Site Servicing and Stormwater Management Report – Orleans II Development - 2025 Mer Bleue Road – Phase 2A

Project # 160401242

Prepared for:
SmartREIT (Orleans II) Inc. and Mer Bleue Shopping Centres Limited

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Sign-off Sheet

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1.0 INTRODUCTION

Stantec Consulting Ltd. has been commissioned by SmartREIT (Orleans II) Inc. to prepare a servicing study in support of Site Plan Control submission of the proposed commercial development located at 4100 Innes Road and 2025 Mer Bleue Road. The site is situated southeast of the intersection of Innes Road and Mer Bleue Road within the City of Ottawa. The property location is indicated in **Figure 1**. The proposed commercial development comprises approximately 8.35ha of land, of which 0.85ha will be constructed as part of the current Phase 2A of the development. This report incorporates minor infrastructure revisions to the existing Phase 1 development for the purposes of construction of Phase 2A. 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 background studies noted in **Section 2.0**, and as per consultation with City of Ottawa staff.

Figure 1: Location Plan

2.0 BACKGROUND

Documents referenced in preparation of the design for the 4100 Innes Road/ 2025 Mer Bleue Road Commercial Development include:

- Geotechnical Investigation – Proposed Pharand Lands-Commercial Developments – Innes Road, Patersongroup Consulting Engineers, April 24, 2006.
- Geotechnical Investigation – Proposed Commercial Development – Phase 1 – Innes Road at Mer Bleue Road, Patersongroup Consulting Engineers, March 2, 2017.
- Phase 1 Environmental Site Assessment, Proposed Commercial Property, Pharand Lands – Innes Road at Mer Bleue Road, Patersongroup Consulting Engineers, April 28, 2006.
- City of Ottawa Sewer Design Guidelines, City of Ottawa, October 2012.
- City of Ottawa Design Guidelines – Water Distribution, City of Ottawa, July 2010.
- Gloucester and Cumberland East Urban Community Expansion Area and Bilberry Creek Industrial Park Master Servicing Update, Stantec Consulting Ltd., July 2006.
- Pharand Lands, Innes Shopping Centres Limited – City of Ottawa, Stantec Consulting Ltd., February 22, 2012.
- Site Servicing and Stormwater Management Report – Orleans II Development – 2025 Mer Bleue Road – Phase 1, Stantec Consulting Ltd., March 7, 2017.

3.0 WATER SUPPLY SERVICING

3.1 BACKGROUND

The proposed development comprises two commercial buildings, and above ground parking areas. The site is located on the east side of Mer Bleue Road and south of the intersection with Innes Road. Phase 2A will be serviced via 200mm watermain stub constructed as part of Phase 1. The full development will be serviced via existing 300mm watermain stub located approximately 150m within the site originating from Mer Bleue road, and ultimately connecting to the 600mm watermain on Innes Road opposite from Wildflower Drive.

The property is located within the City's Pressure Zone 2E. Proposed ground elevations of the site vary from approximately 88.3m to 89.6m. Under normal operating conditions, hydraulic gradelines vary from approximately 127.3m to 130.8m as confirmed through boundary conditions as provided by the City of Ottawa (see **Appendix A.3**).

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/m^2 of commercial space was used for the proposed site. It is predicted that such 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 (including Phase 2A) was determined to be 0.55 L/s. The maximum daily demand (MXDY) is 1.5 times the AVDY (commercial property), which totals 0.99 L/s. The peak hour demand (PKHR) is 2.8 times the MXDY, totaling 1.48 L/s.

Ordinary construction for buildings J, K, L, M and N and non-combustible construction for building H was considered in the assessment for fire flow requirements according to the FUS Guidelines. Buildings J, K, L, M and H were considered to be fully equipped with an automatic sprinkler system conforming to NFPA 13. Based on calculations per the FUS Guidelines (**Appendix A.2**), the maximum required fire flows for Buildings J, K, L, M N and H are 83 L/s, 67 L/s, 67L/s, 67L/s,100 L/s and 183 L/s respectively.

3.3 HYDRAULIC MODEL RESULTS

A hydraulic model of the water supply system was created by Stantec based on current boundary conditions to assess the proposed watermain layout under the above demands and during fire flow scenarios. Results of the hydraulic modeling demonstrate that adequate flows are available for the subject site, with on-site pressures ranging from **54 psi** to **60 psi** under normal operating conditions. These values are within the normal operating pressure range as defined by MOECC and City of Ottawa design guidelines (desired 50 to 70 psi and not less than 40 psi). Results of the hydraulic model analysis can be found in **Appendix A.4**.

A fire flow analysis was carried out using the hydraulic model to determine the anticipated amount of flow that could be provided for the proposed development under maximum day demands and fire flow requirements per the FUS methodology. Results of the modeling analysis indicate that flows in excess of 11,000L/min (183 l/sec) can be delivered while still maintaining a residual pressure of 140 kPa (20 psi). Results of the hydraulic modeling are included for reference in **Appendix A.4**.

3.4 SUMMARY OF FINDINGS

Based on the results of the hydraulic analysis, the proposed water servicing will provide sufficient capacity to sustain required domestic demands such that normal operating pressures remain within City of Ottawa required limits. The model indicates that this rate can be achieved at all locations while still maintaining the minimum residual pressure per City requirements.

4.0 WASTEWATER SERVICING

4.1 BACKGROUND

Phase 2A of the development will be serviced by the 250mm diameter stub constructed as part of Phase 1, which is serviced via an existing 250mm diameter sanitary sewer installed within an easement that runs parallel to Innes Road, and south of the existing commercial development to the east (see **Drawing SSP-1**). The sewer directs flows to an existing 525mm diameter municipal main at the intersection of Lanthier Drive and Vanguard Drive, and ultimately to the Tenth Line Road pumping station.

For detailed information regarding the wastewater servicing and pump station improvements for the area, please refer to the *Gloucester and Cumberland East Urban Community Expansion Area and Bilberry Creek Industrial Park Master Servicing Update* (Stantec, July 2006).

4.2 DESIGN CRITERIA

As outlined in the City of Ottawa Sewer Design Guidelines and the MOE'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 – 200mm dia. for residential areas, 250mm for commercial areas
- Average Wastewater Generation – 50,000L/ha/day
- Peak Factor – 1.5 (Harmon's) (Commercial)
- 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 gravity sewers which will direct the wastewater flows (approx. 4.5 L/s with allowance for infiltration) to the existing 250mm diameter sanitary sewer. The proposed drainage pattern is detailed on **Drawing SA-1**. A sanitary sewer design sheet for the proposed service lateral is included in **Appendix B.1**. Full port backwater valves are to be installed on all sanitary services within the site to prevent any potential surcharge from the downstream sanitary sewer from impacting the proposed property.

5.0 STORMWATER MANAGEMENT

5.1 OBJECTIVES

The objective of this stormwater management plan is to determine the measures necessary to control the quantity/quality of stormwater released from the proposed development to criteria established by the *Pharand Lands – Innes Shopping Centres Limited – Serviceability Study* Stantec, February 2012) for the region, and to provide sufficient detail for approval and construction.

5.2 SWM CRITERIA AND CONSTRAINTS

Criteria were established by combining current design practices outlined by the City of Ottawa Design Guidelines (2012), and through consultation with City of Ottawa staff. The following summarizes the criteria, with the source of each criterion indicated in brackets:

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 on major & minor drainage system (City of Ottawa).
- Enhanced quality control (80% TSS removal) to be provided on-site for the development.

Storm Sewer & Inlet Controls

- Proposed site to discharge the existing 1350mm diameter storm sewer stub at the intersection of Wildflower Drive and Innes Road at the northern boundary of the subject site (City of Ottawa).
- Proposed storm sewers to be sized to service existing and future commercial/light industrial developments to the south and east of the subject site as per background reports (Pharand Lands Serviceability Study).
- Minor system inflow to be restricted for all contributing areas to 50L/s/ha (Pharand Lands Serviceability Study).
- Minor system inflow for municipal ROW contributing areas to be limited to 100L/s/ha (Pharand Lands Serviceability Study).
- 100-year HGL boundary condition at the site outlet sewer of 81.342m (BCIP Report, Appendix I for node W19).
- 100-year Storm HGL to be a minimum of 0.30 m below building foundation footing (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).
- No overland flow is to be permitted from internal sites to the municipal ROW (Pharand Lands Serviceability Study).
- Sites to provide minimum storage of 200 m^3/ha or sufficient storage to contain 100-year storm event on-site, whichever is greater (Pharand Lands Serviceability Study).
- Road storage to be maximized where possible to provide 130 m^3/ha of storage (Pharand Lands Serviceability Study).
- Maximum depth of flow under either static or dynamic conditions shall be less than 0.35m (City of Ottawa)
- Provide adequate emergency overflow conveyance off-site (City of Ottawa)

5.3 STORMWATER MANAGEMENT

5.3.1 Allowable Release Rate

Based on background information, the peak post-development discharge from the entirety of the development to the minor system is to be limited to 50L/s/ha of contributing area. Peak post-development discharge from municipal Rights-of-Way within the development are to be limited to 100L/s/ha. Peak release rates for the current phase, existing tributary areas and future developments are summarized in **Table 1** below:

Table 1: Target Release Rates

Development Site	Area (ha)	Target Flow Rate (L/s)
Phase 1 - Private	2.718	135.9
Phase 1 – Public	1.294	129.4
Phase 2A & 2B	2.508	125.4
Phase 3	1.826	91.3
Existing (EX104A)	4.205	214.235 (Per CIMA Stormwater Management Report)
Future – Private	11.189	559.5
Future – Public	1.434	143.4
Future External (South of Vanguard Road)	16.830	841.5
Total	**41.464**	**2240.6**

5.3.2 Modeling Rationale

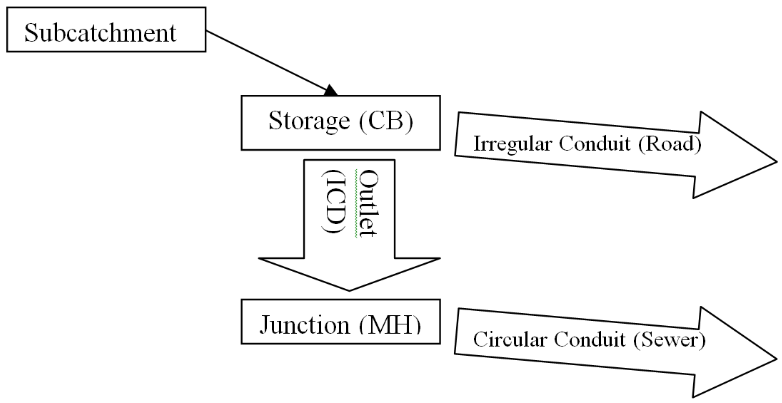
A comprehensive hydrologic modeling exercise was completed with PCSWMM, accounting for the estimated major and minor systems to evaluate the storm sewer infrastructure. The use of PCSWMM for modeling of the site hydrology and hydraulics allowed for an analysis of the systems response during various storm 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
- 3-hour Chicago Storm distribution for the 2, 5, and 100-year analysis with static boundary condition at the site outlet to model 'worst-case' scenario in regards to on-site HGLs.
- To 'stress test' the system a 'climate change' scenario was created by adding 20% of the individual intensity values of the 100-year Chicago 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 x 2 (length of overland flow path measured from high-point to high-point) for street (double-sided) catchments, and by 225 x the subcatchment area otherwise.
- Number of catchbasins based on servicing plan (**Drawing SSP-1**)
- Catchbasin inflow restricted with inlet-control devices (ICDs) as necessary to maintain inflow target rate and maximize use of surface storage.
- Surface ponding in sag storage where no well-defined road cross section exists calculated based on grading plans (**Drawing GP-1**). For storage on roads with defined cross-sections, active storage was modeled based on actual conduit flow using a cross-section as detailed on **Drawing DS-1**.

5.3.2.1 SWMM Dual Drainage Methodology

The proposed subdivision 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 orifice link objects (or outlets) 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 an additional constant depth of 0.45m represent depth of surface water over the storage node (to be limited to a maximum of 0.35m during the 100-year event). The additional 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 diameter and discharge coefficient taken from manufacturer's specifications for the chosen ICD model.

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. All storm events used to demonstrate adherence to SWM target outflow rates for the site were run with a free-flowing outlet condition to be conservative with respect to the maximum expected release rate from the site. Within previous reports for the region, a fixed backwater condition was used at the obvert of the receiving 1800mm diameter sewer within Wildflower Drive. As this elevation is below the invert of the 1350mm sewer outgoing from the subject site, the 1350mm sewer was considered as free flowing for the purposes of this model.

5.3.3 Input Parameters

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

Appendices C2-C3 summarize the modeling input parameters and results for the subject area; an example input and output file are provided for the 100-year 3hr Chicago 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.

5.3.3.1 Hydrologic Parameters

Table 2 presents the general subcatchment parameters used:

Table 2: 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	0.25
Dstore Imperv. (mm)	1.57
Dstore perv. (mm)	4.67
Zero Imperv. (%)	0

Table 3 presents the individual parameters that vary for each of the proposed subcatchments.

Table 3: Subcatchment Parameters

Name	Outlet	Area (ha)	Width (m)	Slope (%)	Imperv. (%)
BLDG_H	BLDG_H-S	0.652	146.7	1.5	100.0
BLDG_J	BLDG_J-S	0.056	16.7	1.5	100.0
BLDG_K	BLDG_K-S	0.057	13.6	1.5	100.0

BLDG_L	BLDG_L-S	0.062	34.9	1.5	100.0
BLDG_M	BLDG_M-S	0.088	45.7	1.5	100.0
BLDG_N	BLDG_N-S	0.038	15.1	1.5	100.0
C100A	100A	0.128	102.5	2	75.7
C101A	101A	0.170	141.6	1.3	77.1
C102A	102A	0.174	146.2	1.1	77.1
C103A	103A	0.164	133.8	1.2	77.1
C104A	104A	0.209	166.0	1	75.7
C105A	105A	0.449	288.7	1.2	62.9
L106A	106A	0.028	27.0	3	100.0
L106B	106A	0.023	24.0	3	100.0
L107A	107	0.023	142.1	2	100.0
L107B	107A	0.144	27.0	3	100.0
L203A	203A	0.299	68.6	1.5	100.0
L203B	203B	0.066	70.7	2	100.0
L203C	203C	0.050	32.1	1.5	100.0
L204A	204A	0.319	70.2	1.5	100.0
L204B	204B	0.372	85.0	1.5	100.0
L205A	205	0.374	81.8	1.5	87.1
L207A	207A	0.162	30.9	3.5	42.9
PHASE_2	201	2.184	269.4	1.5	24.3
U100A	U100A-OF	0.026	5.9	3	32.9
U100B	U100B-OF	0.078	13.6	3	31.4
U100C	U100C-OF	0.123	25.6	2	31.4
TOTAL		**6.521**			**58.0**

Table 4 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 an additional buffer depth to allow for demonstration of overland flow in the climate change event scenario. The buffer is unused during other modeled events. Storage curves noted as 'functional' are set not to provide any additional storage for the node, as storage will occur within the major system conduit (transect) connecting the storage nodes within the model. Storage curves for roof areas are based on roof design spreadsheets included as **Appendix C1**.

Table 4: Storage Node Parameters

Name	Invert	Rim	Depth	Storage Curve	Curve Name
100A	86.25	88.40	2.15	FUNCTIONAL	*
100A-1	88.56	88.66	0.10	FUNCTIONAL	*
101A	86.48	88.73	2.25	FUNCTIONAL	*
101A-1	88.67	88.77	0.10	FUNCTIONAL	*
102A	86.56	88.81	2.25	FUNCTIONAL	*
102A-1	88.69	88.79	0.10	FUNCTIONAL	*
103A	86.66	88.91	2.25	FUNCTIONAL	*
103A-1	88.84	88.94	0.10	FUNCTIONAL	*
104A	86.81	89.06	2.25	FUNCTIONAL	*
104A-1	88.92	89.02	0.10	FUNCTIONAL	*
105A	86.44	89.12	2.68	FUNCTIONAL	*
107A	87.25	89.40	2.15	TABULAR	107
203A	87.30	89.45	2.15	TABULAR	203A
203B	87.30	89.45	2.15	TABULAR	203B
203C	86.50	89.18	2.68	TABULAR	203C
204A	87.30	89.45	2.15	TABULAR	204A
204B	87.30	89.45	2.15	TABULAR	204B
205	86.49	89.45	2.96	TABULAR	205
BLDG_H-S	103.00	103.30	0.30	TABULAR	BLDG_H
BLDG_J-S	103.00	103.30	0.30	TABULAR	BLDG_J
BLDG_K-S	103.00	103.30	0.30	TABULAR	BLDG_K
BLDG_N-S	103.00	103.30	0.30	TABULAR	BLDG_N
BLDG_M-S	103.00	103.30	0.30	TABULAR	BLDG_M
BLDG_L-S	103.00	103.30	0.30	TABULAR	BLDG_L

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 and overland flow corridors representing roadways.

Storm sewers were modeled to confirm flow capacities and hydraulic grade lines (HGLs) in the ultimate condition with consideration of flow contributions from future areas. The detailed storm sewer design sheet is included in **Appendix C1**.

Tables 5 and 6 below present the parameters for the outlet and orifice link objects in the model, which represent ICDs and flow controlled roof drains. All IPEX tempest orifices were assigned a discharge coefficient of 0.572. Roof release discharge curves assume a standard Watts Model R1100 Accuflow controlled release roof drain as noted in the calculation sheets in **Appendix C1**. The number of roof notches are to be confirmed with the building mechanical engineer. Inflows from future undeveloped areas have been modeled as direct constant inflows to the minor system at the given 50L/s/ha or 100L/s/ha inflow rate as applicable. It is assumed that no major system spillage will occur from future areas to those within the Phase 1 and Phase 2A development area during design storm events.

Flows from the northern portion of the site (catchment areas L20X) are also proposed to discharge to the surface at a storm stub east of MH203 and via a 1.0m wide flat bottom channel constructed as part of Phase 1. An inlet pipe at the downstream end of the channel complete with an orifice plate permits ponding within the channel in the undeveloped Phase 2B lands until such a time as they are developed. At such time, the storm sewer inlet will be extended to connect to the stub east of MH203. Storage volume lost through filling of the channel (approx. 720.90m^3) will be converted to a future underground SWM storage facility. It is assumed that any increase in estimated runoff from Phase 2B areas due to increase in imperviousness will be stored on the surface in future ponding areas within the proposed parking lot. Surface ponding within the channel will permit reduction of peak site discharge to the required levels:

Table 5: Orifice Parameters

Name	Inlet	Outlet	Inlet Elev.	Type	Diameter
204A-O	204A	204	87.30	IPEX Tempest	0.076
204B-O	204B	204	87.30	IPEX Tempest	0.076
203A-O	203A	203	87.30	IPEX Tempest	0.076
203B-O	203B	203	87.30	IPEX Tempest	0.076
205-O	205	204	86.79	IPEX Tempest	0.076
105A-O	105A	105	86.44	IPEX Tempest	0.140
107-O	107A	107	87.25	IPEX Tempest	0.108
201-O	201	STC	84.74	CIRCULAR	0.200
201CB-O	203C	201CM	86.50	IPEX Tempest	0.076

Table 6: Outlet Parameters

Name	Inlet	Outlet	Inlet Elev.	Curve Name
100A-O	100A	100	86.25	*
100A-O1	100A	100	86.25	*
101A-O	101A	101	86.48	*
101A-O1	101A	101	86.48	*
102A-O	102A	102	86.56	*
102A-O1	102A	102	86.56	*
103A-O	103A	103	86.66	*
103A-O1	103A	103	86.66	*
104A-O	104A	104	86.81	*
104A-O1	104A	104	86.81	*
BLDG_H_OUT	BLDG_H-S	106	103	BLDG_H_O
BLDG_J_OUT	BLDG_J-S	207	103	BLDG_J_O
BLDG_K_OUT	BLDG_K-S	206	103	BLDG_K_O
BLDG_L_OUT	BLDG_L-S	201CB	103	BLDG_L_O
BLDG_M_OUT	BLDG_M-S	201CB	103	BLDG_M_O
BLDG_N_OUT	BLDG_N-S	208	103	BLDG_N_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 the electronic model files on the enclosed CD.

5.3.4.1 Hydrologic Results

The following tables demonstrate the peak outflow from each modeled outfall during the design storm (3hr Chicago 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 U100A-OF through U100C-OF denote uncontrolled flows from the perimeter of the site that, due to grading restrictions, are captured by the existing rights-of-way at the west and north boundaries of the site. These flows will have a minimal contribution to the infrastructure within Innes/Mer Bleue Road, which ultimately discharge to the Wildflower Drive sewer. Peaks from these uncontrolled flows are non-coincident with peaks from surface storage, and as such, flows from MH 100 are the only values considered in meeting the site target release rate. The required storage volume and orifice for the open channel within the future Phase 2B site was determined through iteration of each event, and sized to mirror site release rate targets per the Pharand Lands report.

Table 7: 3 Hour Chicago Event Peak Discharge Rates

Event	Location	Discharge Rate (L/s)
3 Hr Chicago 2-Year	Outlet Sewer (OF-MJ, OF-MN)	2135
	U100A-OF, U100B-OF	7
3 Hr Chicago 5-Year	Outlet Sewer (OF-MJ, OF-MN)	2168
	U100A-OF, U100B-OF	12
3 Hr Chicago 100-Year	Outlet Sewer (OF-MJ, OF-MN)	2231
	U100A-OF, U100B-OF	31
3 Hr Chicago 100-Year + 20%	Outlet Sewer (OF-MJ, OF-MN)	2259
	U100A-OF, U100B-OF	41

5.3.4.2 Hydraulic Results

Table 8 summarizes the HGL results within the site for the 100 year, 3 hour Chicago storm event 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. HGLs during the climate change event are not to exceed adjacent USF elevations (Freeboard of 0.00m). USF elevations are detailed on **Drawing GP-1**.

Table 8: Modeled Hydraulic Grade Line (HGL) Results

STM MH	Adjacent USF (m)	100-year 3hr Chicago		100-year 3hr Chicago + 20%	
		HGL (m)	USF-HGL Clearance (m)	HGL (m)	USF-HGL Clearance (m)
100	-	82.82	-	82.82	-
101	-	83.09	-	83.09	-
102	-	83.41	-	83.41	-
103	-	84.36	-	84.37	-
104	-	84.50	-	84.50	-
105	-	84.95	-	84.96	-
106	87.25	85.66	1.59	85.67	1.58

STM MH	Adjacent USF (m)	100-year 3hr Chicago		100-year 3hr Chicago + 20%	
		HGL (m)	USF-HGL Clearance (m)	HGL (m)	USF-HGL Clearance (m)
107	-	86.87	-	86.87	-
203	-	86.39	-	86.63	-
204	-	86.52	-	86.64	-
205	-	89.42	-	89.44	-
206	88.55	87.21	1.34	87.69	0.86
207	88.65	87.50	1.17	88.02	0.72
208	88.21	87.31	0.90	87.31	0.90

As is demonstrated in the table above, the worst-case scenario results in HGL elevations remain at least 0.30 m below the proposed underside of footings, and HGL elevations remain below the proposed underside of footing elevations during the 20% increased intensity 'climate change' scenario. HGL elevations, as well as required storage volumes and minor system discharge rates are to be reassessed for the site upon detailed design of the Phase 2B property.

Table 9 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 0.35m 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 buildings.

Table 9: Maximum Surface Water Depths

Storage node ID	Structure ID	Rim Elevation (m)	100 year, 3 hour Chicago		100 year, 3 hour Chicago+20%	
			Max Surface HGL (m)	Total Surface Water Depth (m)	Max Surface HGL (m)	Total Surface Water Depth (m)
100A	CB 100A	88.05	88.09	0.04	88.09	0.04
101A	CB 101A	88.28	88.48	0.20	88.52**	0.24
102A	CB 102A	88.36	88.57	0.21	88.70**	0.34
103A	CB 103A	88.46	88.72	0.26	88.76**	0.30
104A	CB 104A	88.61	88.89	0.28	88.92**	0.31
105A	EX CB	88.77	89.01	0.24	89.02	0.25
107A	CB 107A	89.05	89.26	0.21	89.27	0.22

Storage node ID	Structure ID	Rim Elevation (m)	100 year, 3 hour Chicago		100 year, 3 hour Chicago+20%	
			Max Surface HGL (m)	Total Surface Water Depth (m)	Max Surface HGL (m)	Total Surface Water Depth (m)
203A	CB 203A	89.10	89.36	0.26	89.42	0.32
203B	CB 203B	89.10	89.26	0.16	89.26	0.16
203C	CBMH 201	88.83	88.91	0.08	88.94	0.11
204A	CB 204A	89.10	89.42	0.32	89.44	0.33
204B	CB 204B	89.10	89.37	0.27	89.43	0.33
205	CBMH 205	89.10	89.42	0.32	89.44	0.34

**Adjacent Phase 2B and Future development areas to be designed to ensure no impacts on adjacent proposed building opening elevations.

5.3.5 Results

Table 10 demonstrates that the proposed stormwater management plan provides adequate attenuation storage to meet the target peak outflow rates for the site.

Table 10: Summary of Total 5 and 100 Year Event Release Rates

Storm Event	Site Peak Discharge (L/s)*	Target Discharge Rate (L/s)*
3 Hour Chicago 2-Year	2135.4	2240.6
3 Hour Chicago 5-Year	2167.6	2240.6
3 Hour Chicago 100-Year	2230.7	2240.6
3 Hour Chicago 100-Year+20%	2259.0	-

*Includes discharge from future, uncontrolled and external drainage areas.

5.4 WATER QUALITY CONTROL

On-site quality control measures are expected for the proposed redevelopment per section 3.1.2.5.2 of the Gloucester Cumberland EUC & BCIP Servicing Update. It is assumed that enhanced protection (80% removal of suspended solids) will be required for the site similar to existing areas of the BCIP. As a result, an oil grit separator has been installed on-site as part of Phase 1 to treat runoff from impervious areas directed to catchbasins on-site. The oil-grit separator unit is privately maintained, and located as shown on **Drawing SSP-1**. The OGS unit was been modeled in the PCSWMM for Stormceptor model provided by the manufacturer (Imbrium Systems) to validate whether the unit is appropriate for the area. The OGS unit was sized to ensure that should the area noted as Phase 2B be proposed to be paved in a future site plan submission that the OGS design will be adequate based on the increased impervious area to the unit. The analysis included as part of **Appendix C.4** indicates that the unit provides a minimum of 80% TSS removal for the site, meeting water quality objectives for the downstream Bilberry Creek. The downstream SWMF additionally provides quality control to normal protection criteria (70% TSS removal). As the majority of impervious surfaces are directed to the on-site OGS unit, suspended solids within runoff generated by the site are not anticipated to have a deleterious impact on downstream watercourses.

6.0 GRADING AND DRAINAGE

The proposed development phase measures approximately 0.85 ha in area. The topography across the site is relatively flat, and currently drains from west to east. 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 maintains emergency overland flow routes for flows deriving from storm events in excess of the maximum design event to the proposed municipal rights-of-way at the southern and eastern boundaries of the development, and ultimately to Innes Road as depicted in **Drawing GP-1.** Future areas to the south and east of the development are anticipated to maintain overland flow routes to the future Vanguard Road extension.

7.0 UTILITIES

As the subject site is bound to the east and west by an existing commercial business park, and by a municipal right-of-way to the north, Hydro, Bell, Gas and Cable servicing for the proposed development should be readily available. Pole mounted Hydro infrastructure and a gas main exist along Mer Bleue 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

Environmental Compliance Approval (ECAs, formerly Certificates of Approval (CofA)) under the Ontario Water Resources Act are not expected to be a requirement for the current phase as approval was obtained for storm and sanitary sewer usage within the north-south and east-west shared access routes as a part of Phase 1 of the development, the current property is of a single parcel of non-industrial use, and discharges to approved sewer stubs constructed as part of Phase 1 with the intent of development of the current phase.

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.

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

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

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

10.0 GEOTECHNICAL INVESTIGATION AND ENVIRONMENTAL ASSESSMENT

A geotechnical Investigation Report was prepared by Patersongroup dated March 2, 2017. The report summarizes the existing soil conditions within the entirety of the development 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 7 boreholes and 11 test pits (probeholes) distributed across Phase 1, and 4 test pits across Phase 2A. In general soil stratigraphy consisted of topsoil underlain by a silty clay crust layer, followed by a grey silty clay deposit. Bedrock/inferred bedrock elevations range from depths of 0.3 to 7.6m below ground surface. Groundwater Levels were measured in November 2016, and vary in elevation from 0.97m to 2.13m below ground surface. Three potable wells (one dug and two drilled) exist within the proposed site, and are to be abandoned prior to construction per Ontario Regulation 903.

Grade raise fill restriction recommendations for grading within 6m of a building footprint was identified as 2m, and a 2.5m raise in parking areas and access lanes.

The required pavement structure for proposed hard surfaced areas are outlined in **Table 11** and **12** below:

Table 11: 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
400	Subbase - OPSS Granular B Type II
-	Subgrade – Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or bedrock.

Table 12: 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 or OPSS Granular B Type I or II material placed over in situ soil or bedrock.

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 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 development will be serviced by a network of gravity sewers which will direct wastewater flows to the existing 250mm dia. sanitary sewer stub constructed as part of Phase 1. The proposed drainage outlet to the east has sufficient capacity to receive sanitary discharge from the site based on the findings of the Gloucester and Cumberland EUC Master Servicing Update.

11.3 STORMWATER SERVICING

The proposed stormwater management plan is in compliance with the goals specified through consultation with the City of Ottawa. On-site catchbasins and connected ICDs have been proposed to limit peak storm sewer inflows to downstream storm sewers to 50L/s/ha for privately owned areas and 100L/s/ha for municipal ROWs as determined by background reports. The downstream receiving sewer has sufficient capacity to receive runoff volumes from the site based on the findings of the Gloucester and Cumberland EUC Master Servicing Update.

11.4 GRADING

Grading for the site has been designed to provide an emergency overland flow route as per City requirements and reflects the recommendations made in the Geotechnical Investigation Report prepared by Patersongroup. Erosion and sediment control measures will be implemented during construction to reduce the impact on existing facilities.

11.5 UTILITIES

Utility infrastructure exists within overhead lines within the Mer Bleue Road ROW at the western boundary of the proposed site. It is anticipated that existing infrastructure will be sufficient to provide a means of distribution for the entirety of the development. Exact size, location and routing of utilities will be finalized after design circulation.

11.6 APPROVALS/PERMITS

An MOECC Environmental Compliance Approval is not expected to be required as approval was obtained for the receiving storm and sanitary sewers as part of Phase 1. No other approval requirements from other regulatory agencies are anticipated.