Servicing and Stormwater Management Brief – Wellings of Stittsville Phase 2, 20 Cedarow Court

Project # 160401511

Prepared for:
Nautical Lands Group

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Sign-off Sheet

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1.0 INTRODUCTION

Stantec Consulting Ltd. has been commissioned by Nautical Land Group. to prepare the following servicing study in support of the development at 20 Cedarow Court located within the City of Ottawa. 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 and commercial development comprises approximately 2.29ha of land and proposes construction of a 284 unit, six storey residential building (Phase 2 and 3), a second 200 unit (Phase 4), six storey residential building, as well as commercial buildings, all of which are proposed to be connected via one level of underground parking. The site will be constructed in two phases, beginning with building phases 2 and 3 located adjacent to Hazeldean Road, and ultimately constructing phase 4. 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 20 Cedarow Court Development include:

- Kanata West Master Servicing Study, Stantec Consulting Ltd., Cumming Cockburn Limited / IBI, October 1, 2014.
- Carp River PCSWMM Model Documentation Draft Report, City of Ottawa, March 2016.
- Geotechnical Investigation, Proposed Mixed Use Development Wellings of Stittsville – Phase 2 20 Cedarow Court, Ottawa, Ontario, Paterson Group, March 7, 2019.
- Geotechnical Plan Review, Proposed Mixed Use Development Wellings of Stittsville – Phase 2 20 Cedarow Court, Ottawa, Ontario, Paterson Group, August 12, 2021.
- Servicing and Stormwater Management Brief-5731 Hazeldean Road, Stantec Consulting Ltd., March 22, 2017
- Tree Conservation Report – 5731 Hazeldean Road, IFS Associates, March 11, 2016.
- City of Ottawa Sewer Design Guidelines, City of Ottawa, October 2012.f
- City of Ottawa Design Guidelines – Water Distribution, City of Ottawa, July 2010.

3.0 WATER SUPPLY SERVICING

3.1 BACKGROUND

The proposed development comprises two residential apartment buildings with commercial space fronting Hazeldean Road, and complete with associated infrastructure and underground 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 site will be serviced at two connection points via a proposed 200mm diameter connection to the existing stub within the Fringewood Avenue ROW at the eastern quadrant of the site, and a 300mm diameter connection to the existing 300mm diameter watermain within Cedarow Court along the western boundary of the site. The stub on Fringewood Avenue connects directly to the existing 762mm feedermain within Hazeldean Road immediately south of the site. The plumbing through the buildings will be looped.

3.2 WATER DEMANDS

Water demands for the development were estimated using the Ministry of Environment's Design Guidelines for Drinking Water Systems (2008) and the Ottawa Design Guidelines – Water Distribution (2010). A daily rate of 28,000 L/gross ha/day has been applied for commercial building space, whereas the residential facility demand was estimated at 350L/person/day with an estimated population of 1.4 persons/unit for bachelor or one bedroom apartments and 2.1 persons/unit for two bedroom apartments. See **Appendix A.1** for detailed domestic water demand estimates.

The average day demand (AVDY) for the entire site was determined to be 3.2 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 7.86 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 17.19 L/s.

Non-combustible construction with 2-hour fire separation between each floor was considered in the assessment of the fire flow requirements for the site according to the FUS Guidelines. The FUS Guidelines indicate that low hazard occupancies include apartments, dwellings, dormitories, hotels, and schools, and as such, a low hazard occupancy / limited combustible building contents credit was applied. A sprinkler system conforming to NFPA 13 was considered, and a credit applied per FUS Guidelines. Based on calculations per the FUS Guidelines (**Appendix A.2**), the maximum required fire flows for this development is 167 L/s (10,000 L/min) occurring at the proposed six-storey apartment building fronting Hazeldean Road ROW. Two hydrants located in proximity to both buildings siamese connections are proposed on the subject site. The existing hydrants along the northeastern boundary and the two proposed on site hydrants will provide amply fire flow.

3.3 PROPOSED SERVICING

Per boundary conditions provided by the City of Ottawa and an approximate elevation on-site of 104.7m, adequate domestic water supply is available for the subject site with pressures ranging from 44.9m (75.4psi) to 56.4m (80.3psi). These values are within the normal operating pressure range as defined by the MECP and City of Ottawa design guidelines (desired 50-80 psi and not less than 40 psi). A pressure check once construction is completed is required to determine if pressure reducing valves are needed.

The boundary conditions for the proposed development under maximum day demands were initially provided under an assumed fire flow demand of 267L/s. As such, it can be confirmed that the system will maintain a residual pressure which is in excess of the required 140 kPa (20 psi) under the required fire flow demand of 167L/s. The above demonstrates that the existing watermain within Fringewood Avenue and Cedarow Court can provide adequate fire and domestic flows in excess of flow requirements for the subject site. An existing hydrant is located approximately 18m northeast of the subject site and at least one proposed hydrant is to be located within 45m of the building fire department connection (siamese) per OBC requirements.

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 the boundary conditions as provided by the City of Ottawa staff, fire flows are available for this development based on FUS guidelines and as per the City of Ottawa water distribution guidelines.

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, which will ultimately outlet to the Kanata West Pump Station (see **Drawing SSP-1**).

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) – 28,000L/gross ha/day of building space
- Average Wastewater Generation (Residential) – 280L/cap/day
- Peak Factor (Commercial) – 1.5 (Max Day Demand per MOE Design Guidelines for Drinking Water Systems)
- Peak Factor (Residential) – 4.0 (Harmon's)
- Extraneous Flow Allowance – 0.33 l/s/ha (conservative value)
- Manhole Spacing – 120 m
- Minimum Cover – 2.5m
- Population density for single-bedroom and bachelor apartments – 1.4 pers./apartment
- Population density for two-bedroom apartments – 2.1 pers./apartment

4.3 PROPOSED SERVICING

The proposed site will be serviced by a gravity sewer which will direct the wastewater flows (approx. 10.4 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. A proposed excavation cross section of the Hazeldean Road connection to the existing 675mm diameter sanitary sewer has been included on **Drawing SSP-1.** Extra precaution should be taken to ensure no damages are made to the existing 762 backbone watermain located 2.9m south of the proposed sanitary connection. Additional construction details will be included by the contractor prior to construction.

The proposed drainage pattern is in accordance with the Kanata West Master Servicing Study (KWMSS) for Hazeldean Road and is detailed on **Drawing SAN-1.** Sanitary flows will ultimately be discharging to the downstream Kanata West Pump Station. A Sanitary sewer design sheet is included in **Appendix B.1**. Excerpts of the overall sanitary system discharging to the Kanata West Pump Station based on the KWMSS are included in **Appendix B.2.** It is noted that peak ultimate sanitary discharge to the KWPS is likely to be far lower than that indicated within the KWMSS design sheet, as current operational parameters estimating peak flow from residential uses have decreased from 350L/person/day to 280L/person/day, and commercial lands contributions have decreased from 50,000L/ha/day to 28,000L/ha/day. As a result, it is assumed that there is ample capacity within the downstream conveyance network and KWPS to receive any additional flows from that originally assumed for the area (50,000L/ha/day x 2.29ha x 1.5 P.F. = approximately 2.0L/s).

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.

5.2 SWM CRITERIA AND CONSTRAINTS

Criteria were 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)
- 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, MECP)
- Site discharge to be controlled to pre-development rates (City of Ottawa)

Storm Sewer & Inlet Controls

- Size storm sewers to convey the 2-year storm event under free-flow conditions using City of Ottawa I-D-F parameters (City of Ottawa)
- Minimum sewer inlet capture rates to be set such that no ponding occurs at the end of the - 2-year event (City of Ottawa)
- Request made by the client to not allow ponding to occur in the 100-year event
- 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)

5.2.1 Pre-Development Conditions

A background report for 20 Cedarow Court Commercial Development was completed on April 6, 2009 by Novatech Engineering for the proposed property. Currently, a large portion of the site is pervious, and sheet drains north west towards Poole Creek. Based on topography, existing drainage is directed through the site for properties on Cedarow Court adjacent to the subject lands. The additional runoff will be returned to the Cedarow Court storm sewer and was not included in the overall area contributing to the pre-development rate. The sewers on Cedarow Court were analyzed based on 2K mapping data corroborated by field investigation, and the additional flows were determined not to impact the downstream 525mm diameter storm sewer. The design sheet and area map for the Cedarrow Court sewer can be found in **Appendix C.6.**

The site discharge will be conveyed to the approved outlet located at the northwestern boundary of the subject site. The outlet was constructed as part of Wellings of Stittsville Inc. and Extendicare Inc. Phase 1 and was sized to convey flows from both sites. Excerpts from the Wellings of Stittsville Phase 1 servicing and stormwater management brief can be found in **Appendix C.7**.

A lumped catchment PCSWMM model was created for the subject site based on a site area of 2.3ha, and utilizing an existing SCS curve number of 82 per background documents (Carp River Full Restoration PCSWMM Model). Additional subcatchment parameters were defined based upon recent topographical survey of the property:

Area (ha)	Width (m)	Slope (%)	Imperv. (%)	Subarea Routing
2.29	143	1.0	0.0	Outlet

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

Storm Event	Peak Outflow Rate (L/s)
2-Year	17.9
5-Year	43.4
10-Year	69.8
25-Year	111.6
50-Year	142.4
100-Year	182.1

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

5.3 STORMWATER MANAGEMENT DESIGN

5.3.1 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 available flat rooftops, and in a subsurface storage unit to ensure that peak flows after construction will not exceed the target discharge rates.

Runoff from the site is captured via catchbasins and roof drains and conveyed to a hydrodynamic separator for water quality treatment before entering an underground storage unit for quantity control. The storage unit is restricted by an ICD at the downstream end while the roof runoff is controlled via roof drains discharging through the internal building plumbing. Eight interconnected tanks are proposed to act as subsurface storage for the development. Each tank is capable of storing up to 79m^3 (20,000 gallons) of runoff for a total allowable storage of 633 m^3. The underground storage unit is sized assuming that the entirety of the roof area is available to capture and store water up to150mm in depth during the 100-year storm event.

In case of subsurface storage tank failure, overflows are managed first via installed weir wall within MH101 to address orifice blockage, followed by surface vents/openings at each tank in series to the surface to ensure failure of an individual tank does not cause failure of the system at large. Flows are then recaptured by the remaining tank cells in operation. Building internal pumping to the building storm outlet upstream of the hydrodynamic separator has been air-gapped with provision for overflow per OBC requirements.

As the proposed invert of the hydrodynamic separator lies above anticipated downstream 100-year HGL of the subsurface storage tanks, no tailwater concerns are noted for design of the separator. The proposed hydrodynamic separator maintains an internal overflow weir for large storm events for protection of building internal plumbing, and will not impede inflow to the downstream storage tanks.

The site discharge will be conveyed to the previously approved outlet location at the western boundary of the site which ultimately directs flow into Poole Creek. The existing outlet is designed to convey flows from the proposed site as well as the existing adjacent site to the northeast, Wellings of Stittsville Inc. and Extendicare Inc Phase 1.

The site will be constructed in two phases, including build out of the underground parking structure. As the first phase is built, the entirety of the storm water storage tanks will be constructed.

5.3.2 Modeling Rationale

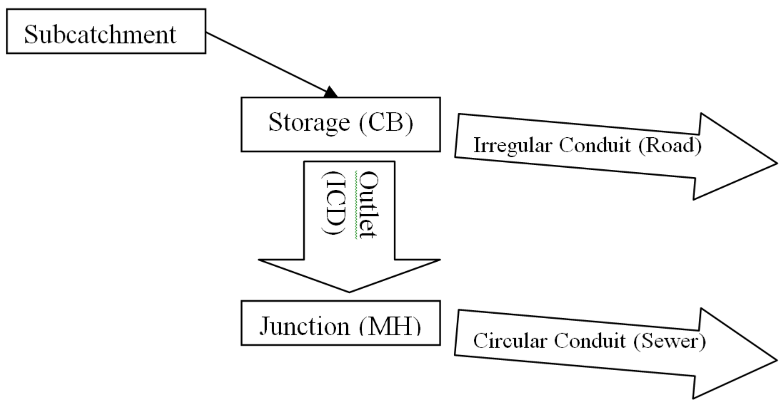
A comprehensive hydrologic modeling exercise was completed with PCSWMM, accounting for the estimated major and minor systems to evaluate the storm sewer infrastructure. The use of PCSWMM for modeling of the site hydrology and hydraulics allowed for an analysis of the systems response during various storm 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 storage volume.
- 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. It is of note that the 100-Year floodplain elevation of the Creek at the site discharge point will not affect upstream HGLs or storage volumes provided.
- 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.
- 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**)

5.3.2.1 SWMM Dual Drainage Methodology

The proposed site is modeled in one modeling program as a dual conduit system (see **Figure 2**), 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 2: 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(all catch basins on top of the underground structure will not have any surface storage). An additional 0.3m 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.

Inlet control devices, as represented by orifice links, use a user-specified discharge coefficient to approximate manufacturer's specifications for the chosen ICD model. Discharge rates from the rooftops are based on the quantity of roof drains provided in the site plan per roof level. The roof drains are modelled using outlets with rating curves which specifies the outflows per roof level.

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

5.3.2.2 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 99.8m to enable free-flowing model condition for the site outlet. The elevation of the water level within Cedarow Court was conservatively set to an obvert of the receiving sewer at 102.17m.

5.3.3 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 C2 and C3 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 5.1.014.

5.3.3.1 Hydrologic Parameters

Table 1 presents the general subcatchment parameters used:

Table 1: General Subcatchment Parameters

Parameter	Value
Infiltration Method	Horton
Max. Infil. Rate (mm/hr)	76.2
Min. Infil. Rate (mm/hr)	13.2
Decay Constant (1/hr)	4.14
N Impervious	0.013
N Pervious	0.2
Dstore Imperv. (mm)	1.57
Dstore perv. (mm)	4.67
Zero Imperv. (%)	0

Table 2 presents the individual parameters that vary for each of the proposed subcatchments.

Table 2: Subcatchment Parameters

Name	Outlet	Area (ha)	Width (m)	Slope (%)	Imperv. (%)
EXT-1	CB509-S	0.069	95	1.5	38.6
ROOF_10	ROOF-10-S	0.281	136	1.5	100.0

ROOF_11	ROOF-11-S	0.011	21	1.5	100.0
ROOF_12	ROOF-12-S	0.013	15.6	1.5	100.0
ROOF_3	ROOF-3-S	0.110	130	1.5	100.0
ROOF_4	ROOF-4-S	0.035	46	1.5	100.0
ROOF_5	ROOF-5-S	0.110	130	1.5	100.0
ROOF_8	ROOF-8-S	0.010	21	1.5	100.0
ROOF_9	ROOF-9-S	0.005	15	1.5	100.0
ROOF1_2	ROOF-1-2-S	0.095	95	1.5	100.0
ROOF6_7	ROOF-6-7-S	0.096	95	1.5	100.0
UGPK_1	TANKS	0.144	115	2.0	77.1
UGPK_2	TANKS	0.152	122	2.0	80.0
UGPK_3	TANKS	0.060	60	2.0	58.6
UGPK_4	TANKS	0.120	95	2.0	70.0
UGPK_5	TANKS	0.110	85	2.0	70.0
UGPK_6	TANKS	0.022	60	15.0	100.0
UGPK_7	TANKS	0.112	78	2.0	78.6
UGPK_8	TANKS	0.062	42	2.0	75.7
UGPK_9	TANKS	0.032	42	2.0	100.0
UNC-1	OF1	0.078	78	2.0	41.4
UNC-2	OF2	0.515	25	1.0	8.6
UNC-3	OF3	0.069	122	2.0	61.4
UNC-4	CB509-S	0.052	90	2.0	37.1

Table 3 summarizes the storage node parameters used in the model. Storage curves for each node have been created based on available volumes within each roof top or subsurface storage as applicable. Rim elevations for each node correspond to the rim elevation of the associated area's roof top drain or catch basin plus maximum depth of storage. Catch basins located above underground parking areas flow uncontrolled to the underground storage tank and provide no quantity storage for events up to the 100-year design event. No quantity storage has been assumed for model conservatism for the water balance BMP described in **Section 5.3.6**

Storage volumes and release rates for the underground storage tank were obtained through PCSWMM hydrologic/hydraulic modeling:

Table 3: Storage Node Parameters

Name	Invert El. (m)	Rim Elev. (m)	Depth (m)	Coefficient	Exponent	Constant (m^2)	Curve Name	Storage Curve
CB509-S	102.56	104.79	2.23	0	0	0	*	FUNCTIONAL
ROOF-10-S	114	114.15	0.15	0	0	0	ROOF10	TABULAR
ROOF-11-S	114	114.15	0.15	0	0	0	ROOF11	TABULAR
ROOF-12-S	114	114.15	0.15	0	0	0	ROOF12	TABULAR
ROOF-1-2-S	114	114.15	0.15	0	0	0	ROOF1and2	TABULAR
ROOF-3-S	114	114.15	0.15	0	0	0	ROOF3	TABULAR
ROOF-4-S	114	114.15	0.15	0	0	0	ROOF4	TABULAR
ROOF-5-S	114	114.15	0.15	0	0	0	ROOF5	TABULAR
ROOF-6-7-S	114	114.15	0.15	0	0	0	ROOF6and7	TABULAR
ROOF-8-S	114	114.15	0.15	0	0	0	ROOF8	TABULAR
ROOF-9-S	114	114.15	0.15	0	0	0	ROOF9	TABULAR
TANKS	99.7	103.31	3.61	0	0	222	*	FUNCTIONAL

5.3.3.2 Hydraulic Parameters

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

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**.

PCSWMM output hydrographs from Phase 1 for each storm event were used at manhole structure 100 in the current PCSWMM model to accurately represent to total outflow from both properties at the headwall.

Table 4 below presents the parameters for the orifice and outlet link objects in the model, which represent ICDs and restricted roof release drains respectively. The underground storage orifice was assigned a discharge coefficient of 0.61. The tank is designed with a 75mm ICD to restrict flows during the 2-year event, as well as a weir to allow additional flows to be directed towards the outlet during larger storm events. The weir is placed in manhole structure 1000 and designed with a width of 0.5m (see **Table 4** for invert elevation).

The roof release discharge curves assume the use of standard Watts Model R1100 Accutrol controlled release roof drains as noted in the calculation sheets in **Appendix C**. The number of roof notches for each roof level was confirmed with the building mechanical engineer. Details for the IPEX ICDs and Watts drains are included as part of **Appendix C**.

Table 4: Outlet/Orifice Parameters

Name	Inlet	Outlet	Inlet Elev.	Type	Diameter (m)
CISTERN-O	TANKS	100	99.95	CIRCULAR	0.075
			102.25	WEIR	0.50
ROOF1-2-O	ROOF-1-2-S	TANKS	114	ROOF-1-2-O	-
ROOF3-O	ROOF-3-S	TANKS	114	ROOF-3-O	-
ROOF4-O	ROOF-4-S	TANKS	114	ROOF-4-O	-
ROOF5-O	ROOF-5-S	TANKS	114	ROOF-5-O	-
ROOF6-7-O	ROOF-6-7-S	TANKS	114	ROOF-6-7-O	-
ROOF8-O	ROOF-8-S	TANKS	114	ROOF-8-O	-
ROOF9-O	ROOF-9-S	TANKS	114	ROOF-9-O	-
ROOF10-O	ROOF-10-S	TANKS	114	ROOF-10-O	-
ROOF11-O	ROOF-11-S	TANKS	114	ROOF-11-O	-
ROOF12-O	ROOF-12-S	TANKS	114	ROOF-12-O	-

5.3.4 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 provided.

5.3.4.1 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 OF1 to OF4 denote uncontrolled flows from the perimeter of the site that, due to grading restrictions, are captured by the existing ROW on Fringewood Avenue at the eastern boundary, Poole Creek at the north boundaries of the site, Hazeldean Road to the south and Cedarow Court Row to the west. The adjacent site on the eastern boundary (2500 Wellings Private) has sufficient capacity to capture minor uncontrolled flows from subcatchment UNC1. Flows from area UNC3-OF will have a minimal contribution to the infrastructure within Hazeldean Road. Based on existing external and proposed grading, subcatchments EXT-1 and UNC-4 are proposed to drain to a swale and runoff is to be captured in the subdrain. Connection to the existing 300mm diameter storm sewer on Cedarow Court is proposed to direct the flows captured from the subdrain. The storm sewer along Cedarow Court ultimately discharges to Poole Creek upstream of the proposed site. Appendix C.6 provides calculations based of general

Results of the PCSWMM model run have been provided in **Appendix C**. Peaks from the uncontrolled flows with the exception of UNC-2 are non-coincident with peaks from the subsurface storage tank/weir, and as such, flows from the conduit downstream of the subsurface storage tank (conduit C2) and UNC-2 have been considered in meeting the site pre-development release rate target. The required subsurface storage tank volume was determined through iteration of each event and sized to mirror the site release rate target.

Table 5: Site Peak Discharge Rates

Event	Location	Discharge Rate (L/s)	Target (L/s)
2-Year 12 Hour SCS	Outlet Headwall	16.3	17.9
5-Year 12 Hour SCS	Outlet Headwall	25.6	43.4
10-Year 12 Hour SCS	Outlet Headwall	57.2	69.8
25-Year 12 Hour SCS	Outlet Headwall	90.1	111.6
50-Year 12 Hour SCS	Outlet Headwall	113.8	142.4
100-Year 12 Hour SCS	Outlet Headwall	150.3	182.1
100-Year 12 Hour SCS +20%	Outlet Headwall	308.8	-

*Post-development flows are a sum of the hydrographs from conduit C2 and outfall OF2

5.3.4.2 Hydraulic Results

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. The USFs elevations have been considered at 0.5m below the lowest top of basement slab elevation of the proposed buildings. As the proposed building perimeter foundation drain will be disconnected from the storm sewer and pumped to the surface, the proposed building footings will not be hydraulically connected to the underground storage tank. The ramp drain is to be pumped to the storage tanks. The maximum hydraulic grade line (HGL) of the underground storage tank reaches 102.48m and 102.68m during the 100 year and 100year +20% event. The HGL elevations in both scenarios remain at least 0.30 m below the proposed surface elevations as the lowest elevation of the connected catch basins within the aboveground parking structure are at 104.18m.

Table 6 presents the maximum total surface water depths (static ponding depth + dynamic flow) above the top-of-grate of the catch basin discharging to the Cedarow Drive sewer for the 100-year design storm and climate change storm. Based on the model results, no surface ponding is anticipated within the swale/subdrain within area UNC-4.

Table 6: 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)
CB507-S	CB 507	104.44	102.74	0.00	102.78	0.00

5.3.5 Water Quality Control

On-site water quality control is required to provide 80% TSS removal prior to discharging to Poole Creek. A Stormceptor unit STC300 is proposed upstream of the underground storage tank. Runoff from roof top areas are considered clean and were assumed as pervious when calculating the total imperviousness of the contributing catchment area to the stormceptor. Design calculations for the Stormceptor indicate that the selected model will provide greater than 80% TSS removal on an annual basis. The Stormceptor unit will be 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.4.**

5.3.6 Water Balance

The KWMSS and Carp River Watershed Study report identify that the site is located within a low groundwater recharge area. The Watershed Study in particular recommends a minimum of 73mm per year of infiltration (or 1171m^3/yr for the 2.29ha site) for water balance purposes and to support Poole Creek baseflow. As such, it is proposed that roof runoff from Phase 4 buildings (areas Roof-8 through Roof-12) be directed to an infiltration trench BMP composed of clear stone be located within the Poole Creek regulation limit corridor (but outside of any limit of hazard lands and top of stable slope line as determined by Paterson/MVCA) to provide baseflow to the creek during the inter-event period. The BMP is to tie in behind the orifice control for the subsurface storage tanks to allow overflow via perforated pipe for larger storm events to be controlled prior to release to the creek. Inverts of the BMP have been set to avoid high groundwater elevations and provide a minimum offset of 1.0m from anticipated bedrock elevations. Sizing of the BMP has been provided within **Appendix C.8**, and demonstrates that sufficient storage exists below the perforated pipe drain to sequester runoff from up to the 25mm storm event for connected areas, and provide up to 2675m^3 of annual infiltration.

6.0 GRADING AND DRAINAGE

The proposed development site measures approximately 2.29 ha in area. The topography across the site decreases from south to north, with a change in elevation of approximately 1.5 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 10.0**) 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**.

7.0 UTILITIES

Utility infrastructure exists within the Hazeldean Road ROW at the south property boundary of the proposed site. 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 will be finalized after design circulation.

8.0 APPROVALS

As the site will be discharging to an existing storm sewer outlet, will remain under singular ownership, and will not drain industrial lands or industrial land uses, exemption from the Ontario Ministry of Environment, Conservation and Parks (MECP) Environmental Compliance Approval (ECA) process is expected for works within the subject site.

The outlet headwall has been previously approved under the neighboring property. The ECA application number is NUMBER 7185-ARZMHZ.

The Mississippi Valley Conservation Authority (MVCA) will need to be consulted in order to obtain municipal approval for site development, and permits acquired for any proposed fill within the Poole Creek regulatory limit.

Requirement for a MECP Permit to Take Water (PTTW) for sewer construction is unlikely for the site as the proposed works are above the groundwater elevations shown in the geotechnical report. Building excavation areas, however, will likely be within the groundwater table and may require a PTTW. The geotechnical consultant shall confirm at the time of application that a PTTW is not required.

9.0 EROSION CONTROL DURING CONSTRUCTION

Erosion and sediment controls must be in place during construction. 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.

10.0 GEOTECHNICAL INVESTIGATION

A geotechnical investigation was completed by Paterson Group Ltd. in March of 2019. 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 29 boreholes distributed across the proposed site. In general soil stratigraphy consisted of topsoil underlain by a hard to very stiff silty clay, followed by very stiff to stiff silty clay layer over a glacial till layer.

Groundwater Levels were measured on January 29, 2019 and vary in elevation from 1.7 to 3.2m 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 2.0 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 at-grade parking areas and access lanes are outlined in **Table 7** and **Table 8** below:

Table 7: Recommended Pavement Structure – At-Grade 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 – In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

Table 8: Recommended 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 – In situ soil, or OPSS Granular B Type I or II material placed over in situ soil.

11.0 CONCLUSIONS

11.1 WATER SERVICING

Based on the supplied boundary conditions for existing watermains and estimated domestic and fire flow demands for the subject site, it is anticipated that 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. Fire flows greater than those required per the FUS Guidelines are available for this development.

11.2 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. 10.4 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 the Hazeldean Road sewer.

11.3 STORMWATER SERVICING

The proposed stormwater management plan is in compliance with the criteria established for the site. Rooftop and subsurface storage have been designed to limit outflows from the subject site to calculated predevelopment levels. Poole Creek is located downstream of the site and has sufficient capacity to receive runoff volumes from the site based on anticipated peak flows and detention times for the subsurface storage tank servicing the development.

11.4 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 prepared by Paterson Group (March, 2019) . Erosion and sediment control measures will be implemented during construction to reduce the impact on existing facilities.

11.5 UTILITIES

Utility infrastructure exists within the Hazeldean Road ROW at the south property boundary of the proposed 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.

11.6 APPROVALS/PERMITS

MECP Environmental Compliance Approval is not expected to be required for the proposed site works. A Permit to Take Water is not anticipated to be required for pumping requirements for sewer installation, however, will likely be a requirement for building excavation. The Mississippi Valley Conservation Authority will need to be consulted in order to obtain municipal approval for site development. No other approval requirements from other regulatory agencies are anticipated.