# Taggart – Loblaws Subdivision Stormwater Management Facility Design Report

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**June 11, 2013**

6th Submission: January 28, 2012
5th Submission: November 6, 2012
4th Submission: August 10, 2012
3rd Submission: April 27, 2012
2nd Submission: November 11, 2011
December 12, 2007
1604-00402/83

# TAGGART – LOBLAWS SUBDIVISION
# STORMWATER MANAGEMENT FACILITY DESIGN REPORT

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# 1.0 INTRODUCTION AND BACKGROUND

## 1.1 OVERVIEW

Stantec Consulting Ltd. has been retained to complete the design of the Taggart - Loblaws Subdivision Stormwater Management Facility in the northern portion of the Kanata West Community. The facility will provide end-of-pipe treatment for a future development area located between Highway 417, Didsbury Road, and the Carp River. The tributary lands, as outlined in **Figure 1** include the future Transitway, and portions of Campeau Drive and Didsbury Road. The combined 30 ha tributary area as provided on **Figure 2** will consist of commercial and mixed-use developments. The location of the stormwater management facility (SWMF) and tributary catchments are consistent with the information presented in the June 16, 2006 Kanata West Master Servicing Study (KWMSS) by Stantec Consulting Ltd. and IBI Group.

### 1.1.1 Submission Revisions

Following an initial consultation with the City of Ottawa in 2007, the decision was made to design the SWMF such that it remains outside of the existing floodplain. In so doing, the proposed works do not cause any reduction in floodplain storage per existing conditions. Despite recent developments in the Kanata West Development Area, and in order to be most flexible with various phasing scenarios for the Carp River Restoration Plan (CRRP), the intent of this decision has been maintained and the pond lies outside of the regulatory floodplain line.

The previous submission of this report was submitted for third party review by Greenland Engineering on November 6, 2012; third party review comments were received by Stantec in an e-mail dated December 20, 2012. This submission addresses the third party review comments with the significant changes being the revision of the boundary condition used for determining pond and sewer HGLs and a 0.25m increase in the pond permanent pool elevation to raise the outlet above the normal water level of the Carp River. A response letter summarizing the changes made to address the third party review comments is included in the correspondence **Appendix G**.

The enclosed design revision has also included modifications to accommodate drainage from the future West Transitway (east of Didsbury Road). It is proposed by others that in the future the Transitway will be serviced through connection to the sewers within Roger Nielson Way (see Drawing SWM-1). Pipe sizes in Roger Nielson way were upsized to convey the additional estimated 10-year flows from the future West Transitway. Due to the pipe upsizing the inlet invert was lowered and required the pond bottom to be lowered by 0.05m to 90.20m in order to provide sufficient sediment storage within the forebay. Both the pipe size changes and pond bottom lowering are included in the following design. While, the drainage area extents for the Transitway east of Didsbury Road are not included in the drawings or models the areas were included in the sewer design sheet and hydrographs were added to the PCSWMM model.

A flow bypass structure was also added to the proposed sewer network to divert flows greater than the 25mm quality event runoff (1100L/s) directly to the pond maincell and bypass the forebay in order to reduce re-suspension of solids during higher intensity events.

### 1.1.2 Background

The Kanata West Development area and the Carp River have been studied in detail over the past several years. From the City's website [URL: http://ottawa.ca/en/env_water/tlg/alw/brs/subwatershed/completed/carp/restoration/chronology/index.html], the following bullet point chronology shows the progress of studies related to the Carp River over the past 11 years (note: this list was prepared by City staff, not by Stantec):

*The Carp River Restoration Plan has been under development for more than eleven years, beginning with the Carp River Watershed/Subwatershed Study in 2000:*

- *August 2000 - Contract awarded for Carp Watershed/Subwatershed Study*
- *March 2003 – Council approves Kanata West Concept Plan*
- *January 2005 - Council approves the Carp Watershed/Subwatershed Study*
- *June 2006 - Kanata West Class Environmental Assessment (EA) Notices of Completion issued*
- *July 2006 – Part II Order requests received*
- *April 2007 - The Minister of Natural Resources stated that the objectives of the provincial natural hazards policy have been met in the Carp River Restoration Plan*
- *2007 - Auditor General conducts audit of Carp River watershed and related projects in response to call to Fraud and Waste Hotline*
- *January 15, 2008 – City staff member informs TSH Environmental and Engineering Services of concerns with modeling*
- *January 24, 2008 – TSH Environmental and Engineering Services confirm errors in model*
- *January 25, 2008 – City advises Ministry of the Environment (MOE)*
- *February to April 2008 – Consultants of record, TSH and CH2M Hill, correct models*
- *April 23, 2008 – Auditor General releases Audit of the Carp River Watershed and Related Projects at City Council*
- *Recommendations were referred to the Agriculture and Rural Affairs Committee meeting of May 8, 2008 and the Planning and Environment Committee (PEC) meeting of May13, 2008. The issue of potential conflict of interest with respect to consultants and the community lands development recommendations was referred to the Corporate Services and Economic Development Committee meeting of May 6, 2008*
- *May 14, 2008 – Council directed staff to proceed with the Third Party Review.*
- *June 25, 2008 – Council approved the amended terms of reference for the Third Party Review.*

- *July 14, 2008 –Ministry of Transportation of Ontario (MTO) and MOE invited to participate on the Project Advisory Committee.*
- *July 21, 2008 – Minister of Environment issues an order under the Environmental Assessment Act imposing conditions with respect to the 22 projects within the Kanata West Development area.*
- *August 8, 2008 – Greenland International Consulting Ltd. was retained to conduct Third Party Review.*
- *February 13, 2009 – Project Advisory Committee received draft Third Party report.*
- *March 16, 2009 – Updated Existing Conditions model released*
- *April 9, 2009 – Technical Briefing on final draft of Third Party Review Report*
- *April 9, 2009 – Final draft of Third Party Report posted on City website for comment*
- *April 20, 2009 – Public comment period ends; comments forwarded to Greenland*
- *May 12, 2009 – PEC recommends approval of Third Party Review to Council*
- *May 27, 2009 – Council approves Third Party Review*
- *November 24, 2009 – Ministry of Environment advises that conditions 1 and 2 of the Minister's Order have been satisfied and that the City can proceed to put new notices of completion for the seven projects that are the subject of Part II Order requests in July 2006.*
- *July 30, 2010 – New Notices of Completion are posted July 30, 2010.*
- *August 29, 2010 – Part II Order requests received*
- *March 30, 2011 – Minister of Environment dismisses Part II Order requests and confirms that an individual Environmental Assessment is not required for the seven disputed projects. The Minister further directs that Stormwater Management Ponds 1, 2, and 5 not proceed until such time as the stormwater management models are calibrated and validated*
- *September 2011 – Model Validation Report completed and filed with Ministry. Models are validated and confirm the approach in the Environmental Assessments was valid. Stormwater Management Ponds 1, 2, and 5 may proceed to design and construction*

Because the models have now been validated, the EAs deemed valid, and all reports and models filed with the MOE, we are now submitting this report to propose proceeding with the development of the subject lands.

## 1.2 PURPOSE

This report has been prepared in order to demonstrate that the proposed design of the Taggart – Loblaws Subdivision SWMF adheres to all of the established criteria presented in the background documents. The report includes hydrologic and hydraulic analyses, a detailed trunk sewer alignment, revised transportation corridors, detailed design of the SWMF, and provides guidelines for future development within the tributary catchments.

## 1.3 BACKGROUND RESOURCES

The following documents were referenced in the preparation of this report:

- *Carp River Model Calibration Validation Exercise, Final Report*, Greenland International Consulting Ltd., July 13, 2011
- *Carp River, Poole Creek and Feedmill Creek Restoration – Class Environmental Assessment,* Totten Sims Hubicki Associates and Parish Geomorphic, June 2006
- *Carp River, Poole Creek and Feedmill Creek Restoration – Update and Amendment*, Delcan, July 2010
- *Carp River Watershed/Subwatershed Study,* Robinson Consultants, December 2004
- *Flow Characterization and Flood Level Analysis, Carp River, Feedmill Creek and Carp River,* CH2MHILL, July 2005
- *Kanata West Master Servicing Study,* Stantec Consulting Ltd and IBI. Group, June 2006
- *Implementation Plan Kanata West Development Area*, Delcan, July 2010
- *Third Party Review – Carp River Restoration Plan*, Greenland International Consulting Ltd., March 2009

These documents were prepared in accordance with the Planning & Design process of the Class Environmental Assessment (EA). The recommended stormwater management (SWM) plan presented in these documents included a system of seven wet pond facilities to provide end-of-pipe treatment, as provided on **Figure 5.3** (an excerpt from the Kanata West Master Servicing Study, see Figures section at the end of this report).

Additional documents referenced in designing the SWM Plan for the Taggart – Loblaws Subdivision SWMF include:

- *City of Ottawa Sewer Design Guidelines*, 2$^{nd}$ Ed., City of Ottawa, October 2012
- *Stormwater Management Planning and Design Manual*, Ministry of the Environment (Ontario), March 2003
- *Kanata West Overall Monitoring Plan*, City of Ottawa, http://ottawa.ca/en/env_water/tlg/alw/brs/subwatershed/completed/carp/restoration/kanatawest/index.html

# 2.0 STORMWATER MANAGEMENT DESIGN

The following sections describe the stormwater management plan for the future developments within the SWMF drainage area as provided by the background documents and agency literature.

## 2.1 PROPOSED CONDITIONS

Following construction of the SWMF, construction of trunk storm sewers through both the Taggart and Loblaws properties will begin. These commercial properties will consist mainly of retail buildings and the associated asphalt parking areas. Typical percent imperviousness values for these sites are approximately 85%.

The sites will be designed using the “dual drainage” principle, whereby the minor (pipe) system is designed to convey the peak rate of runoff based on a 5 year return period storm capture rate for most areas, and a 10 year capture rate for arterial road and Transitway areas. Runoff from larger events is conveyed by both minor (pipe) and major (overland) channels such as roadways and walkways safely off-site without impacting proposed or existing downstream properties. Inlet control devices (ICDs) will be specified to limit the inlet capture rate to the minor system and thus control the hydraulic grade line during major storms. Solid covers will be installed on all manholes located in ponding areas to limit inflows to the minor system to that of the ICDs.

Major runoff from developments is generally directed towards the Carp River. Details of the major system flow direction are provided on **Drawing SWM-1**. Water depths shall not exceed 300 mm, including ponding depth and major system overflow depth, and street cross-sections and profiles shall be designed to ensure conveyance of the peak 100 year design storm.

## 2.2 STORMWATER MANAGEMENT PLAN

### 2.2.1 Trunk Storm Sewer Sizing

Trunk sewer sizing and alignment provided in the Master Servicing Study was determined using preliminary street alignments and catchment boundaries. **Drawing SWM-1** provides a revised sewer alignment that is consistent with up-to-date information pertaining to the major roadway and Transitway corridors and property ownership. Trunk sewers were sized using the Rational Method to convey peak flow from 5 year and 10 year return periods from non-arterial and arterial roadways, respectively. In those sewers passing from an arterial roadway to a non-arterial roadway, a blended rate was used, as provided on the storm sewer design sheets in **Appendix A**.

#### 2.2.1.1 Standing Water in Downstream Sewers

Due to a combination of grade raise constraints within developments upstream and minimum cover requirements over the storm sewers at the pond inlet, storm sewers were designed with standing water during periods of Normal Water Level in the pond. The extent of standing water will reach 22 metres upstream of MH103 towards MH104 and 14 metres upstream of MH110 towards MH111.

It is anticipated that the introduction of standing water will increase sedimentation within the affected storm sewers. As requested by the City, the proposed storm sewer system has been modeled with sediment depths up to 25% of the pipe diameter. Maintenance and monitoring frequency of these sewers will need to be increased, in particular during the construction phases, as mentioned in Section 6.0.

#### 2.2.1.2 Stormwater Pumping Station for Rona Properties

Due to grading constraints within the Rona properties, identified as areas A1 and A2 in drawing SWM-1, a stormwater pumping station is proposed. A very large volume of fill would be required to raise the grade of this site to permit gravity flow to SWM Pond 3 and it is not possible to drain the lands to the existing Broughton Pond as there is no available capacity within the pond. As a result a pumping station was proposed. This station will serve to provide storm servicing to the property without excessive grade raise and will also isolate the property from an elevated hydraulic gradeline during rare events. Flow rates from the pumping station will be limited to 150 L/s (~ 20 L/s/ha) determined from Modified Rational Method calculations for the draft site plan available at the time in order to maximize the use of available storage and minimize the pump size requirements. Sag storage will be provided in sufficient quantity to store the 100 year storm event. The current site plan consists of a commercial flat roof building with a roof area of approximately 1.2ha, the remainder of the site is proposed to be parking lots and some small retail buildings. These conditions are reflected in the revised hydrologic and hydraulic modeling below. Detailed design of the pumping station and required storage will be provided in support of site plan application for the respective properties.

The detailed design of the Rona site may consider onsite quantity and quality control options with direct discharge to the Carp River as an alternative to pumping. However, the facility design has accommodated pumped flows from the site. Should the Rona flows not be pumped to the SWM facility, the allowable release from the Rona site would have to be determined by re-analyzing the SWM facility without the Rona site flows and determining the available difference between the total allowable flow and the revised pond discharge.

### 2.2.2 Water Quality Control

The Taggart-Loblaws SWMF is required to achieve a ‘normal’ level of treatment of urban runoff according to MOE criteria – representing a 70% removal of total suspended solids (TSS). This criterion was established by the Carp River Watershed/Subwatershed Study (Robinson Consultants, December 2004, p.152):

*For facilities discharging to Carp River or any other tributaries within the subwatershed area, Level 2 control, suitable for warmwater species is required. This level of water quality control requires that 70% of total suspended solids in the incoming stormwater be removed by stormwater Best Management Practices.*

The facility, as outlined in Section 3.4, has in fact been designed with sufficient permanent pool and extended detention storage to provide an enhanced level of treatment (80% removal). The end-of-pipe facility will be designed according to the recommendations of the Ministry of the Environment Stormwater Management Planning and Design Manual, as provided in Section 3.4, and therefore no quality control infrastructure is required within each of the individual developments tributary to the pond.

In order to minimize re-suspension of settled solids during higher intensity events, the pond inlet has been designed such that flows above the 25mm event will be bypassed to the main cell of the pond. The bypass structure is located in STM102 and consists of a 2.6m wide rectangular weir with an invert of 93.00m. See **Drawings SP-1 and DS-3** for bypass details.

### 2.2.3 Water Quantity Control

In the ultimate condition the SWM facility is primarily for water quality control however, it is also required that the facility match post-to-pre flow rates for the 10 year event. Additionally, based on discussions with the City of Ottawa it may be required that in the interim condition, the pond be designed to provide 100 year post-to-pre quantity control. The design presented herein is for the interim condition and therefore assumes that a higher level of quantity control will be required. It is noted that the allowable outflow rate to meet the 10-year post-to-pre requirement is 2.38 $m^3/s$, as calculated from the KWMSS XPSWMM model (see **Appendix F.1** for the stage-discharge graph from the pond). As discussed further in section 2.6.1 the 100-year pre-development release rate for the tributary areas is 3.06 $m^3/s$ and designing for control to this target rate meets the ultimate condition flow control requirements in the 10-year event as well (see **Appendix E.1** 12hr SCS storm discharge rate summary table).

Control within the individual developments (with the exception of Arterial, Transitway, and Rona properties) is required to reduce the peak flow entering the minor system to the 5 year inflow rate (10 year for arterials and Transitways) and to store a specified areal volume rate (see section 4.0 for prescriptive detailed design criteria for future development). As indicated in Section 2.2.1.2 above, the Rona properties, identified as areas A1 and A2 on Drawing SWM-1, will control discharge to a combined maximum rate of 150 L/s (19.8 L/s/ha) and storage in sufficient quantities to completely store the 100 year storm event. This reduced release rate is related to the proposed installation of a stormwater pump station, as described above.

## 2.3 RAINFALL DISTRIBUTIONS

In order to determine the worst-case rainfall distribution for the minor and major systems, two different design storm distributions were considered. As an additional method of evaluating the performance of the proposed infrastructure, rainfall data from three historical rainfall events in the region were modeled, as well as climate change scenario storms (20% increase of City IDF curves) used to stress test the design.

### 2.3.1 Design Storms

Two design storm distributions were modeled over the drainage area. The Chicago distribution was selected due to its tendency to generate high peak flows in urban catchments, similar to the proposed developments. Since the Carp River models (CH2MHILL, Greenland) identified the 12 hour SCS distribution as the critical design event for that waterbody, the hydrologic and hydraulic response to this distribution was also of interest. While the SCS distribution is typically used to evaluate rural catchments, it was useful as a method of comparing results with those presented in the Master Servicing Study, and Carp River modeling.

To determine relevant critical design events, 1500 storms were run for the hydrology portion only and the results were analyzed to select additional events for further analysis (see section 2.5.2 for more information).

Design storms were selected to meet several requirements as listed below. Note that only design storms with available boundary conditions were modeled in the fixed backwater scenario and all storms were modeled in the free-flow outlet scenario:

1) Storms previously modeled in DDWSMM in earlier design submissions which were obtained from CH2MHILL's Carp River modeling and differ from the typical distribution.
    - 2 year 12hr SCS (Fixed & Free)
    - 5 year 12 hr SCS (Fixed & Free)
    - 10 year 12hr SCS (Free)
    - 100 year 12hr SCS (Fixed & Free)
2) Design storms required by the City based on City of Ottawa IDF curves
    - 100 year 3hr Chicago (Fixed & Free)
    - 100 year 6hr Chicago (Fixed & Free)
3) Other design storms required by the City, and additional storms
    - 25mm Chicago (Free)
    - 100 year 3hr Chicago (different r-value) (Fixed & Free)
    - 100 year 48hr Chicago (Fixed & Free)
    - 25 year 12hr SCS (Fixed & Free)
4) Design storms which resulted in maximum peak runoff for 5 year and 10 year return periods and were used to set inlet rates for subcatchment areas.
    - 5 year 6hr SCS (Free)
    - 10 year 5hr Chicago (Free)

### 2.3.2 Historical Storms and Climate Change Storms

In addition to the design storm distributions, three historical rainfall events were also modeled:

- July 1979
- August 1988

- August 1996

While these events do not establish a level-of-service for the proposed infrastructure, they are used to provide an increased level of comfort with the proposed stormwater management plan.

As per City of Ottawa 2012 Sewer Design Guidelines, drainage systems are to be 'stress tested' for climate change scenarios by using a 20% increase in the City's 100 year storm intensities. Therefore, three climate change storm scenarios are modeled:

- 20% increase of 100 Year 12 hour SCS used by CH2MHILL
- 20% increase of City of Ottawa's 100 year 3 hour Chicago storm
- 20% increase of City of Ottawa's 100 year 6 hour Chicago storm

All hydrologic and hydraulic modeling files are available on the enclosed compact disc located in the appendices**.**

## 2.4 MODEL SELECTION AND DEVELOPMENT

### 2.4.1 Model Selection

Earlier design submissions for the proposed site used DDSWMM for hydrologic modeling with the resulting minor system inflow hydrographs imported into the XPSWMM modeling software for hydraulic analysis. The previous submission revised the modeling approach to conduct hydrologic and hydraulic modeling within the same model using PCSWMM. Internal comparison of the previous DDSWMM model to the PCSWMM model under different storm scenarios indicates that hydrographs generated for each subcatchment are comparable between the two models. This is to be expected since both DDSMWM and PCSWMM use the EPA-SWMM method of generating hydrographs. PCSWMM uses the EPA-SWMM 5.0.022 computational engine for analysis and the included models can also be opened and reviewed using the free EPA-SWMM GUI.

### 2.4.2 Model Structure

The storm sewer network modeled in PCSWMM includes only trunk sewers and does not include detailed design storm sewers for the development blocks. Major flows are routed overland via roadways modeled with the appropriate ROW transects. Note that a generic transect is used for flows which are routed through development blocks. A lumped storage is assigned to each of the subcatchments as discussed in **section 2.4.4**. Note that the pond forebay bypass structure was not included in the model scenarios but was separately analyzed to determine the weir elevation and assess the impact on the storm sewer hydraulic grade line. Copies of all PCSWMM models are included on the enclosed CD.

### 2.4.3 Model Parameter Selection

Model parameters used in the hydrologic and hydraulic calculations within the PCSWMM model are based on City of Ottawa design criteria and were applied as outlined below. Calculated parameter values that vary per subcatchment, node, or link are included in the parameter summary tables in **Appendix C.1.**

#### 2.4.3.1 Hydrologic Parameter Selection

Imperviousness ratios were assigned based on a conversion of runoff coefficients for all areas where actual or proposed paved areas were not known. Actual impervious areas were available for Didsbury Road and drainage areas B and C, therefore, impervious ratios have been calculated for these areas. Runoff coefficients for all other areas were determined based on proposed or anticipated land use as specified in the KWMSS and the City of Ottawa Sewer Design Guidelines:

- Residential – High Density, C = 0.60
- Core Area (urban downtown), C = 0.80
- Commercial, C= 0.80
- Employment Lands, C= 0.70
- Collector Road/Transitway R.O.W., C = 0.7
- Parkland/Open Space/Hydro/Pipeline Corridor, C = 0.20

Infiltration calculations are based on Horton Infiltration with the following standard values assumed for all subcatchments, as per section 5.4.5.5 of the City of Ottawa Sewer Design Guidelines:

- Final Infiltration Rate = 13.2 mm/hr
- Initial Infiltration Rate = 76.2 mm/hr
- Decay Constant = 4.14 $hr^{-1}$

Other parameters assigned for all subcatchments include the following as per the City of Ottawa Sewer Design Guidelines:

- N Imperv = manning's n for impervious areas = 0.013
- N Perv = Manning's n for pervious areas = 0.25
- Dstore Imperv = depression storage for impervious areas = 1.57
- Dstore Perv = depression storage for pervious areas = 4.67
- Subcatchment width is calculated as per Section 5.4.5.6

#### 2.4.3.2 Hydraulic Parameter Selection

Exit losses at manholes were set for all pipe segments based on the flow angle through the structure. Exit losses were assigned as per City guidelines (Appendix 6b).

**Table 2-1: Exit Loss Coefficients for Bends at Manholes**

| Degrees | Coefficient |
|---|---|
| 11 | 0.060 |
| 22 | 0.140 |
| 30 | 0.210 |
| 45 | 0.390 |
| 60 | 0.640 |
| 90 | 1.320 |
| 180 | 0.020 |

Other parameters applied within the model include the following:

- Manning's n = 0.013 *(Section 6.1.8.1)*
- Orifice Discharge Coefficient = 0.61
- Weir Discharge Coefficient = 1.76

Free flow scenarios were run using the full St.Venant Dynamic Wave equations including a dampened inertial term, while the backwater scenarios were run while ignoring the inertial term for stability purposes (i.e. a Diffusion Wave simulation).

### 2.4.4 Subcatchment Storage

As noted in section 2.2.3 above, since the design of commercial properties can accommodate in excess of the KWMSS-required 50 $m^3$/ha, commercial developments were assumed to provide 200 $m^3$/ha. In addition, the Rona properties will limit discharge to 150 L/s (19.8 L/s/ha) and provide sufficient storage to completely contain the 100 year event. The result of this increased storage is a reduction in overland flow.

### 2.4.5 Inlet Rates

Inlet flow rates for each subcatchment were sized based on the maximum 5 and 10 year peak inflow rates for residential/commercial/non-arterial roadway land uses, and arterial roadways/Transitways, respectively. A series of 5 and 10 year SCS and Chicago design storms were run to determine the peak 5-year or 10-year flows to be captured by the minor system. These flow rates were then assigned to their respective outlet in the model. A summary of the inlet rate for each subcatchment is included in the Parameter Summary Table in **Appendix C.1**.

### 2.4.6 Model Scenarios

In order to examine a range of conditions, multiple model scenarios were run. As indicated in section 2.3 above, 13 design storm distributions and three historical storms were used in order to test the design. As per City of Ottawa criteria, three climate change scenarios were also run to stress test the system. For each storm distribution four alternative scenarios were modeled as described below:

1) Sedimented pipes with a static backwater
2) Sedimented pipes with a free flow outlet condition

Due to the submerged sewer outlet to the pond, the City requires that the proposed sewer network be modeled under the condition of sediment accumulation within the pipe network for pipes with inverts below the pond permanent pool (92.25m). The modeled sediment is to have a depth up to 25% of the pipe diameter. The sedimented pipe scenario modeled sediment in all pipes with inverts below the permanent pool elevation.

The KWMSS requires that model backwater conditions start at the MVC 1983 flood level of 93.98m for all 100 year storm events. Backwater conditions for the 2 year, 5 year and 25 year return period 12hr SCS storms were obtained from Greenland Consulting Engineers in an e-mail on January 10, 2013.

**Table 2-2** below summarizes the boundary conditions applied for each of the return periods analyzed. Note that Greenland did not complete any modeling for the 10-year event therefore it was not included in the fixed backwater scenarios.

**Table 2-2: Summary of Backwater Elevations from Carp River**

| Storm Event | Backwater Elevation |
|---|---|
| 2 year 12hour SCS | 92.92 |
| 5 year 12 hour SCS | 93.15 |
| 25 year 12 hour SCS | 93.40 |
| All 100 year events | 93.98 |

## 2.5 MODEL RESULTS

### 2.5.1 Comparison of Model Scenarios

As noted in the previous section, four categories of model scenarios were run for each storm event. For the purposes of assessing HGL results the backwater scenarios for the 100 year 3 hour Chicago and 100 year 12 hour SCS storms have been used however, for all other purposes (such as calculating pond outflows etc.) only the free flow conditions have been considered. Free flow conditions were used since the backwater scenarios resulted in high flow rates into the pond from the Carp River, obscuring the true outflow rate and total discharge volumes from the pond.

### 2.5.2 Hydrologic Modeling Results

The PCSWMM base model was used for hydrologic computations of runoff hydrographs in 5-year and 10-year events. In the initial development of the model, approximately 1500 storm scenarios were run in order to determine the maximum peak runoff generated in each of the 5-

year and 10-year storm events. These values were then used to set the capture rates for each of the subcatchments. For subsequent revisions to the model and updates of capture rates, the hydrologic computations were only repeated for the storm events which yielded the highest runoff rates in the initial analysis. The storm events which generated the highest peak runoff were the 5-year 6 hour SCS storm and the 10-year 5 hour Chicago storm. A summary of the peak flows for each of the subcatchments in each storm scenario is included on the enclosed CD. **Appendix C.1** includes a summary table of the peak 5-year and 10-year runoff rates used for capture rates.

Hydrologic modeling was completed as part of the dual drainage modeling in each PCSWMM model scenario to determine major system flow rates and depths of flow. Detailed hydrologic model parameters for each subcatchment are included in **Appendix C.1**. The output files, as provided in **Appendix D and Appendix C.3**, demonstrate that the maximum flow depth (including surface ponding) occurring down any street section is less than 0.30 m. The only exceptions occur in the commercial developments (areas A1, B and C) and existing areas E1 and E2, where peak flows are assumed to flow down a single street segment, when in reality, flow will be conveyed by multiple segments or sheet draining over the surface, thereby reducing flow depths.

### 2.5.3 Hydraulic Modeling Results

Hydraulic boundary conditions for the Future Carp River were obtained from Greenland Consulting Engineers.

Utilizing PCSWMM and the various storm events mentioned above, a hydraulic grade line was computed for all of the described events. **Table 2-3** below presents the clearance between the proposed ground elevations and the 100 year hydraulic grade line for the critical 100 year Chicago and SCS distributions with a fixed pond backwater elevation of 93.98m. Future detailed design stages will require that a minimum clearance of 0.3 m is provided between all underside-of-footings and HGLs. It is noted that the HGL clearance at STM117 and CBMH118 is less than 0.30 m but remains below the ground surface. The low clearance at these structures is due to static backwater conditions from the existing Broughton SWMF (HWL=93.60) and existing low grades. Re-grading of this area is proposed as part of the Didsbury Road works however, the extent of the grade raise was limited by the requirement to maintain the overland flow through this area. Nevertheless, there are no service connections to this section of sewer and none are expected in the future so clearance with USFs should not be a concern. Furthermore, both the existing church and future commercial buildings are slab-on-grade buildings and as such basement flooding is not a risk.

**Table 2-3: 100 year Hydraulic Grade Line Results along Trunk Sewers**

| | | 100yr 3hr Chicago | | 100yr 12hr SCS | |
|---|---|---|---|---|---|
| **Node ID** | **Ground Elevation (m)** | **HGL (m)** | **Clearance (m)** | **HGL (m)** | **Clearance (m)** |
| CBMH108 | 97.42 | 95.21 | 2.21 | 95.12 | 2.30 |
| CBMH122 | 96.79 | 94.88 | 1.91 | 94.85 | 1.94 |
| CBMH121 | 96.01 | 94.98 | 1.03 | 94.40 | 1.61 |
| CBMH123 | 97.59 | 95.58 | 2.01 | 95.57 | 2.02 |
| CBMH118 | 94.05 | 93.87 | 0.18 | 93.82 | 0.23 |
| CBMH120 | 94.50 | 94.26 | 0.24 | 94.02 | 0.48 |
| CBMH119 | 94.17 | 94.03 | 0.14 | 93.91 | 0.26 |
| STM115 | 95.80 | 94.92 | 0.88 | 94.90 | 0.90 |
| STM116 | 96.07 | 94.92 | 1.15 | 94.91 | 1.16 |
| STM114 | 95.50 | 94.89 | 0.61 | 94.87 | 0.63 |
| STM117 | 93.80 | 93.66 | 0.14 | 93.65 | 0.15 |
| STM105 | 96.09 | 94.94 | 1.15 | 94.87 | 1.22 |
| STM106 | 96.52 | 95.02 | 1.50 | 94.93 | 1.59 |
| STM107 | 96.82 | 95.17 | 1.65 | 95.08 | 1.74 |
| STM104 | 95.86 | 94.75 | 1.11 | 94.67 | 1.19 |
| STM111 | 95.60 | 94.83 | 0.77 | 94.82 | 0.78 |
| STM109 | 95.00 | 94.64 | 0.36 | 94.62 | 0.38 |
| STM112 | 95.72 | 94.85 | 0.87 | 94.83 | 0.89 |
| STM110 | 95.43 | 94.72 | 0.71 | 94.70 | 0.73 |
| STM113 | 95.82 | 94.85 | 0.97 | 94.83 | 0.99 |
| STM102 | 95.00 | 94.28 | 0.72 | 94.26 | 0.74 |
| STM103 | 95.22 | 94.49 | 0.73 | 94.46 | 0.76 |
| STM101 | 95.00 | 94.13 | 0.87 | 94.13 | 0.87 |
| CAMPEAU_STUB | 96.50 | 96.05 | 0.45 | 96.05 | 0.45 |
| STUB | 95.11 | 94.85 | 0.26 | 94.83 | 0.28 |
| CBMH120A | 94.42 | 94.01 | 0.41 | 94.01 | 0.41 |
| TW-D2 (STM107A) | 95.38 | 94.66 | 0.72 | 94.22 | 1.16 |
| J4 (STM102-2) | 97.00 | 95.18 | 1.82 | 95.09 | 1.91 |

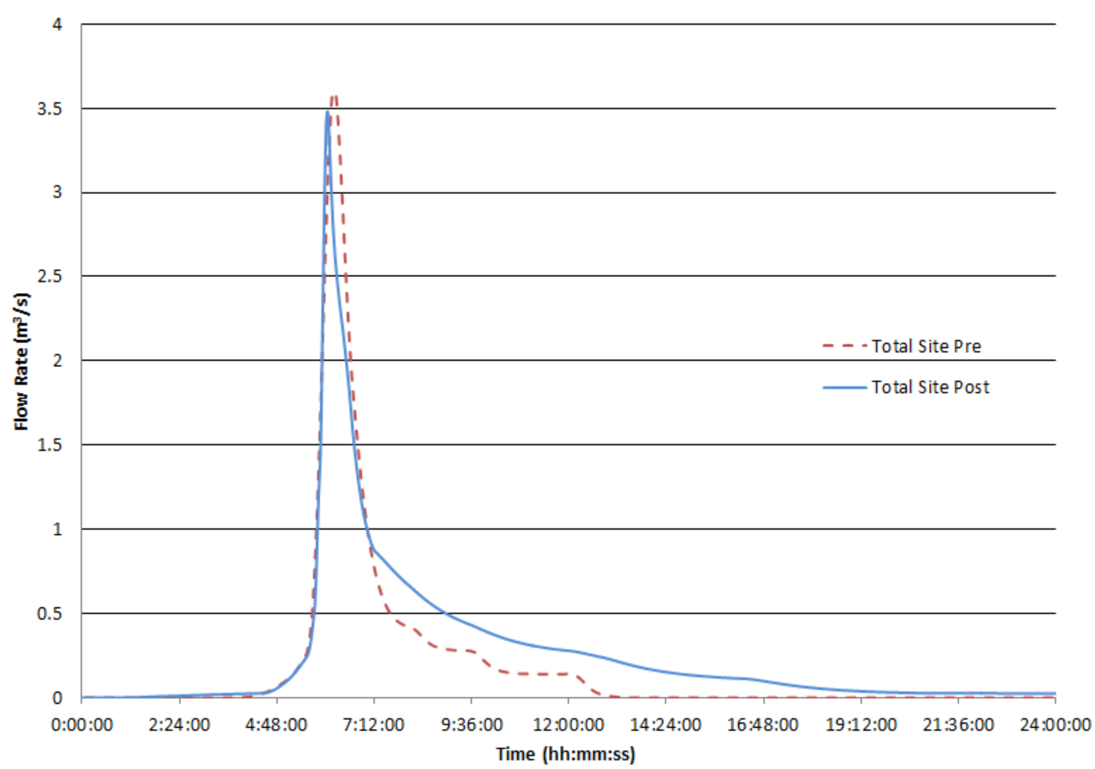
## 2.6 WATER QUANITITY CONTROL REQUIREMENTS

### 2.6.1 Interim Conditions 100 year Post-to-Pre Requirement

As per discussion with the City of Ottawa it may be required that the pond design provide sufficient quantity control in the interim condition to limit the total site peak flow rate to the 100year pre-development rate. The pre-development rate for the site areas tributary to the SWM pond was obtained from XPSWMM modeling completed by CH2MHILL as part of their report “Flow Characterization and Flood Level Analysis: Carp River, Feedmill Creek and Poole Creek” July 2005.

Based on the CH2MHILL report and drainage area drawings the Pond 3 tributary areas fall within the KTC-2 and KTC-4 drainage areas. The XPSWMM model inputs for these drainage areas were revised to include: reduced drainage areas to reflect only the portion of these areas that is tributary to the pond (including the additional West Transitway area east of Didsbury Road); modified catchment width to reflect the revised catchment shape; revised time of concentration. Runoff hydrographs were generated for the 100year 12hour SCS storm distribution. Hydrographs from each of the two drainage areas were then summed to determine the total 100-year pre development runoff rate for the pond tributary areas. The resulting 100year pre-development peak flow rate is 3.59m$^3$/s (see **Appendix B**). **Figure 2.1** below shows the pre and post development hydrography for the 100 year 12 hour SCS storm.

**Figure 2.1: 12 hr SCS Total Site Pre and Post-development Hydrographs**



In order to meet a post-development peak flow rate of $3.59m^3/s$ during the 100 year 12 hour SCS storm event and accommodate the additional future West Transitway flows, the pond spillway configuration has been modified from the previous design. The spillcrest is now set at an elevation of 93.55m per the KWMSS (previously 93.40m in order to contain the 10 year event) and has a width of 20 m. Post-development modeling in PCSWMM indicated that with the proposed pond configuration, the peak flow rate from the total site during the 100 year 12 hour SCS storm can be restricted to $3.48m^3/s$ and therefore would meet the 100year post-to-pre requirement. See **Appendix B** for calculations and hydrographs.

### 2.6.2 Carp River Model Calibration and Validation Report Requirements

Greenland Consulting was commissioned by the City of Ottawa to update hydrologic and hydraulic models for the Carp River that were created as part of the Carp River Restoration Plan (CRRP). The report provides inflow and outflow “target hydrographs” in its Appendix E for each of the KW ponds. Section 9.7 “Target Hydrographs” of that report indicates that the peak flow and total runoff volumes exiting the pond should not be exceeded, otherwise the proposed model must be integrated into the current Carp River corridor model to assess impacts. Below is an excerpt from page 58 of that report:

> *“As development proceeds, there could be some changes in land use from the information available during the preparation of the KWMSS. Therefore, during the review of the individual development application, the hydrology report would provide a summary of the inputs from all the contributing area to the SWM facility. The composite hydrograph entering the SWM facility would have to match peak flow and runoff volume for the approved hydrograph from the KWMSS. Once the peak flow or runoff volume values are exceeded the new information would be used to replace the target hydrograph once it has been demonstrated that there are no impacts to the Carp River.”*

Due to the various factors that affect the shape of hydrographs it is not likely that hydrographs will match exactly when using different modeling scale, different storm distributions, time steps, models, etc. We can however summarize the basic hydrograph characteristics that the above-excerpt specifies. **Table 2-5**, below, shows the peak flow and total volume, calculated from the “target hydrograph” from the calibration report, and the two 100 year events that were modeled in this report.

It is shown in **Table 2-5** that the pond successfully reduces the pond and total site peak flow to below the pre-development target rate and the ultimate allowable pond outflow rate in the 100 year 12 hour SCS storm. The proposed pond outflow volume in the 100 year 12 hour SCS storm is greater than the target pond outflow volume from the Calibration Report however, the total runoff volume to the Carp River is only 3% greater than the target runoff volume to the Carp River. This increased volume is largely due to the addition of runoff from a portion of the West Transitway east of Didsbury road. This area was originally intended to be conveyed to the existing Broughton Pond and was not initially intended to be conveyed to Pond 3 and so was not included in the pond’s tributary area for the Calibration Report. While peak flow targets from pond 3 are still met there is an estimated 688m3 of additional volume released from Pond 3, however, this volume would also have been released to the Carp River from the Broughton Pond. It is noted that the additional volume is released to the Carp River over approximately 25 hours during the extended detention release and not during the peak discharge and therefore should have minimal impact on Carp River peak water levels.

**Table 2-4: Summary of Peak Outflows and Total Volumes, 100 Year Events**

| | Pond Outflow | | Total Site Flow | |
|---|---|---|---|---|
| | Peak Flow (cms) | Total Volume (cu.m) | Peak Flow (cms) | Total Volume (cu.m) |
| Allowable Pond Outflow (Calibration report) | 3.19 | 20204 | 4.87 | 22934 |
| 100 year, 12 Hour SCS storm Pre-development Flow (this report) | - | - | 3.59 | - |
| 100 Year, 12 Hour SCS storm Post-development Flow (this report) | 2.97 | 22892 | 3.48 | 23621 |

**Figure 2.2** provides a comparison of the peak outflow from Pond 3 as presented in the Carp River Model Calibration Validation Final Report and the results from detailed modeling presented herein. These release rates are based on a 12 hour SCS rainfall distribution.

**Figure 2.2: Pond Discharge Comparison – 12 hr SCS Distribution**

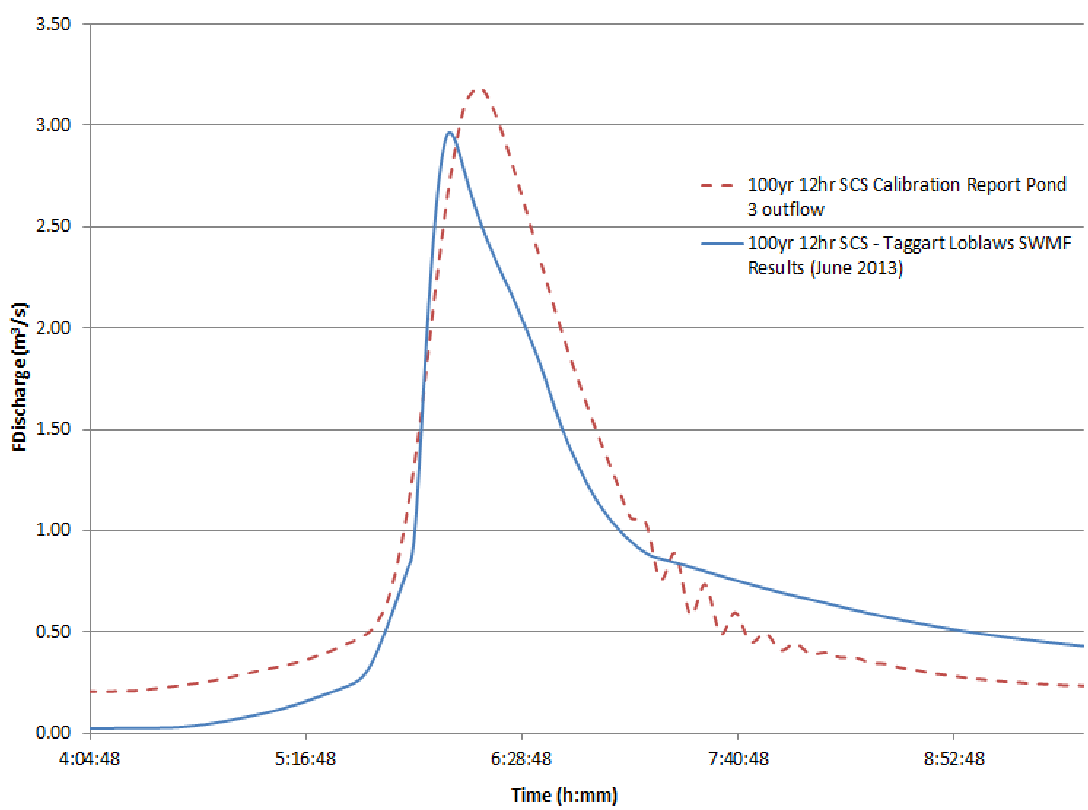
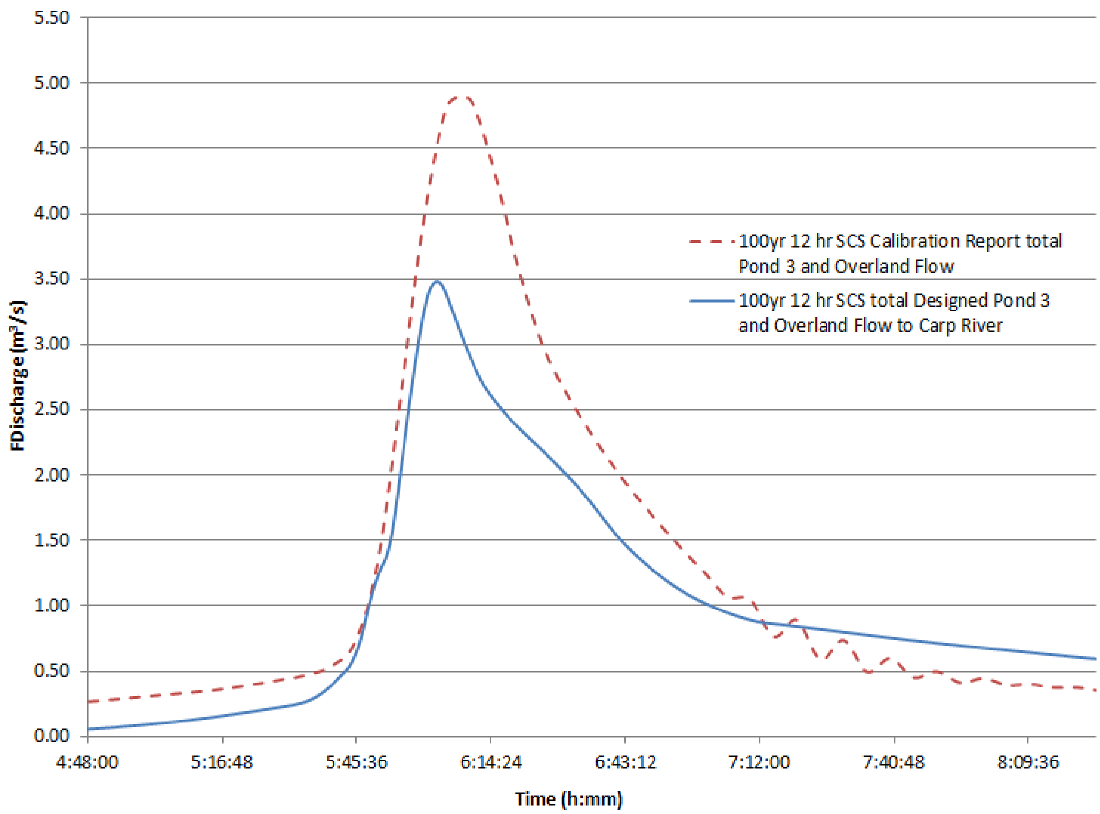


**Figure 2.3** below is a comparison of pond and major system outflows to the Carp River as presented in the Carp River Model Calibration Validation Final Report, with the results from detailed modeling. It is noted that the peak flows for the detailed modeling are below those from the Calibration Report.

**Figure 2.3: Total Discharge to Carp River Comparison – 12 hr SCS Distribution**



## 2.7 WATER BALANCE

The Carp River Watershed/Subwatershed study listed infiltration rates ranging from 262 mm/yr to 73 mm/yr in areas of high and low recharge, respectively. Detailed geotechnical work completed for the Kanata West Master Servicing study and the Fairwinds Subdivision modified these rates to 100 mm/yr and 50 mm/yr respectively for lands within the Kanata West Development, as provided on **Figure 5.4** (excerpt from KWMSS). The Taggart-Loblaws subdivision is identified as having an estimated infiltration rate between 50-70mm/yr. Infiltration requirements were established to provide base flow to the Carp River in post-development conditions.

The soils within the Taggart-Loblaws property were described by JD Paterson and Assoc. Ltd. to be topsoil overlying silty clay and clayey silt.

Future development sites are required to include infiltration measures in order to meet the 70mm/year infiltration rate indicated in the KWMSS. Calculations for the SWM pond area of the site are included in this report as well as a discussion of the proposed infiltration measures. Future developments will be required to install either infiltration trenches in a buffer area around the site, infiltration pits in grassed curb islands, or other infiltration technologies.

### 2.7.1 Pre-Development Infiltration

Pre-development infiltration rates were determined based on the KWMSS requirement of 50-70mm/year over the site area. The upper limit of 70mm/year was used to ensure sufficient infiltration over the site. Therefore, at 70mm/year over the pond block site area and Campeau Drive extention, Future Transitway and Roger Neilson Way with a total area of 4.8ha, the resulting total pre-development infiltration from the pond block area is 3,360m$^3$/year.

### 2.7.2 Post-Development Infiltration

In the Kanata West Master Servicing Study the criteria for infiltration is presented as follows:

*"Post development infiltration rates are to be increased by 25 percent above the pre-development rate. This rate of infiltration has been established to compensate for those areas (ie. Roadway corridors) that can not (sic) provide infiltration."*

However, the 25% augmentation is to account for roadways which have already been included in the infiltration volume calculations. Therefore, through discussion with MVC the 25% increase in the infiltration rate is not applicable. The target rate thus remains 70mm/year, or 3,360m$^3$/year (0.11L/s).

Due to the impermeable nature of the marine clay throughout most of the site infiltration measures within the clay will not meet infiltration requirements. Therefore, it is proposed that infiltration measures are applied within the shallow overburden above the top of the clay layer. Infiltration calculations should be completed on a site specific basis based on geotechnical investigations at the location for the infiltration trenches or basins.

For the purposes of this report, calculations were completed only for the SWM pond block and the future roadway areas including Campeau Drive, the Tranistway, and Roger Neilson Way. Detailed calculations are included in **Appendix E.3**.

Calculations for the infiltration to be provided within the SWM pond block indicate a required infiltration volume of 3,360m$^3$/year. Infiltration calculations for the SWM pond block assumed that the 362m$^3$ excess permanent pool could provide a source for continual infiltration. It was assumed that the average annual groundwater level near the Carp River is approximately equal to the elevation at the limit of the wetland which is approximately 92.40m. Therefore, there is approximately 0.10m of head available for infiltration. If only the excess permanent pool volume is infiltrated in order to ensure that the MOE required permanent pool remains available in the pond, then the pond would need to be drained down and recharged 10 times to meet the annual infiltration requirement. Rainfall data obtained from the City of Ottawa for 2009-2011 was used to determine the average number of rainfall events greater than 6mm (as recommended by the MVC) which occur over one year. The analysis indicated an annual average of 47 rainfall events greater than 6mm. Additionally, the pond has a tributary area of approximately 29.9ha and would only require capture of 1.2mm of runoff over the entire tributary area in order to recharge the pond permanent pool. Therefore, the pond permanent pool could be recharged with sufficient frequency to provide the required volume for infiltration. City of Ottawa rainfall data is included in Appendix **E.4**.

In order to meet the infiltration requirement it is proposed to install two Big-O pipe systems below the permanent pool of the pond and outlet below the normal water level of the Carp River. They systems would draw water from the bottom of the pond but above the top of the sediment accumulation at an elevation of 91.20m and would run reverse sloped to a manhole located in the pond berm. A 200mm diameter big-O pipe would then drain to the Carp River and outlet below the permanent pool at an invert of 91.70mThe flow rate would be controlled by a 13mm opening at the manhole outlet. Details of the system are included on **Drawing POND-1**. The outflow from each system would range from 0.08L/s at normal Carp River water levels (92.40m) to 0.34L/s with low Carp River levels (defined as the top of the habitat pool riffle elevation of 91.37). Therefore, with the two systems in place the base flow rate to the Carp River would range from 0.16L/s to 0.68L/s. The proposed configuration would therefore provide the required base flow to the Carp River. The length and diameter of the Big-O pipe should also provide sufficient cooling of the baseflow to prevent thermal impacts.

#### 2.7.2.1 Infiltration Measures for Future Development Blocks

Infiltration for future development blocks will also need to meet the 70mm/year infiltration requirement. The 25% augmentation specified in the KWMSS will not be required as the roadway areas have been accommodated with the infiltration from the pond block. It is recommended that infiltration techniques applied to each development area should be designed on a site by site basis in consultation with the geotechnical engineer.

It is also recommended that, regardless of the infiltration technique, the depth should be kept fairly shallow and limited to the top strata of the soil column and must be above the groundwater level. Infiltration is intended to provide base flow to the Carp River and therefore need not be extended into the clay.

# 3.0 Detailed Facility Design Components

The following sections describe the stormwater management design approach and hydraulic results for the temporary stormwater management facility.

## 3.1 DESIGN APPROACH

The proposed SWMF is designed to meet the quality control requirements outlined above and to achieve all physical design criteria established for wet pond facilities by the Ministry of the Environment. These physical design criteria are provided in the MOE's Stormwater Management Design and Planning Manual (March 2003). As indicated in Section 1.2, the intent is to proceed with design and construction of the SWM facility in advance of the design and construction of the tributary developments.

From the preceding sections, the general design approach for the SWMF is as follows:

1. Provide MOE Enhanced water quality treatment, thereby establishing the permanent pool and extended detention volumes
2. Size inlet structure and forebay based on generated inflow and MOE guidelines
3. Compare peak flow rates to the Carp River
4. Consider environmental and operations and maintenance concerns in orientation and design of all pond components

### 3.1.1 Forebay Design

The purpose of the forebay is to act as the primary settling zone in the pond for the initial influx of coarser sediment and associated pollutants flushing off the sewershed. The forebay should be designed to provide sufficient cross-section and length to reduce velocities and promote settling, minimize resuspension of settled solids, minimize percentage of overall permanent pool, provide sufficient sediment storage for infrequent clean out (>10 years), and have adequate accessibility and bottom-treatment for maintenance operations.

The required forebay characteristics dictate a required forebay settling length and dispersion length of 33.4 m, and 36.2 m respectively, and the provided length is approximately 79.0 m. The resulting length to width ratio of the proposed configuration is approximately 3.6:1, also meeting MOE design recommendations. The provision of 2.3 m depth within the forebay provides for 0.7 m sediment accumulation prior to recommended cleanout while maintaining 1.6 m permanent pool depth, thereby minimizing the risk of scour and re-suspension. The designed sediment storage volume provided in the bottom 0.7 m corresponds to an estimated sediment removal frequency of approximately 10 years for the forebay, meeting MOE guidelines.

The forebay overflow berm to the main cell of the pond will be submerged 0.55 m below the permanent pool, and will be a 3 m wide clay berm.  A forebay drawdown chamber has been provided to allow for dewatering of the forebay via the main cell outlet structure for maintenance or winter operation purposes, as provided on **Drawing POND-2**.  See section 6.0 for details of the operations, maintenance, and monitoring component of the design.

Detailed forebay calculations are provided in **Appendix E.1**.

## 3.2 POND GRADING AND STORAGE DESIGN

Mild side slopes for safety (max 3:1) have been provided throughout the facility and along the forebay berms.  These slopes can be varied throughout to promote a less-engineered aesthetically pleasing design, where sufficient room is available.  Fencing is proposed along the top of the armour stone retaining walls for additional safety.

### 3.2.1 Coordination with Carp River Restoration

The Carp River Restoration EA proposes that the bank elevation between the Carp River and the proposed facility be limited to the 10 year elevation in the River to increase the available floodplain storage during rare events.  However, the KWMSS also specifies that quantity control is to be provided up to the 10 year storm. In order to achieve this, the spill elevation of the pond would be set to 93.55 m as per the KWMSS.

In the interim condition however, the pond is required to provided quantity control up to the 100year event and restrict 100 year post-development flows to the 100 year pre-development rate. As discussed in section 2.6.1 the pond spill elevation was set to 93.55m and 20m wide to restrict post-development flows to the pre-development rate.

Since the current SWMF plan remains outside of the Regulatory Floodplain, the bank between the SWMF and River would be higher than the future 10 year Carp River water level.  Following the Carp River Restoration works, the pond berm on the south side can be lowered to the 93.55m elevation of the spillway such that the pond berm would tie in with the Carp River bank. The requirement and details for any alterations to the bank would be dependent on the final detailed design of the CRRP to be completed by AECOM.

### 3.2.2 Water Quality Control

Maximum permanent water depths within the forebay and main cell of the facility are 2.3 m. As stated in Section 2.2.2 above, the required level of treatment for the SWMF is ‘normal’ or 70% as per the Carp River Watershed/Subwatershed Study (Robinson Consultants, December 2004). However, due to the lower pond bottom and increased permanent pool, sufficient volume is provided to meet an ‘enhanced’ level of treatment or 80% TSS removal. **Table 3-1** illustrates how the proposed end-of-pipe SWMF design provides this level of treatment.  A flow bypass structure is proposed within STM102 to divert flows greater than the 25mm event runoff directly to the pond main cell to minimize resuspension of settled solids in the pond forebay.

### 3.2.3 Water Quantity Control

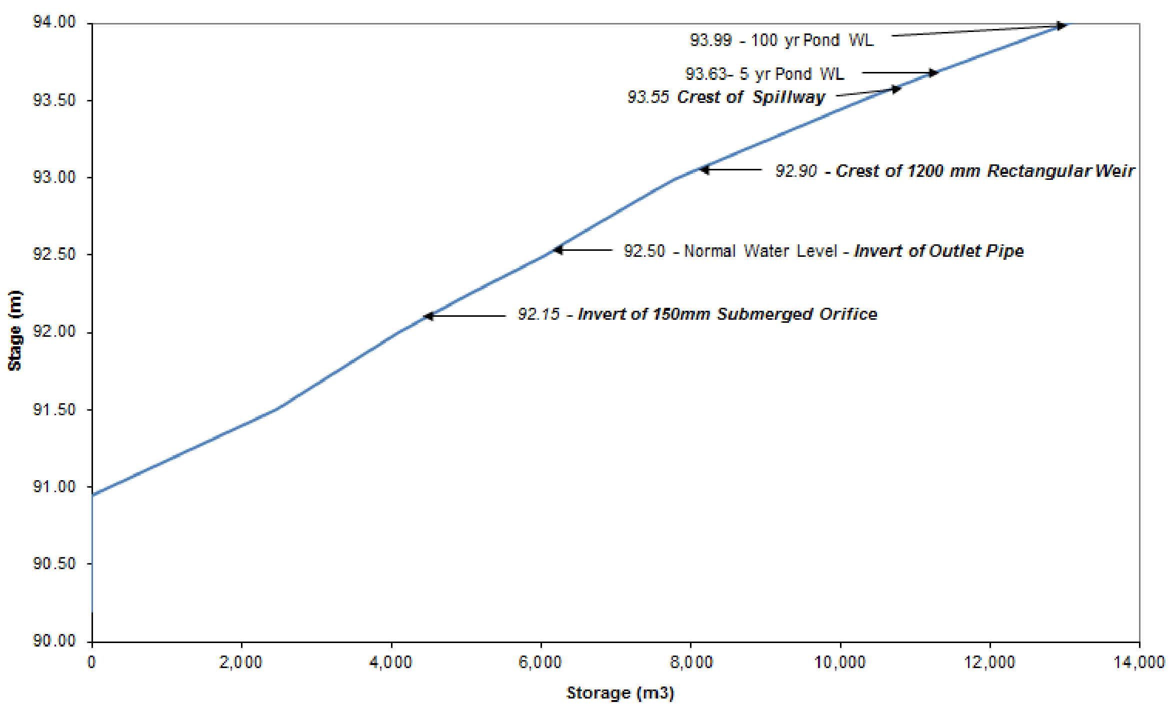
Peak inflow to the storm sewers is to be controlled locally to reduce the peak to either the 5 year or the 10 year peak flow depending on whether the area is non-arterial or arterial, respectively. Furthermore, runoff from the Rona property will be controlled to a combined release rate of 150 L/s.

Despite not being required to provide quantity control for the ultimate condition, the facility is required to provide quantity control up to the 100 year event in the interim condtition. The first 0.40 m of active storage is controlled by a 150 mm orifice. This orifice is submerged to an invert elevation of 92.15 to limit clogging by floatables. Since the invert elevation of the outlet sewer is 92.50 m, it is actually the outlet sewer that establishes the NWL. The orifice will drawdown the water quality volume over a period of 31.5 hours and provides flow augmentation for the fish habitat function of Carp River. The secondary and tertiary pond outlets occur via a 1.2 m x 1.0 m (WxH) concrete weir opening at invert 92.90 and a 20 m grassed spillway at invert 93.55 m. The first two outlets discharge to a common chamber which subsequently discharges to the Carp River via a 750 mm storm sewer and channel. The third outlet discharges directly to the Carp River. Combined outflow hydrographs from these conduits are presented in **Appendix E.1**.

### 3.2.4 Stage-Storage Relationship

The stage-storage relationship for the entire facility was established using the average end area method as presented in **Appendix E**. **Figure 3.1** summarizes the stage-storage curve.

**Figure 3.1: Stage-Storage Relationship for SWM Facility**



## 3.3 OUTLET DESIGN

The following subsections describe the pond outlet design. **Drawing POND-3** illustrates the outlet structure in plan, profile and sectional view.

The outlet will be located opposite the inlet, and will drain to the Carp River. A concrete outlet structure will house the required extended detention orifice, weir, and outlet pipes, including the associated maintenance infrastructure (i.e. stop log guides, etc.).

### 3.3.1 Extended Detention Control

The design of the required outlet structure incorporates a dual control configuration. Firstly, a 150 mm submerged orifice provides an approximate 31 hour extended detention for quality control. The entire extended detention volume is stored between 92.50 m and 92.90 m, as calculated below.

Required Storage:
Tributary Area = 29.9 ha
Extended Detention Storage = 40 $m^3$/ha
Storage = 29.9 x 40 = 1196 $m^3$

Provided Storage:
Pond area at NWL = 4214 $m^2$
Pond area at 92.90m = 4657 $m^2$
Provided Depth = 92.90 - 92.50 = 0.40 m
Provided Extended Detention Active Volume = 1,372 $m^3$

The reverse-slope pipe from the pond to the outlet structure will promote release of cooler water and prevent blockage of the outlet pipe from ice and floatables. Above an elevation of 93.0m, water will also enter the outlet chamber via a 2.0m opening covered with vertical grating (see **Drawing POND-3** for more detail).

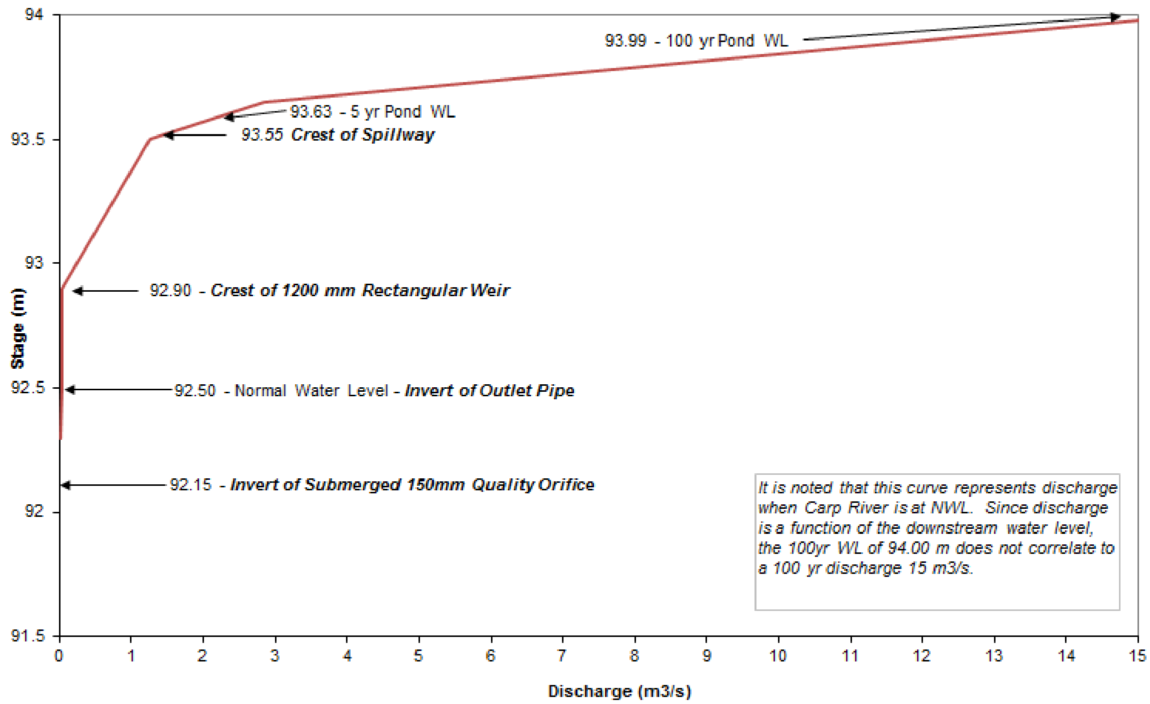
### 3.3.2 Grassed Spillway

The spillway location is separate from the outlet control structure, coinciding with the intersection of the future Carp River Corridor and the MVC Regulatory Floodplain. As per the KWMSS the spill elevation would be set at the 10 year pond water level resulting in a spill elevation of 93.55 m. In the interim condition the SWMF may also be required to control outflow rates to meet the 100-year predevelopment rate. Therefore the spill elevation is set to 93.55 m but is restricted to a width of 20m to restrict flow to 100-year pre-development rates. The spillway acts as a broad-crested weir, approximately 20 m wide. Should the stormwater management facility outlet structure clog or be subject to rare rainfall events (beyond the 10-year event), the spillway is designed to safely convey runoff to the Carp River. The sizing of the spillway is such that during a 100-year event, the total flow from the SWMF is restricted such that the total site flows meet the 100-year pre-development target rate (see **Section 2.6.1**).

### 3.3.3 Stage-Discharge Relationship

A stage-discharge relationship was estimated using standard orifice and weir equations as outlined in **Appendix B**. The resulting stage-discharge relationship is presented in **Figure 3.2**.

**Figure 3.2: Stage-Discharge Relationship for SWM Facility**




## 3.4 POND PERFORMANCE

As **Table 3-1** indicates, the water quality objectives of the SWM basin are met by providing extended detention of 24-48 hours, and exceeding the MOE recommended water quality volumes. A comparison of pond parameters to MOE Table 4.6 criteria is included in **Appendix E.2.**

**Table 3-1: SWM Facility Operational Characteristics**

| SWM Basin Parameters | Basin Value |
|---|---|
| Total Contributing Area [Including Pond Area] | 29.90 ha |
| Imperviousness of Contributing Area [of Sewershed Area] | 73.3 % |
| Unit Area Storage Volume Requirements as per SWMPD Manual | 230.5 $m^3$/ha |
| Required Total Water Quality Volume | 6,892$m^3$ |
| Wet Pond Bottom Elevation | 90.20 m |
| Required Permanent Pool Volume | 5,696 $m^3$ |
| Permanent Pool Volume Provided | 6,059 $m^3$ |
| Permanent Pool Elevation (Outlet Invert) | 92.50 m |
| Permanent Pool Surface Area | 4,214 $m^2$ |
| Required Extended Detention Volume | 1,196 $m^3$ |
| Extended Detention Volume Provided | 1,371 $m^3$ |
| Peak Release Rate for Extended Detention | 0.030 $m^3$/s |
| Extended Detention Drawdown Time | 31.5 hours |
| Extended Detention Elevation (Weir Crest) | 92.90 m |
| 5 Year Storm Maximum Ponding Level (SCS) | 93.63 m[A] |
| 5 Year Storm Peak Pond Release (SCS) | 0.84 $m^3$/s[B] |
| 25 Year Storm Maximum Ponding Level (SCS) | 93.71 m[A] |
| 25 Year Peak Pond Release (SCS) | 2.78 $m^3$/s[B] |
| 100 Year Storm Maximum Ponding Level (SCS) | 93.99 m[A] |
| 100 Year Storm Peak Pond Release (SCS) | 2.90 $m^3$/s[B] |
| Top of Berm (minimum grade of surrounding properties) | 95.00 m |
| **Forebay Parameters** | |
| Forebay Bottom Elevation | 90.20 m |
| Sediment Accumulation Depth | 0.70 m |
| Forebay Depth from Permanent Pool | 2.3 m |
| Required Forebay Length | 79 m |
| Actual Forebay Length | 79 m |
| Clean Out Frequency | 9 yrs |
| **Outlet Parameters** | |
| Quality Orifice Size (Orifice #1) | 150 mm |
| Quality Orifice Invert (Orifice #1) [submerged] | 92.15 m |
| Quality Weir Crest Length (Weir 1#) | 1200 mm |
| Quality Weir Crest Elevation (Weir #1) | 92.90 m |
| Emergency Weir Crest Length (Weir#2) | 20 m |
| Emergency Weir Crest Elevation (Weir#2) | 93.55 m |

A. A stage-history boundary condition obtained from the Carp River HEC-RAS model was used for backwater conditions. Note that values for the free flow conditions are different.
B. Peak flow from model, combining outflow hydrographs from conduits C41 (outflow pipe from outlet structure) and C38 (overflow spillway).

## 3.5 OTHER CONSIDERATIONS

Additional key design notes include the following:

- A 3 m wide access road with 1.0m shoulders has been provided for ease of inspection and maintenance of the inlet, forebay and main cell. The access road will have an engineered base consisting of granular 'A' and granular 'B' for durability and strength, while the surface will be asphaltic concrete for erosion protection. The route has been designed with a minimum slope to facilitate maintenance equipment manoeuvrability.
- A sediment storage area has been provided to the north of the SWM facility. The storage area has been sized to accommodate the estimated 608$m^3$ of sediment build-up in the forebay between cleanings (see pond design calculations in **Appendix E**). The area is set to accommodate a maximum sediment depth of 1.0m with 3:1 side slopes. An earth berm is proposed around the sediment storage area to prevent transport of sediment outside of the storage area.

# 4.0 CRITERIA AND CONSTRAINTS FOR FUTURE DEVELOPMENT

The following section discusses the criteria and constraints to be utilized during detailed design of the developments within the 26.6 ha catchment area tributary to the SWM facility.

## 4.1 STORMWATER MANAGEMENT DESIGN CRITERIA

The following design guidelines were established through review of the background documentation, supplemented with current design practices outlined by the City of Ottawa Sewer Design Guidelines (2004) and Ministry of the Environment Stormwater Management and Planning and Design Manual (2003) guidelines:

- Design using the dual drainage principle
- Maximum 100 year water depth of 0.30 m in road sags, including overflow spill depth
- Average sag storage of 200 $m^3$/ha to be provided in commercial properties with the exception of Rona properties which must provide storage sufficient to completely contain 100 year flow to 150 L/s
- Average sag storage of 50 $m^3$/ha to be provided along roadways; 40 $m^3$/ha on arterials and Transitways
- 100 year hydraulic grade line (HGL) to be a minimum 0.30 m below building foundation footing
- Design inlets within developments and along local roadways to capture the 5 year peak flow unless otherwise noted (e.g. for Rona properties and arterials).
- Design storm sewers along local roadways to convey 5 year peak flow under free-flow conditions using 2004 City of Ottawa I-D-F parameters
- Design inlets and storm sewers along arterial roadways to capture and convey 10 year peak flow under free-flow conditions using 2004 City of Ottawa I-D-F parameters
- Account for tributary external drainage through the system
- Provide adequate emergency overflow conveyance off-site
- Provide quality control for at least an MOE ‘normal’ level of protection
- Design and submit a detailed Erosion Control Plan, as outlined in Section 6.2

# 5.0 Carp River Floodplain Compensation

The Carp River Restoration Plan completed by TSH in 2009 included HEC-RAS modeling to determine the new floodplain limit of the Carp River. Model cross sections for the Carp River along the Taggart-Loblaws river frontage indicate that the property boundary of the Taggart-Loblaws lands would need to be offset by 3.8m to accommodate the new floodplain limit. It was agreed that alternatively the property lines could remain unchanged and the volume of floodplain that would be contained within the 3.8m offset could be compensated for elsewhere on the property. This alternative was preferred by Taggart and as such the volume of floodplain required to be compensated was calculated from the HEC-RAS cross sections shown on **Drawing DET-1** and was determined to be approximately $260m^3$. Note that this compensation volume is for the Taggart-Loblaws lands from the north side of Highway 417 to the south side of the Campeau Drive extension but does not include the area lost from the construction of the Transitway.

Floodplain compensation is proposed in three areas of the Taggart-Loblaws lands; on the north and south sides of the future transitway, and south of the proposed SWM pond, as shown on **Drawing DET-1**. The cross-sections for these areas are shown on **Drawings DET-2** and **DET-3** and were used to calculate the available floodplain volume at a flood elevation of 93.58m as per the modeled river cross section near the compensation areas (RS = 41725.5). The volumes were calculated for the cut area within the existing property line, therefore although the drawings show cuts beyond the property line this has not been included in the additional floodplain volume totals. The cut area within the Carp River corridor is proposed only so that the grades may be lowered to allow water to flow to and drain from the compensation areas.

Floodplain compensation calculations have been reviewed and approved by Mississippi Valley Conservation, and a permit to remove fill within the regulation limit of the Carp River was issued on December 10, 2012.

Note that the compensation calculations do not include any compensation for the Rona property. Any compensation requirements for the Rona site shall be determined at the detailed site design stage.

# 6.0 Operations and Maintenance

As with any SWM facility, maintenance will be required to ensure the operational efficiency and functionality of the facility. It is noted that until such a time as the ownership is transferred to the City of Ottawa, the developer will be responsible for any maintenance works. The preparation of an Operations, Maintenance and Monitoring Manual for the SWMF will be required at a later date and prepared under separate cover.

## 6.1 MAINTENANCE PROGRAM

The following summarizes the key components to be considered for the maintenance program:

- **Inventory of Stormwater System Components** (e.g. conveyance locations, elevations, outfalls, contributing drainage area, receiving watercourse, control structure components and specifications, material types, vegetative species and other pertinent information)
- **Periodic and Scheduled Inspections** (performed on a regular basis as determined in consultation with the city, MOE and conservation authority)
- **Maintenance Scheduling and Performance** (e.g. vegetation/landscaping maintenance, trash and debris removal, dewatering, sediment removal and disposal, pond and stream bank stabilization, inlet/outlet structure repairs, equipment testing/troubleshooting etc.)
- **Documentation and Reporting** (e.g. frequent inspection reports to the municipality and conservation authority during construction)

### 6.1.1 Storm Sewer Cleanout

Increased maintenance frequency is anticipated for the proposed storm sewers due to the extent of standing water. In order to allow for proper cleaning of the sewers, stop logs are proposed within the outlet structure and the outlet headwall. As required by the City of Ottawa, the design also includes a method of isolating the sewers from the pond. Gains installed to hold the grating on the inlet headwall can be used to frame in stop-logs and thereby allow standing water to be pumped from the sewers. In this scenario accumulated sediment would be removed via a vacuum truck, rather than washed into the forebay.

### 6.1.2 Monitoring

Monitoring of the proposed SWMF will be conducted to ensure proper hydraulic and water quality performance of the facility as designed. The monitoring program will be as specified by the City of Ottawa for the Kanata West Area. The standardized SWM monitoring program was obtained from the City of Ottawa's web publication of the Kanata West Overall Monitoring Plan which outlines the following monitoring plan:

1. The stormwater monitoring program shall be implemented for a minimum of two years and shall continue to be implemented until such time that the MOE's Ottawa District Office provides written notice that the program may be discontinued;

2. Monitoring will be provided during the period between May 1st and October 31st for water quality and water level as well as general performance. Water quality would be established for both baseflow and rainfall event conditions;
3. Routine operational inspections would be conducted over the life of the facility, to confirm: general site conditions (erosion/landscaping); ensure monitoring equipment is functioning appropriately; and that orifices and weirs are not clogged with debris. Routine cleaning of any blockages would be done during the inspections, if required;
4. Pond water levels would be monitored using continuous recording water level loggers to determine drawdown characteristics of the facility (typically 24 - 48 hours). No flow monitoring is proposed in this program;
5. Water quality samples would consist of composite samples (collected utilizing automated equipment or by grab sampling) taken at the facility inlet and outlet during/after specified rainfall events. The routine inspection would typically be conducted coincident with sample collection activities;
6. Water quality parameters would include the Total Suspended Solids (TSS) and Total Phosphorous (TP) and Water Temperature (spot measurement);
7. The sample collection, preservation, handling and analytical methods including detection limits, would be documented. Sample analysis to be conducted per Standard Methods or approved equivalent at a CALA certified laboratory. Field and lab QA/QC procedures would reflect standard sampling protocols (i.e. samples delivered within 12 hours to lab following collection);
8. Provide monitoring (sampling, water levels) during the following events over a two year period to ensure that the facility is performing as designed:
    - Two small rainfall events ( less than 7mm)
    - Two medium rainfall events (7-15mm)
    - Three large rainfall events (greater than 15mm)
9. The requirement is for a minimum of the 7 noted events of the size specified. Monitoring activities would typically capture a number of other events during the process, as the amount of rainfall from an event is not predictable in advance. All events that are monitored/sampled, are to be reported in the annual reports. The sampling of all the events within a one year period, although possible, is not acceptable as the intent is a two year program;
10. Rainfall measurements should be obtained from the nearest available City of Ottawa rain gauge(s) to the facility;
11. A topographic/bathymetric survey should be undertaken after 3-5 years of operation to determine sediment deposition and sediment deposition rates. This would allow for more concise forecast of forebay cleanout frequency; either confirming or revising the frequency in final SWM report.

http://ottawa.ca/en/env_water/tlg/alw/brs/subwatershed/completed/carp/restoration/kanatawest/index.html

### 6.1.3 Sediment Removal and Storage

A SWMF access road has been provided along the south and east sides of the proposed SWMF. The access road will permit access to all proposed structures within the SWMF. The pond forebay area has been designed to be long and narrow such that sediment removal activities can be conducted with access from only one side of the forebay. The maximum distance to reach the bottom of the west side of the forebay from the proposed access road is approximately 25 m. Stop log access hatches and guides are provided in the outlet and drawdown structures to facilitate sediment removal and maintenance activities.

Sediment removal is estimated to be required at 9 year intervals or at a sediment accumulation depth of 0.75m. Sediment accumulation within the forebay is variable and dependent on many factors including the condition of the tributary lands and the frequency of rainfall events therefore sediment removal may be required at shorter or longer intervals than 9 years. Monitoring of sediment depth should be conducted regularly and a bathymetric survey should be completed at regular intervals to confirm sediment accumulation rates. Sediment accumulation within the main cell will occur at lower rates and therefore require less frequent clean-outs than the forebay cell.

A temporary sediment storage/drying area has been provided near the pond inlet structure to facilitate sediment removal and provide short-term storage for drying of sediment prior to disposal. Any sediment removed from the site to be disposed of off-site must be analysed for Ontario Regulation 347 criteria. Analytical results must meet inert fill requirements if the sediment is to be used for land application, and meet non-hazardous requirements as per the TCLP leachate test in O.Reg. 347 if it is to be disposed of in a municipal landfill.

## 6.2 EROSION AND SEDIMENT CONTROL

In order to control erosion and migration of sediment-laden runoff off site during construction, an erosion and sediment control plan will be required for the SWMF, following the general principles outlined in **Drawing DS-1**. Therefore, an appropriate inspection and maintenance program is necessary that will be prepared by the contractor, and will consider phasing of construction and the following goals:

- Protection from migration of sediment-laden runoff entering Carp River, or its floodplain
- Construction of the outlet works must completed with the approval of the Mississippi Valley Conservation
- No instream works, and minimization of disturbance to floodplain vegetation
- No work in or around the river between March and July of any given year
- Immediate stabilization of exposed soil and/or stockpiles
- Frequent inspection of all controls during construction of the pond and after significant rainfall events (greater than 13 mm) for sediment accumulation and erosion
- Immediate repair of all noticeable erosion, with investigation into the cause so implementation of mitigation measures to prevent recurrence will be more successful
- Maintenance of the erosion control measures in good repair during pond construction
- Preparation of weekly inspection reports during active pond construction outlining the condition of erosion control works, their overall performance, and any actions such as repairs, replacement or modification

The SWM Facility will be constructed in advance of completion of the tributary drainage area, and therefore will not operate as designed until complete build-out. In addition, there is increased potential for high TSS inflow during subdivision construction therefore an increased clean-out frequency during build-out is likely. Individual developments will be responsible for their own sediment and erosion controls during construction, as not to impact the trunk infrastructure.

Permanent erosion control measures are required for the pond spillway due to high flow velocities (up to 4.5m/s) during high intensity storm events. It is proposed that the spillway be lined with either rip-rap with d50 of 450mm or Terrafix P42 synthetic fibre mat and seeded with fescue per the manufacturer's specifications.

## 6.3 GROUNDWATER GRADIENTS AND CONSTRUCTION REQUIREMENTS

At the request of the Ministry of Environment, an analysis was completed by Paterson Group to assess the groundwater flow gradient in the Pond 3 area. Groundwater gradients in the area surrounding the proposed SWM facility were assessed by Paterson Group based on available borehole data. Paterson prepared a memo *Taggart Stormwater Management Facility (KW Pond 3) Consultation on Infiltration Potential and Pond Construction* dated May 9, 2013, which summarizes the results of the investigation and recommendations (see Appendix F). Results of the investigation indicated that an overburden groundwater gradient exists in the southerly direction towards the SWM facility as well as from the facility towards the river, dependent upon groundwater levels. Paterson has provided details for sidewall stabilization/protection measures to ensure the long term stability of the facility which are summarized in the attached Paterson memo and illustrated in **Drawing Pond-1**.

With regards to construction of the SWM facility Paterson recommends that measures are implemented to drawdown the groundwater around the excavation area during construction to avoid seepage into the excavation. Although available test pit data suggests that the soil in the pond area is stiff clay, Paterson indicated that weaker clays are known to be present in the surrounding area and it is strongly recommended that the excavation is inspected during construction to assess the pond bottom and sidewalls for the potential of basal heave. Recommendations for any additional base or sidewall treatment against basal heave are to be provided based on site inspection by the geotechnical engineer. Please refer to the attached memo prepared by Paterson for further details. In keeping with Paterson's recommendation the following note has been added to all engineering drawings:

"The minimum permissible factor of safety for basal heave for the firm to stiff clay is 2.0. The sidewalls and base of the stormwater management pond should be examined by a geotechnical engineer during excavation in order to accurately assess the potential for basal heave and make the necessary recommendations to remedy minor basal instability along the base of the excavation, should it be encountered."

# 7.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this report, the following conclusions can be drawn:

- Water quality control will be provided by the Taggart-Loblaws Subdivision Stormwater Management Facility for an 'Enhanced' level of protection
- The SWMF can be constructed outside of both the Carp River Corridor and Regulatory Floodplain
- Post-development infiltration targets for the pond block and roadways can be achieved through the use of subsurface infiltration trenches used to convey base flow to the Carp River.
- Post-development flows meet the target requirements of peak flow and total runoff volume as established in the Carp River Restoration Plan.

Based on the findings of this report, the following recommendations are advised:

- Future developments will ensure that a clearance of at least 0.30 m is achieved between building footings and the 100 year hydraulic grade line
- 100 year ponding depths have been maintained at a maximum 0.30 m in road sags.
- Future developments shall follow those criteria provided in Section 4.0 above

All of which is respectfully submitted;

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