Servicing and Stormwater Management Brief – 5731 Hazeldean Road (Phase 2)

Project # 160401195

Prepared for:
Wellings of Stittsville Inc. and Extendicare (Canada) Inc.

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Sign-off Sheet

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Table of Contents

1.0 INTRODUCTION 1.4

2.0 BACKGROUND 2.1

3.0 WATER SUPPLY SERVICING 3.1
3.1 BACKGROUND 3.1
3.2 WATER DEMANDS 3.1
3.3 HYDRAULIC MODEL RESULTS 3.2
3.4 SUMMARY OF FINDINGS 3.5

4.0 WASTEWATER SERVICING 4.1
4.1 BACKGROUND 4.1
4.2 DESIGN CRITERIA 4.1
4.3 PROPOSED SERVICING 4.1

5.0 STORMWATER MANAGEMENT 5.1
5.1 OBJECTIVES 5.1
5.2 SWM CRITERIA AND CONSTRAINTS 5.1
5.2.1 Pre-Development Conditions 5.2
5.3 STORMWATER MANAGEMENT DESIGN 5.3
5.3.1 Rationale for Design and Servicing Deviations 5.3
5.3.2 Design Methodology 5.3
5.3.3 Modeling Rationale 5.4
5.3.4 Input Parameters 5.6
5.3.5 Model Results 5.9
5.3.6 Water Quality Control 5.12
5.3.7 Infiltration Targets 5.12
5.3.8 Thermal Controls 5.15
5.3.9 Monitoring Plan 5.15
5.3.10 Contingency Plan 5.16
5.4 SUMMARY OF FINDINGS 5.17

6.0 POOLE CREEK EROSION ANALYSIS 6.1
6.1 2017 BACKGROUND 6.1
6.2 INTRODUCTION 6.1
6.3 EROSION ANALYSIS METHODOLOGY 6.1
6.4 EROSION ANALYSIS RESULTS 6.2

7.0 GRADING AND DRAINAGE 7.4

8.0 UTILITIES 8.1

9.0 APPROVALS 9.1

10.0 EROSION CONTROL DURING CONSTRUCTION 10.1

11.0 GEOTECHNICAL INVESTIGATION 11.1

12.0 CONCLUSIONS 12.1
12.1 WATER SERVICING 12.1
12.2 SANITARY SERVICING 12.1
12.3 STORMWATER SERVICING 12.1
12.4 GRADING 12.1
12.5 UTILITIES 12.1
12.6 APPROVALS/PERMITS 12.2

LIST OF TABLES

Table 1: Hydraulic Analysis Results (AVDY) 3.3
Table 2: Hydraulic Analysis Results (PKHR) 3.4
Table 3: Hydraulic Analysis Results (MXDY + FF) 3.5
Table 4: General Subcatchment Parameters 5.6
Table 5: Subcatchment Parameters 5.7
Table 6: Storage Node Parameters 5.8
Table 7: Outlet/Orifice Parameters 5.9
Table 8: Site Peak Discharge Rates 5.10
Table 9: Modeled Hydraulic Grade Line Results 5.11
Table 10: Maximum Surface Water Depths 5.11
Table 11: Summary of Infiltration from LID Features 5.13
Table 12: Expected Seasonal Variation of Groundwater Levels 5.13
Table 13: Estimated Maximum Groundwater Mounding below Infiltration Gallery 5.14
Table 14: Site Peak Discharge Rates/Targets 5.17
Table 15: Summary of Critical Velocities and Threshold Exceedances in Poole Creek 6.2
Table 16: Pavement Structure – Car Only Parking Areas 11.2
Table 17: Pavement Structure – Access Lanes and Heavy Truck Parking Areas 11.2

LIST OF FIGURES

Figure 1 Location Plan 1.5
Figure 2: Hydraulic Model Overview 3.2
Figure 3: Schematic Representing Model Object Roles 5.5

LIST OF APPENDICES

APPENDIX A WATER SUPPLY SERVICING ... A.1
A.1 Domestic Water Demand Estimate ... A.1
A.2 Fire Flow Requirements Per FUS ... A.2
A.3 Hydraulic Analysis Results ... A.3

APPENDIX B WASTEWATER SERVICING ... B.1
B.1 Sanitary Sewer Design ... B.1

APPENDIX C STORMWATER MANAGEMENT ... C.1
C.1 Storm Sewer Design Sheet And Roof Storage Calculations ... C.1
C.2 SAMPLE PCSWMM Model Input (12HR 100YR SCS) ... C.2
C.3 SAMPLE PCSWMM Model Output (12HR 100yr SCS) ... C.3
C.4 Oil/Grit Separator Sizing Calculations ... C.4

APPENDIX D POOLE CREEK EROSION ANALYSIS ... D.1

APPENDIX E LID DESIGN AND WATER BALANCE ... E.1
E.1 LID design Report ... E.1
E.2 Groundwater Mounding Calculations ... E.2

APPENDIX F GEOTECHNICAL INVESTIGATION ... F.1

APPENDIX G PRODUCT SPECIFICATIONS ... G.1

APPENDIX H CORRESPONDENCE ... H.1

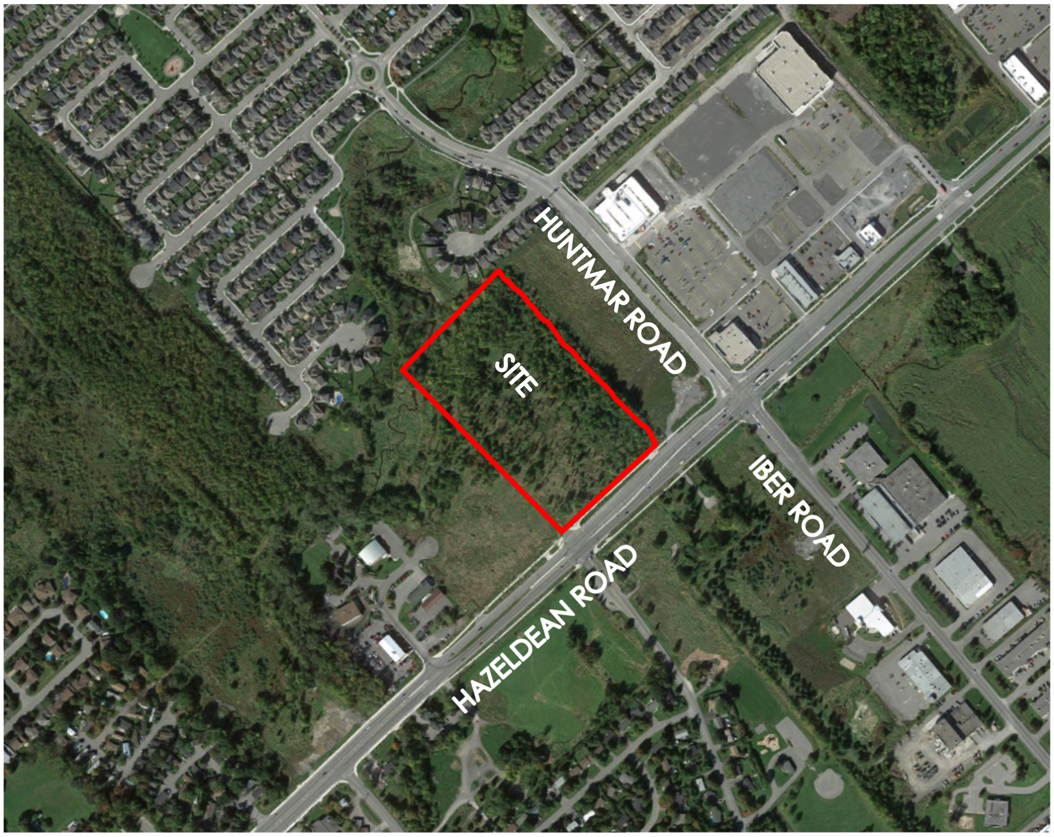
1.0 INTRODUCTION

Stantec Consulting Ltd. was originally commissioned by Wellings of Stittsville Inc. and Extendicare (Canada) Inc. to prepare a servicing study in support of the development at 5731 Hazeldean Road located within the City of Ottawa's Kanata West master plan area. The development site plan application was subsequently approved by the City of Ottawa, and proceeded to construction of the first phase of development comprising the southerly Wellings building (a 5-storey, 185 unit residential facility), a portion of the underground parking lot originally proposed across the majority of the site, as well as associated storm and sanitary sewer and watermain servicing.

Extendicare (Canada) Inc. is seeking to revise the original site plan application in consideration of a similar 256 unit, four storey long term care building within a smaller footprint to permit aboveground parking in lieu of the connected underground parking lot, and removal of all but one of the ancillary commercial/institutional buildings originally proposed to front Hazeldean Road.

The subject property is located northwest of the intersection of Huntmar Road and Hazeldean Road. The property location is indicated in **Figure 1**. The proposed mixed use residential, commercial and institutional development comprises approximately 2.88ha of land, and proposes construction of a 256 unit, four storey long term care facility, an existing 185 unit, 5-storey residential building, as well as a proposed two-storey commercial building, with associated parking and access areas. The intent of this report is to provide a servicing scenario for the site that is free of conflicts, provides on-site servicing in accordance with City of Ottawa design guidelines, and utilizes the existing local infrastructure in accordance with the guidelines outlined in background documents, and as per consultation with City of Ottawa.

Figure 1 Location Plan



2.0 BACKGROUND

Documents referenced in preparing the site design for the 5731 Hazeldean Road Development include:

- Kanata West Master Servicing Study, Stantec Consulting Ltd., Cumming Cockburn Limited / IBI, October 1, 2014.
- Servicing and Stormwater Management Brief – 5731 Hazeldean Road, Stantec Consulting Ltd., March 22, 2017.
- Carp River PCSWMM Model Documentation Draft Report, City of Ottawa, March 2016.
- Geotechnical Investigation, Proposed Mixed Use Development 5731 Hazeldean Road, Ottawa, Ontario, Paterson Group, February 5, 2016.
- Tree Conservation Report – 5731 Hazeldean Road, IFS Associates, March 11, 2016.
- City of Ottawa Sewer Design Guidelines, City of Ottawa, October 2012.
- City of Ottawa Design Guidelines – Water Distribution, City of Ottawa, July 2010.

3.0 WATER SUPPLY SERVICING

3.1 BACKGROUND

The proposed development comprises one commercial building, an existing residential apartment building and a long term care facility (institutional), complete with associated infrastructure and parking. The site is located west of Huntmar Drive, north of Hazeldean Road, and south of Poole Creek, and lies within the City's 3W pressure zone. The proposed buildings will be serviced via an existing 250mm diameter connection to the 762mm feedermain within the Hazeldean Road ROW at the southwest quadrant of the site.

3.2 WATER DEMANDS

Water demands for the development were estimated using the Ministry of Environment's Design Guidelines for Drinking Water Systems (2008). A daily rate of 5 L/m2 of commercial building space has been applied for commercial building space, whereas the long term care facility and apartment dwelling demand was estimated at 350L/person/day with an estimated population of 1.0 and 1.8 persons/unit (average apartment density) respectively. It is predicted that commercial facilities will be operated 12 hours per day. See **Appendix A.1** for detailed domestic water demand estimates.

The average day demand (AVDY) for the entire site was determined to be 2.53 L/s. The maximum daily demand (MXDY) is 1.5 times the AVDY for commercial property demand and 2.5 times the AVDY for residential demand, which equates to 6.18 L/s. The peak hour demand (PKHR) is 1.8 times the MXDY for commercial property and 2.2 times the MXDY for residential properties, totaling 13.51 L/s.

Individual calculation sheets based on FUS guidelines were prepared based on proposed site plan information for all on-site buildings. Calculations assume that the commercial buildings are to be of non-combustible construction and are to maintain at minimum 2-hour firewalls separating buildings from underground parking areas. The long term care facility is also intended to be of non-combustible construction, and the existing 5-storey apartment is of ordinary (combustible) construction with 2-hour fire walls separating the unit into areas of no more than 9,000m^2 based on building code requirements. Based on calculations per the FUS Guidelines (**Appendix A.2**), the maximum required fire flows for this development are 267 L/s occurring at the proposed five-storey apartment building. See building architectural plans (by others) for locations of 2-hour fire walls within the proposed buildings.

3.3 HYDRAULIC MODEL RESULTS

A hydraulic model of the off-site water supply system was created in the H20MAP Water software based on boundary conditions as provided by City of Ottawa staff on March 8, 2016 (see **Appendix A.3**). The model was tested under three different domestic demand conditions: average day (AVDY), peak hour (PKHR) and maximum day plus fire flow (MXDY + FF). Spreadsheets within **Appendix A.3** present the model output results for each demand analysis.

Figure 2: Hydraulic Model Overview

Typical operating pressures are anticipated to range between 572 kPa (82.9 psi) and 558 kPa (80.9 psi) based on the local ground elevations and pipe hydraulic conditions. The range of anticipated operating pressures exceeds the recommended pressure range of 275 kPa to 552 kPa (40 to 80 psi), as recommended by the City of Ottawa's Water Distribution Design Guidelines. Pressure reducing valves are recommended for all proposed buildings within the development in order to satisfy the design guidelines.

Table 1: Hydraulic Analysis Results (AVDY)

Node ID	Demand (L/s)	Elevation (m)	Pressure (psi)
1	0.14	104.74	80.9
10	1.35	103.40	82.8
11	0	103.82	82.2
12	0	103.74	82.3
13	0	103.55	82.6
14	1.04	104.53	81.2
2	0	104.66	81.0
3	0	104.58	81.1
4	0	104.08	81.8
5	0	104.12	81.7
6	0	104.22	81.6
7	0	103.35	82.8
8	0	103.33	82.9
9	0	103.28	82.9

Peak hour operating pressures are anticipated to range between 507 kPa (73.5 psi) and 520 kPa (75.4 psi) based on the local ground elevations and pipe hydraulic conditions. The resultant pressures are also within the allowable pressure range of 275 kPa to 552 kPa (40 to 80 psi), even under consideration of an estimated loss of 5 psi per storey within the multi-storey buildings. Requirements for internal jet pumps will ultimately be required to be confirmed by the building mechanical engineer.

Table 2: Hydraulic Analysis Results (PKHR)

Node ID	Demand (L/s)	Elevation (m)	Pressure (psi)
1	0.39	104.74	73.5
10	7.42	103.40	75.2
11	0	103.82	74.6
12	0	103.74	74.7
13	0	103.55	75.0
14	5.70	104.53	73.6
2	0	104.66	73.6
3	0	104.58	73.7
4	0	104.08	74.4
5	0	104.12	74.3
6	0	104.22	74.1
7	0	103.35	75.3
8	0	103.33	75.3
9	0	103.28	75.4

The City of Ottawa's design guidelines for water distribution systems require a minimum pressure of 140 kPa (20 psi) to be maintained at all points in the distribution system under a condition of maximum day and fire flow demand. A fire flow analysis was carried out using the hydraulic model to determine the anticipated amount of flow that could be provided at each of the nodes in the proposed development under maximum day demands while still maintaining a residual pressure of 140 kPa (20 psi). This was accomplished using a steady-state maximum day demand scenario along with the automated fire flow simulation feature of the software. The boundary condition at a fire flow of 267L/s was applied for conservatism.

The fire flow results presented in **Appendix A.3** show that the required fire flows are available at all locations within the proposed development. All nodes resulted with residual pressures at or in excess of 20 psi.

Table 3: Hydraulic Analysis Results (MXDY + FF)

Node ID	Fire-Flow Demand (L/s)	Residual Pressure (psi)	Available Flow (L/s)
11	100	44.8	257.5
14	100	41.9	221.0
2	267	43.5	645.6
6	267	31.6	379.4
9	267	18.3	285.0

3.4 SUMMARY OF FINDINGS

The proposed development is located in an area of the City's water distribution system that has sufficient capacity to provide both the required domestic and emergency fire flows. Based on computer modeling results by others, fire flows are available for this development per fire flow demands per the City of Ottawa water distribution guidelines. Pressure reducing valves are recommended for all proposed buildings within the development in order to satisfy the design guidelines for average day operating pressures.

4.0 WASTEWATER SERVICING

4.1 BACKGROUND

The site will be serviced via an existing 675mm dia. sanitary sewer located within the Hazeldean Road ROW south of the site and west of the intersection of Hazeldean Road and Huntmar Drive (see **Drawing SSP-1**). Discharge from the site has been accounted for in the Kanata West Master Servicing Report prepared by Cumming Cockburn Limited/IBI Group and Stantec Consulting Ltd for the overall area.

4.2 DESIGN CRITERIA

As outlined in the City of Ottawa Sewer Design Guidelines and the MECP's Design Guidelines for Sewage Works, the following criteria were used to calculate estimated wastewater flow rates and to size the sanitary sewers:

- Minimum Velocity – 0.6 m/s (0.8 m/s for upstream sections)
- Maximum Velocity – 3.0 m/s
- Manning roughness coefficient for all smooth wall pipes – 0.013
- Minimum size – 250mm dia. for commercial areas
- Average Wastewater Generation (Commercial) – 5L/day/m2 of building space
- Average Wastewater Generation (Residential) – 350L/day/person
- Peak Factor (Commercial) – 1.5 (Max Day Demand per MOE Design Guidelines for Drinking Water Systems)
- Peak Factor (Residential) – 3.94 (Harmon's)
- Extraneous Flow Allowance – 0.28 l/s/ha (conservative value)
- Manhole Spacing – 120 m
- Minimum Cover – 2.5m

4.3 PROPOSED SERVICING

The proposed site will be serviced by a gravity sewer which will direct the wastewater flows (approx. 10.3 L/s with allowance for infiltration) to the existing 675mm dia. Hazeldean Road sanitary sewer. A backflow preventer will be required for the on-site building in the event of surcharge of the sanitary sewer, and will be coordinated with building mechanical engineers. The proposed drainage pattern is in accordance with the Kanata West Master Servicing Report for Hazeldean Road and is detailed on **Drawing SAN-1.** Sanitary flows will be discharging to the downstream Kanata West Pump Station. A Sanitary sewer design sheet is included in **Appendix B.1**.

5.0 STORMWATER MANAGEMENT

5.1 OBJECTIVES

The objective of this stormwater management plan is to determine the measures necessary to control the quantity of stormwater released from the proposed development to established criteria, and to provide sufficient detail for approval and construction. The proposed development will discharge treated and controlled stormwater runoff to Poole Creek using the previously approved & constructed outfall for the site.

5.2 SWM CRITERIA AND CONSTRAINTS

Criteria for the development were initially established by combining current design practices outlined by the City of Ottawa Design Guidelines (2012), Ministry of Environment, Conservation, and Parks (MECP) and Mississippi Valley Conservation Authority (MVCA). The following summarizes the criteria, with the source of each criterion indicated in italics:

General

- Use of the dual drainage principle (City of Ottawa)
- Wherever feasible and practical, site-level measures should be used to reduce and control the volume and rate of runoff (City of Ottawa)
- Site-level infiltration measures to be implemented to meet infiltration criteria of minimum 50 mm/yr (MVCA)
- Assess impact of 100-year event outlined in the City of Ottawa Sewer Design Guidelines, and climate change scenarios with a 20% increase of rainfall intensity, on major & minor drainage system (City of Ottawa)
- Quality control to be provided for 80% TSS removal (MVCA, MOECC)
- Site discharge to be controlled to pre-development rates (MVCA, City of Ottawa)
- Site design to mitigate erosion impacts on Poole Creek (City of Ottawa)

Storm Sewer & Inlet Controls

- Size storm sewers to convey the 5-year storm event under free-flow conditions using City of Ottawa I-D-F parameters (City of Ottawa) with the exception of the outlet sewer from the proposed underground storage facility.
- Minimum sewer inlet capture rates to be set such that no ponding occurs at the end of the 5-year event (City of Ottawa)
- Hydraulic Grade Line (HGL) analysis to be conducted using the 100-year 12 hour SCS storm distribution (City of Ottawa).
- 100-year Storm HGL to be a minimum of 0.30 m below building foundation footing otherwise foundation drains will be pumped (City of Ottawa)

Surface Storage & Overland Flow

- Building openings to be a minimum of 0.30m above the 100-year water level (City of Ottawa)
- Maximum depth of flow under either static or dynamic conditions shall be less than 0.30m (City of Ottawa)
- Subdrains required in swales where longitudinal gradient is less than 1.5% (City of Ottawa)
- Provide adequate emergency overflow conveyance off-site (City of Ottawa)

Criteria for the development area remain unchanged for the proposed site plan modifications.

5.2.1 Pre-Development Conditions

A lumped catchment PCSWMM model was created for the subject site based on a site area of 2.9ha, and utilizing an existing SCS curve number of 80 per background documents. Additional subcatchment parameters were defined based upon topographical survey of the property:

Area (ha)	Width (m)	Slope (%)	Imperv. (%)	Subarea Routing
2.90	161.1	1.0	0.0	Outlet

Based on the above and during the 2 through 100-year 12hr SCS events (MTO Distribution curves), peak pre-development outflow rates from the subject site were identified per the tables below:

Storm Event	2-Year	5-Year	10-Year
Peak Outflow Rate	17.7L/s	43.9L/s	66.2L/s

Storm Event	25-Year	50-Year	100-Year
Peak Outflow Rate	103.7L/s	136.5L/s	176.3L/s

PCSWMM model input and output files for the predevelopment scenario are included within **Appendix C.**

5.3 STORMWATER MANAGEMENT DESIGN

5.3.1 Rationale for Design and Servicing Deviations

5.3.1.1 Deviation from Kanata West MSS

Per the findings of the Kanata West MSS, stormwater outflows from the proposed site were intended to be directed to the storm sewer within Huntmar Drive, and in turn directed to the downstream Fairwinds temporary pond 5. The MSS had assumed that the entire area of land west of Huntmar Drive and bound by Poole creek to the north and Hazeldean Road to the south was to be directed to the Huntmar Drive sewer, however, the proposed site forms only part of the tributary area, within lands owned by others blocking direct access to the storm sewer within Huntmar Drive. Rather than encumbering the adjacent property, and to avoid considerable connection fees associated with the outlet from the Kanata West Owners Group (KWOG), a separate outlet for the site to Poole Creek has been considered. As the downstream Pond 5 discharges to Poole Creek as well, by restricting flows to predevelopment levels, and assessing the erosive potential of such flows for the Poole Creek reach between the site outlet and that of the downstream Pond 5, no deleterious effects to the downstream watercourse are expected. Additionally, this option provides additional potential to supplement baseflows to Poole Creek in accordance with recommendations from the MVCA.

5.3.1.2 Deviation from Standard SWM Design

The proposed SWM design includes three LID measures (portions of which were previously constructed) to encourage on-site infiltration and water re-use for irrigation. It is recognized that these measures are not currently standard SWM controls and when they are used for water balance purposes are not traditionally included in SWM calculations due to concerns over longterm reliability. The proposed SWM design has included some of the storage and infiltration/reuse rates from these measures in the supporting analysis as discussed in the following sections. However, the analysis has also included simulations assuming that these measures fail in order to assess the potential associated impacts. The benefit of including some of the storage and infiltration losses associated with the LID measures was that the end-of-pipe underground storage component of the infiltration gallery was able to be reduced by 30% as compared to previous design requirements when no credit was assigned for the LID measures. As discussed later in this report, a monitoring plan was previously developed and will continue to be implemented to ensure that constructed LID measures are performing as designed.

5.3.2 Design Methodology

The intent of the stormwater management plan presented herein is to mitigate negative impacts that the proposed development might have on the receiving watercourse (Poole Creek), while providing adequate capacity to service the proposed buildings, underground parking and access areas. The proposed stormwater management plan is designed to detain runoff on the rooftop, surface and in the subsurface (StormTech chamber) to ensure that peak flows after construction will not exceed the target discharge rates and erosion mitigation requirements.

Runoff from the site is captured via catchbasins, landscaping drains and roof drains and conveyed to an existing hydrodynamic separator for water quality treatment followed by a previously constructed underground storage unit for quantity control. The storage unit is restricted by an ICD at the downstream end and is an open bottom unit designed to also promote infiltration. Roof runoff is controlled via roof drains discharging through the internal building plumbing to rainwater harvesting tanks. Two rainwater harvesting tanks are proposed for each building. Two of the four tanks have already been installed to service the existing residential building. Each rainwater tank is capable of storing up to 91m^3 of runoff (approximately 32mm of rainfall) beyond which it will overflow into the storm sewer and be conveyed to the storage unit. The underground storage unit is sized assuming that the rainwater harvesting tanks are available at the start of the rainfall event.

Additional infiltration will be achieved on-site through the existing implementation of a bioswale along the east side of the site. Relocation of a portion of the existing bioswale is necessary to permit site plan modifications. The granular subbase of the swale was previously sized to store runoff from its tributary area. An overflow drain is also provided to convey excess water to the underground storage unit.

The site discharge will be conveyed to the approved outlet location for the adjacent CMHC lands to the west of the subject site. The outlet will be sized to convey flows from both sites. Utilizing this location addresses concerns regarding an additional outlet to Poole Creek and prevents disturbance of the natural area to the north of the site.

5.3.3 Modeling Rationale

A comprehensive hydrologic modeling exercise was completed with PCSWMM, accounting for the estimated major and minor systems to evaluate the storm sewer infrastructure. The use of PCSWMM for modeling of the site hydrology and hydraulics allowed for an analysis of the systems response during various storm events. Surface storage estimates were based on the final grading plan design (see **Drawing GP-1**). The following assumptions were applied to the detailed model:

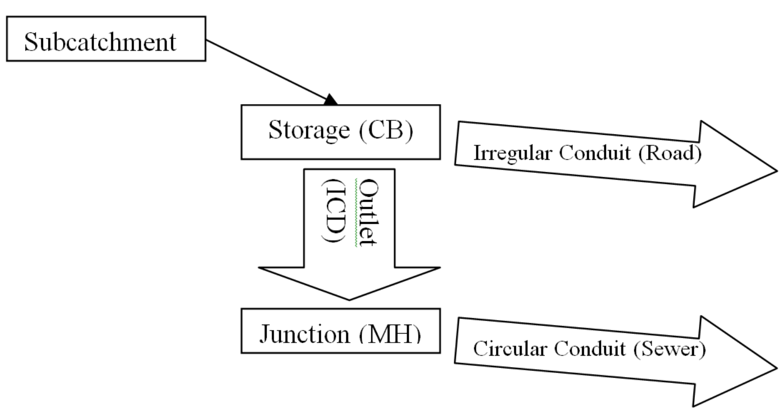
- Hydrologic parameters as per Ottawa Sewer Design Guidelines, including Horton infiltration, Manning's 'n', and depression storage values
- 12-hour SCS Storm distribution for the 100-year analysis to model 'worst-case' scenario in regards to on-site HGLs.
- 12hr SCS distributions (2 and 100-year events) with free flowing boundary condition to model 'worst-case' scenario in regards to site discharge rates to meet target rate.
- To 'stress test' the system a 'climate change' scenario was created by adding 20% of the individual intensity values of the 100-year SCS storm event at their specified time step.

- All LID measures were designed outside of PCSWMM (as documented in the report and calculations included in **Appendix E**) in order to allow routing of LID overflows to the next downstream LID which cannot be done in PCSWMM where an LID is defined as part of a given subcatchment. Total design storage and calculated infiltration losses were then input into PCSWMM as storage nodes with separate outlets for infiltration losses.
- Percent imperviousness calculated based on actual soft and hard surfaces on each subcatchment, converted to equivalent Runoff Coefficient using the relationship C = (Imp. x 0.7) + 0.2
- Subcatchment areas are defined from high-point to high-point where sags occur. Subcatchment width (average length of overland sheet flow) determined by dividing subcatchment area by subcatchment length (length of overland flow path measured from high-point to high-point).
- Number of catchbasins based on servicing plan (**Drawing SP-1**)
- Catchbasin inflow restricted with inlet-control devices (ICDs) as necessary to maintain inflow target rate and maximize use of surface storage where possible.
- Surface ponding in sag storage calculated based on grading plans (**Drawing GP-1**).

5.3.3.1 SWMM Dual Drainage Methodology

The proposed site is modeled in one modeling program as a dual conduit system (see **Figure 3**), with: 1) circular conduits representing the sewers & junction nodes representing manholes; 2) irregular conduits using street-shaped cross-sections to represent the sawtoothed overland road network from high-point to low-point and storage nodes representing catchbasins. The dual drainage systems are connected via outlet link objects (or orifices) from storage node (i.e. CB) to junction (i.e. MH), and represent inlet control devices (ICDs). Subcatchments are linked to the storage node on the surface so that generated hydrographs are directed there firstly.

Figure 3: Schematic Representing Model Object Roles



Storage nodes are used in the model to represent catchbasins as well as major system junctions. For storage nodes representing catchbasins (CBs), the invert of the storage node represents the invert of the CB and the rim of the storage node is the top of the CB plus the maximum above ground storage depth. An additional buffer depth has been added to rim elevations to allow routing from one surface storage to the next, and is unused where no spillage occurs between ponding areas. Ponding at low points is represented via storage area-depth curves for each individual storage node to match ponding volumes demonstrated on the grading plan **Drawing GP-1**. Storage volumes exceeding the sag storage available in the node will route through the connected irregular conduit to the next storage node and continue routing through the system until, ultimately, flows either re-enter the minor system or reach the outfall of the major system.

Inlet control devices, as represented by orifice links, use a user-specified discharge coefficient to approximate manufacturer's specifications for the chosen ICD model.

Subcatchment imperviousness was calculated via impervious area measured from **Drawing SSP-1**.

5.3.3.2 Boundary Conditions

The detailed PCSWMM hydrology and the proposed storm sewers were used to assess the peak inflows and hydraulic grade line (HGL) for the site. The elevation of the outlet sewer at MH100 immediately upstream of Poole Creek has been set conservatively to be above the 100-Year water elevation of the Creek per MVCA Flood Risk Mapping at an invert elevation of 99.7m to enable free-flowing model condition for the site outlet.

5.3.4 Input Parameters

Drawing SD-1 summarizes the discretized subcatchments used in the analysis of the proposed site, and outlines the major overland flow paths. The grading plans are also enclosed for review.

Appendices A1 to A3 summarize the modeling input parameters and results for the subject area; an example input and output file are provided for the 100-year 12hr SCS storm. For all other input files and results of storm scenarios, please examine the electronic model files located on the CD provided with this report. This analysis was performed using PCSWMM, which is a front-end GUI to the EPA-SWMM engine. Model files can be examined in any program which can read EPA-SWMM files version 5.1.010.

5.3.4.1 Hydrologic Parameters

Table 4 presents the general subcatchment parameters used:

Table 4: General Subcatchment Parameters

Parameter	Value
Infiltration Method	Curve Number
Drying Time (days)	7

Curve Number	80
N Impervious	0.013
N Pervious	0.2
Dstore Imperv. (mm)	1.57
Dstore perv. (mm)	4.67
Zero Imperv. (%)	0

Table 5 presents the individual parameters that vary for each of the proposed subcatchments.

Table 5: Subcatchment Parameters

Name	Outlet	Area (ha)	Width (m)	Slope (%)	Imperv. (%)
EXT_1	EXT1-OF	0.073	16.4	33.3	0.0
EXT2	EXT2-OF	0.051	11.5	2.0	80.0
L104A	ST104A-S	0.031	32.1	2.0	81.4
L201A	L201A-S	0.226	88.7	2.0	62.9
L201B	L201B-S	0.156	76.0	3.0	78.6
L201C	L201C-S	0.186	83.0	2.5	80.0
L201D	L201D-S	0.142	48.0	1.0	100.0
L202A	L202A-S	0.343	77.1	1.5	100.0
ST107A	ST107A-S	0.282	225.0	1.5	72.9
ST108A	ST108A-S	0.404	90.8	1.5	100.0
ST108C	ST108C-S	0.062	14.0	1.5	100.0
ST108D	108	0.368	82.7	1.2	37.7
ST110A	110	0.075	16.8	0.8	7.1
ST110C	110	0.030	26.6	10.0	100.0
ST110D	110	0.074	16.7	0.8	7.1
ST111A	ST111A-S	0.256	107.5	0.8	65.7
ST111B	111	0.037	88.0	0.8	100.0
ST111C	ST111C-S	0.044	36.8	1.5	61.4
ST507A	ST507A-S	0.051	25.8	1.5	81.4
ST508A	508	0.096	158.9	1.0	0.0

Table 6 summarizes the storage node parameters used in the model. Storage curves for each node have been created based on volumes presented for each individual ponding area within

Drawing GP-1. Rim elevations for each node correspond to the rim elevation of the associated area's catchbasin plus maximum depth of storage plus 0.30m to allow for demonstration of overland flow in the climate change event scenario. The 0.30m buffer is unused during other modeled events.

Storage volumes and release rates for the rainwater harvesting tank, bioswale/rain garden, and infiltration basin were originally obtained through iterations between design sizing calculations (final sizing attached in **Appendix E**) and PCSWMM hydrologic/hydraulic modeling.

Table 6: Storage Node Parameters

Name	Invert El. (m)	Rim Elev. (m)	Depth (m)	Coefficient	Curve Name	Storage Curve
108	97.24	104.37	7.13	1000	RWHtank	TABULAR
508	101.06	102.85	1.79	1000	ST508A-S	TABULAR
L201A-S	102.24	103.97	1.73	1000	L201A-S	TABULAR
L201B-S	101.83	103.56	1.73	1000	L201B-S	TABULAR
L201C-S	101.83	103.56	1.73	1000	L201C-S	TABULAR
L201D-S	102.81	104.31	1.50	0	*	FUNCTIONAL
L202A-S	115.75	115.90	0.15	1000	L202A	TABULAR
ST104A-S	101.52	103.62	2.10	1000	ST104A-S	TABULAR
ST107A-S	101.13	103.23	2.10	1000	ST107A-S	TABULAR
ST108A-S	118.60	118.75	0.15	1000	ST108A	TABULAR
ST108C-S	110.40	110.55	0.15	1000	ST108C	TABULAR
ST111A-S	101.86	104.26	2.40	1000	ST111A-S	TABULAR
ST111C-S	101.95	104.05	2.10	0	*	FUNCTIONAL
ST507A-S	101.57	103.67	2.10	1000	ST507A-S	TABULAR
TANK	100.10	103.37	3.27	1000	TANK	TABULAR

5.3.4.2 Hydraulic Parameters

As per the Ottawa Sewer Design Guidelines (OSDG 2012), Manning's roughness values of 0.013 were used for sewer modeling and overland flow corridors representing roadways.

Storm sewers were modeled to confirm flow capacities and hydraulic grade lines (HGLs) in the proposed condition. The detailed storm sewer design sheet is included in **Appendix C**.

Table 7 below presents the parameters for the orifice and outlet link objects in the model, which represent ICDs and restricted roof release drains respectively. CB leads modeled as orifices were assigned a discharge coefficient of 0.65. The roof release discharge curves assume the use of standard Zurn model Z-105-5 controlled release roof drains as noted in the calculation sheet in **Appendix C**. The number of roof notches for each building area is to be confirmed with the building mechanical engineer. Details for the IPEX ICDs and Zurn drains are included as part of **Appendix G**.

Table 7: Outlet/Orifice Parameters

Name	Inlet	Outlet	Inlet Elev.	Type	Diameter
L201A-O	L201A-S	201	102.24	IPEX TEMPEST	0.127
L201B-O	L201B-S	201	101.83	IPEX TEMPEST	0.095
L201C-O	L201C-S	201	101.83	IPEX TEMPEST	0.108
OR1	TANK	102	100.1	CIRCULAR	0.11*
OR2	TANK	102	100.7	CIRCULAR	0.15*
OR3	TANK	102	101	CIRCULAR	0.15*
ST104A-O	ST104A-S	104	101.52	IPEX TEMPEST	0.083
ST107A-O	ST107A-S	107	101.13	CIRCULAR	0.2*
ST109C-O	L201D-S	201	102.81	CIRCULAR	0.2*
ST111A-O	ST111A-S	111	101.86	IPEX HF	0.076*
ST111C-O	ST111C-S	111	101.95	CIRCULAR	0.2*
ST111C-O1	ST111C-S	111	101.95	CIRCULAR	0.2*
OL-1	TANK	P_OF1	100.10	0.66L/s	-
OL-2	508	P_OF2	101.06	0.3L/s	-
ST507A-O	ST507A-S	TANK	101.57	IPEX LMF 95	-
ST108A-O	ST108A-S	108	118.60	ROOF	-
ST108B-O	ST108B-S	108	115.75	ROOF	-
ST108C-O	ST108C-S	108	110.40	ROOF	-

*Denotes existing ICD to remain

5.3.5 Model Results

The following section summarizes the key hydrologic and hydraulic model results. For detailed model results or inputs please refer to the example input file in **Appendix C.2 and C.3** and the electronic model files on the enclosed CD.

5.3.5.1 Hydrologic Results

The following tables demonstrate the peak outflow from each modeled outfall during the design storm (12hr SCS 2-100yr) events. A free-flowing outfall condition has been modeled for these events to be conservative with respect to site peak release rates. Outfalls EXT1-OF and EXT2-OF denote uncontrolled flows from the perimeter of the site that, due to grading restrictions, are captured by the existing right-of-way/Poole Creek at the south and north boundaries of the site. Flows from area EXT2 will have a minimal contribution to the infrastructure within Hazeldean Road. Peaks from these uncontrolled flows are non-coincident with peaks from the subsurface storage tank/weir, and as such, flows from the outlet headwall are the only values considered in meeting the release rate target. The required subsurface storage tank volume was originally determined through iteration of each event, and sized to mirror the site release rate target.

Table 8: Site Peak Discharge Rates

Event	Location	Discharge Rate (L/s)	Previously Approved (L/s)	Target (L/s)
2-Year 12 Hour SCS	Outlet Headwall	16.5	15.2	17.7
5-Year 12 Hour SCS	Outlet Headwall	42.3	38.6	43.9
10-Year 12 Hour SCS	Outlet Headwall	70.7	64.5	66.2
25-Year 12 Hour SCS	Outlet Headwall	98.3	98.7	103.7
50-Year 12 Hour SCS	Outlet Headwall	117.9	116.2	136.5
100-Year 12 Hour SCS	Outlet Headwall	133.1	136.3	176.3
100-Year 12 Hour SCS +20%	Outlet Headwall	243.3	317.0	-

5.3.5.2 Hydraulic Results

Table 9 summarizes the HGL results within the site for the 100 year storm events and the 'climate change' scenario storm required by the City of Ottawa Sewer Design Guidelines (2012), where intensities are increased by 20%. The City of Ottawa requires that during major storm events, the maximum hydraulic grade line be kept at least 0.30 m below the underside-of-footing (USF) of any adjacent units connected to the storm sewer during design storm events. As the proposed building perimeter drain and ramp drains will be disconnected from the storm sewer and pumped to the surface, USFs are considered at 0.3m below the lowest finished floor elevation of the building.

Table 9: Modeled Hydraulic Grade Line Results

STM MH	Proposed Elev. (m)	100-year 12hr SCS		100-year 12hr SCS + 20%	
		HGL (m)	USF-HGL Clearance (m)	HGL (m)	USF-HGL Clearance (m)
103	104.20	101.91	2.29	102.36	1.84
104	104.20	101.92	2.28	102.37	1.83
105	104.20	101.93	2.27	102.39	1.81
106	104.20	101.93	2.27	102.40	1.80
107	104.20	101.94	2.26	102.41	1.79
108	104.20	101.97	2.23	102.47	1.73

As is demonstrated in the table above, the worst-case scenario results in HGL elevations remain at least 0.30 m below the proposed surface elevations, and HGL elevations remain below the proposed surface elevations during the 20% increased intensity ‘climate change’ scenario.

Table 10 presents the maximum total surface water depths (static ponding depth + dynamic flow) above the top-of-grate of catchbasins for the 100-year design storm and climate change storm. Based on the model results, the total ponding depth (static + dynamic) does not exceed the required 0.30m maximum during the 100-year event. Total ponding depths during the climate change scenario are below adjacent building openings and should not impact the proposed building.

Table 10: Maximum Surface Water Depths

Storage node ID	Structure ID	Rim Elevation (m)	100 year, 12hr SCS		100 year, 12hr SCS +20%	
			Max HGL (m)	Total Surface Water Depth (m)	Max HGL (m)	Total Surface Water Depth (m)
ST104A-S	CB 506	103.32	102.10	0.00	103.44	0.12
ST107A-S	CB 505	102.93	102.23	0.00	102.97	0.04
ST111A-S	CB 501	103.96	104.18	0.22	104.22	0.26
ST111C-S	CB 504	103.75	102.03	0.00	102.43	0.00
ST507A-S	CB 507	103.37	103.42	0.05	103.44	0.07
508	CB T 508	102.60	102.04	0.00	102.41	0.00
L201A-S	CB 1000	103.62	103.81	0.19	103.83	0.21
L201B-S	CB 1002	103.21	103.40	0.19	103.45	0.24
L201C-S	CB 1003	103.21	103.39	0.18	103.43	0.22

5.3.6 Water Quality Control

On-site water quality control is required to provide 80% TSS removal prior to discharging to Poole Creek. A Stormceptor unit STC3000 was previously proposed upstream of the underground storage/infiltration basin to treat site runoff. The Stormceptor was initially sized to provide greater than 80% TSS removal in the 25mm event and act as pre-treatment for the storage/ infiltration basin thereby reducing maintenance requirements of the facility and improving long-term performance.

The initial sizing for the unit assumed an overall contributing area of 2.72ha at a 70% overall site imperviousness for an overall impervious area of 1.90ha. The initial sizing also conservatively considered rooftop areas (generally consisting of clean runoff) within the 70% overall value. The current proposal considers a contributing area of 2.71ha at an overall imperviousness of 73.9% for an overall impervious area of 2.00ha. Given that much of the impervious area relates to building runoff, and further treatment of runoff by the downstream storage/infiltration basin has not been considered, it is assumed that the minor increase in impervious area for treatment by the OGS will have negligible effect on TSS removal from the site at large.

The Stormceptor unit will remain privately maintained. The location and general arrangement of the Stormceptor unit is indicated on **Drawing SD-1**. Detailed sizing calculations for the Stormceptor unit are included in **Appendix C.5**

5.3.7 Infiltration Targets

The MVCA requires that BMP measures be implemented on-site to meet the minimum infiltration target rate of 50 mm/yr (as identified in the Kanata West Master Servicing Study, Stantec, 2006). For a site area of 2.9ha with an average imperviousness of 70% the total annual infiltration requirement is therefore 1,015m^3/yr. The KWMSS also requires a 25% augmentation to site infiltration requirements to account for off-site road areas for which no infiltration measures were required. Therefore, the total site infiltration target is 1,269 m^3/yr. Past correspondence with the MVCA indicated that the target infiltration rates were in fact "target hydrograph volume reduction rates".

The LID bioswale and infiltration gallery initially proposed for the site provide significant opportunity for stormwater infiltration. Infiltration calculations completed for the design and sizing of these LID measures were used to approximate an expected annual infiltration rate. Water balance calculations for a continuous rainfall scenario from August 2, 2009 to March 1, 2012 (see **Appendix E**) were used to determine an average daily infiltration rate over a one year period. The average rate was estimated to be 44m^3/day. Note that this rate is averaged over 365 days per year and would underestimate summer months and overestimate winter. Nevertheless, the average annual infiltration that could be provided through the LID measures would be approximately 16,262 m^3/yr. Therefore, only about 10% of the total possible infiltration is required to meet the infiltration target for the site.

The infiltration contribution from the bioswale and infiltration gallery is included in **Table 11** below based on calculations within the approved 2017 Servicing and Stormwater Management Brief. As there are no proposed modifications to the size of either LID feature, it is assumed that even though contributing impervious areas may differ from that reported in 2017, the smaller of either facility is more than capable of meeting target infiltration values for the proposed site. Note that this summary does not include infiltration resulting from the rainwater harvesting reuse for irrigation.

Table 11: Summary of Infiltration from LID Features

LID Feature	Estimated Total Annual Infiltration (m3/yr)
Bioswale	2,568
Infiltration Gallery	13,694
Total Infiltration	**16,262**

5.3.7.1 Potential Groundwater Mounding

Groundwater levels at the site were measured by Paterson Group during two separate site visits and are summarized in the attached Paterson memos in **Appendix F**. Based on the results of the groundwater monitoring Paterson Group prepared a memo discussing the variation in groundwater level measurements and anticipated seasonally high and normal groundwater levels. The results for the boreholes near the LID features are summarized in **Table 12** below. The complete memo dated January 25, 2017 is included in **Appendix F**.

Table 12: Expected Seasonal Variation of Groundwater Levels

Borehole Number	Ground Elevation (m)	Long-term Groundwater Levels		Seasonally High Groundwater Level	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
BH 1	102.93	3.7	99.23	3.2	99.73
BH 2	103.02	3.7	99.32	3.2	99.82
BH 3	103.07	3.7	99.37	3.2	99.87
BH 4	103.15	3.7	99.45	3.2	99.95
BH 5	103.22	3.7	99.52	3.2	100.02

BH 6	103.25	3.7	99.55	3.2	100.05
BH 7	102.91	4.5	98.41	4.0	98.91

Since the clearance from the bottom of the infiltration tank to the groundwater table is less than 1.0m the potential for groundwater mounding was considered. Groundwater mounding calculations were completed for both the seasonally high groundwater condition and the normal groundwater condition. However, per the Paterson memo, the seasonally high level is expected to occur during March-April, as such historical rainfall data was used to establish the average rainfall event volume for March-April. The analysis indicated approximately a 10mm event. The duration of infiltration for the infiltration gallery was obtained from the PCSWMM hydraulic model based on the modeled time for the infiltration gallery to empty. No PCSWMM model was run for the 10mm event so the 2-year event was used as a conservative estimate. These durations were input into the groundwater mounding calculation spreadsheet in **Appendix E**. It is noted that the calculations are based on the Hantush (1967) equation for groundwater mounding and use the hydraulic conductivity (measured by Paterson and summarized in the attached memo from September 2016) as the recharge rate and typical specific yield for silty clay. It is also noted that spreadsheet inputs and results are in imperial units. **Table 13** below summarizes the results of the groundwater mounding calculations.

Table 13: Estimated Maximum Groundwater Mounding below Infiltration Gallery

Groundwater Conditon	Mounding Height (m)	Mounding Elevation (m)	Distance to Bottom of Infiltration Gallery (m)
Long-term (99.23)	0.31	99.54	0.56
Seasonally High (99.73)	0.26	99.99	0.11

It is noted the above mounding depths are still below the bottom of the infiltration gallery. Should a larger rainfall even occur during the seasonally high groundwater condition there could be potential for the groundwater mound to extend into the infiltration gallery. However, there is a storm sewer outlet proposed at the bottom of the infiltration gallery (Invert =100.10m per attached **Drawing SSP-1**) which will limit the maximum groundwater height to the bottom of the infiltration gallery. Once the mounding reaches the bottom, the stored stormwater would discharge only through the controlled outlets and would not infiltrate. Since the groundwater mounding is caused only by infiltrating stormwater and not by external sources, there should be no loss of storage volume due to groundwater mounding.

5.3.8 Thermal Controls

The MVCA and MECP confirmed that Poole Creek is designated as a "cool-water fish habitat". As the proposed development will increase the amount of impervious area on the site and roof top detention will increase water temperatures, thermal mitigation measures are required for the site.

As the majority of heat transfer from paved surfaces occurs during the first flush (considered as the initial 10mm of each design event), storage of the 10mm event has been given priority. With exception of the rooftop areas, the site is designed with minimal surface storage. All runoff will be captured and detained in the underground storage unit which will allow for heat dumping into the surrounding ground and granular material. Similarly, runoff conveyed through the granular subbase of the bioswale will experience cooling. Roof discharge will be the most thermally impacted water as it will be retained on the rooftops for several hours. This water will be discharged to the underground rainwater harvesting tank and will inlet at the bottom of the tank such that if the tank is full, the cooler water will be discharged first through the overflow. With 167m^3 of storage available in each of the rainwater harvesting tanks, the only occurrence where roof discharge would not experience any temperature mitigation via mixing or detention would be when total rainfall exceeds approximately a 2-year event. The reverse temperature mitigation effect (warming water during cold weather) would also occur with these measures as ground temperatures would warm the runoff.

5.3.9 Monitoring Plan

In addition to monitoring requirements identified by the MECP in the Environmental Compliance Approval (ECA) previously obtained for site stormwater management works, the site will require regular monitoring of the LID measures installed on the site. A detailed monitoring program for the site was previously developed through consultation with the City of Ottawa and MVCA. In general, the monitoring plan requires pre-construction, during construction and post-construction monitoring and includes the following:

- Installation of water level loggers in both the rainwater harvesting tank, infiltration gallery, and bioswale (monitoring "well" to be installed) to assess frequency of overflow and drawdown rates and compare with design values;
- Installation of temperature logger in the outlet manhole from the site to monitor temperature of the storm discharge. The temperature logger cannot be installed at the outlet to Poole Creek as this outlet will include discharge from the CMHC lands as well and would not be representative of the subject site;
- Collection of water quality samples upstream and downstream of the proposed OGS unit;
- Visual inspection of all LID systems at least once per month and following all large rainfall events. Including observations for:

- Debris accumulation on the surface
- Measurement of/inspection for sediment accumulation in rainwater harvesting tank and infiltration gallery
- Presence of ponded water on the surface of the bioswale beyond design duration
- Outlet/inlet blockages of tanks and OGS

The detailed monitoring plan is included in **Appendix H**.

5.3.10 Contingency Plan

It is recognized that the proposed stormwater management plan is considered a "pilot project" by the City of Ottawa and has allowed for credit from the LID measures toward the stormwater management design. As such the monitoring plan for the site will be critical in assessing the performance of the system. Should either the pre-construction monitoring result in findings that will impact the function of the system, then additional assessment of the design will be required to assess system performance and determine whether additional storage is required. Additional storage would be provided by expanding the size of the proposed infiltration gallery. This assessment would be required prior to constructing the facility. A memo will be issued to the City of Ottawa outlining the monitoring results and confirming whether there is any need for expansion of the infiltration system.

Similarly, post-construction monitoring will assess the performance of the system. Data analysis and reporting will be completed and review whether any retrofits to the system are required. The greatest benefit to the SWM design is the storage available in the rainwater harvesting system. It is estimated that the greatest impact to the system storage requirements would be if this system does not operate as designed and this entire volume cannot be relied upon for the SWM system. This would result in the need for an additional 335m^3 to be added to the infiltration gallery.

Post-construction monitoring will include groundwater level monitoring and water level monitoring within the infiltration gallery. Results will be monitored to ensure no storage volume is lost as a result of groundwater influences and storage volume would be adjusted as necessary. However, it is anticipated that since the infiltration gallery design includes and outlet at the bottom of the storage area, there should be no significant loss of volume caused by seasonal groundwater fluctuations or mounding.

5.4 SUMMARY OF FINDINGS

Based on the preceding, the following conclusions can be drawn:

- The proposed stormwater management plan is in compliance with the criteria established for the site and the 2012 City of Ottawa Sewer Guidelines.
- Inlet control devices are proposed to limit inflow from the site area into the minor system to maximize the use of surface storage.
- Subsurface storage has been provided to further limit site outflows to the peak site discharge rate determined via PCSWMM model (See **Table 14** below).
- The storm sewer hydraulic grade line is maintained at least 0.30 m below finished ground elevations during design storm events.
- All dynamic surface water depths are less than 0.35 m during all design storm events.
- Quality control is provided by a Stormceptor model STC3000 upstream of the underground storage facility to maintain water quality objectives outlined in the background reports.

Table 14: Site Peak Discharge Rates/Targets

Storm Event	Site Peak Discharge Rate (L/s)	Target Discharge Rate (L/s)
2-Year 12 Hour SCS	16.5	17.7
5-Year 12 Hour SCS	42.3	43.9
10-Year 12 Hour SCS	70.7	66.2
25-Year 12 Hour SCS	98.3	103.7
50-Year 12 Hour SCS	117.9	136.5
100-Year 12 Hour SCS	133.1	176.3
100-Year 12 Hour SCS +20%	243.3	-

6.0 POOLE CREEK EROSION ANALYSIS

6.1 2017 BACKGROUND

The following section describes erosion analyses performed within the 2017 Servicing and Stormwater Management Brief for the subject site, and were completed to enable approval and construction of the outfall to Poole Creek for development stormwater outflows. The Poole Creek outfall has been previously approved and constructed on the basis of a continuous model simulation for historical rainfall over the catchment at approximately 66.8% total imperviousness. A revised continuous model for the development has noted an increase in peak discharge during the historical rainfall of 2.1L/s. Given the minimal impact based on the current revised site plan, results of the previous erosion analysis are anticipated to remain consistent with the present.

6.2 INTRODUCTION

The planned outlet for the proposed development site was originally for the site to discharge to the Huntmar Road storm sewer and ultimately drain to the Kanata West Pond 5 before discharging to the Carp River. The current design proposes to combine the site discharge with that from neighbouring CMHC land to be released at the approved CMHC outlet location in Poole Creek.

As Poole Creek has been identified to be sensitive to erosion an analysis of existing and proposed flows in the creek was completed. The analysis used the City of Ottawa Carp River PCSWMM model to estimate bankfull flows for the reaches downstream of the site and to assess critical velocity exceedances.

6.3 EROSION ANALYSIS METHODOLOGY

No fluvial geomorphology assessment was completed for the downstream Poole Creek reaches to determine critical velocities or erosion thresholds for the reaches. As such, the Carp River model was run to determine the maximum velocity in the channel at bank full flow based on the creek cross-sections in the Carp River Model. The model was initially run for the 2-year SCS event however these flows overtopped bankfull for most of the reaches. The 25mm event produced close to bankfull flows for most of the Poole Creek reaches examined. Maximum velocities were then extracted for each reach and used as critical velocities in the assessment of frequency and duration of exceedances for both existing and proposed conditions.

It is noted that the post-restoration Carp River model was not used in this analysis as the level of details included in the model required a simulation time step that resulted in impractical model runtimes for the continuous simulation. Comparison of the existing flow regime in Poole Creek to the ultimate condition in the Post-restoration model indicated that the total flow in Poole Creek is less in the ultimate condition as a significant drainage area is diverted to the Carp River in the ultimate condition. As such, erosion in the creek was assumed to be worse under the existing condition and so the addition of the proposed discharge would have the most potential to impact the creek using this model compared to the ultimate condition.

All continuous model simulations were run using historical rainfall from August 2, 2009 to January 3, 2011 as included in the Carp River model time series files.

To assess the proposed condition the proposed development discharge hydrograph was imported into the upstream node of the outlet reach (model node PJ035). The site area was subtracted from the subcatchment areas in the Carp River model and imperviousness values were recalculated per the adjustments summarized in **Appendix D**. It is noted that the site outflow hydrograph does account for storage on the building roof-tops, surface ponding and end-of-pipe infiltration gallery but does not account for infiltration losses through any of the Proposed LID measures or storage from the bioswale or rainwater harvesting tanks. The LID measures were not included in the analysis to provide a conservative analysis that would represent the conditions should the LID measures fail 100%. The inclusion of the LID measures was examined initially however the runoff generated during the rainfall period that was assessed was negligible. As such it was determined that perhaps the rainfall period did not include large enough events to generate significant runoff from the site when the storage and losses from the LID measures is included.

6.4 EROSION ANALYSIS RESULTS

The critical velocity assumed for each reach and the frequency and duration of exceedances for the existing and proposed conditions are summarized in **Table 15** below. A total increase resulting from the proposed development discharge is also provided.

Table 15: Summary of Critical Velocities and Threshold Exceedances in Poole Creek

Poole Creek Segment Model ID	**PC035**	**PC034**	**PC033**	**PC032**	**PC031**
25mm event Maximum Velocity(m/s):	0.916	1.107	0.7199	0.5083	1.055
Existing Conditions Carp River Model					
Maximum Velocity(m/s):	0.9175	1.109	0.7836	0.6452	1.198
Minimum Velocity(m/s):	0.5351	0.6875	0.2843	0.3674	0.4464
Mean Velocity(m/s):	0.6202	0.781	0.3699	0.3905	0.5565
Duration of Exceedances(h):	0.137	1.563	232.1	229.1	230.4
Number of Exceedances:	4	13	24	24	24
Carp River Model with Proposed Site Discharge					
Maximum Velocity(m/s):	0.9174	1.11	0.7838	0.6455	1.198

Minimum Velocity(m/s):	0.5351	0.6875	0.2843	0.3674	0.4464
Mean Velocity(m/s):	0.6202	0.7811	0.37	0.3906	0.5566
Duration of Exceedances(h):	0.183	0.4591	232.8	229.9	231.1
Number of Exceedances:	4	7	24	24	24
Increase with Proposed Development					
Maximum Flow(m^3/s):	-0.01%	0.09%	0.03%	0.05%	0.00%
Minimum Flow(m^3/s):	0.00%	0.00%	0.00%	0.00%	0.00%
Mean Flow(m^3/s):	0.00%	0.01%	0.03%	0.03%	0.02%
Duration of Exceedances(h):	33.58%	-70.63%	0.30%	0.35%	0.30%
Number of Exceedances:	0.00%	-46.15%	0.00%	0.00%	0.00%

The results in **Table 15** above indicate that the site impact to Poole Creek is minimal. While the percent increase to the duration of erosion in Section PC035 (the site outlet reach) seems significant it is noted that the existing condition duration of exceedance is very low and the total duration of exceedance only increased by 0.046 hours (or 2.8 minutes) over the whole continuous simulation period. The next downstream reach (PC034) shows a reduction in the number and duration of exceedances, this appears to be due to a very slight change in the shape of the flow hydrographs. Again, the total duration of exceedances in this reach is very low in both the existing and proposed condition. In all other reaches evaluated the number of exceedances does not change and the duration increase is less than one hour over the 12456 hours simulated in the continuous model. It is also noted that the model routing error is reported to be 0.3% which is approximately equal to the difference in the exceedance duration downstream of PC034. For these reasons, the analysis would suggest that the proposed development site does not have a significant impact to erosion in Poole Creek.

7.0 GRADING AND DRAINAGE

The proposed development site measures approximately 2.88 ha in area. The topography across the site decreases from south to north, with a change in elevation of approximately 1.4 m to the top of bank of the existing Poole Creek. A detailed grading plan (see **Drawing GP-1**) has been provided to satisfy the stormwater management requirements, adhere to permissible grade raise restrictions (see **Section 11.0**) for the site, and provide for minimum cover requirements for storm and sanitary sewers where possible. Site grading has been established to provide emergency overland flow routes required for stormwater management in accordance with City of Ottawa requirements.

The subject site in its majority maintains emergency overland flow routes for flows deriving from storm events in excess of the maximum design event to Poole Creek as depicted in **Drawings GP-1, SD-1**.

8.0 UTILITIES

Utility infrastructure exists within the Hazeldean Road ROWs at the south property boundaries of the proposed site, as well as within the site servicing the existing Wellings building. Overhead utility poles are located along the south side of Hazeldean Road. It is anticipated that existing infrastructure will be sufficient to provide a means of distribution for the proposed site. Exact size, location and routing of utilities for the revised site plan will be finalized after design circulation.

9.0 APPROVALS

The current ECA Approval for stormwater management and sewage worksl is required to be amended to suit currently proposed site layout, surface storage, rooftop storage, and site discharge rates.

The Mississippi Valley Conservation Authority (MVCA) will need to be consulted in order to obtain municipal approval for the revised site plan development.

Requirement for a MECP Permit to Take Water (PTTW) is unlikely for the site as the proposed works are above the groundwater elevations shown in the geotechnical report. The geotechnical consultant shall confirm at the time of application that a PTTW is not required.

10.0 EROSION CONTROL DURING CONSTRUCTION

Erosion and sediment controls must be in place during construction. It is noted that the site stormwater management includes Low Impact Development features which are designed to infiltrate. The two features that rely on infiltration are the proposed bioswale and infiltration gallery. These features shall be constructed after all other site grading and construction is complete to protect the backfill media from sediment generated during construction. The bioswale depression may be excavated during construction if it is required for stormwater management/drainage during construction. However, these areas should not be excavated to the depth of the infiltration areas until all other site works are complete. Any accumulated sediment will need to be excavated from these areas and infiltration testing shall be completed before placing clear stone and geotextiles etc.

The following recommendations to the contractor will be included in contract documents.

1. Implement best management practices to provide appropriate protection of the existing and proposed drainage system and the receiving water course(s).
2. Limit extent of exposed soils at any given time.
3. Re-vegetate exposed areas as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with plastic or synthetic mulches.
6. Provide sediment traps and basins during dewatering.
7. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.
8. Plan construction at proper time to avoid flooding.
9. Installation of a mud matt to prevent mud and debris from being transported off site.
10. Installation of a silt fence to prevent sediment runoff.

The contractor will, at every rainfall, complete inspections and guarantee proper performance. The inspection is to include:

11. Verification that water is not flowing under silt barriers.
12. Clean and change silt traps at catch basins.

Refer to **Drawing EC/DS-1** for the proposed location of silt fences, straw bales, and other erosion control structures.

For detailed construction monitoring requirements refer to the attached LID Monitoring Plan in **Appendix H**.

11.0 GEOTECHNICAL INVESTIGATION

A geotechnical investigation was completed by Paterson Group Ltd. in February of 2016. The report summarizes the existing soil conditions within the subject area and construction recommendations. For details which are not summarized below, please see the original Paterson report.

Subsurface soil conditions within the subject area were determined from 25 boreholes distributed across the proposed site. In general soil stratigraphy consisted of topsoil underlain by a stiff to very stiff silty clay, followed by sandy silt deposit over a dense glacial till layer.

Groundwater Levels were measured on January 28, 2016 and vary in elevation from 1.1 to 4.5m below the original ground surface. It is expected that construction occur below the existing groundwater table and therefore a permit to take water may be required as well as requirements for damp proofing or foundation waterproofing may be required.

A permissible grade raise restriction of 1.0 m has been recommended within the Paterson Group report. The grade raise restrictions were accounted for in the grading design of the property.

The required pavement structure for the local roadways is outlined in **Table 16** and **Table 17** below:

Table 16: Pavement Structure – Car Only Parking Areas

Thickness (mm)	Material Description
50	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
150	Base – OPSS Granular A Crushed Stone
300	Subbase - OPSS Granular B Type II
-	Subgrade – Either fill, in situ soil, select subgrade material or OPSS Granular B Type I or II material placed over in situ soil or fill.

Table 17: Pavement Structure – Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course – HL-8 or Superpave 19.0 Asphaltic Concrete
150	Base – OPSS Granular A Crushed Stone
450	Subbase - OPSS Granular B Type II
-	Subgrade – Either fill, in situ soil, select subgrade material or OPSS Granular B Type I or II material placed over in situ soil or fill.

12.0 CONCLUSIONS

12.1 WATER SERVICING

Based on the results of the hydraulic analysis presented in the report, the proposed servicing in this development will provide sufficient capacity to sustain both the required domestic demands and emergency fire flow demands of the proposed site. Based on computer modeling results, fire flows greater than those required per the FUS Guidelines are available for this development.

12.2 SANITARY SERVICING

The proposed sanitary sewer network is sufficiently sized to provide gravity drainage of the site. The proposed site will be serviced by a gravity sewer which will direct the wastewater flows (approx. 10.3 L/s) to the existing 675mm dia. Hazeldean Road sanitary sewer. The proposed drainage pattern is in accordance with the Kanata West Master Servicing Report for Hazeldean Road.

12.3 STORMWATER SERVICING

The proposed stormwater management plan is in compliance with the criteria previously established for the site. Rooftop and subsurface storage has been designed to limit outflows from the subject site to calculated predevelopment levels. All surface water depths are less than 0.35 m during the 100-year storm event. The downstream Poole Creek has sufficient capacity to receive runoff volumes from the site based on anticipated peak flows and detention times for the existing subsurface storage tank servicing the development.

12.4 GRADING

Grading for the site has been designed to provide an emergency overland flow route as per City requirements and reflects the grade raise restrictions recommended in the Supplemental Geotechnical Investigation, Proposed Mixed Use Development – 5731 Hazeldean Road, Paterson Group, February 2016. Erosion and sediment control measures will be implemented during construction to reduce the impact on existing facilities.

12.5 UTILITIES

Utility infrastructure exists within the Hazeldean Road ROWs at the south property boundaries of the proposed site, as well as existing plant servicing the Wellings building on-site. Overhead poles are located along the south side of Hazeldean Road. It is anticipated that existing infrastructure will be sufficient to provide a means of distribution for the proposed site. Exact size, location and routing of utilities will be finalized after design circulation.

12.6 APPROVALS/PERMITS

The existing MECP Environmental Compliance Approval will be required to be amended to suit the revised site plan with regards to surface storage, rooftop storage, and peak site discharge rates. A Permit to Take Water is not anticipated to be required for pumping requirements for sewer installation. The Mississippi Valley Conservation Authority will need to be consulted in order to obtain municipal approval for the revised site plan. No other approval requirements from other regulatory agencies are anticipated.