water in them were observed in 3 of the 5 facilities monitored. This suggests that improvements to control manhole design, material specifications, or construction and inspection practices are warranted. Leakage of water from the inlet side to the outlet side of the control manholes was observed in 2 of the 5 facilities indicating that sealing of the joints between components used to construct the manholes needs to be improved or given more attention during construction. Sediment accumulation at the bottom of control manholes was also observed in 3 of 5 facilities monitored. While sediment accumulation was not found to be impacting the hydrologic performance of the facilities, it was causing issues with function of the control manholes for evaluating drainage times. Design improvements that would address sediment accumulation issues include incorporating a sedimentation pretreatment device (e.g. manhole with Goss trap or an oil and grit separator) upstream of the infiltration facility and sumps in control manholes to help prevent inlet pipes from clogging. Another improvement to the design of control manholes that would better facilitate inspection and maintenance activities is including an outlet with a valve through the weir plate that can be operated from the outlet side, so the system could be drained via gravity without the use of a pump.

It is strongly recommended that such facilities be thoroughly inspected by the construction project manager, system designer or ultimate owner/manager of the infrastructure prior to assumption. Inspection procedures should include continuous water level monitoring over several storm events or a synthetic runoff test to determine if the system is functioning as designed. Contracts that include construction of such stormwater infrastructure should also include conditions whereby any defects or deficiencies revealed through final inspection can be corrected prior to assumption. Visual inspection of the stormwater infiltration systems at a time when the facilities were filled to a level above the elevation of the outflow sewer pipe invert would have revealed that leakage around the weir plate was occurring in some cases.

Monitoring also indicated that the post-development infiltration rate of the soils underlying the stormwater infiltration systems is lower than expected resulting in slower than expected drainage of stored stormwater and in some cases, more frequent occurrence of overflow than expected. In the case of the Bramport system, it was observed that the portion of the gravel bed below the chamber system that has no outlet does not drain between storm events. In both the Elgin Mills Crossing and Mayfield Industrial Park infiltration systems it was observed that infiltration rates decreased exponentially as water levels (i.e. hydraulic head) in the systems declined. Observed infiltration rates when the systems were full or nearly full were approximately 2.5 times higher than when the system was half full or less. This supports the concept of designing stormwater infiltration systems on fine-textured soils to maintain hydraulic head in the water storage reservoirs for longer than the typical target of 48 to 72 hours to help maximize the drainage rate and thereby, the volume of water infiltrated on an annual basis. On low permeability, clayey silt soils like those occurring at the Mayfield Industrial Park site this would translate to designing infiltration trench systems that never fully drain between storm events. Observed infiltration rates for the Elgin Mills Crossing and Mayfield Industrial Park infiltration systems did not exhibit significant seasonal variation. This is consistent with observations from underground stormwater infiltration facilities by other researchers (Roseen et al., 2009). This is due to the fact that the base of each system is approximately

2.5 to 3.5 m below ground surface and therefore is well insulated from surface temperature fluctuations throughout the year.

Since the infiltration systems examined require longer than 72 hours to fully drain after filled to capacity, the control manholes contain standing water for much of the year, which raises the question of whether or not they provide mosquito breeding habitat. While no mosquito larvae were observed through grab samples from control manholes at each site, in systems to be located on low permeability soils it is recommended that the type of covers used on control manholes should contain no holes at all, or screens to help ensure mosquitoes cannot enter them.

Although these systems did not achieve their drainage time design objectives, two of the three stormwater infiltration systems evaluated provided substantial reductions in runoff volume from their roof drainage areas. In the case of the Elgin Mills Crossing infiltration chamber system, even though it was observed to be leaking a small amount of water around the weir plate and draining slower than expected, monitoring indicates that during a normal precipitation year, the system reduces runoff from the roof drainage area in the order of 90% and infiltrates the volume of water necessary to match what would have infiltrated over the lot area prior to development. In the case of the Mayfield Industrial Park infiltration trench system, it was possible to conclude that the system is not achieving the design objective of fully compensating for the loss of infiltration caused by development of the site. In a normal precipitation year it is estimated that Mayfield Trenches 1 and 3 reduce runoff from their respective roof drainage areas in the order of 16% and Mayfield Trench 4 reduces roof runoff by approximately 36%. It was estimated through modeling that water storage capacity of the trenches would need to be between 1.9 and 4.5 times greater than the trenches currently provide in order to infiltrate the targeted volume of roof runoff that would mitigate the loss of infiltration on an average annual basis. If the Mayfield trenches had been sized as such, it is estimated that they would reduce runoff from their respective roof drainage areas in the order of 53 to 61%. While the Mayfield Industrial Park infiltration trenches are draining more slowly than expected, results of this evaluation suggest that if they had been designed based on better knowledge of the permeability of the native subsoil, they could fully compensate for the loss of infiltration caused by the development through infiltration of roof runoff alone. This finding is significant considering that infiltration practices are widely considered to have limited effectiveness on fine-textured soils.

Considering these favorable results, it is recommended that in locations with fine textured subsoil and seasonally high water table or bedrock surface at least 3.5 metres below ground surface, design criteria for stormwater management systems servicing industrial, commercial and institutional developments, where roof area represents 50% or greater of the lot area, should include maintaining average annual pre-development infiltration volume over the lot through infiltration of roof runoff, where feasible.

While results from the Elgin Mills Crossing and Mayfield Industrial Park systems support the implementation of stormwater infiltration systems on fine textured soils, monitoring results from the Bramport infiltration chamber system suggest that little or no runoff reduction benefit is being achieved by designing the system to provide the opportunity for infiltration, as was anticipated at the time of its design. It is possible that in the Bramport case, the infiltration chamber system has raised the seasonally high water table or that a perched water table has been created that does not dissipate or drains so slowly that the inter-event periods during this monitoring study were not long enough to observe substantial change in water levels. While results from this study could not confirm whether or not a perched water table has been created, they highlight the importance of careful consideration of the underlying stratigraphy and predevelopment water table elevation when deciding on suitability of the site for stormwater infiltration practices.

Topics of interest for future research on the performance of underground stormwater infiltration systems include the following:

1. *Medium to long term evolution of hydrologic performance.* This study examined hydrologic performance of underground stormwater infiltration practices located on fine textured soils within a short time period following their construction (less than 5 years post-construction). Little information is currently available in published reports regarding their performance over longer periods of service. Monitoring drainage times of stormwater infiltration practices located on fine-textured soil over the medium term (10 years post-construction) to long term (20 years post construction) to examine how hydrologic performance evolves over time is of interest to better understand the useful lifespan of such facilities and possibly to evaluate what maintenance or rehabilitative procedures could be undertaken to restore their effectiveness, other than complete reconstruction.
2. *Presence of mosquito larvae.* While grab samples from control manholes of the underground infiltration practices examined in this study indicated no presence of mosquito larvae, considering that such practices located on fine-textured soil may contain standing water for much of the year, additional sampling of a larger number of facilities is of interest to evaluate whether or not they provide mosquito breeding habitat.
3. *Routine operation and maintenance*. Monitoring sediment accumulation in pretreatment structures and control manholes of underground stormwater infiltration practices is of interest to determine accumulation rates, evaluate quality and disposal options and estimate cost of inspection and maintenance over the lifespan of the facilities.

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