MINTO COMMUNITIES INC. & MATTAMY HOMES LTD.

BARRHAVEN SOUTH URBAN EXPANSION AREA

EXISTING CONDITION REPORT

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EXECUTIVE SUMMARY

Minto Communities Inc. (Minto) and Mattamy Homes (Mattamy), herein referred to as the Owner, have retained a Consultant Team to prepare a series of comprehensive documents to support the Barrhaven South Urban Expansion Area (BSUEA) Community Design Plan (CDP), which will be implemented as an Amendment to the City of Ottawa Official Plan (OP). J.L. Richards & Associates Limited (JLR) has been retained to prepare a Master Servicing Study (MSS) to outline water, wastewater, storm, and stormwater management servicing strategies for the BSUEA. In advance of preparing the MSS, an Existing Conditions Report is required to evaluate and assess existing water resources and servicing infrastructure in and around the BSUEA, and to identify constraints and opportunities that will provide the baseline conditions of an Environmental Management Plan (EMP).

The BSUEA consists of 121 ha of land that is bounded by Cambrian Road to the north and Barnsdale Road to the south. The BSUEA is located immediately west of the current City of Ottawa Urban Boundary and approximately 360 m east of Borrisokane Road. The subject site is an undeveloped mix of former agricultural land and forested areas spread generally across the southern portion of the site. It contains two (2) ongoing aggregate extraction operations in the northern area of the site, extending west to the Borrisokane Road / Highway 416 corridor. Woodlots and pasture areas are adjacent to the site to the north, and recent urban development abuts the eastern boundary. The topography across the site has significant undulations and there is a knoll in the BSUEA located approximately 1 km north of Barnsdale Road. At the intersection of Borrisokane Road and Barnsdale Road, southwest of the site, there is an existing waterbody that appears to be man-made.

Based on interpreting the LiDAR data and the topographical survey compiled by Stantec (Geomatics), the BSUEA is tributary to three (3) sub-watersheds: the Jock River, Mud Creek via the Thomas Baxter Municipal Drain, and the Rideau River via the Hawkins Municipal Drain. The investigations completed by the Team's biologist did not identify any formal conveyance channels within the BSUEA. The absence of channels or formal watercourses was confirmed by the LiDAR data. The absence of such waterways is generally indicative of areas exhibiting particularly low surface runoff potential.

Catchment areas within the BSUEA have been delineated based on LiDAR data compiled on behalf of the City of Ottawa, as well as any surface infrastructure such as roads, which tend to

form a drainage path barrier, and culverts, where flow will concentrate into a point source. An understanding of the existing conditions in terms of capacity of the culverts and expected flow rates can be achieved from an accurate existing conditions hydrological model.

The SWMHYMO hydrological model for the BSUEA is a single event model for the purpose of estimating flood flows at the boundary control points of the site for storm events ranging from the 1:2 year and 1:100 year recurrence. The model was developed based on generally accepted standard hydrological parameters that fairly represent the land use, soil characteristics, types of flow paths, and topography. The simulated area included the BSUEA, as well as the lands up to the external boundary points: Cambrian Road, Borrisokane Road, and Barnsdale Road.

The SWMHYMO model results show relatively low flow rates at each of the culverts due to the high infiltration capability of the soils in the area and the low impervious ratio. In general, during larger and infrequent events, runoff will increase as the infiltration capability decreases due to saturation of the soils.

The water budget of a given area is often studied by hydrologists and hydrogeologists to better understand the movement of water through its various stages. Natural watershed systems maintain a balance between precipitation, runoff to water bodies, infiltration to the groundwater system, evaporation from water surfaces, or transpiration from vegetation, completing the natural cycle back into atmospheric moisture and precipitation. The analysis of the water budget is often carried out to establish the baseline condition and assess any future changes in infiltration to subsurface water-bearing zones and, consequently, to surface runoff resulting from a significant land cover modification such as urbanization. It is also used to predict changes to the hydrological cycle that will result from urbanization of lands.

An understanding of the water budget within a given study area can be gained through the use of a continuous hydrological model. The PCSWMM software platform was used for this study, which included simplified groundwater and snowmelt modules that allowed the continuous simulation of the water budget, including the elements of evapotranspiration, the groundwater table (lower saturated zone), snowfall, and snowmelt.

A long term simulation was carried out for the site and was based on 30 years of hourly precipitation and temperature. Based on the long-term simulation results, infiltration accounts for approximately 40% of the total annual precipitation of 844 mm, while surface runoff accounts for approximately 0% on a yearly average. Specifically, surface runoff accounts, on average, for 1 mm per year out of the total annual precipitation of 844 mm.

A review of the existing and planned infrastructure (sanitary sewers, watermains, storm sewers, and stormwater management facilities) in the vicinity of the BSUEA was undertaken to assess available servicing opportunities and constraints for the BSUEA.

The existing Greenbank Road trunk sanitary sewer was identified as the wastewater outlet for the existing Barrhaven South Community sanitary sewer system, which includes the Cambrian Road and River Mist Road trunk sanitary sewers. Based on the theoretical peak flow calculations, the existing Greenbank Road trunk sanitary sewer has sufficient capacity to accommodate the anticipated peak flows generated within the BSUEA. Furthermore, the total peak flow at downstream MH 522A (located at the intersection of Half Moon Bay Road and the existing Greenbank Road, immediately south of the Jock River) is anticipated to be less than 590 L/s. This was made a requirement by Stantec based on their 2013 Wastewater Collection System Assessment, which concluded that downstream wastewater infrastructure, including the South Nepean Collector and the West Rideau Collector, could accommodate a peak flow of 590 L/s from the Barrhaven South Community and the BSUEA, with no risk of surcharging and/or basement flooding. Measures to mitigate infiltration into the sanitary sewer system in the BSUEA are anticipated based on existing sewer and groundwater elevations.

Potable water will be provided to the BSUEA predominantly via extensions of existing and planned feedermains located along the existing Greenbank Road, Cambrian Road, River Mist Road, the future realigned Greenbank Road and a future Collector Road. Findings presented in the City of Ottawa Water Supply Optimization Study Report prepared by Delcan in 2008 recommended a Pressure Zone reconfiguration consisting of an expanded Zone 3C within areas previously identified as Pressure Zone BARR, including the Barrhaven South Community and the future BSUEA. Several water distribution projects were identified in the 2013 Infrastructure Master Plan to achieve the recommended zone reconfiguration. Certain growth areas, including the BSUEA, were identified as having low pressures notwithstanding the full zone reconfiguration. Hydraulic simulations showed that pressures less than 40 psi are anticipated along the future water distribution system servicing the elevated portion of the BSUEA. As a result, it is anticipated that a localized portion of the BSUEA may require a booster pumping station or, alternatively, oversized services combined with jet pumps. This condition, as well as water servicing specifics such as watermain looping and sizing of the feedermains for the BSUEA, will be addressed at the MSS stage by a hydraulic network analysis (HNA).

The existing Barrhaven South Community is currently serviced by two (2) stormwater management facilities (SWMFs) known as the Corrigan Pond and the Todd Pond. Three (3) additional SWMFs are planned to service the Barrhaven South Community in accordance with the Jock River Subwatershed Study and the Barrhaven South Master Servicing Study. These planned SMWFs are referred to as the Clarke Pond, Greenbank Pond and Cedarview Pond. Although all five (5) existing/planned SWMFs in Barrhaven South have residual water quality volumes and have available residual capacity in their associated trunk infrastructure, all existing and planned storm sewer systems in Barrhaven South have relatively high hydraulic grade line elevations. The Corrigan and Todd Pond tributary areas also have major system ponding depths which are generally at or exceeding the maximum allowable depth of 0.35 m.

Given the limitations of the existing minor and major systems in the Barrhaven South Community, opportunities to provide stormwater servicing and stormwater management for the BSUEA via the three (3) planned SWMFs (Clarke, Cedarview and Greenbank) and their associated trunk storm sewers, will be reviewed in detail and evaluated as part of the BSUEA Master Servicing Study. Additionally, the servicing of a 5.6 ha parcel of land owned by Mattamy via the Todd Pond will be further reviewed. This parcel was previously allocated to the Todd Pond under an ‘ultimate’ scenario as part of the 2015 Clarke Pond Model Keeper Analysis completed by JFSA. Should a new wet pond facility be required, then the naturally low-lying topography at the southeastern corner of the Study Area would be the optimal location. This pond could provide both stormwater quantity and quality control for the BSUEA before outletting to the existing Greenbank Road trunk storm sewer.

B1.0 INTRODUCTION

Minto Communities Inc. (Minto) and Mattamy Homes (Mattamy), herein referred to as the Owner, have retained a Consultant Team to prepare a series of comprehensive documents to support the Barrhaven South Urban Expansion Area (BSUEA) Community Design Plan (CDP), which will be implemented as an Amendment to the City of Ottawa Official Plan (OP). J.L. Richards & Associates Limited (JLR) has been retained to prepare a Master Servicing Study (MSS) to outline water, wastewater, storm, and stormwater management servicing strategies for the BSUEA. In advance of preparing the MSS, an Existing Conditions Report is required to evaluate and assess existing water resources and servicing infrastructure in and around the BSUEA, and to identify constraints and opportunities that will provide the baseline conditions of an Environmental Management Plan (EMP).

B2.0 STUDY AREA

B2.1 Location

The BSUEA consists of 121 ha of land that is bounded by Cambrian Road to the north and Barnsdale Road to the south. The BSUEA is located immediately west of the current City of Ottawa Urban Boundary and approximately 360 m east of Borrisokane Road, as shown in **Figure 2-1**.

The BSUEA CDP area shown in **Figure 2-1** represents the latest boundary limits. Although there has been no change to the overall CDP area of 121 ha, prior to these limits being established under a provisional agreement with the City of Ottawa, the boundary limit along the existing Greenbank Road was further west, approximately following an extension of River Mist Road. A corresponding area on the west side was then included, following the boundary of property PIN 045921577. The new CDP boundary, as presented in **Figure 2-1**, will allow for a more logical development pattern, with improved phasing opportunities and more efficient connection to surrounding development.

Figure 2-1: BSUEA Location Plan

B2.2 Characteristics

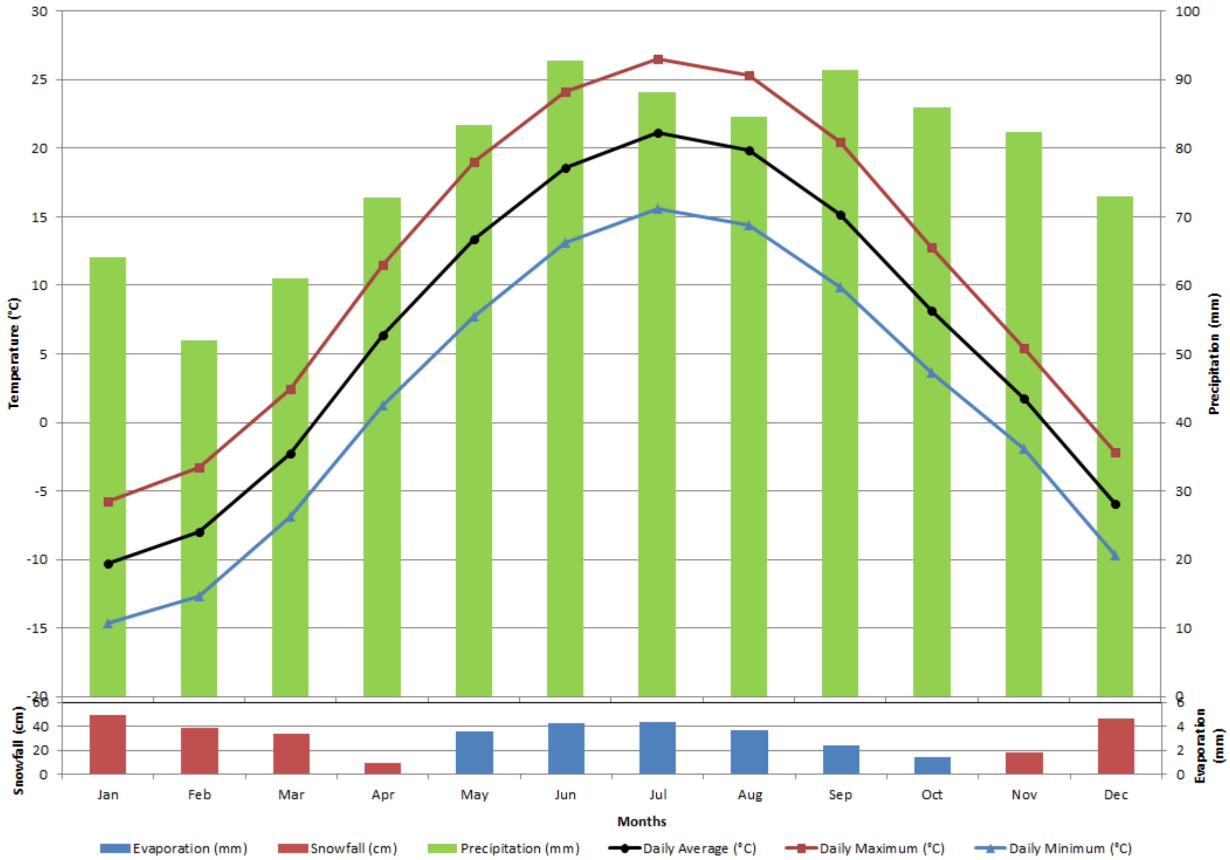
The subject site is an undeveloped mix of former agricultural land and forested areas. It contains two (2) ongoing aggregate extraction operations (Brazeau and Drummond). The southern portion of the site is generally former agricultural land, with some woodlot areas. The aggregate extraction operations are in the northern area of the site, extending west to the Borrisokane Road / Highway 416 corridor. Woodlots and pasture areas are adjacent to the site to the north, and recent urban development abuts the eastern boundary. The topography across the site has significant undulations, with elevations generally varying from approximately 111 m at the peak to approximately 99 m at the lowest point. However, there is also a knoll in the BSUEA south of the Brazeau lands where the existing ground is above 111 m. At the intersection of Borrisokane Road and Barnsdale Road, southwest of the site, there is an existing waterbody that appears to be man-made.

B2.3 Climate

Environment Canada collects meteorological data at two (2) weather stations within the vicinity of the study area: the Ottawa CDA at the Experimental Farm (16 km to the northeast) and the Ottawa MacDonald Cartier International Airport (10 km to the northeast). The averages of the data collected at these two (2) stations provide some information on the climate expected in the study area (**Figure 2-2**).

The average temperature for the BSUEA varies between about -10°C in the winter and over 20°C in the summer. The daily maximum and minimum averages vary between 3.7 and 5.5 degrees from the average, with the greatest difference observed in the summer months. Evaporation is high in the summer, and negligible in the winter. Snowfall is highest during the December/January period. Generally, precipitation volumes are higher in the summer months, although the season variation is not as pronounced compared to temperature: the seasonal differences are usually no more than 40 mm and for three quarters (75%) of the year the variation in precipitation is no more than 20 mm.

Figure 2-2: Environment Canada, Ottawa Climate Normals



B3.0 BACKGROUND DOCUMENTS

There are a large number of documents and reports that have been prepared regarding the lands covered by the BSUEA and/or lands surrounding the site. The documents include subwatershed studies of the Jock River, covering the areas north of the site, and Mud Creek, covering the south of the site, and servicing strategies for the surrounding urban development lands northeast of the site.

The Reports listed below have been summarized in **Appendix 'B1'**. The Reports can be classified in the following four (4) categories:

1. Reports covering a large geographical area, including the site:

- *Jock River Reach One Subwatershed Study Final Report*. Stantec Consulting Ltd. (June 2007). City of Ottawa.
- *Jock River Subwatershed Report 2010: A report on the environmental condition of the Jock River*. Rideau Valley Conservation Authority. (April 2011).

- *Mud Creek Subwatershed Study*. City of Ottawa, Planning and Growth Management. (October 2015).
- *Mud Creek Subwatershed Existing Conditions Report Phase I Summary*. City of Ottawa Land Use and Natural Systems Policy Group. 2011.
- *Jock River Reach 2 & Mud Creek Subwatershed Study Existing Conditions Report (Final)*. Marshall Macklin Monaghan Limited and Water & Earth Sciences Associates. (May 2009). City of Ottawa.
- *Lower Rideau River Subwatershed Report 2012: A report on the environmental condition of the Lower Rideau River*. Rideau Valley Conservation Authority. (February 2013).
- *Lower Rideau Watershed Strategy Final Report*. Aquafor Beech / Robinson Consultants Inc. (September 2005). Rideau Valley Conservation Authority.
- *Barrhaven South Official Plan Amendment Submission*. David McManus Engineering Ltd. (August 2008). Minto Communities Inc.
- *Aggregate Resource Designated Lands Study, 3882 Barnsdale Road, Ottawa, Ontario*. Paterson Group Inc. (February 2012). Minto Communities Inc.

2. Site Servicing Strategies for adjacent urban development sites:

- *Barrhaven South Master Servicing Study Final Report*. Stantec Consulting Ltd. (June 2007). City of Ottawa.
- *2013 Infrastructure Master Plan Wastewater Collection System Assessment.* Stantec Consulting Ltd. (September 17, 2013). City of Ottawa.
- *City of Ottawa 2013 Water Master Plan.* Stantec Consulting Ltd. (September 20, 2013). City of Ottawa.
- *Infrastructure Master Plan*. City of Ottawa. (November 2013).
- *Barrhaven South Master Servicing Study Addendum Draft Report*. Stantec Consulting Ltd. (November 2014). City of Ottawa.
- *Barrhaven MSS Draft Addendum* – Revised Potable Water and Sanitary Servicing Analysis for the BSUEA. Stantec Consulting Ltd. (October 19, 2016).
- *Manotick Master Drainage Plan Phase I*. A.J. Robinson and Associates Inc. (October 1993). Township of Rideau.
- *Manotick Master Drainage Plan Phase II*. Robinson Consultants Inc. (February 1996). Township of Rideau.

3. Surrounding Infrastructure Reports:

- *Corrigan Stormwater Management Facility Stormwater Management Report and Design Brief*. IBI Group (July 2010). McNeil Farm Limited.

- Stormwater Management Barrhaven South. IBI Group (October 2014). Minto Communities Inc.
- Letter Report (June 9, 2015) entitled “Respond to the City of Ottawa Comment – Corrigan SWM Facility” addressed to J.L. Richards & Associates Limited.

- *Todd Pond and Clarke Pond Model Keeper Update Analysis / Update of Half Moon Bay South Subdivision Plan. Letter to David Schaeffer Engineering Limited*. J.F. Sabourin and Associates Inc. July 25, 2013.
- *Todd Pond Model Keeper Analysis Re-Assessment of Existing System Capacity*. J.F. Sabourin and Associates Limited. (April 2015). City of Ottawa.
- *Site Servicing Report Quinn’s Pointe 3872 Greenbank Road*. J.L. Richards & Associates Limited. (July 2015). Minto Communities Inc.
- *Thomas R. Baxter Municipal Drain Engineers Report*. Graham, Berman & Associates Ltd. (May 1967). Township of Nepean.

4. Other background documents:

- *Final Geotechnical Investigation Proposed Barrhaven South Community 3872 Greenbank Road Ottawa*. Paterson Group Inc. (July 2014). Minto Communities Inc.
- *Supplementary Geotechnical Assessment Proposed Quinn’s Pointe Development Barrhaven South 3872 Greenbank Road – Ottawa. Letter to Minto Communities Inc*. Paterson Group Inc. May 19, 2015.
- *Geotechnical Recommendations Greenbank Road Trunk Services Extension Quinn’s Pointe Development - Ottawa. Letter to Minto Communities Inc*. Paterson Group Inc. July 23, 2015.
- *Summary of the Jock River Reach 2 Subwatershed Existing Conditions*. City of Ottawa Land Use and Natural Systems Policy Group. (2011).

B4.0 GEOLOGY AND PHYSIOGRAPHY

A geotechnical investigation was carried out by Paterson Group Inc. (Paterson) to assess general subsoil and groundwater conditions within the BSUEA and to provide geotechnical recommendations for the design of municipal services associated with the proposed development. A field program, consisting of 30 boreholes and 125 test pits, was carried out in November and December 2015. A supplemental program followed in July 2016 with 6 boreholes and 36 permeameter tests. The boreholes/test pits were advanced to depths ranging from 2.6 m to 9.8 m below existing ground surface. The test hole locations for the current investigation were distributed in a manner to provide general coverage of the BSUEA and adjacent properties. Previous investigations had been carried out in 2003 and 2011, which consisted of 27 test pits that were advanced to depths ranging from 2.7 m to 7.0 m. Detailed findings and recommendations have been compiled in the Report entitled “Geotechnical Investigation Proposed Barrhaven South Urban Expansion Area - Community Design Plan, Paterson Group Inc., January 26, 2017. The general findings of the above-noted Report are summarized in Sections B4.1 to B4.3 (below).

B4.1 Surficial Geology

Mapping reviewed by the geology team indicated that overburden soils at the site consist of primarily glaciofluvial deposits to the north with reworked glaciofluvial sand deposits to the west and southwest. The results of the sub-surface investigations were generally consistent with the mapping. Overburden thicknesses were found to be around 10 m to 17 m across the site. The overburden generally consisted of topsoil over silty sand to a sandy silt deposit.

B4.2 Bedrock

Based on digital geological mapping produced by Natural Resources Canada, sourced from the Geological Survey of Canada, the bedrock in this area consists of dolomite of the Oxford formation with an overburden drift thickness of 15 m to 25 m depth. Bedrock was not encountered in any of the boreholes or test pits ordered across the site.

B4.3 Aquifer Systems and Groundwater

Aquifer systems are geological material which permits the movement of groundwater under hydraulic gradients. The overburden soils at the site are, according to the geotechnical report, relatively deep and consist of moderate to high hydraulic conductivities. Three (3) aquifer systems are present across the site: the underlying bedrock aquifer, the deeper overburden aquifer, and a shallower overburden aquifer.

Monitoring wells were installed in the overburden to assess the hydrogeological characteristics across the site. The results of the testing suggest that the permeable overburden aquifer is acting as a recharge area for the esker and bedrock aquifers. The saturated conditions are considered by the geotechnical team to be representative of the long term water table and will vary in level seasonally and with precipitation events.

Groundwater levels were found to be generally highest in the centre of the site with groundwater elevations of up to 101 m above sea level. However, the analysis by Paterson found that, when compared to potential grading on the site, groundwater levels could be within 3.5 to 4.5 m of the surface at the southeastern corner of the site around the existing Greenbank Road / Barnsdale Road intersection.

B5.0 SITE HYDROLOGY AND HYDRAULICS

B5.1 General

Based on interpreting the LiDAR data and the topographical survey compiled by Stantec (Geomatics), the BSUEA is tributary to three (3) sub-watersheds: the Jock River, Mud Creek via the Thomas Baxter Municipal Drain, and the Rideau River via the Hawkins Municipal Drain. The area tributary to the above-noted sub-watersheds has been depicted in Figure 5-1. Based on the study area limits, approximately 99 ha is tributary to the Jock River while areas of approximately 11 ha and approximately 12 ha are tributary to Mud Creek and Rideau River, respectively. The investigations completed by the Team's biologist did not identify any formal conveyance channels within the BSUEA. The absence of channels or formal watercourses was confirmed by the LiDAR data. The absence of such waterways is generally indicative of areas exhibiting particularly low surface runoff potential. As a result, runoff generated from the site is carried as overland sheet flow to the natural low points at the culvert outlet locations.

B5.2 Water Crossings

A total of thirteen culvert crossings were inventoried, which can be grouped into the following two (2) categories:

1. Boundary: There are five (5) culvert crossings acting as boundary control points to the area being modelled. Two (2) are located along Cambrian Road to the north of the BSUEA and three (3) are along Barnsdale Road to the south of the BSUEA. These five (5) crossings form the downstream boundary of the modeled area, where overland sheet flow forms a concentrated point flow external to the site;
2. Internal to the model: There are eight (8) culvert crossings that are internal to the area being modelled. These crossings currently provide conveyance across access routes to property and farmland along Borrisokane Road and Barnsdale Road.

The inventoried crossings have been summarized in **Table 5-1** and **Table 5-2**, below, while **Figure 5-2** depicts the location of the culvert crossings along with the general site hydrology. It should be noted that none of the inventoried water crossings are within the BSUEA limits, although BDR-C5 is on the boundary of the BSUEA.

Figure 5-1: Study Area and Drainage Divide

Table 5-1: Inventory of Model Boundary Water Crossings

Culvert ID	Location	Type	Size (mm)
CR-C1	On Cambrian Road, 910 m east of Borrisokane Road, carries Clarke West Municipal Drain	Circ. CSP	1650
CR-C2	On Cambrian Road at Borrisokane Road	Circ. CSP	N/A
BDR-C4	On Barnsdale Road, 50 m west of Borrisokane Road	Circ. CSP	1200
BDR-C5	On Barnsdale Road, 500 m west of the existing Greenbank Road	Circ. CSP	500
BDR-C6	On Barnsdale Road, 60 m west of the existing Greenbank Road	Circ. CSP	400

It should be noted that culvert CR-C2 was not included as part of the topographical survey and size is currently unknown.

The 2014 Barrhaven South Master Servicing Study Draft Addendum (Draft 2014 BSMSSA) prepared by Stantec, notes that water crossing CR-C1 is to be replaced with storm sewers when the Clarke West Municipal Drain is enclosed as part of the adjacent development and the Clarke Stormwater Management Facility is constructed. The Draft 2014 BSMSSA also indicated that culvert CR-C2 is to be maintained, and will accommodate flows from the existing catchment area south of Cambrian Road up to the 1:100 year event. Should future development occur south of the woodlot draining to CR-C2, grading and servicing from the future development area in the vicinity of the woodlot should be developed to maximize overland sheet flow drainage (not channelized) towards the woodlot.

Table 5-2: Inventory of Model Water Crossings (Internal)

Culvert ID	Location	Type	Size (mm)
CVR-C1	East of Borrisokane Road along the north corner of the BSUEA	Circ. CSP	500
CVR-C2	East of Borrisokane Road at Field Entrance	Circ. CSP	450
CVR-C3	East of Borrisokane Road at Field Entrance	Circ. CSP	400
CVR-C4	Borrisokane Road Crossing north of Barnsdale Road	Circ. CSP	1200
BDR-C1	Viewbank Road Crossing	Circ. CSP	400
BDR-C2	Field Entrance Crossing South of Barnsdale Road	Circ. CSP	400
BDR-C3	Field Entrance Crossing South of Barnsdale Road	Circ. CSP	500
BDR-C7	Barnsdale Road Crossing close to the existing Greenbank Road Intersection	Circ. CSP	500

Table 5-2, above, summarizes the various culvert crossings within the BSUEA. As shown above, all the culverts are 500 mm in diameter or less with the exception of CVR-C4, which is 1200 mm in diameter.

Figure 5-2: SWMHYMO Site Drainage Plan

B5.3 Watershed Delineation

Catchment areas within the BSUEA have been delineated based on LiDAR data compiled on behalf of the City of Ottawa, as well as any surface infrastructure such as roads, which tend to form a drainage path barrier, and culverts, where flow will concentrate into a point source. The delineated catchment areas are split by the three (3) watershed areas: Jock River to the north of the site, Mud Creek to the south and the Rideau River watershed to the east. The catchment areas are summarized in Table 5-3 and are shown in Figure 5-2.

Table 5-3: Subcatchment Summary

Watershed	Downstream Culvert	Catchment ID	Area (ha)
Jock River	CR-C2	A1	18.6
	CR-C1	A2	35.4
		A3	12.7
		A4	13.9
		A6	11.7
		A7	15.5
	CVR-C1	A5	12.7
		A8	42.2
	Overland flow	A9	5.1
		A13	24.6
		A14	22.4
		Subwatershed Total	**214.8**
Mud Creek	CVR-C2	A10	5.5
	CVR-C3	A11	15.2
	CVR-C4	A12	60.0
	BDR-C4	A15	3.5
		Subwatershed Total	**84.2**
Rideau River	BDR-C5	A16	2.9
	BDR-C7	A17	9.8
		Watershed Total	**12.7**
		Total	**311.7**

B5.4 Hydrological Modeling

Since the 1980s, hydrological models have become the principal tool used to calculate flood flows under extreme conditions and assess flooding impacts along water bodies and within urban areas.

An understanding of the existing conditions, in terms of capacity of the culverts and expected flow rates, can be achieved from an accurate existing conditions hydrological model. To simulate the hydrological response of a given area, a hydrologic model should ideally be calibrated to actual monitored flows to ensure a true representation of the physical characteristics of the catchment area during the recorded storm events. For this project, surface flows were not monitored within the limits of the BSUEA and, consequently, the hydrological model was not calibrated. The model was developed based on generally accepted standard hydrological parameters that fairly represent the land use, soil characteristics, types of flow paths, and topography.

The SWMHYMO hydrological model for the BSUEA is a single event model for the purpose of estimating flood flows at the boundary control points of the site for storm events ranging from the 1:2 year and 1:100 year recurrence. The simulated area included the BSUEA, as well as the lands up to the external boundary points: Cambrian Road, Borrisokane Road and Barnsdale Road. Figure 5-2 shows the extent of the simulated areas and the drainage areas delineated for the project.

B5.4.1 Aggregate Extraction Pits

Parts of the BSUEA continue to be used for aggregate extraction. These existing aggregate extraction pits act as large basins and will contain runoff on site until the trapped stormwater is lost to either infiltration or evaporation, or is pumped off site. This runoff condition is the result of man-made activities: originally, runoff would have left the site and contributed to flows at the boundary culvert. As a result, the pre-development condition model for the site was developed assuming the pre-extraction condition based on topography as per the plans submitted to the Ministry of Natural Resources and Forestry (MNRF), where available, or as a constantly sloped surface to the surrounding natural ground level where the original topography is unknown.

B5.5 Model Parameters

The site was modeled using the SCS Curve Number (CN) methodology to predict runoff. The CN number is selected based on the soil types in the area and the land cover, which in this case is predominately rural in nature with either agricultural or woodlot classifications. The site consists of ‘unimproved lands’, which consist of lands that are undeveloped open space with no current use. This classification was also used for the quarry areas. Land uses for the model area are shown in Figure 5-3, while a summary of the hydrological parameters is shown in Table 5-4.

Table 5-4 - Model Parameter Summary

Drainage Area ID	Area (ha)	CN*	Initial Abstraction (mm)	Time to Peak (hrs)	% Impervious
A1	18.6	52	23.7	0.41	
A2	35.4	49	17.9	0.59	
A3	12.7	71	8.7	0.43	
A4	14.0	47	27	0.33	
A5	12.7	49	17.3	0.39	
A6	11.7	50	15.6	0.27	
A7	15.5	52	16.7	0.72	
A8	42.3	49	15.8	0.74	
A9	5.1	49	17.9	0.25	
A10	5.5	50	14.7	0.27	
A11	15.2	56	14.2	1.27	
A12	60.0	51	17.2	0.95	
A13	24.6	48	16.4	0.5	
A14	22.4	40	24.8	1.5	
A15	3.5	67	11.1	0.39	
A16	2.9	37	26.9	0.24	
A17	9.8	44	21.1	0.42	
A18	0.5	69	9.4	See Note 1	26%
A19	1.0	76	10.2	See Note 1	20%
A20	4.8	49	18.6	0.32	
A21	1.5	63	13.3	See Note 1	27%

Note 1: Areas A18, A19 and A21 are simulated using the Standhyd command and therefore do not use the Time to Peak parameter. The impervious ratio parameters for these drainage areas are shown.

The soil types in the area are predominately soils with high infiltration and low runoff potential, even when saturated. The Hydrologic Soil Group (HSG) Types A and AB dominate the site across nearly three quarters (75%) of the modeled area. HSG Type C, which are sandy clay loam soils with low infiltration rates when thoroughly saturated, and have a corresponding relatively higher runoff, occurs across approximately 1% of the site. Two percent (2%) of the area is impervious due to roads or dwellings while mid-range soils make up the remainder of the modeled area. Soil types by geotechnical description are depicted in Figure 5-4.

The model schematic is contained in **Appendix 'B2'** which also includes a detailed description of all the model parameters, including Time to Peak, Initial Abstraction, etc.

Figure 5-3: Land Cover in SWMHYMO Model

Figure 5-4: Soils in SWMHYMO Model

B5.5.1 Storm Distribution

The hydrological response of the BSUEA and abutting lands was simulated under a 6 hour, 12 hour and 24 hour SCS Type II storm distribution. The SCS Type II storm distribution was developed by the American Soil Conservation Service and is generally used for estimating flows in rural areas. The critical storm event under pre-development conditions, with the highest peak runoff, was found to occur under the 12 hour SCS Type II storm distribution.

B5.6 Modeling Results

The pre-development SWMHYMO simulation results, predicting flows at each of the culverts for the critical storm event, are shown in Table 5-5, below. The estimated capacity and level of service of each culvert is also provided. The details of culvert CR-C2, crossing Cambrian Road at Borrisokane Road, could not be obtained in the field due to obstructions and/or structural failure. Hence, the capacity and level of service at this culvert could not be confirmed.

Table 5-5: Hydrological Simulation Results at Culvert Locations (12 hour SCS Type II storm)

Culvert ID	Flow (m^3/s) at culvert location for return period (recurrence)						Estimated Culvert Capacity (m^3/s)	Estimated Level of Service (years)
	1:2 yr	1:5 yr	1:10 yr	1:25 yr	1:50 yr	1:100 yr		
CR-C1	0.3	0.7	1.0	1.6	2.0	2.5	5.5	1:100
CR-C2	0.2	0.4	0.7	1.0	1.3	1.6	N/A	N/A
CVR-C1	0.1	0.3	0.5	0.8	1.0	1.3	0.4	1:5
CVR-C2	0.0	0.1	0.1	0.2	0.2	0.3	0.2	1:25
CVR-C3	0.0	0.1	0.2	0.2	0.3	0.4	0.3	1:50
CVR-C4	0.2	0.4	0.6	0.9	1.1	1.4	2.6	1:100
BDR-C1	0.0	0.0	0.1	0.1	0.1	0.2	0.2	1:100
BDR-C2	0.0	0.1	0.1	0.1	0.2	0.2	0.2	1:50
BDR-C3	0.1	0.1	0.1	0.2	0.2	0.3	0.5	1:100
BDR-C4	0.2	0.4	0.6	0.9	1.2	1.5	2.6	1:100
BDR-C5	0.0	0.0	0.0	0.0	0.0	0.1	0.3	1:100
BDR-C6	0.0	0.0	0.1	0.1	0.2	0.2	0.2	1:100
BDR-C7	0.1	0.1	0.1	0.2	0.3	0.4	0.3	1:50

The SWMHYMO model results show relatively low flow rates at each of the culverts, due to the high infiltration capability of the soils in the area and the low impervious ratio. In general, during larger and infrequent events, runoff will increase as the infiltration capability decreases due to saturation of the soils.

The culverts crossing the main boundary roads to the site, CR-C1, CR-C2, BDR-C4, BDR-C5 and BDR-C6, have high estimated levels of service (in excess of 1:100 years), with the exception of two (2) culverts on Barnsdale Road and the culvert on Cambrian Road, which is unrecorded. Since these roads are classified as arterial or collector roads, they would be expected to have a higher level of service compared to the culverts internal to the site or under access roads south of Barnsdale Road.

B5.6.1 Hydraulics and Floodplain Mapping

There are no well-defined watercourses on the site which are suitable to hydraulic analysis or to prepare floodplain mapping. Flow conveyance for the majority of the site occurs by means of overland sheet flow, and flow in small, intermittent channels. The floodplain of the Jock River subwatershed extends close to the site but remains within the northwestern quadrant of the Cambrian Road / Borrisokane Road intersection (**Figure 5-1**). The floodplains of the Rideau River and Mud Creek do not extend to the site.

B6.0 WATER BUDGET

The water budget of a given area is often studied by hydrologists and hydrogeologists to better understand the movement of water through its various stages. Natural watershed systems maintain a balance between precipitation, runoff to water bodies, infiltration to the groundwater system, evaporation from water surfaces, or transpiration from vegetation, completing the natural cycle back into atmospheric moisture and precipitation. The analysis of the water budget is often carried out to establish the baseline condition and assess any future changes in infiltration to subsurface water-bearing zones and, consequently, to surface runoff resulting from a significant land cover modification such as urbanization. It is also used to predict changes to the hydrological cycle that will result from urbanization of lands. Figure 6-1 summarizes graphically the natural hydrological cycle describing the continuous movement of water on, above, and below the surface.

In simple terms, the general water budget assesses the change in water storage resulting from the difference in water inputs and outputs for a given area or watershed. To estimate the water budget, the following simplified equation is used:

$$P = ET + SW + GW + \Delta S$$

Where:

- P is precipitation;
- ET is evapotranspiration;
- SW is surface runoff;
- GW is groundwater; and
- ΔS is the difference in water storage.

As a fundamental requirement, the water budget developed for the existing conditions will be used as the baseline condition when assessing the effectiveness of any stormwater management (SWM) strategies under the post-development condition. Future SWM strategies should adhere to the recent provincial guidance from the MOECC in their publication entitled "Expectation interpretation bulletin, February 2015", as well as relevant provincial policy statement. One of the objectives of the upcoming Master Servicing Study will be to develop a storm servicing strategy that will incorporate mitigation measures to minimize the water deficit in an effort to offset the potential impact of urbanization.

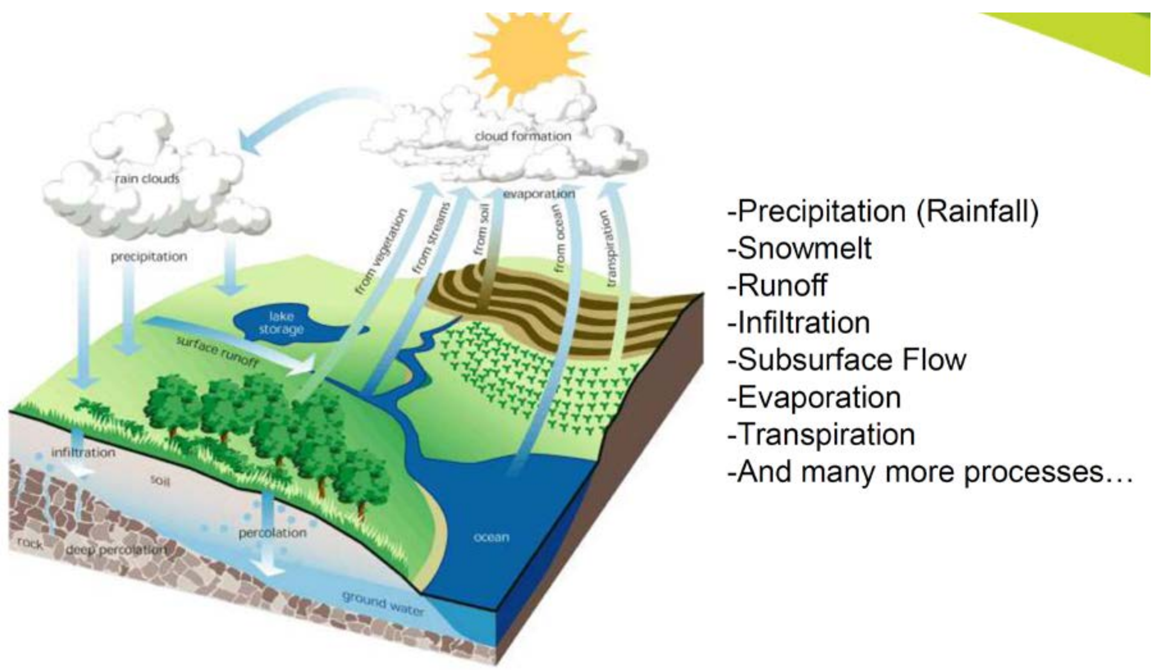


Figure 6-1: Water Budget Cycle

B6.1 Continuous Simulation Modeling

An understanding of the water budget within a given study area can be gained through the use of a continuous hydrological model. The Toronto and Region Conservation

Authority (TRCA), in their publication entitled “Stormwater Management Criteria, August 2012”, recommend the use of a continuous model such as QUALHYMO or PCSWMM.

The PCSWMM model to be used for this study includes simplified groundwater and snowmelt modules that allow the continuous simulation of the water budget, including the elements of evapotranspiration, the groundwater table (lower saturated zone), snowfall, and snowmelt. Due to its capabilities and versatility, the PCSWMM modeling tool will also be used to assess the impact of development and effectiveness of the proposed Low Impact Development (LID).

B6.1.1 Infiltration Parameters

Saturated field hydraulic conductivity of the surficial soils for the southern and eastern parts of the study area was measured on-site by Paterson. The saturated field hydraulic conductivity (Kfs) blocks ranged from $2.7\text{x}10^{-7}$ m/s to less than $8\text{x}10^{-5}$ m/s. The Ontario Ministry of Municipal Affairs and Housing (MMAH) Building Code Supplementary Standard SB-6 (2012) provides guidance on converting saturated field hydraulic conductivity (Kfs) to infiltration rate via the Percolation Time (an extract of this Supplementary Standard is included in **Appendix 'B3'**). For permeable gravel sand mixtures, which have saturated field hydraulic conductivities in the range of $1\text{x}10^{-4}$ to $1\text{x}10^{-6}$ m/s, the percolation rate is in the range of 12 to 4 mins/cm, which is equivalent to 50 to 150 mm/hr. Therefore, the range of infiltration rates across the extent tested by Paterson varies between 49.8 mm/hr and 106.8 mm/hr.

In the PCSWMM model, subcatchments were delineated based on the saturated field hydraulic conductivity blocks and maximum, median, and minimum infiltration values selected for each block. Field testing was not available to determine the Saturated Infiltration coefficient for the Horton infiltration method (minimum or final infiltration rate). Consequently, the values of the final infiltration capacity were estimated based on the initial infiltration capacity and the soil type. **Table 6-1,** below, summarizes the various ranges of saturated field hydraulic conductivities and their corresponding range of maximum infiltration rates as well as the selected final infiltration coefficient used in the water budget model.

It should be noted that a decay constant, k, of 4.14 hr^{-1}, was used in the PCSWMM model to mimic the decreasing infiltration capacity of the soil over time during wet weather events.

Table 6-1: Horton Coefficients for Water Budget Model

Range of Kfs (m/s)	Max Range Infiltration Values (mm/hr)	Estimated Soil Groupings (based on infiltration values)[1]	Range of Expected Final Infiltration Values (mm/hr)[1]	Selected Final Infiltration Coefficient (mm/hr)
$2.7x10^{-7}$ – $9.9x10^{-7}$	49.8 – 50.0	Moist Loam Soils	3.8 - 7.6	4
$1.0x10^{-6}$ – $9.9x10^{-6}$	50.0 – 53.2	Moist Loam soils	3.8 - 7.6	4 - 7
$1.0x10^{-5}$ – $8.0x10^{-5}$	53.2 – 106.8	Moist to Dry Sandy Soils	7.6 – 11.4	7 - 11

B6.1.2 Precipitation and Evaporation

The continuous simulation model was run using precipitation data from the Environment Canada weather stations at Ottawa Macdonald-Cartier International Airport and the Experimental Farm in Ottawa (approximately 10 km and 15 km from the BSUEA respectively). In excess of 30 years of hourly data, between January 1, 1960 and October 31, 1990, was used to undertake the water budget analysis. The average annual rainfall during this period was 844 mm, while maximum and minimum daily temperatures from the same weather stations and time period were also used.

The model undertakes evaporation calculations based on average monthly rates of evaporation (as reported by Environment Canada, refer to Section **B2.3**) and from the available standing water that is "trapped" in natural depressions within the subcatchment surfaces as a result of the initial abstraction depth.

B6.1.3 Groundwater

PCSWMM analyzes groundwater flow for each subcatchment independently. It represents the subsurface region beneath a subcatchment as an unsaturated upper zone that lies above a lower saturated zone. The elevation of the lower saturated zone, the water table, varies in time depending on the rates of flow into and out of the lower saturated zone. The flow to the lower saturated zone is controlled by percolation, which is dictated by the soil characteristics. The upper unsaturated zone receives water via infiltration from surface runoff. Evapotranspiration occurs from the upper unsaturated zone; it can also occur from the lower saturated zone, depending on root depth. If the water table, or elevation of the lower saturated zone, reaches the surface level then all the soil is saturated and infiltration will no longer occur.

[1] Akan, A. O., *Urban Stormwater Hydrology: A Guide to Engineering Calculations.* Lancaster: Technomic Publishing Co., Inc., 1993 (extract contained in **Appendix 'B3'**).

For the purposes of modelling the water budget's existing condition, subcatchments were delineated by each hydrologic soils group (HSG) within the BSUEA. Properties currently operating as aggregate pits were delineated as separate subcatchments. The soil parameters associated with each HSG are detailed in **Appendix 'B3'**.

B6.1.4 Snowmelt

Snowmelt is an additional mechanism by which runoff may be generated in a continuous simulation model. The current PCSWMM software platform utilizes the Canadian SWMM snowmelt routines with extensions for long term continuous modelling.

Snowfall rates are determined directly from hourly precipitation data by using a pre-set temperature: snowfall will occur when the temperature is below the pre-set point and rainfall when above. Snowmelt is handled differently by PCSWMM, depending on the occurrence of rainfall. During rain on snowmelt events, the model takes into account the rainfall intensity and the air temperature as well as the saturation vapour pressure. When snowmelt occurs without any rainfall, the snowmelt is linearly proportional to the air temperature, which varies with the user supplied melt coefficients. The parameters used in the snowmelt module are detailed in **Appendix 'B3'**.

For the existing condition model it has been assumed that all snow occurs on pervious land cover and there is no snow removal.

B6.1.5 Continuous Simulation Results

A long term simulation was carried out for the maximum, minimum, and median of each of the infiltration ranges of each block. The summary of the simulation results is given in **Table 6-2**, below, which has been grouped based on the above noted ranges of infiltration. These results show that there is no significant variation within the range of values for each set of infiltration values. This is expected, as the peak rainfall events over a 30-year simulation period are generally lower than the infiltration rates (ranging between ±50 mm/hr and ±105 mm/hr) within the frequent event ranges and are, therefore, not affected. Only the infrequent large peak rainfall events will be affected by the change in infiltration within the range of values.

Table 6-2: Comparison of Results with Range of Infiltration Values

Annual Water Budget Components	Minimum Infiltration	Median Infiltration	Maximum Infiltration
Precipitation (mm/year)	844	844	844
Evapotranspiration (mm/year)	506	506	506
Infiltration (mm/year)	336	337	338
Surface Runoff (mm/year)	2	1	1

Since there is negligible variability in the simulation results shown above, only the median infiltration values will be considered going forward with the water budget analysis. Based on the medium infiltration, the continuous simulation results have been summarized for each of the three (3) subwatersheds as shown in **Table 6-3**, below.

Table 6-3: Water Budget Continuous Simulation Results

Annual Water Budget Components	Annual Average Depth (mm)				Budget %
	To Jock River	To Mud Creek	To Rideau River	Area Weighted Total	
Precipitation	844	844	844	844	100%
Evapotranspiration	506	506	506	506	60%
Infiltration	338	336	336	337	40%
Surface Runoff	1	2	2	1	0%

Based on the above long-term simulation results, infiltration accounts for approximately 40% of the total annual precipitation of 844 mm, while surface runoff accounts for approximately 0% on a yearly average. Specifically, surface runoff accounts, on average, for 1 mm per year out of the total annual precipitation of 844 mm, representing approximately 0.1%.

Although the average annual total depth of runoff is virtually non-existent percentage wise, the model computed surface flows when precipitation depths were encountered that exceeded a 1:5 year recurrence or when peak intensities were greater than a 1:2 year recurrence. Based on the review of the 30-year simulation results, a peak flow of 3.27 m^3/s was computed as the largest flow by the model. This peak flow was generated from the overall BSUEA and sheet flow draining towards the three (3) sub-watershed outlets; a peak flow of 2.44 m^3/s was computed as being directed to the Jock River as summarized in **Table 6-4**, below.

The event resulting in the largest peak runoff occurred on August 4, 1988 and is one of the historical storms listed in the City of Ottawa Sewer Design Guidelines to be simulated to assess the performance of drainage systems. However, the large events are infrequent in nature; the peak flow generated on August 4, 1988 was experienced once during the 30-year simulation. **Table 6-4**, below, summarizes the five (5) greatest and smallest peak flow events generated by the model with the associated rainfall depth and peak intensity as well as its corresponding frequency.

Table 6-4: Top Five Largest and Smallest Peak Flow Events

Date	Peak Runoff to Jock (m^3/s)	Peak Runoff to Mud (m^3/s)	Peak Runoff to Rideau (m^3/s)	Storm Duration (hrs)	Peak Rainfall Intensity (mm/hr)	Equivalent Return Period	Total Rainfall Volume (mm)	Equivalent Return Period
August 4, 1988	2.44	0.40	0.43	4	37.4	10 year	77.0	100 year
August 8, 1973	0.42	0.15	0.16	7	30.2	5 year	60.5	10 year
June 16, 1981	0.35	0.14	0.15	2	36.9	10 year	38.3	5 year
June 11, 1973	0.34	0.14	0.15	3	33	5 year	42.4	5 year
August 8, 1973	0.31	0.07	0.10	8	30.2	5 year	60.5	10 year
June 11, 1976	0.02	0.01	0.02	2	24.6	2 year	24.9	2 year
Sept. 12, 1963	0.01	0.01	0.01	8	10.2	< 2 year	52.4	5 year
Sept. 11, 1986	0.01	0.01	0.01	14	14	< 2 year	42.9	5 year
July 1, 1979	0	0	0	5	14.0	< 2 year	26.3	< 2 year

Note: shaded columns above highlight the computed flows during two (2) historical storms.

The following conclusions can be drawn from the statistics summarized in the above table:

- The largest event, from a volumetric perspective, occurred on August 4, 1988 with a recorded volume of 77 mm over four (4) hours, which, compared to the City of Ottawa IDF curve, has a frequency greater than a 1:100 year; and
- From a peak intensity perspective, the August 4, 1988 event was the largest with a recorded peak intensity of 37 mm/hr, which, compared to the City of Ottawa IDF curve, has a frequency of 1:10 year.

Three (3) events were recorded with a peak intensity of between 36 and 37 mm/hour, which, given that this is approximately a 1:10 year return period event, is true to the statistics. Only two (2) of these recorded any runoff as the total volume of runoff was insufficient to trigger flow.

In terms of rainfall volume, none of the events over the 30-year simulation period were greater than a 1:10 year event, other than the one which resulted in the largest peak flow and one other 24-hour period, which did receive more rainfall than the 1:100 year volume; however, the peak intensity was 20 mm/hr, less than a 1:2 year event. Due to the low peak intensity, this other 1:100 year event did not trigger any runoff flows.

The City of Ottawa has three (3) historical events that are used to assess how a system would function under historical extreme events. The 1988 event triggered the largest runoff flow from the BSUEA; however, the 1979 event did not result in any runoff. The simulation uses hourly rainfall data that averages the intensities within the hour period and creates an average peak, which will be of less significance when compared to any peak intensities identified in the 5-minute data.

A preliminary high level scenario of a potential future water budget was simulated for a development having a total imperviousness of 65%, of which 58.5% is Directly Connected Impervious area (i.e., 90% of the total imperviousness was assumed to be directly connected) without any Best Management Practices to promote infiltration. The results indicate that the percentage of infiltration will reduce from 40% (existing condition) to 28% (post-development without BMPs) of the overall annual water budget. Simulation results have also shown reductions in evapotranspiration as a result of the decrease in vegetation which, in turn, generates increases in surface runoff. Consequently, stormwater management measures promoting infiltration will be necessary to meet the target of maintaining pre-development infiltration levels.

B7.0 EXISTING INFRASTRUCTURE AND SERVICING SUMMARY

B7.1 Trunk Wastewater Servicing

In 2007, as part of the Barrhaven South Master Servicing Study (BSMSS), Stantec modelled the existing South Nepean Collector (SNC) and West Rideau Collector (WRC) trunk sewers to determine whether they could both accommodate peak flows from new development in Barrhaven South, up to the current Urban Boundary. Their analysis consisted of the following two (2) scenarios:

- Peak flows for existing development were established based on monitored flow data while peak flows for future development in Barrhaven South were calculated based on theoretical flow generation parameters given in the 2004 City of Ottawa Sewer Design Guidelines[2]; and

[2] As the BSMSS was completed in 2007, the 2004 Sewer Design Guidelines were current at that time. The Design Guidelines were updated in 2012; however, the 2007 BSMSS still references the 2004 version.

- Peak flows for both existing and future development were calculated based on the theoretical flow generation parameters given in the 2004 Ottawa Sewer Design Guidelines.

Modeling results showed that under Scenario 1, the existing SNC and WRC could accommodate the additional flow generated from Barrhaven South; however, under Scenario 2, the SNC and WRC showed risk of surcharging and basement flooding at certain locations.

In September 2013, Stantec completed a City-wide Wastewater Collection System Assessment to identify areas in need of wastewater collection system upgrades to accommodate projected growth to 2031 and 2060. This assessment served as the basis of the wastewater component of the City's 2013 Infrastructure Master Plan (IMP). To complete the assessment, Stantec developed a hydrodynamic model using PCSWMM that included all existing and planned (approved) sanitary trunk sewers (i.e., greater than 450 mm in diameter), pumping stations, and associated wastewater infrastructure in the City. Simulations were carried out based on monitored flow data obtained from 31 locations throughout the City under dry weather conditions and wet weather flow parameters established from two (2) historical rainfall events and one synthetic storm event (the 1:100 year storm). Flow generation for future development areas was based on average flow rates consistent with the 2012 Design Guidelines. Based on the simulation results, the 2013 Stantec Assessment did not identify any growth-related areas at risk nor recommend system upgrades in Barrhaven South under both the 2031 and 2060 planned growth horizons. Consequently, no wastewater projects were identified within Barrhaven South as part of the November 2013 IMP.

In 2014, Stantec issued a draft addendum to their 2007 BSMSS. In this addendum, herein referred to as the Draft 2014 BSMSSA, the peak flow projected in 2007 was re-calculated to account for actual peak flow values from developments that had been approved and/or constructed since the completion of the 2007 BSMSS. These developments included Mattamy's Half Moon Bay and Half Moon Bay South and Tamarack Homes' The Meadows at Half Moon Bay. The Draft 2014 BSMSSA also considered the design parameters prescribed in the 2012 Design Guidelines for calculating both existing and future development peak flows in Barrhaven South. The total peak sanitary flow re-calculated for the Barrhaven South Community as part of the Draft 2014 BSMSSA was found to be significantly less than the peak flow allocated for Barrhaven South as part of the 2013 Wastewater Collection System Assessment by Stantec. Consequently, as indicated in the Draft 2014 BSMSSA, Stantec concluded that downstream wastewater infrastructure, including the SNC and WRC, could accommodate the peak flows from the Barrhaven South Community with no risk of surcharging or basement flooding. **Table 7-1** provides the peak flows calculated for

Barrhaven South, up to the current Urban Boundary, as part of the three (3) aforementioned studies. These peak flows were calculated at downstream sanitary maintenance hole 522A, located at the intersection of the existing Greenbank Road and Half Moon Bay Road, immediately south of the Jock River.

Table 7-1: Total Sanitary Peak Flow for Barrhaven South

Study	Barrhaven South Peak Flow (l/s)
2007 BSMSS	388
2013 IMP Model	590
Draft 2014 BSMSSA	412

B7.1.1 Existing Sanitary Sewers

Wastewater servicing for existing development in Barrhaven South was designed in general conformance with the 2007 BSMSS and the Draft 2014 BSMSSA (refer to the Sanitary Servicing Plan prepared by Stantec in **Appendix 'B4'**). The southerly portion of the existing Barrhaven South Community is serviced by the following three (3) main trunk sanitary sewers as shown on **Figure 7-1**:

- <u>Existing Greenbank Road Trunk Sanitary Sewer</u>
  This sewer extends approximately 2 km south of the Jock River up to the current Urban Boundary. At its upstream end, at the Urban Boundary, this sewer is 600 mm in diameter, with a 0.25% slope, an obvert elevation of 91.77 m and is 6.7 m below existing ground.

- <u>Cambrian Road Trunk Sanitary Sewer</u>
  This sewer extends approximately 1 km west of the existing Greenbank Road up to the future realigned Greenbank Road. At its upstream end, at the future realigned Greenbank Road, this sewer is 500 mm in diameter, with a 0.20% slope, an obvert elevation of 89.53 m and is 4.1 m below existing ground.

- <u>River Mist Road Trunk Sanitary Sewer</u>
  This sewer extends approximately 1.7 km south of Cambrian Road to the southern limit of the current Urban Boundary (within Minto's Quinn's Pointe subdivision). At its upstream end, at the Urban Boundary, this sewer is 200 mm in diameter, with a 0.32% slope, an obvert elevation of 100.44 m and is 3.3 m below existing ground. As its downstream end, at Cambrian Road, this sewer is 375 mm in diameter.

Figure 7-1: Wastewater Servicing Plan

The Cambrian Road and River Mist Road trunk sanitary sewers convey wastewater to the existing Greenbank Road trunk sanitary sewer, which outlets to the SNC and, ultimately, to the WRC.

A review of the trunk sanitary sewers in Barrhaven South was undertaken to evaluate whether there is available capacity in the existing sanitary sewer system to service additional lands outside the current Urban Boundary. Design sheets for the three (3) existing trunk sanitary sewers listed above were reviewed to identify the limiting pipe reach in terms of available residual capacity. It should be noted that the review was based on theoretical peak flow calculations and not monitored flow data. Based on the theoretical peak flows presented in the Draft 2014 BSMSSA Sanitary Sewer Design Sheet prepared by Stantec (a copy of which is provided in **Appendix 'B4'**), the limiting residual capacity in each trunk sanitary sewer is shown in Table 7-2, below.

Table 7-2: Available Residual Capacity in Existing Trunk Sanitary Sewers

Trunk Sanitary Sewer	Limiting Pipe Reach	Residual Capacity (L/s)
Existing Greenbank Road	MH 45 to MH45A	295.4
Cambrian Road	MH13A to MH15A	51.4
River Mist Road	MH 102A to MH17A	17.8

Furthermore, as part of the detailed design of the Quinn’s Pointe subdivision, tributary areas were reassigned from the existing Greenbank Road trunk sanitary sewer to the River Mist Road trunk sanitary sewer due to a change in the alignment of River Mist Road that was implemented following the issuance of the Draft 2014 BSMSSA. As a result, a higher peak flow is conveyed to the River Mist Road trunk sanitary sewer in comparison to the allocated flow in the Draft 2014 BSMSSA. Therefore, the available residual capacity in the River Mist Road trunk sanitary sewer is slightly less than that indicated in Table 7-2 above, as summarized in **Table 7-3**.

Table 7-3: River Mist Road Trunk Sanitary Sewer Available Residual Capacity

Quinn’s Pointe Allocation per Draft 2014 BSMSSA	10.40 L/s
Peak flow per Quinn’s Pointe detailed design	13.77 L/s
Difference	3.37 L/s additional
River Mist Residual Capacity per Table 7-2	17.8 L/s
River Mist Revised Residual Capacity	**14.4 L/s**

It should be noted that the critical pipe reach on the existing Greenbank Road trunk sanitary sewer is located downstream of the Cambrian Road and River Mist Road trunk sanitary sewers (refer to the Sanitary Servicing Plan in **Appendix 'B4'**). Should future development require the use of the entire theoretical residual capacities of the River Mist Road and Cambrian Road trunk sanitary sewers (i.e., 14.4 L/s and 51.4 L/s, respectively), then the allowable peak flow that could be conveyed by the existing Greenbank Road trunk sanitary sewer downstream of Cambrian Road is 229.6 L/s (i.e., 295.4 – 51.4 – 14.4 = 229.6 L/s).

B7.1.2 Planned Trunk Sanitary Sewers

There are also trunk sanitary sewers in Barrhaven South within the current Urban Boundary that have yet to be constructed, as shown in the Draft 2014 BSMSSA. These planned trunk sanitary sewers are identified by dashed lines in **Figure 7-1** and are described as follows:

- <u>Future Cambrian Road Trunk Sanitary Sewer</u>
  This sewer is to extend approximately 400 m west of the existing Cambrian Road trunk sanitary sewer to the future Collector Roadway located approximately 500 m east of Borrisokane Road. As per the Draft 2014 BSMSSA, this sewer is proposed to be 375 mm in diameter, with a 0.40% slope and an upstream obvert elevation of 91.38 m at the future Collector, which is 2.1 m below existing ground.

- <u>Future Collector Road Trunk Sanitary Sewer</u>
  This sewer is to extend approximately 550 m south of Cambrian Road along a future Collector located between the future realigned Greenbank Road and Borrisokane Road. As per the Draft 2014 BSMSSA, this sewer is to be 300 mm in diameter, with a 0.75% slope and an upstream obvert elevation of 95.00 m, which is 5.0 m below existing ground.

- <u>Future Realigned Greenbank Road Trunk Sanitary Sewer</u>
  This sewer is to extend approximately 800 m south of Cambrian Road along the future realigned Greenbank Road. As per the Draft 2014 BSMSSA, this sewer is proposed to have an upstream diameter of 250 mm, with a 1.30% slope and an obvert elevation of 96.10 m north of Dundonald Drive, which is 6.9 m below existing ground.

Similar to the existing trunk sanitary sewers, these planned (future) trunk sanitary sewers in Barrhaven South will convey wastewater to the existing Greenbank Road trunk sanitary sewer which ultimately outlets to the SNC.

B7.2 Water Servicing

B7.2.1 Existing Conditions

The existing Barrhaven South Community is part of the City's Pressure Zone BARR which is fed from Pressure Zone 2W via the following two (2) source transmission mains:

1. The Woodroffe Avenue 1200 mm diameter watermain (from Baseline Road to Fallowfield Road); and
2. The Fallowfield Road 762 mm diameter watermain.

The Barrhaven Reservoir, located off of Fallowfield Road, acts as balancing storage for Zone 2W and feeds the Barrhaven Pumping Station (BPS), which boosts water to the Pressure Zone BARR.

B7.2.2 Future Zone Reconfiguration

Findings presented in the City of Ottawa Water Supply Optimization Study Report prepared by Delcan in 2008, and more recently in the Draft 2014 BSMSSA, showed that a reconfiguration of pressure zones within the South Urban Community (which includes the Barrhaven South Community) would eliminate the need for a new pumping station, minimize system fragmentation (dead-ends), and generally improve system reliability and operation. This recommended zone reconfiguration consists of an expanded Zone 3C within areas originally identified as Pressure Zone BARR, including the Barrhaven South Community and the future BSUEA. The City elected to move forward with the zone reconfiguration and has, therefore, identified several water distribution projects in their 2013 IMP that are required to achieve the recommended zone reconfiguration and meet future growth and reliability needs up to the 2031 planning horizon within the South Urban Community. These IMP projects are as follows:

- Barrhaven Reservoir Pumping Station Upgrade;
- Fallowfield Road Watermain Upgrade (from the Barrhaven Reservoir to Borrisokane Road);
- Barrhaven Pumping Station Upgrade;
- Barrhaven-3C Feedermain (from Foxfield at Holitman to Via Chianti Grove); and
- Existing Greenbank Road Watermain (from north of Jockvale to south of the Jock River).

Refer to extracts from the 2013 IMP provided in **Appendix 'B5'** for details on the above-noted projects.

It should be noted that as part of the City of Ottawa 2013 Water Master Plan analyses, Stantec has highlighted a particular constraint in terms of providing water servicing for the BSUEA. Hydraulic simulation results for growth areas within the South Urban Community up to the 2031 planning horizon have shown that low pressures (i.e., less than 40 psi) are anticipated along the future water distribution system that will be servicing the elevated portion of the BSUEA (refer to Figure 5-5: Residual Pressure Distribution – SUC – 2031 provided in **Appendix 'B5')**. As a result, a new booster pumping station and oversized services or jet pumps are anticipated to be required in order to provide adequate water servicing within the BSUEA.

B7.2.3 Existing Watermains and Operating Pressures

Water servicing for existing development in the Barrhaven South Community was developed in general conformance with the 2007 BSMSS and the more recent Draft 2014 BSMSSA (refer to the Water Servicing Plan prepared by Stantec in **Appendix 'B5'**). Potable water is supplied to the southerly portion of the existing Barrhaven South Community by the following existing feedermains, as shown in **Figure 7-2**:

- <u>Existing Greenbank Road 406 mm Diameter Feedermain</u>
  This feedermain extends approximately 2 km south of the Jock River up to the southerly limit of the current Urban Boundary.

- <u>Existing Cambrian Road 406 mm Diameter Feedermain</u>
  This feedermain extends from the existing Greenbank Road to the future realigned Greenbank Road.

- <u>Existing River Mist Road 305 mm Diameter Feedermain</u>
  This feedermain extends the entire length of River Mist Road from River Run Avenue up to the southerly limit of the current Urban Boundary. The actual alignment of this feedermain was slightly revised from the Draft 2014 BSMSSA, which showed a 305 mm diameter feedermain along Alex Polowin Avenue, from Kilbirnie Drive up to the southerly limit of the Urban Boundary (refer to **Figure 7-2** and the Draft 2014 BSMSSA Water Servicing Plan provided in **Appendix 'B5'**).

B7.2.4 Future Watermains and Operating Pressures

As part of the long-term infrastructure needs, there will also be a requirement to extend existing feedermains that are planned but are yet to be constructed within the current Urban Boundary. These future planned watermains, which have been identified in the Draft 2014 BSMSSA, include the following and are shown as dashed lines on **Figure 7-2**.

Figure 7-2: Water Servicing Plan

- Future 305 mm diameter feedermain on the future realigned Greenbank Road
  This feedermain is to extend by approximately 2 km south of Cambrian Road, past the current Urban Boundary and about 500 m east to River Mist Road to complete a 305 mm diameter watermain loop.

- Future 406 mm diameter feedermain on Cambrian Road
  This feedermain is to extend from the future realigned Greenbank Road to Borrisokane Road.

- Future 305 mm diameter feedermain on future Collector Road
  This feedermain, located along the future Collector Road between Borrisokane Road and the future realigned Greenbank Road, is to extend approximately 550 m south of Cambrian Road to the current Urban Boundary and approximately 430 m east to the future realigned Greenbank Road feedermain to complete a system loop.

In terms of the existing operating pressures and hydraulic grade lines (HGL) in Pressure Zone BARR under domestic and fire flow conditions, the following information was extracted from the Draft 2014 BSMSSA:

- The HGL in Pressure Zone BARR is dependent on the water levels in the Moodie Drive Tank, the pump discharge pressure, and the system head losses. When those elements are factored in, the HGL is found to typically vary from a minimum of 147 m to a maximum of 160 m under fire flow and domestic demands.
- Ground elevations in Pressure Zone BARR range between a minimum elevation of 84 m and a maximum elevation of 117 m. The range in Barrhaven South is between a minimum of 83 m and a maximum of 110 m, while the ground elevations in the BSUEA range between 99 m and 111 m.
- During basic day demand and under the maximum operating HGL of 160 m, pressures in Barrhaven South are expected to range between 489 kPa (71 psi) and 751 kPa (109 psi), assuming no system head loss. As noted in Section 5.2.1 of the Draft 2014 BSMSSA, low lying areas will experience pressures greater than 80 psi (552 kPa).
- During peak hour and under the minimum operating HGL of 147 m, pressures in Barrhaven South are expected to range between 365 kPa (53 psi) and 627 kPa (91 psi), assuming no system head losses.

B7.2.5 Future Operating Pressures in the BSUEA

Water servicing needs in the BSUEA will be investigated and evaluated as part of the upcoming Master Servicing Study (MSS) for the BSUEA. The existing and planned trunk feedermains, as depicted on **Figure 7-2**, will serve as the backbone of the future trunks within the BSUEA. The Draft 2014 BSMSSA has to be finalized before the BSUEA MSS can begin. The Draft 2014 BSMSSA will evaluate the performance of the water distribution system at a preliminary level, to determine whether the system can support the urbanization of the BSUEA, referred to as Area 7. This work will be finalized early in 2017, in advance of adopting the Community Design Plan. Results from this undertaking will be extracted and included in this section of the Existing Condition Report when made available.

As previously noted, Barrhaven South, as well as the BSUEA, will be serviced from an expanded Zone 3C by future watermain extensions. As per the Draft 2014 BSMSSA, potable water will originate from the Barrhaven Pumping Station where a constant 147 m discharge HGL will be maintained under the various range of demands. Table 7-4 summarizes the operating pressures in Barrhaven South and the BSUEA (information extracted from the Draft 2014 BSMSSA).

Table 7-4: Residual Pressure during Basic and Peak Hour Demands

Area	Zone	Basic Day Maximum Pressure kPa (psi)	Peak Hour Minimum Pressure kPa (psi)
Barrhaven South	3C	372 (54) - 607 (88)	296 (43) - 503 (73)

B7.3 BSUEA Stormwater Servicing and Stormwater Management

A preferred stormwater management (SWM) strategy for Barrhaven South up to the current Urban Boundary was first identified in the Jock River Reach One Subwatershed Study prepared by Stantec in June 2007. This preferred SWM strategy included five (5) wet pond facilities that would provide quality control (to an MOE enhanced level of protection, 80% suspended solids removal) before discharging to the Jock River. The 2007 Jock River Reach One Subwatershed Study served as the basis for the 2007 BSMSS, also prepared by Stantec, which outlined tributary areas and trunk storm sewers to convey stormwater to the five (5) wet pond facilities. These five (5) facilities were strategically located south of the Jock River and referred to as Corrigan, Todd, Clarke, Borrisokane and Greenbank (refer to **Appendix 'B6'** for a copy of the 2007 BSMSS Conceptual SWM Storm Servicing Plan, Drawing ST1, prepared by Stantec).

A Draft Addendum to the 2007 BSMSS was subsequently issued by Stantec in 2014 to address changes in SWM criteria established in the 2012 Design Guidelines and associated Technical Bulletins. The Draft 2014 BSMSSA also incorporated the detailed designs of the Corrigan and Todd Ponds and their tributary infrastructure, which were completed since the issuance of the 2007 BSMSS.

The following provides an overview of existing and planned stormwater infrastructure for both minor and major systems in Barrhaven South that is associated with each of the five (5) designated wet pond facilities. The minor storm system is comprised of street gutters, catch basins, inlet control devices, junction chambers, and storm sewers and conveys stormwater during frequent storm events (i.e., generally up to the 1:5 year storm event). The major system is comprised of swales, roadway sags, streets, etc. and generally conveys stormwater flows overland, up to the 1:100 year storm event, when flows are in excess of the minor system capacity.

B7.3.1 Corrigan Pond

In 2008, the detailed design of the Corrigan Pond was carried out by the IBI Group (IBI). Construction of the Pond was completed in 2010. The Corrigan Pond is located south of the Jock River, east of the existing Greenbank Road, and currently services a tributary area of approximately 148 ha within the eastern portion of the existing Barrhaven South Community, as shown on the Drainage Schematic (dated June 2015) prepared by IBI, and provided in **Appendix 'B6'**.

Minor System

Existing trunk storm sewers that outlet to the Corrigan Pond include the following, as shown in **Figure 7-3** and on the Drainage Schematic prepared by IBI provided in **Appendix 'B6'**:

- <u>Existing Tucana Way Trunk Storm Sewer</u>
  This sewer extends approximately 450 m south of the Corrigan Pond to Cambrian Road.

- <u>Existing Cambrian Road Trunk Storm Sewer</u>
  This sewer extends approximately 400 m west of Tucana Way to the existing Greenbank Road.

- <u>Existing Greenbank Road Trunk Storm Sewer</u>
  This sewer extends from Cambrian Road to approximately 1.7 km south of Cambrian Road, at the southern limit of the current Urban Boundary. At its upstream end, at the Urban Boundary, this sewer is 1800 mm in diameter, with a 0.16% slope, an obvert elevation of 94.07 m and is about 3.9 m below existing ground.

Figure 7-3: Stormwater Servicing Plan

It should also be noted that there is another storm sewer along the existing Greenbank Road, north of Cambrian Road. This storm sewer serves as an overflow; diverting flows of approximately 6.8 m^3/s of stormwater directly to the Jock River during the 1:100 year storm event (refer to IBI's Drainage Schematic provided in **Appendix 'B6'**).

In terms of hydraulic grade line (HGL) along the existing trunk storm sewers, simulation results provided in IBI's 2014 Stormwater Management Barrhaven South Report indicate that although certain portions of the trunk storm sewers experience surcharging under the 1:100 year storm event (SCS Type II), the minimum required freeboard of 30 cm to the underside of footing (USF) of each unit is still achieved throughout the system. The minimum freeboard in the system is 0.32 m at MH 104B on the existing Greenbank Road.

In terms of water quality analysis, the latest June 9, 2015 letter issued by IBI confirmed that the existing Corrigan Pond exceeds the required water quality volume to achieve an MOE enhanced level of protection for the current 147.5 ha tributary area, which is inclusive of the most recent detailed design of Minto's Quinn's Pointe subdivision. The existing Corrigan Pond has 1700 m^3 of residual permanent pool storage and 100m^3 of residual extended detention storage.

Major System

Major system flows generated from existing development located south of Cambrian Road and east of the existing Greenbank Road are tributary to the Corrigan Pond. Since overland flow is not permitted to cross arterial roadways (i.e., Cambrian Road and the existing Greenbank Road) as per the Design Guidelines, major system flows generated within these lands are conveyed overland to designated storage areas where they are released, at a restricted flow rate, back into the minor system of the Corrigan Pond. These storage areas consist of a parking lot and park which are identified as Areas B5D and B6, respectively, on the Dual Drainage Cascading Flow Figure prepared by IBI and provided in **Appendix 'B6'**. This Figure also shows that major system flows generated from existing development located to the north of Cambrian Road and to the east of the existing Greenbank Road outlet directly to the Jock River.

Major system flows generated in Minto's Quinn's Pointe subdivision are also ultimately conveyed to the Corrigan Pond. Since the subdivision is located to the west of the existing Greenbank Road, major system flows are first conveyed and captured by a dry pond located at the southern limit of Quinn's Pointe where they are released, at a restricted rate, to the Corrigan Pond minor system. As per the July 2015 Site Servicing Report for Quinn's Pointe prepared by JLR, the maximum major system street ponding depth in the subdivision is 35 cm (static and dynamic), in accordance with City Design Guidelines.

With the exception of the Quinn's Pointe subdivision, the remainder of the major system flow generated from lands to the west of the existing Greenbank Road is conveyed to the Todd Pond.

B7.3.2 Todd Pond

The detailed design of the Todd Pond was carried out by J.F. Sabourin and Associates Inc. (JFSA) in 2008. It is located in the northeast quadrant of the Cambrian Road and River Mist Road intersection. It is noted that the existing Todd Pond configuration is slightly different than the proposed configuration outlined in the 2007 BSMSS since the existing pond is not bisected by Cambrian Road.

The Todd Pond currently services a tributary area of approximately 141 ha within the existing central portion of Barrhaven South. This tributary area includes the Half Moon Bay, The Meadows and Half Moon Bay South subdivisions as shown in the Overall Drainage Area to Todd Pond (Figure 2) prepared by JFSA, and included in **Appendix 'B6'**. The latest model keeper analysis for the Todd Pond, completed by JFSA in April 2015, confirmed that the existing Todd Pond exceeds the MOECC's water quality volume requirement to achieve an enhanced level of protection for its tributary area based on the detailed designs of developments in Barrhaven South. The Todd Pond has approximately 6900 m^3 of residual permanent pool storage.

Minor System

Existing trunk storm sewers servicing the southerly portion of the Todd Pond tributary area include the following, as shown on **Figure 7-3**:

- <u>Existing River Mist Road Trunk Storm Sewer</u>
  This sewer extends from the Todd Pond to approximately 1.2 km south of Cambrian Road.

- <u>Existing Cambrian Road Trunk Storm Sewer</u>
  This sewer extends from the Todd Pond to approximately 500 m west of River Mist Road.

For the existing Half Moon Bay and a portion of The Meadows and Half Moon Bay South subdivisions, HGL analyses of the existing storm sewers tributary to the Todd Pond were completed at detailed design and approved based on the 1:100 year 3-hour Chicago design storm only. Based on this design storm, these developments meet the City's minimum freeboard requirement of 30 cm between the HGL and the USF, as indicated in the April 2015 Todd Pond Model Keeper Analysis.

The HGL analyses for these existing/approved developments did not, however, consider the 1:100 year 24-hour SCS Type II design storm and 1:100 year + 20% climate change stress test. As a result, under these two (2) designs storms, the minimum 30 cm freeboard is not always achieved, as noted by JFSA in the 2015 Todd Pond Model Keeper Analysis. However, the HGL was identified by JFSA as always below the finished basement floor elevation of existing units, as shown in Figure I-1: Lots with less than 30 cm freeboard, prepared by JFSA and provided in **Appendix 'B6'**. As indicated on Figure I-1, the minimum freeboard was found to be 23 cm above USF during the 1:100 year 24-hour SCS Type II design storm. Although the 30 cm freeboard for HGL and USF criterion was not respected, JFSA concluded that the existing units were not at risk of basement flooding.

As part of the latest Todd Pond Model Keeper Analysis completed in April 2015, JFSA analysed an 'ultimate' tributary area to the Todd Pond, which included minor system flows from 5.6 ha of future development lands, owned by Mattamy, located outside of the current Urban Boundary and within the BSUEA (refer to Figure 3, prepared by JFSA and provided in **Appendix 'B6',** for ultimate tributary area boundary). It is noted that the HGL analysis results summarized on Figure I-1, prepared by JFSA, include this ultimate drainage area and therefore existing basements would not be at risk of flooding with the addition of the 5.6 ha area. JFSA concluded that, in order for the Todd Pond to accommodate minor system flows from the additional 5.6 ha of future development area (based on an imperviousness of 56%), the current 30 m long Todd Pond outlet weir would have to be extended to a length of 48 m.

Major System

Major system flows generated from existing subdivisions west of the existing Greenbank Road including Half Moon Bay, The Meadows and Half Moon Bay South are conveyed to the Todd Pond. Since the Todd Pond is located immediately north of Cambrian Road, which is an arterial roadway, all major overland flow up to the 1:100 year storm was designed to be captured by the minor system just south of Cambrian Road so that the flows are conveyed to the Todd Pond without crossing Cambrian Road overland. Furthermore, the minor system also captures major system flow up to the 1:100 year storm at the northern property limit of Half Moon Bay South and The Meadows so that overland flow is not conveyed between subdivisions during the 1:100 year storm, with the exception of 2.33 m^3/s conveyed from Half Moon Bay South at Penant Avenue to Regatta Avenue in The Meadows.

As noted in the JFSA 2015 Todd Pond Model Keeper Analysis, some of the existing road sag storage areas exceed the maximum allowable ponding depth of 30 cm (static and dynamic for new developed areas) during the 1:100 year storm, in contravention of the 2012 Design Guidelines. These road sags are identified on Figure I-2, prepared by JFSA and provided in **Appendix 'B6'.** It is also noted that some of these road storage areas exceed a maximum ponding depth of 35 cm (static and dynamic) during the 1:100 year storm, in contravention of more recent guidelines, specifically, Technical Bulletin PIEDTB-2016-01.

Major system flows generated by Minto's Quinn's Pointe subdivision were originally identified to flow overland to the Half Moon Bay South subdivision and, ultimately, to the Todd Pond as per the 2007 BSMSS. Quinn's Pointe was allocated a total allowable major overland flow rate of 1.2 m^3/s (divided into 0.9 m^3/s and 0.3 m^3/s at two (2) outlets). However, the 2014 Stormwater Management Report for Quinn's Pointe (formerly referred to as Barrhaven South Phase 1), prepared by IBI, showed that the proposed Quinn's Pointe development would exceed the allowable major system release rate of 1.2 m^3/s to Half Moon Bay South. Consequently, an alternative SWM solution was developed where the 1.2 m^3/s major system release rate to existing development was not required. This solution consisted of a dry pond, located at the low-lying southeastern corner of Quinn's Pointe, to which major system flows were conveyed and released back into the minor system. This SWM approach was implemented in JLR's detailed design for the subdivision, the construction of which has recently been completed.

The 5.6 ha of future development lands owned by Mattamy outside of the current Urban Boundary within the BSUEA were also accounted for as part of the major overland flows tributary to the Todd Pond. In JFSA's 2015 Todd Pond Model Keeper Analysis, both minor and major system flows generated from these future lands were allocated under the 'ultimate' scenario – refer to Figure 3, prepared by JFSA and provided in **Appendix 'B6'**. It is noted that the ponding depths shown on Figure I-2, prepared by JFSA, consider this ultimate drainage area. The analysis concluded that in order for the Todd Pond to be able to accommodate minor and major flows from these future lands, the Todd Pond outlet weir would have to be extended from 30 m to 48 m in length.

B7.3.3 Clarke Pond

As per the Draft 2014 BSMSSA, the Clarke Pond is proposed to be located approximately 250 m north of Cambrian Road and approximately 780 m west of River Mist Road. It is intended to service the southwestern portion of Barrhaven South up to the southern limit of the current Urban Boundary. The analysis and sizing of the Clarke Pond was initiated by JFSA in 2011 and has since been updated in 2013 to incorporate the detailed designs of developments in Barrhaven South that were completed since 2011.

Minor System

The following planned trunk storm sewers were identified in the Draft 2014 BSMSSA to convey stormwater generated from the southern portion of Barrhaven South (up to the current Urban Boundary) to the Clarke Pond, as shown in **Figure 7-3**:

- Future Realigned Greenbank Road Trunk Storm Sewer
  This sewer is to extend approximately 1.2 km south of the Clarke Pond to the limit of the current Urban Boundary. At its upstream end, at the Urban Boundary, it is anticipated that, as per the Draft 2014 BSMSSA, this sewer will have a diameter of 825 mm, a slope of 1.0%, an obvert elevation of 102.95 (invert of 102.11), and a cover depth of 5.1 m.

- Future Collector Road Trunk Storm Sewer
  This sewer is to extend from the Clarke Pond to approximately 350 m south of Cambrian Road. At its upstream end, it is anticipated that, as per the Draft 2014 BSMSSA, this sewer will have a diameter of 1200 mm, a slope of 0.55%, an obvert elevation of 92.20 m (invert of 90.98), and a cover depth of 5.8 m.

- Future Cambrian Road Trunk Storm Sewer
  Planned sewers on Cambrian Road are to convey stormwater either towards the planned future realigned Greenbank Road trunk sewer or to the planned future Collector Road trunk storm sewer.

Due to the grade raise constraints within the northern portion of Barrhaven South and the normal water level in the Jock River, it is anticipated that the majority of the planned storm sewers outletting to the Clarke Pond will be partially submerged, as shown on the Draft 2014 BSMSSA Drainage Area Plan provided in **Appendix 'B6'**. Maintenance solutions to minimize the accumulation of sediments within the submerged sewers are to be addressed at detailed design, as indicated in the Draft 2014 BSMSSA.

Major System

Areas bounded by Cambrian Road and the future realigned Greenbank Road are to provide on-site storage to contain major system flow up to the 1:100 year storm such that the flows do not cross these arterials. Major system flows generated from lands located north of Cambrian Road and west of the future realigned Greenbank Road are to be conveyed directly to the Clarke Pond, as shown on the Draft 2014 BSMSSA Drainage Area Plan prepared by Stantec and provided in **Appendix 'B6'**.

B7.3.4 Cedarview Pond

Cedarview Pond is a future SWM facility that is proposed to be located immediately south of the Jock River, west of the future realigned Greenbank Road as per the Draft

2014 BSMSSA. The detailed design of the Pond has not yet been initiated; however, it is intended to serve as an outlet for planned minor system storm sewers servicing the northwestern portion of Barrhaven South (refer to Cedarview Pond Drainage Area Plan prepared by Stantec, provided in **Appendix 'B6'**). Furthermore, most major system flow generated within the Cedarview Pond tributary area is intended to be conveyed overland directly to the Jock River as shown on Stantec's Cedarview Drainage Plan in **Appendix 'B5'**.

B7.3.5 Greenbank Pond

Greenbank Pond is a future SWM facility that is proposed immediately south of the future realigned Greenbank Road and southwest of the Jock River as per the Draft 2014 BSMSSA. The detailed design of the Pond has been initiated by David Schaeffer Engineering Limited (DSEL) as part of Mattamy's Half Moon Bay Phase 7 Subdivision. The Greenbank Pond is to serve as an outlet for planned minor system storm sewers servicing the central northern portion of Barrhaven South - refer to the Draft 2014 BSMSSA Greenbank Pond Drainage Area Plan prepared by Stantec and provided in **Appendix 'B6'**. Furthermore, major system flow for tributary lands located south of the future realigned Greenbank Road is intended to be conveyed overland to the Greenbank Pond, while major system flow to the north of the future realigned Greenbank Road is intended to be conveyed overland directly to the Jock River, as per the Greenbank Pond Drainage Plan provided in **Appendix 'B6'**.

B8.0 OPPORTUNITIES AND CONSTRAINTS

B8.1 Drainage Network

The review of LiDAR data and the Headwater Drainage Feature Assessment concluded that there were no waterways or watercourses within the limits of the BSUEA. The lack of waterways is generally indicative of the low frequency in the flow generation potential for a given area. Based on the measured saturated hydraulic conductivities and corresponding calculated infiltration rates, the surface water flow component accounts for 1.2% of the total annual precipitation depth of 844 mm. As part of storm servicing, some constraints such as modification to drainage may be reviewed and/or permitted as part of the MSS. However, these modifications will need to follow the appropriate levels of approvals. Any modification to the hydrological regime of either Mud Creek (Thomas Baxter Municipal Drain) or Rideau River (Hawkins Municipal Drain) will need to be completed in accordance with the Drainage Act. Minor adjustments to the drainage divide between sub-watersheds are generally permitted while a significant diversion may trigger the requirement of additional investigations. This requirement would permit the evaluation of the impact on the natural environment and the identification of mitigation measures. Should there be an adjustment contemplated with the drainage divide, the MSS would then be the vehicle that would determine whether the proposed mitigation measures are adequate or, alternatively, if there will be a need to complete a Schedule C Class EA.

B8.1.1 Area Tributary to the Jock River

Historically, three (3) municipal drains provided conveyance to the Jock River. However, these drains are being enclosed and abandoned as the urban development extends south. The former Todd Municipal Drain is now enclosed downstream of the Todd Pond, the Clarke East Municipal Drain is now partially enclosed, and there are plans for the Clarke West Municipal Drain to be partially enclosed as development proceeds. Where partial enclosure is proposed, the works will be subject to the requirements of the Federal Fisheries Act. Modification or abandonment of these municipal drains will need to be completed in accordance with Section 78 of the Drainage Act, chapter D.17, R.S.O. 1990. Based on topography, approximately 99 ha of the study area (refer to the drainage limits depicted in **Figure 5-1**) is tributary to the Jock River.

B8.1.2 Area Tributary to Mud Creek

The Thomas Baxter Municipal Drain is immediately south of the Study Area at Barnsdale Road and carries flow from Barnsdale Road to Mud Creek. There are no plans to abandon or enclose the drain. Based on topography, approximately 11 ha of the southwestern portion of the study area (refer to drainage limits depicted on **Figure 5-1**) is tributary to Mud Creek via the above-noted Drain.

B8.1.3 Area Tributary to the Rideau River

The area tributary to the Rideau River drains via roadside ditches along Barnsdale Road and the existing Greenbank Road which in turn drains to the Hawkins Municipal Drain system to the east of the existing Greenbank Road and south of Barnsdale Road. Based on topography, approximately 12 ha of the southeastern portion of the study area (refer to the drainage limits depicted in **Figure 5-1**) is tributary to the Rideau River via the above-noted Drain.

B8.2 Aquatic Habitat and Natural Heritage

Natural heritage features have been identified by Dillon Consulting in their March 2016 report based on the fieldwork undertaken. The report was prepared to identify, amongst other features, wetlands and fish habitat. Based on this undertaking, there were no watercourses found within the study area, and the only water features identified were man-made ponds resulting from aggregate extraction.

B8.3 Floodplains

Lands which are part of the BSUEA are not restricted by any regulation floodplains.

B8.4 Water Quantity Control

Water quantity criteria will be affected by site constraints and municipal and provincial requirements. Water quantity control for lands that drain to the Jock River will be governed by available capacity in existing and planned minor and major systems in the existing Barrhaven South Community. Since the existing and planned infrastructure was designed to service lands up to the current Urban Boundary, there is limited or no available capacity in existing and planned minor and major systems. Table 8-1 summarizes the minimum residual capacity of trunk sewers, the minimum freeboard between USF, and the 1:100 year HGL and maximum street ponding depth for each stormwater management facility (SWMF) outlet. As shown in **Table 8-1** below, all minor systems are either close to or exceeding the minimum allowable freeboard between HGL and USF of 0.30 m as required by the Design Guidelines. Additionally, the major system through which the Todd Pond outlets does not meet the maximum allowable street ponding depth (static and dynamic) of 35 cm per Technical Bulletin PIEDTB-2016-01.

For lands that drain to Mud Creek, flows will need to be limited to the capacity of the Thomas Baxter Drain. Any additional flow discharge to the Thomas Gamble Drain should not aggravate the risk of flooding. Any modifications to this Drain or to its flow regime will need to be assessed and approved in accordance with Section 78 of the Drainage Act, Chapter D.17, R.S.O., 1990.

In addition, Barnsdale Road is classified as an arterial roadway, which means that major flow from events up to the 1:100 year may not be conveyed across it.

Similarly, lands tributary to the Rideau River will be subject to the capacity limits of the downstream watercourse, namely, the Hawkins Municipal Drain as well as the overland flow conveyance restrictions due to the arterial road. Any additional flow discharge to the Hawkins Municipal Drain should not aggravate the risk of flooding. Any modifications to this Drain will need to be assessed and approved in accordance with Section 78 of the Drainage Act, Chapter D.17, R.S.O., 1990.

Table 8-1: Available Capacity in Existing and Planned Minor and Major Storm Systems in Barrhaven South

Outlet (SWMF)	Sewer Status	Minimum available capacity in trunk storm sewers	Minimum clearance (freeboard) between 1:100 yr & USF	Maximum street ponding (static + dynamic)
Corrigan Pond	Existing	735 L/s (MH 105-106A) (1)	0.32 m @ MH 104B on the existing Greenbank Road (2)	0.35m (3)
Todd Pond	Existing	136 L/s (MH 643-640) (5)	0 m (i.e., no freeboard, HGL is 0.23 m above USF and 0.02 m below basement floor) (5)	11 roadway sags with flow depth in excess 400 mm (5)
Clarke Pond	Planned	237 L/s (MH 211-211A) (6)	Not available - conceptual HGL at 0.08 m below T/G of MH 304 (4)	Information has not been reported
Cedarview Pond	Planned	165 L/s (MH 506-505) (6)	Not available, conceptual HGL at 0.20 m below T/G of MH 304 (6)	Information has not been reported
Greenbank Pond	Planned	167 L/s (MH415-409) (6)	Not available, conceptual HGL at 0.47m below T/G of MH N403A (6)	Information has not been reported

References
(1) IBI Storm Sewer Design Sheet Dated May 2010
(2) IBI SWM Barrhaven South October 2014
(3) JLR Quinn's Pointe Site Servicing Report Dated July 29, 2016
(4) JFSA July 25, 2013 Todd Pond and Clarke Pond Model Keeper Analysis / Update of Half Moon Bay South Subdivision Plan
(5) JFSA April 2015 Todd Pond Model Keeper Re-Assessment of Existing System Capacity
(6) Stantec Draft 2014 BSMSSA

Within the BSUEA, the realigned Greenbank Road will be classified as an arterial road; major overland flow generated by events up to 1:100 year will, therefore, not be able to cross the realigned Greenbank Road. Since the realigned Greenbank Road will bisect the BSUEA, this restriction on overland flow conveyance will have implications for stormwater management within the development.

B8.5 Water Quality Control

Water quality treatment will need to be provided in accordance with the relevant subwatershed studies. The Jock River Reach One Subwatershed Study states that, for areas tributary to the Jock River, an enhanced level of water quality treatment, in accordance with the MOE 2003 SWMPDM, is required. It is noted that currently all existing and planned SWMFs in Barrhaven South exceed the MOECC water quality storage requirement to achieve an enhanced level of protection for their current drainage areas, as summarized in **Table 8-2,** below.

Table 8-2: Water Quality Storage Assessment for Existing and Planned SWMFs

SWMF	SWMF Status	Water Quality Storage (m^3) in Excess of Minimum MOECC Requirements	
		Wet Permanent Pool	Extended Detention Volume
Corrigan Pond	Existing	1700 (1)	100 (1)
Todd Pond	Existing	6936 (2)	Information has not been reported
Clarke Pond	Planned	2804 (3)	Information has not been reported
Cedarview Pond	Planned	211 (4)	Information has not been reported
Greenbank Pond	Planned	582 (4)	Information has not been reported

Notes
(1) As per IBI June 9, 2015 Letter Report - Corrigan SWM Facility
(2) As per JFSA April 2015 Todd Pond Model Keeper Re-Assessment of Existing System Capacity (Appendix E)
(3) As per JFSA July 25, 2013 Todd Pond and Clarke Pond Model Keeper Analysis / Update of Half Moon Bay South Subdivision Plan
(4) As per Stantec Draft 2014 BSMSSA

The Mud Creek Subwatershed Study does not make any specific water quality protection requirements but states that surface water contamination from point and non-point sources must be managed. Additionally, the Lower Rideau Subwatershed Study has set an objective to manage the quality and quantity of non-point source runoff and to manage surface and groundwater contamination from point source discharges.

B8.6 Infiltration

The Hydrogeological Study undertaken by Paterson confirms that, in general, soils within the BSUEA have a moderate to high hydraulic conductivity (refer to Figure C-1 and Figure C-2, **Appendix 'B3'**). The moderate are observed in the eastern portion while the highest are observed in the northwestern portion of the study area. Such soils are considered to act as a recharge zone for underlying aquifers. The presence of soils with high hydraulic conductivity means that the site is suitable for providing groundwater recharge and appropriate practices must be implemented to maximize the rate of infiltration.

Non-structural Best Management Practices such as discharging impervious runoff, including roof runoff, to lawns are required regardless of soil capacity, in accordance with the Jock River Reach One Subwatershed Study. In addition, the study should consider the potential to incorporate LID in areas that are conducive to infiltration, to reduce the anticipated infiltration deficit and maintain pre-development infiltration levels.

Based on fulfilling the objective of maintaining pre-development infiltration levels, storm servicing solutions incorporating infiltration measures will be developed at the MSS in order to achieve a water balance. These infiltration measures will be incorporated within all areas of the BSUEA, striving to maintain pre-development infiltration levels for the Jock River, Rideau River (via the Hawkins Municipal Drain), and Mud Creek (via the Thomas Baxter Municipal Drain) sub-watersheds. As part of storm servicing, some constraints such as modification to drainage may be reviewed and/or permitted as part of the MSS. However, these modifications will need to follow the appropriate levels of approvals. Any modification to the hydrological regime of either Mud Creek (Thomas Baxter Municipal Drain) or Rideau River (Hawkins Municipal Drain) will need to be completed in accordance with the Drainage Act.

Any future redevelopment of the sand and gravel resource area, as per Official Plan policy 3.7.4, should ensure that the groundwater recharge function be maintained.

The Mud Creek Subwatershed Study identified that there are significant groundwater recharge areas including the Kars Esker, a Mississippi-Rideau Sourcewater Protection Region Significant Groundwater Recharge Area, across the majority of the western portion of the BSUEA. These pre-development recharge conditions must be maintained. A water balance will be required to demonstrate adherence to this requirement. The Mud Creek Subwatershed Study also recommends avoiding infiltration of poor quality runoff from paved surfaces without pre-treatment, and to consider the use of LID and Best Management Practices for stormwater management quality and quantity control.

B8.7 Existing Servicing Infrastructure

The following opportunities and constraints have been identified for the existing and planned servicing infrastructure in Barrhaven South. These will be reviewed and evaluated in greater detail as part of the BSUEA MSS.

B8.7.1 Wastewater Servicing

Wastewater servicing for the BSUEA is ultimately governed by the available residual capacities in the existing downstream trunk sanitary sewers in Barrhaven South. Based on the theoretical available residual capacities for existing trunk sanitary sewers identified in Section B7.1 of this Report, an approximate equivalent residential development area was calculated using an average peak flow rate per hectare of 1.77 L/s/ha. This average peak flow rate is based on an average unit density of 34 units/ha (as per the 2006 Barrhaven South CDP) a population density of 2.7 persons/unit, an average flow rate of 350 L/cap/day, a residential peaking factor of 4.0, and an infiltration allowance of 0.28 L/s/ha in accordance with the Design Guidelines. It is noted that a peaking factor of 4.0 is relatively high; however, it provides a more conservative estimate. Based on an average peak flow rate of 1.77 L/s/ha, it appears that the existing trunk sanitary sewers could service the following future residential development:

Table 8-3: Available Tributary Area Capacity in Existing Trunk Sanitary Sewers

Trunk Sanitary Sewer	Residual Capacity (L/s)	Equivalent Residential Development Area (ha)
Existing Greenbank Road	295.4	167
Cambrian Road	51.4	29
River Mist Road	14.4	8

As indicated in Section **B7.1** of this Report, the existing Greenbank Road trunk sanitary sewer serves as the main outlet for the existing Barrhaven South sanitary sewer system, which includes the Cambrian Road and River Mist Road trunk sanitary sewers. Based on the theoretical peak flow calculations summarized in **Table 8-3** above, the existing Greenbank Road trunk sanitary sewer has sufficient capacity to accommodate the peak flows generated within the BSUEA. It can accommodate approximately 167 ha of additional residential area, which is more than the 121 ha area of the BSUEA. Furthermore, based on the conclusions of the 2013 IMP wastewater model, total peak flows conveyed downstream of the existing Greenbank Road trunk sanitary sewer at MH 522A (located at the intersection of Half Moon Bay Road, immediately south of the Jock River) should be limited to 590 L/s to prevent potential adverse impacts to the HGL in the existing downstream sanitary sewer system. Stantec concluded in the 2013 Wastewater Collection System Assessment that downstream wastewater infrastructure, including the SNC and WRC, could accommodate a peak flow of 590 L/s from the Barrhaven South Community and the BSUEA, with no risk of surcharging and/or basement flooding.

It should also be noted that based on the depth of the existing Greenbank Road trunk sanitary sewer at the current Urban Boundary (i.e., obvert elevation of 91.77 m) and the groundwater elevations observed by Paterson within the eastern portion of the BSUEA (i.e., ± 95.00 m), it is anticipated that a portion of the trunk sanitary sewer near the eastern limit of the BSUEA will be lower than the groundwater table. This could result in increased infiltration into the sewer system and lowering of the groundwater table.

B8.7.2 Water Servicing

Low pressures (i.e., less than 40 psi) are anticipated during domestic water demand conditions in the BSUEA in localized areas where the existing ground elevation is relatively high. Further opportunities and constraints will be identified upon completion of a detailed Hydraulic Network Analysis (HNA) as part of the BSUEA MSS.

B8.7.3 Stormwater Servicing and Stormwater Management

Stormwater servicing and SWM opportunities and constraints associated with each existing and planned SWMF outlet in the Barrhaven South Community are summarized in Table 8-4.

Table 8-4: Stormwater Servicing and SWM Opportunities and Constraints in Barrhaven South

SWMF Outlet	Opportunity	Constraint
Corrigan Pond	Available residual capacity in existing sewers and pond storage volume	Max Ponding depth of 35 cm & min HGL freeboard of 32 cm
Todd Pond	Available residual capacity in existing sewers pond storage volume	Max. ponding depth & min. HGL freeboard not being met
Clarke Pond	Available residual capacity in planned sewers and pond storage volume	Minimal HGL freeboard anticipated
Cedarview Pond	Available residual capacity in planned sewers and pond storage volume	Minimal HGL freeboard anticipated
Greenbank Pond	Available residual capacity in planned sewers and pond storage volume	Minimal HGL freeboard anticipated
Notes Refer to **Table 8-1** and **Table 8-2 above** for details		

Although all five (5) existing/planned SWMFs in Barrhaven South have residual water quality volumes, as shown in **Table 8-2**, and have available residual capacity in their associated trunk infrastructure, as shown in **Table 8-1**, all storm sewer systems in Barrhaven South are ultimately limited by high HGL elevations. The Corrigan and Todd Ponds are also limited by the major system ponding depths which are generally at or exceeding the maximum allowable depth of 0.35 m. Although the planned trunk storm sewers are anticipated to have high HGL elevations as per the Draft 2014 BSMSSA, opportunities to provide stormwater servicing and SWM for the BSUEA via the three (3) planned wet pond facilities (Clarke, Cedarview and Greenbank) and their associated trunk storm sewers will be reviewed in detail and evaluated as part of the BSUEA Master Servicing Study (MSS).

In addition, as noted in Section **B7.3.2** of this Report, both minor and major system flows generated from the 5.6 ha parcel of land owned by Mattamy within the BSUEA (i.e., east of the future realigned Greenbank Road) were allocated to the Todd Pond under an 'ultimate' scenario as part of the 2015 Clarke Pond Model Keeper Analysis completed by JFSA. Their analysis showed that the Todd Pond can provide adequate water quality control for this future development land if the length of the facility outlet weir is extended from 30 m to 48 m. Servicing this 5.6 ha land parcel via the Todd Pond would, however, require that the City accept a deviation from their Guidelines with respect to minimum freeboard to the underside of footing, as well as to the maximum street ponding depth.

B9.0 SUMMARY AND RECOMMENDATIONS

The following provides a series of recommendations to be considered when preparing the BSUEA MSS, in the context of the opportunities and constraints discussed above.

B9.1 Wastewater Servicing

Assuming that the Brazeau and Drummond lands (totalling ± 45 ha) will ultimately be developed as commercial/employment, it is anticipated that the northern portion of the BSUEA will be serviced by the planned trunk sanitary sewers along the Future Collector Road and/or the future realigned Greenbank Road, which both converge at the Cambrian Road trunk sanitary sewer (refer to **Figure 7-1**). It should be noted that **Figure 7-1** indicates obvert elevations at the upstream end of the Future Collector and future realigned Greenbank Road trunk sanitary sewers, in accordance with the Draft 2014 BSMSSA; however, opportunities to reduce the slope and lower the upstream obvert elevation of these future sewers will be reviewed as part of the MSS. Revisions to the design of these future sewers at the MSS stage are anticipated, to improve the serviceability of the existing low-lying aggregate pit areas within the northern portion of the BSUEA.

It is anticipated that the 5.6 ha parcel of land owned by Mattamy east of the future realigned Greenbank Road within the BSUEA will be serviced by the existing River Mist Road trunk sanitary sewer as per the existing Half Moon Bay Phase 3 design. It is also anticipated that the Mattamy lands west of the future realigned Greenbank Road will outlet to the River Mist Road trunk sanitary sewers either via the Half Moon Bay or Quinn's Pointe local sanitary sewers. Based on the natural topography of the remaining 66 ha of the BSUEA, it is anticipated that these lands could outlet by gravity to the existing Greenbank Road trunk sanitary sewer.

To minimize infiltration into the existing Greenbank Road sanitary sewer system where it is anticipated to be below the groundwater table (i.e., eastern portion of the BSUEA), the following measures can be assessed and implemented:

- Concrete pressure pipe (CPP);
- Higher class PVC pipes;
- Higher percentage silica fume in maintenance hole structures;
- Cretex seals in maintenance holes; and

- Clay seals along the service trench.

B9.2 Water Servicing

Water servicing for the BSUEA will be developed to fulfill regulatory requirements under both domestic and fire flow conditions. It is anticipated that there will be an area in the BSUEA that will not fulfill the minimum pressure requirements during domestic demand conditions, due to topography. As a result, this localized area may require a booster pumping station or, alternatively, oversized services combined with jet pumps. This condition, as well as water servicing specifics such as watermain looping and sizing of the feedermains for the BSUEA, will be addressed at the MSS stage by a hydraulic network analysis (HNA). The HNA for the BSUEA MSS will be carried out in accordance with the most recent Design Guidelines and Technical Bulletins.

B9.3 Stormwater Servicing and Stormwater Management

The planned trunk storm sewer along the Future Collector Road, which outlets to the future Clarke Pond, is anticipated to have the lowest upstream obvert elevation (at the northern limit of the BSUEA) compared to the other planned and existing trunk storm sewers surrounding the Study Area, as indicated in **Figure 7-3**. For this reason, the Future Collector Road trunk storm sewer would be the optimal minor system outlet for the existing low-lying aggregate pit areas within the northern portion of the BSUEA. This servicing strategy would, however, have to be reviewed in conjunction with JFSA (the Clarke Pond model keeper) to assess whether the planned Future Collector trunk storm sewer and the Clarke Pond could be designed to accommodate these additional lands.

Opportunities to utilize the future Cedarview Pond as a minor system outlet for the BSUEA will also be reviewed and assessed as part of the MSS. It should be noted that since both the future Clarke and Cedarview Ponds are planned to be located north of Cambrian Road, which is an arterial roadway, major system flows up to the 1:100 year would have to be contained upstream of Cambrian Road using a dry pond facility.

Should a new wet pond facility be required to provide SWM for the southern portion of the BSUEA, then the naturally low-lying topography at the southeastern corner of the Study Area would be the optimal location. It is anticipated that this pond could provide both stormwater quantity and quality control and could potentially outlet to the existing Greenbank Road trunk storm sewer. Quantity control would be required so that the integrity of the existing Greenbank Road storm sewer system is maintained. Major system flows could also be conveyed overland to this future wet pond, with the exception of those generated west of the future realigned Greenbank Road. Since this road is designated as an arterial roadway, major system flow up to the 1:100 year event generated within the BSUEA lands west of the future realigned Greenbank Road would either have to be contained within a separate dry pond facility or captured via an oversized minor system and conveyed beneath the future realigned Greenbank Road.

Depending on the preferred SWM selected to service the BSUEA, it is anticipated that diverting some stormwater away from the Thomas Baxter Municipal Drain (Mud Creek) towards the Jock River may occur. The impact of diverting a small proportion of the watershed should have minor significance to the watercourse since the continuous simulation modeling found that there is no surface runoff response in events smaller than a 1:5 year event. The MSS will outline any requirements, triggered by a change in watershed boundaries, to meet the Municipal Class Environmental Assessment process, including if a Schedule C EA is required as a result of the selected SWM strategy.

LID measures to promote infiltration are to be incorporated as part of the overall servicing of the BSUEA to achieve the targeted water balance for each of the three (3) subwatersheds. Special attention should be devoted towards infiltrating “clean water” across the area of the Kars Esker while Best Management Practices (BMPs) to reduce runoff should be included in all locations where there is an increase in imperviousness. BMPs can include draining roofs to rear-yards and increasing vegetation areas.

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APPENDIX 'B1'

SYNOPSIS OF BACKGROUND DOCUMENTS

APPENDIX 'B2'

HYDROLOGICAL MODELING PARAMETERS AND RESULTS

A P P E N D I X ' B 3 '

WATER BUDGET ANALYSIS

APPENDIX 'B4'

WASTEWATER SERVICING

APPENDIX 'B5'

WATER SERVICING

APPENDIX 'B6'

STORMWATER SERVICING