Modeling Retrofitted Extended-Detention Wet Ponds and Wetland Pockets

Alan S. Lam and R. Mark Palmer

The restoration of stressed watercourses in urbanized watersheds can best be achieved by taking advantage of opportunities such as the retrofitting or improvement of existing best management practices (BMPs). Within the Sixteen Mile Creek and Emery Creek Watersheds, two existing on-line stormwater management (SWM) ponds were recently retrofitted for water quality enhancement: the Ninth Line SWM facility (Mississauga) and the Pine Valley SWM pond (Vaughan). These ponds were originally constructed in the 1980's for flood control, but without the pollutant removal capabilities of today's extended-detention wet pond and wetland/wet pocket systems.

Each component of these retrofit BMPs was designed to employ a variety of pollutant removal pathways to increase the efficiency of sediment removal and improve the quality of water discharging downstream. Typical pollutant removal pathways, in order of dominance, are sedimentation, adsorption by vegetation, physical filtration, and nutrient uptake by the wet pond and wetland/wet pocket compartments plants and algae. The original flood control and riparian storage functions were preserved in each retrofit design.

In order to quantify the sediment removal capabilities of each retrofit, the QUALHYMO and STORM computer programs were used with water budget models to design the forebay, wet pond, wet pocket, and extended-detention storage features. Annual sediment loadings and evaporation characteristics, during periods of minimal precipitation, were then used to develop a maintenance schedule for the new retrofit features.

Lam, A.S. and M. Palmer. 1997. "Modeling Retrofitted Extended-Detention Wet Ponds and Wetland Pockets." *Journal of Water Management Modeling* R195-24. doi: 10.14796/JWMM.R195-24.
© CHI 1997 www.chijournal.org ISSN: 2292-6062 (Formerly in *Advances in Modeling the Management of Stormwater Impacts*. ISBN: 0-9697422-7-4)

A dynamic-wave routing analysis was also necessary for the Ninth Line SWM facility. This analysis used the OTTHYMO, QUALHYMO and BOSS-DAMBRK computer programs and confirmed that the relatively flat gradient of the receiving watercourse and two undersized culverts downstream will not cause a backwater effect significant enough to submerge the new outlet structure during the water quality design storm. This analysis also determined the “realtime” drawdown operation of the on-line facility, for both continuous and less frequent event storm simulations, as well as the submergence problems affecting the original flood control outlet structure.

24.1 Introduction

Over the last two decades, substantial evidence has been collected about the pervasive impacts of urbanization on stream hydrology, geomorphology, water quality, habitat and ecology. In response, growing municipalities such as the municipalities of Mississauga and Vaughan have implemented stream restoration and environmental protection strategies for the urban Sixteen Mile Creek and Humber River (which includes Emery Creek) watersheds, respectively, in partnership with the local Conservation Authorities, Government Ministries, community stakeholders, and land development industries.

The restoration of stressed or degraded stream systems is perhaps the most challenging watershed management objective for achieving sustainable “targets” such as water quality enhancement. The restoration of degraded watercourses within urbanized watersheds, to any meaningful degree, can be economically achieved by taking advantage of “opportunities” such as the retrofitting or improvement of existing BMPs in the drainage network. Opportunities for urban retrofitting are limited in developed watersheds, but they can be revealed through detailed on-site evaluations during the preparation of sub-watershed plans. Typically, the best sites for urban retrofits are found at the terminus of a storm drainage system, across or within artificial open channels, adjacent to a natural or engineered open channel, or within an older BMP system, such as a dry stormwater detention pond (Anacostia Restoration Team, 1992).

The range of possible retrofit techniques includes the following:

- source control;
- open channel;
- natural channel;
- off-line storm sewer;
- on-line storm sewer; and
- “end-of-pipe” surface storage.

Each technique differs with respect to where the retrofit would be located within an existing storm drainage system. This chapter presents two retrofit projects which utilized the open channel and end-of-pipe surface storage techniques.

Open channel retrofits are installed in an engineered channel, immediately below a single or network of storm sewer outfalls, and preferably where a road embankment or other structure crosses the watercourse. On-line options typically consist of “extended-detention - shallow wetland” or “wet pocket - wet pond” systems. Alternatively, off-line open channel retrofits employ a flow splitter to divert the target water quality runoff volume to a similar bio-diverse system. However, the retrofit features are located in or adjacent to the floodplain. The first case study presented in this chapter concerns an open channel retrofit in the City of Mississauga - the largest of its kind in the Greater Toronto Area.

Retrofits for surface storage facilities include the modification of older, end-of-pipe dry stormwater detention or flood control ponds to improve their runoff and storage treatment capacity. The new storage created, either by excavation, adding to the height of the embankment, and/or constrictions from low flow orifices, is used to provide:

1. permanent pool(s), such as forebay and wet pond features,
2. extended-detention storage,
3. shallow wetland or wet pocket compartments, or
4. combinations of the above categories.

For example, pond retrofitting has been the primary focus of restoration efforts in the Washington Metropolitan Area (Herson, 1989) and has typically involved converting older, dry stormwater ponds into extended-detention, wet pond and wetland systems. The second case study is located in the City of Vaughan and involves a dry stormwater management pond, which was originally constructed for flood control in 1981.

24.2 Description of Projects

24.2.1 Ninth Line Stormwater Management Facility

In the City of Mississauga, the Lisgar Region Water Quality Management Plan (Winter, 1993) was approved in 1994. This sub-watershed plan set out the best achievable targets for water resources and environmental management for the 26.8 km², warm-water East Tributary of Sixteen Mile Creek. It also identified constraints and opportunities in the sub-watershed to achieve the desired targets. The management targets included:

1. reducing erosion and flooding susceptibilities,
2. maintaining or enhancing base flows to emulate natural conditions,
3. maintaining existing wetlands, and
4. enhancing aquatic/terrestrial habitats and water quality to comply with the Provincial Water Quality Objectives (MOEE, 1994b).

The plan was developed to address stormwater management and drainage requirements for existing activities within the East Tributary sub-watershed and future land use changes.

The relatively flat gradient of the tributary was identified as a constraint during the 1993 study since the watercourse is prone to flooding during more frequent storms (i.e. every summer storm). This affects the on-line SWM open channel facility, upstream of Ninth Line, which often fills to near capacity during rainstorms. Together, the downstream channel slope, two undersized culvert crossings downstream of the SWM facility, and the temporal distribution of rainfall events affect the drawdown operation of this flood control facility.

One of the key recommendations from the Lisgar Region Water Quality Management Plan was to retrofit the 38.9 ha-m flood control/open channel facility, referred to as the Ninth Line SWM facility, for water quality enhancement of runoff from a 13 mm rainfall event. An environmental study to examine various retrofitting options and to design the preferred alternative was completed, so that a number of community developments discharging to the facility could be constructed (Fred Schaeffer Associates (FSA), 1994).

This retrofit project also investigated the steady-state hydraulics of the East Tributary, and submergence problems at the Ninth Line SWM facility's outlet structure caused by the two downstream undersized culverts. It was confirmed that the new retrofitted water quality outlet control structure would not be submerged during the 13 mm design storm event. However, the outlet would be submerged by storms greater than a two year event (FSA, 1994).

A culvert replacement study was conducted by the City for the two downstream structures that were causing problems at the Ninth Line SWM facility (C.C. Tatham Associates (CCTA), 1995). A hydraulic or dynamic wave routing analysis was undertaken for the East Tributary using output data from event storm and continuous storm hydrologic models of the Lisgar Region sub-watershed. While this additional analysis recommended replacement structures for the two undersized crossings, in order to restore the original flood control design outflows and storage water levels at the Ninth Line SWM facility, it also confirmed that the SWM facility's new retrofitted water quality outlet would operate properly during the 13 mm design storm event.

A monitoring program to study the performance of the water quality facility was undertaken. The program will run over five years and results will be published at selected intervals.

24.2.2 Pine Valley Stormwater Management Pond

Recently, the Metropolitan Toronto and Region Conservation Authority (MTRCA) initiated the *Humber River Watershed Management Strategy*, which includes the Emery Creek Watershed within the City of Vaughan. However,

prior to this initiative, the MTRCA and Ministry of Natural Resources (MNR) identified in 1993 preliminary water resources and environmental management targets for the 3.2 km² headwater development area of the warm-water Emery Creek watershed. Stormwater peak flow control and quality enhancement for future developments were two of the target objectives. These conditions were included in the plan of subdivision approval requirements for a number of Woodbridge residential and commercial development applications, north of Highway 7, within the watershed.

Due to existing constraints within the receiving storm sewer system and previously approved functional drainage requirements (ABA, 1981 and 1983) for the future headwater development area, the only option available for peak flow control was on-site within each development block. However, two surface storage options for stormwater quality enhancement were possible, namely: at the source within each development, or by taking advantage of an off-site retrofit end-of-pipe opportunity within an older 16.9 ha-m dry stormwater pond known as the Pine Valley SWM pond. This pond forms part of the Emery Creek drainage network and is located at the end of the storm sewer that services the 3.2 km² headwater area. Through consultation with the City of Vaughan, MTRCA, and MNR, it was agreed that site plan approvals would be granted to all affected developments, provided that the developers would construct the water quality retrofitting improvements to the Pine Valley SWM pond (FSA, 1993).

24.3 Project Goals and Objectives

The primary goals of each retrofit project, which was constructed in 1994, were as follows:

1. retain the original flood control and riparian storage capabilities of the existing facility; and
2. rehabilitate or enhance downstream water quality and associated resources by way of the most appropriate retrofitting works.

The retrofit designs were selected from a variety of options, using best management practice matrix evaluations.

Both selection processes were undertaken in accordance with the ecosystem approach, regulatory policies and guidelines at the time, and the Class Environmental Assessment for Municipal and Wastewater Projects (1993) document. In addition, the following objectives were adhered to in the selection of the designs:

1. retain, as a minimum, the original peak flow shaving and riparian storage design relationships;
2. minimize any impacts to local base flows, relative to existing conditions;
3. optimize suspended solids and heavy metal removal efficiencies;

4. optimize nutrient uptake potential and diversity of plants to provide terrestrial and aquatic habitats;
5. maximize aesthetic features in keeping with the overall master landscape or park use plan for the area;
6. consider the limitations imposed by urban planning issues related to the preferred retrofit option in the overall context of the watershed;
7. consider operation and maintenance costs (i.e. requirements and frequency) as part of the selection process;
8. acceptance by public and regulatory agency;
9. consider construction costs;
10. consider other watercourse structures downstream that may affect the hydraulics of the new retrofit features (e.g. recommendations were provided on the hydraulic capacities of two culvert crossings downstream of the Ninth Line SWM facility, as well as replacement structures, after investigating the submergence effects on the actual versus original design flood control relationships of the facility);
11. implement, after the construction of the preferred design, a post-construction monitoring program at the inlet(s) and outlet to measure water quality - pursuant to the retrofit compliance requirements of the Ministry of Environment and Energy's (MOEE) Certificate of Approval.

At the time of each project, the Province's manual on stormwater management practices (MOEE, 1994a) was not available; however, each project incorporated the design principles specified in that manual.

24.4 Description of Preferred Retrofit Options

24.4.1 Ninth Line Stormwater Management Facility

General

A long list of eleven practical design options was developed from the following open channel retrofit categories:

- an extended-detention basin with no excavational storage improvements (i.e. only 7 mm storm runoff enhancement possible) to the existing facility (*note*: this option had the lowest capital cost but also had many performance disadvantages, relative to the other alternatives, and, in addition, the stormwater enhancement volume was much less than the target storage specified in the Lisgar Region Water Quality Management Plan);
- an extended-detention basin with additional excavation to treat runoff from a 13 mm storm;

- an extended-detention basin (13 mm storm) and creation of wet-land/wet pockets;
- a retention basin (i.e. wet pond covering either one-third, two-thirds, or the entire surface of the facility) and extended-detention storage (13 mm storm); and
- combination of the above open channel retrofit categories.

Each retrofit option included:

1. a control berm and water quality outlet, immediately upstream of the existing flood control outlet structure,
2. a sediment forebay with a multi-dispersed pipe outlet, bio-engineered berm and spillway,
3. a landscape and open space enhancement strategy,
4. a maintenance and operation plan, and
5. a post-construction water quality monitoring program.

As a result of the environmental-assessment public-consultation process, the forebay features were revised to address nuisance concerns (e.g. shape and location of the forebay, appearance and materials to be used). In addition to all of the above features, the preferred retrofit option from the general “combination” category included:

1. additional excavation for extended-detention storage,
2. a wet pond covering about one-third of the basin floor, next to the new control berm and outlet structure,
3. creation of four large wet pockets upstream of the wet pond, and
4. removal of the existing concrete-lined low flow channel through the facility.

Functional Design Parameters

The functional design parameters of the retrofitted Ninth Line SWM facility included:

1. the capture of the storm runoff volume from the first 13 mm rainfall depth (i.e. total volume of 43,000 m³);
2. the detention of the storm runoff from the first 13 mm of rainfall for between 24 and 48 hours;
3. optimized sediment removal associated with the 40 µm criteria, as recommended in the Lisgar Region Water Quality Management Plan;
4. as a minimum, the matching of the quality enhancement criteria (i.e. in terms of first order pollutants and total sediment concentrations), as determined from the approved continuous modeling for the Lisgar Region Water Quality Management Plan; and
5. the retention of the existing flood control capabilities of the facility.

During the hydrologic investigations of the environmental assessment study (FSA, 1994), it was determined that the water level associated with a 2-year storage volume would be about 188.7 m, based on the “ultimate” design storage relationship of the facility. The FSA study also recommended that the quality enhancement storage volume of 43,000 m³ be obtained at a maximum water level of 188.5 m. Rainfall runoff volumes (i.e. 2-year through Regional Storm events) in excess of the retrofitting design storage would then overtop the proposed quality control berm of the retrofitted facility and be controlled by the existing concrete flood control weir. In other words, the approved discharge-storage function of the facility for flood control and riparian storage could be retained with the new retrofit features:

1. the retention of the existing inlet and outlet locations at opposite ends of the facility to prevent short circuiting and minimize potential re-suspension due to the on-line configuration of the facility;
2. the incorporation into the design of a means to provide downstream runoff volume maintenance, if required later on;
3. the provision of a sediment forebay to pre-treat the stormwater runoff by trapping large debris and coarse suspended solids, and reduce the inlet velocity prior to flows entering the SWM facility;
4. the provision of shallow wet pockets to promote a variety of pollutant removal pathways such as sedimentation, adsorption, and nutrient uptake; and,
5. the provision of a permanent pool depth of 1.6 m within the wet pond compartment to further enhance the sediment removal capabilities of the facility.

Table 24.1 summarizes the design volumes and water levels associated with each component of the preferred retrofit option.

Detailed Modeling Procedures Leading to the Design

Each component of the retrofitted SWM facility was designed to employ a variety of pollutant removal pathways to increase the efficiency of sediment removal and improve the quality of the water discharging downstream. Typical pollutant removal pathways, in order of dominance, are: sedimentation, adsorption by vegetation, physical filtration, and nutrient uptake by wet pond and wet pocket plants and algae.

With respect to quantifying pollutant removal by way of sedimentation, the computer models QUALHYMO and STORM were used in the detail design. Since sedimentation is the dominant pollutant removal pathway, it was used as an indicator of water quality enhancement.

Table 24.1 Summary of design volumes and water levels for the Ninth Line SWM facility.

Retrofit Feature	Design Water Level (m)	Maximum Design Depth (m)	Minimum Required Volume (m ³)	Proposed Design Volume (m ³)
Sediment Forebay (1)	188.5	2.0	4,300 ^{A.}	8,900 (4,800 active)
Wet Pond (1)	187.2	1.6	--	27,000
Wet Pocket (4)	187.2 to 187.4	0.7 to 0.9	--	6,900
Extended-detention Storage	188.5	1.3	43,000	61,900 ^{B.}

^{A.} Typically, the active forebay volume should comprise at least ten percent of the entire treatment volume (i.e. 10% of 43,000 m³) within the SWM facility.

^{B.} Additional volume provided as a result of: 1) regrading constraints to provide 7:1 sideslopes within the extended-detention storage compartment, 2) compensating for a future 25 years of $\pm 12,000$ m³ of sediments within the extended-detention compartment, as part of the maintenance program, and 3) providing the balance of the approved design storage below 188.5 m (ie. $\pm 4,300$ m³) which was not present within the existing facility.

QUALHYMO Modeling

The continuous hydrologic model QUALHYMO (Version 2.11) was used to model the retrofitting design and the water quality results were compared with the approved modeling enhancement criteria from the Lisgar Region Water Quality Management Plan. A “pre-retrofit” model of the watershed (i.e. existing Ninth Line SWM facility without the preferred retrofit features) was used as the baseline model and modified (i.e. “post-retrofit” condition) to include the extended-detention storage, wet pond, and wet pocket compartments of the design. This was done to assess whether the preferred design, at a minimum, could match the approved enhancement criteria from the Lisgar Region Water Quality Management Plan. Model changes were in the form of revised stage-storage and stage-discharge curves, while the balance of the original model in terms of land use, rainfall, etc. was not modified.

For the purpose of the analysis, the permanent pool of the forebay and wet pocket volumes were added to the wet pond volume in the “post-retrofit” modeling analysis. Given the low permeability of the soils within the pockets and lined forebay bottom, it was assumed that the forebay and wet pockets would function similarly to the wet pond feature with respect to sediment removal. This assumption was appropriate for the wet pockets given the observed standing water in an existing wet meadow/cattail depression, which had formed along the northern perimeter of the existing SWM facility.

The precipitation records used for the QUALHYMO analysis were from the Lester B. Pearson International Airport for the period May to November, 1972 inclusive. This series of data was used as an “average year” for the QUALHYMO water quality simulations that were used to develop the Lisgar Region Water Quality Management Plan.

STORM Modeling

The continuous hydrologic model STORM was also used in the detail design to determine the overflow volumes and sub-watershed runoff amounts under two series of rainfall data. The first model used the same series of precipitation records from the “post-retrofit” QUALHYMO modeling analysis discussed above. The second STORM analysis used a longer period of rainfall data (i.e. February, 1977 to January, 1982 inclusive) from the Airport, as this series of rainfall data has been previously used by FSA in the design of many stormwater quality ponds throughout the Greater Toronto Area.

The results from both STORM models were used in conjunction with the Chen (1975) methodology, which uses the Camp formula (1946) to predict sediment removal efficiency:

$$F = 1 - \left[1 + \frac{1}{n} \frac{V_s}{Q_r / A} \right]^n \quad (24.1)$$

where:

- F = sediment removal efficiency
- Q_r = average constant release rate from the SWM facility
- A = active storage surface area of the SWM facility
- V_s = settling velocity
- n = turbulence or short circuiting parameter

Fair and Geyer suggest an empirical relationship between performance and the value of “ n ”, which is: $n=1$ (very poor); $n=3$ (good); $n>5$ (very good). When $n = \infty$, the equation reduces to the Chen formula: $F=1-e^{-V_s Q_r / A}$. In this study, a conservative value of 1 was assumed for “ n ”.

The distribution of suspended solid fractions were chosen based on the US National Urban Runoff Pollution Program and were considered representative for the sub-watershed. The distribution is listed in Table 24.2. The fractional distributions and settling velocities of suspended solids used in the STORM analysis were the same distributions used in the QUALHYMO model for the Lisgar Region Water Quality Management Plan.

It was felt that in the event of an overflow, the trap or sediment removal efficiency would be reduced due to the increase in velocity in the Ninth Line SWM facility. The results from the STORM model were used to determine the average

Table 24.2 Distribution of fractions of suspended solids used in the QUALHYMO and STORM modeling.

Class	Total %	Avg. Diameter (mm)	V _s Average mm/s
i	20	4.00	5.5
ii	20	0.40	0.59
iii	20	0.13	0.13
iv	20	0.06	0.025
v	20	0.02	0.0027

annual runoff (R) and overflow volume (R_o). Using a value of 0.5 for trap efficiency (E_t) for events causing overflows, the average annual trap efficiency, E , was calculated as:

$$E = \frac{R_o \times E_t \times (R - R_o) \times F}{R} \quad (24.2)$$

Water Budget Analysis

A water budget analysis was also undertaken to determine the water level fluctuations in the “full” permanent pool storage volumes of the forebay and wet pond compartments of the SWM facility during periods of minimal precipitation. In addition, another simulation was completed assuming both the forebay and wet pond were essentially empty prior to the first rainfall event in each of the years examined. The record from May to September, 1986-1990 (inclusive) was analyzed for both the full and empty condition, because of the unusually dry and hot summer weather experienced in southern Ontario during that time. Daily precipitation data used in the analysis were obtained from the Lester B. Pearson International Airport records. It was assumed that these weather conditions would serve as a good indicator for ensuring that both the wet pond and forebay would not evaporate and therefore detract from the aesthetic and operational characteristics of the retrofitted facility. Also, water level fluctuations were considered to be important for determining what varieties of plants would be best suited to survive under these conditions.

The water budget spreadsheet analysis included the following:

- The basic equation for the water budget:

$$\Delta S = P + R - E - I \quad (24.3)$$

where:

- S = change in storage,
- P = precipitation directly on surface areas of forebay and wet pond,
- R = runoff from the sub-watershed,
- E = evaporation from the forebay and wet pond, and
- I = infiltration through bottom of forebay and wet pond.

- Evaporation rates were calculated from the findings of an earlier study in the Town of Richmond Hill by FSA.
- Daily runoff to the SWM facility was calculated using the volumetric runoff coefficient from the OTTHYMO model for ultimate sub-watershed development conditions from the Lisgar Region Water Quality Management Plan and daily precipitation volumes.
- Infiltration was calculated using a constant permeability coefficient of 8.6 mm/day (i.e. typical of silty clay soils in the bottom of the wet pond compartment). This approach was also conservatively applied in the forebay water budget analysis, even though a hardened bottom was recommended.
- Any overtopping constituted the exceedence of the dead storage volume, which meant that water would rise into the active volume, where it discharges through an outlet pipe.

BOSS-DAMBRK Modeling

The construction of the original Ninth Line SWM facility in 1988 for flood control included a 42.0 m wide stepped concrete weir with a lateral contraction. This flood control weir and the facility's ultimate design storage (which has not been fully constructed yet) was designed in 1983 to provide only peak flow reduction for the 2- to 100-year flood events. The facility was not designed for incremental riparian storage. However, additional storage was included in the original design in order to:

1. maintain the peak time lags and shapes of the 1983 or "pre-development" hydrographs for post-development conditions, during the 2- to 100-year events, and
2. maintain the total floodplain storage for the Regional Storm.

In 1993, the Lisgar Region Water Quality Study identified the relatively flat gradient of the Sixteen Mile Creek East Tributary as a constraint since the watercourse is prone to flooding during frequent storm events. The study concluded that the "pre-retrofit" Ninth Line SWM facility often filled to near-capacity during rain storms and the cause appeared to be two undersized culverts downstream. In addition, backwater from the relatively flat channel gradient was submerging these two culverts and flood control weir of the Ninth Line SWM facility.

During the 1994 retrofit design by FSA, a preliminary steady-state hydraulic analysis of the tributary using the HEC-2 computer program confirmed that the new control berm/outlet structure for water quality enhancement (i.e. 13 mm storm event) would not be submerged as a result of the two downstream undersized culverts. However, given the unsteady flow characteristics of the downstream tributary, a dynamic wave routing analysis was also required. This involved a numerical solution of the Saint Venant equations.

The Ontario Ministry of Natural Resources *Floodway Fringe Analysis Technical Guidelines* recommends the use of a dynamic wave routing model when the average reach slope is less than 0.7 m/km. Downstream of the Ninth Line SWM facility ($\pm 2,430$ m) the average slope is less than 0.6 m/km.

Since the extent of the problem associated with the submergence of the concrete flood control weir (i.e. flood events greater than 2-year return period) and the downstream impact of replacing the two subject crossings was not quantified in the 1994 preliminary analysis, further detailed investigations were necessary. In 1995, the City of Mississauga initiated an MEA Class Environmental Assessment (Schedule 'C') for the reconstruction of Ninth Line. Therefore, further investigations were needed since a replacement of the Ninth Line structure was necessary, in conjunction with the road widening design, in order to achieve the original flood-reduction targets for the Ninth Line SWM facility. These additional investigations used the 1983 hydraulic design results for the SWM facility as target operational parameters. A recommendation to replace the other downstream existing culvert was also necessary to achieve the 1983 or "pre-development" approved design objectives for the SWM facility in terms of maximum outflows and storage water levels.

The City of Mississauga wanted to resolve outstanding flood susceptibility concerns as a result of the two downstream undersized culverts and the flood control operation of the Ninth Line SWM facility (CCTA, 1995). A dynamic wave routing analysis was undertaken using the BOSS-DAMBRK computer program based on the same reach length and crossing scenarios from the original FSA (1994) steady-state analysis. This study also used the same OTTHYMO and QUALHYMO hydrologic models from the Lisgar Region Water Quality Study and FSA investigations in 1993 and 1994, respectively.

24.4.2 Pine Valley Stormwater Management Pond

General

Only two end-of-pipe surface storage retrofit options were practical for the Pine Valley SWM pond. Both options included new sediment forebays at the two existing storm sewer outfalls to the pond, an extended-detention compartment for the water quality storage, a water quality control device fitted to the existing flood

control outlet, and naturalization landscape plans. However, the new water quality storage was constrained by the height of the existing embankment and sideslopes, which had to be maintained. Therefore, this retrofit project could not be designed to treat a specified rainfall volume, as was the case for the Ninth Line SWM facility.

The first option included 16,000 m³ of extended-detention excavation (corresponding to a 9 mm rainfall event) over the entire pond surface. This new cell also included the construction of a single large wet pocket area. The second option included a forebay flow-splitter structure and a bypass open channel through one-third of the pond, as well as 15,000 m³ of additional extended-detention (corresponding to an 8 mm rainfall event) and many small wet pocket features. The purpose of the forebay flow-splitter/bypass channel structure was to direct the first 8 mm of rainfall runoff to the new water quality pond cell. The balance of the minor system flow from the storm sewer would be diverted to the bypass open channel. The primary objective of this diversion concept was to avoid re-suspension in the water quality pond cell during less frequent storms. The second option was agreed to by the regulatory agencies as the preferred retrofit solution.

Functional Design Parameters

The functional design parameters for the preferred retrofit option were:

1. capture the storm runoff from the first 8 mm of rainfall depth in a new 15,000 m³ extended-detention storage compartment, at maximum depth of 1.5 m;
2. detain the water quality runoff volume for 12 hours (minimum) to 48 hours (maximum);
3. provide two new sediment forebays with a maximum depth of 1 m, velocity distribution chamber for the pipe outlets, and energy dissipation structure at the storm sewer outfalls;
4. provide five small wet pockets in the new water quality cell, each with a permanent water depth of from 0.25 m to 0.50 m;
5. provide a water quality control structure, at the existing flood control outlet, consisting of a hickenbottom pipe/gravel jacket structure with an orifice;
6. provide a maintenance access road to the forebay and control structures; and
7. designate an area on the sideslope of the original dry pond, away from the new cell and the tableland of the park property, to dispose of sediment that will be removed from the pond.

Detailed Modeling Procedures Leading to the Design

Similar to the Ninth Line SWM facility retrofit project, water quality enhancement for the Pine Valley SWM pond was assessed using the Chen (1975) methodology.

The STORM computer program was again used to determine the average annual runoff and overflow volume from the retrofit design. The same STORM input parameters from the Ninth Line SWM facility project were also used to analyze the retrofit features of the Pine Valley SWM pond. In addition, a sensitivity analysis was completed to assess sediment removal efficiencies for drawdown times from 12 hours to 48 hours.

24.5 Results and Discussion

24.5.1 Ninth Line Stormwater Management Facility

QUALHYMO and STORM Modeling

As illustrated in Figures 24.1 and 24.2, the QUALHYMO modeling results for the Ninth Line SWM facility retrofit design satisfied the required exceedence criteria from the Lisgar Region Water Quality Management Plan. That is, the hours and numbers of exceedence for first order pollutant (i.e. fecal coliforms) and total sediment concentrations flowing from the outlet of the retrofitted facility were typically less than the exceedence targets from the 1993 environmental assessment document.

Table 24.3 presents the results from the STORM modeling analysis. The detailed results were satisfactory relative to the pre-design target parameters.

Literature values representative of the land uses under ultimate development conditions were used to assess the impacts of sediment loading on the downstream water quality. The assumed sediment loading rates as a function of land use are shown in Table 24.4. These values were determined from recent studies in the Ottawa area by FSA.

The QUALHYMO model was used to estimate the total annual sediment loading in kg/yr, based on ultimate development conditions in the sub-watershed. The result was that approximately 860,000 kg/yr of sediment would discharge into the retrofitted facility. Assuming a conservative sediment specific gravity of 1.2, this yearly mass translates into a volume of about 715 m³/yr. Therefore, if the active storage or extended-detention areas of the facility retained 72% of this volume, about 515 m³/yr of sediment would accumulate in the SWM facility. The balance of sediments removed would be from the forebay, wet pockets, and wet pond compartments.

Figure 24.1 Comparison of QUALHYMO results: ultimate sub-watershed conditions - suspended solids. Sources: Winter et al (1993), and FSA (1994b).

Figure 24.2 Comparison of QUALHYMO results: ultimate sub-watershed conditions - 1st order pollutant. Sources: Winter et al (1993), and FSA (1994b).

Table 24.3 Comparison of STORM modeling sediment removal efficiency results for the Ninth Line SWM facility.

Description of STORM Model	Drawdown Time (hrs)	Avg. Qr (m³/s)	R (mm)	R _c (mm)	Sediment Removal Efficiency (F) (%)	Average Annual Trap Efficiency (E) (%)	
						^A 20 µm Criteria	^B 40 µm Criteria
Detail Design by Fred Schaeffer & Associates Ltd. (1994)							
1972 Rainfall	30	0.58	206	58.4	74.7	68.2	71.7
1977 - 1981 Rainfall	30	0.58	326	95.4	74.7	68.0	71.5
Environmental Study Report (ESR) Pre-Design by Fred Schaeffer & Associates Ltd. (1994)							
1977 - 1981 Rainfall	24	0.50	326	133.5	79.0	67.0	70.5

^A Used in the 1994 FSA Environmental Study Report.

^B Recommended in the Lisgar Region Water Quality Study.

It was calculated during the detail design that for a total detention time of 30 hours, the extended-detention storage of 61,900 m³ (i.e. both series of design rainfall data) would provide approximately 75% removal of suspended particles. This value compared well with the FSA (1994) pre-design sediment removal efficiency of 79%. *It must be remembered that this efficiency was based on the removal of solids by gravitational settling only.* The overall sediment removal for the preferred design option was estimated in the FSA environmental study report to be approximately 7% higher than the gravitational settling efficiency.

Table 24.4 Assumed annual sediment loading rates.

Land Use	Annual Loading (kg/ha/yr)
Commercial	1100
Residential	900
Undeveloped	440

This extra efficiency was derived from Ministry Environment and Energy research performance curves used in the FSA environmental study report for other typical wet pond/wetland facilities. It is noted that these curves have been incorporated into the manual on stormwater management practices in the province of Ontario (MOEE, 1994). The total estimated efficiency of the preferred design option in the FSA report was about 79%.

Water Budget Analysis

Figure 24.3 illustrates the computed water level fluctuation from the forebay and wet pond water budget/analysis assuming both features are filled to capacity prior to the historical period investigated. It was found that minimal fluctuations of the depth of water in the forebay and wet pond compartments may be anticipated. The worst cases occurred when there was no significant rainfall for an entire month in the middle of the summer periods. That is, water levels were computed to be approximately 0.1 m lower than usual during these months.

An additional water budget simulation was completed for the same historical period assuming that both the forebay and wet pond are empty prior to the first rainfall event in May of each year examined. It was estimated that both the forebay and wet pond will be filled to capacity after the first few rainfall events of each annual period examined. These computed water budgets are therefore essentially the same as those illustrated in Figure 24.3.

Figure 24.3 Results of water budget analysis.

BOSS-DAMBRK Modeling

Figure 24.4 presents the computed 13 mm storm event stage depletion for the SWM facility for the ultimate development conditions, if the retrofit extended-detention storage is full prior to the storm. The control berm is computed to be overtopped slightly. However, during a 13 mm storm event when the full retrofit active storage of $\pm 61,000 \text{ m}^3$ is available, the SWM facility is computed to operate properly in accordance with the original (FSA, 1994) design parameters.

Figure 24.4 Ultimate Ninth Line SWM facility depletion stage, 13 mm storm event and initial retrofit storage at 188.5 m.

24.5.2 Pine Valley Stormwater Management Pond

Table 24.5 presents results of the STORM modeling analysis for the retrofit design of the Pine Valley stormwater management pond.

Table 24.5 Summary of STORM modeling results for the Pine Valley SWM pond.

Drawdown Time (hrs)	Qr (m ³ /s)	R (mm)	R° (mm)	F (%)	E (%)
48	0.09	426	190	85	47

Figure 24.5 Sediment removal vs pond size. Pine Valley Pond, 48 hours detention time.

Figure 24.6 Sediment removal vs pond size. Pine Valley.

It was computed that for an average area of 14,300 m² and a detention time of 48 hours, the storage volume of 15,000 m³ will provide 47% removal of suspended particles 20 µm or larger. The computed number of overflow events will be 22. Based on the same annual sediment loading rate data collected in the Ottawa area, that was also used for the Ninth Line SWM facility project, a removal efficiency of 47% is computed to lead to a yearly load of 165 tonnes - as compared to 231 tonnes computed for existing conditions and 321 tonnes computed for full development within the headwater basin.

Based on an average runoff volume coefficient for the Emery Creek watershed of 0.61, the 15,000 m³ of extended-detention storage in the retrofitted pond is computed to correspond to a rainfall event of about 8 mm.

Using the STORM computer program, a sensitivity analysis was undertaken to investigate sediment removal efficiencies relative to various extended-detention volumes and a 48 hours drawdown time.

The computed efficiency of sediment removal versus the size of pond is illustrated on Figure 24.5. A significant improvement is computed to be achieved as the size increases from 16,000 m³ to 25,000 m³. Further improvements if the pond is increased are less significant.

Figure 24.6 illustrates that, for the detention time of 48 hours, the removal efficiency is computed to improve significantly compared to 24 hours.

Acknowledgements

The authors are grateful for the guidance of Allan R. Steedman of Fred Schaeffer & Associates Ltd. during these two projects. We wish to also extend our appreciation to Janice Teare and Bob Levesque with the City of Mississauga for their valuable comments during the environmental assessment study, retrofit design/construction, and culvert replacement analysis. Finally, we also acknowledge gratefully, during the Pine Valley stormwater management pond retrofit project, the assistance from Glen MacMillan with the Metropolitan Toronto and Region Conservation Authority.

References

- Anacostia Restoration Team, 1992. Watershed Restoration Sourcebook: Collected Papers Presented at the Conference; "Restoring Our Home River, Water Quality and Habitat in the Anacostia", held November 6-7, 1991 in College Park M.D., Metro Washington Counc. Gov. D.C.
- Andrew Brodie Associates, 1981 and 1983. Pine Valley Village Master Drainage Plan and Storm Water Management Studies; City of Vaughan.

- Camp, J. R., 1946. Sedimentation and the Design of Settling Tanks, Trans. ASCE, Paper No. 2285, p.p. 895-958.
- Chen, C.N. 1975. Design of Sediment Retention Basins, Trans. National Symposium on Urban Hydrology and Sediment Control, University of Kentucky, p.p. 285-290.
- C. C. Tatham & Associates Ltd., 1996. Sixteen Mile Creek Culvert Improvements Study; City of Mississauga.
- Fair, G.M. and J.C. Geyer, 1954. Water Supply and Waste-Water Disposal, John Wiley and Sons, New York.
- Fred Schaeffer & Associates Ltd., 1993. Design Brief for Retrofitting the Pine Valley SWM Pond To Provide Run-off Quality Control for East Woodbridge Developments Ltd.; City of Vaughan.
- Fred Schaeffer & Associates Ltd., 1994a. Environmental Study Report - Retrofitting of the Ninth Line Storm Water Management Facility; City of Mississauga.
- Fred Schaeffer & Associates Ltd., 1994b. Design Brief - Retrofitting of the Ninth Line Storm Water Management Facility; City of Mississauga.
- Herson, L., 1989. The State of the Anacostia River: 1988 Status Report; Metro. Washington Counc. Gov., D.C.
- Ministry of Environment and Energy, 1994a. Stormwater Management Practices Planning and Design Manual.
- Ministry of Environment and Energy, 1994b. Water Management: Policies, Guidelines, Provincial Water Quality Objectives of the Ministry of Environment and Energy.
- Municipal Engineers Association, 1993. Class Environmental Assessment for Municipal Water and Wastewater Projects.
- Winter Associates and Gore & Storrie, 1993. Lisgar Region Water Quality Study - Final Report; City of Mississauga.