Mahogany Subdivision Phases 2-4 – Functional Servicing Report

Project #160410140

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Sign-off Sheet

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1.0 INTRODUCTION

Minto Group Inc. has commissioned Stantec Consulting Ltd. to prepare the following Functional Servicing Report for Phases 2 through 4 of the Mahogany Subdivision. The subject property is located northwest of the intersection of Century Road and Manotick Main Street within Manotick Village. The overall Mahogany subdivision is currently zoned Development Reserve (DR) and is bordered by Manotick Main Street Road to the east, Century Road to the south, First Line Road and existing rural residential development to the west and existing residential development along Potter Drive to the north. The property is indicated in **Figure 1**. The proposed residential development phases comprise approximately 45.0ha of land and are to contain a mixture of single family units, townhomes, a school block, as well as neighbourhood parks and greenspace consistent with current secondary policies.

Figure 1: Approximate Location of Mahogany Phases 2-4 Draft Plan Area

1.1 OBJECTIVE

The intent of this report is to build on the servicing principles outlined in the Master Serviceability Study for Mahogany in Manotick (MSS) and later updated for Phase 1 of the Mahogany Subdivision through the Servicing Report, Phase 1 – Mahogany Community Manotick to create a servicing strategy specific to the subject property. The report will establish criteria for future detailed design of the subdivision, in accordance with the associated background studies, City of Ottawa Guidelines, and all other relevant regulations

2.0 BACKGROUND

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

- Amendment to the Manotick Servicing Options Study, Robinson Consultants Inc., December 2000.
- Master Serviceability Study for Mahogany Community Village of Manotick (City of Ottawa), David McManus Engineering Ltd., June 2007.
- Mahogany Community, Manotick – Development Concept Plan, FoTenn Consultants, January 2008.
- Village of Manotick Environmental Management Plan – Special Design Area Component, Marshall Macklin Monaghan Limited, June 2006.
- Mahogany Community Stormwater Management Servicing Report, IBI Group, July 2007.
- Servicing Report Phase 1 – Mahogany Community Manotick, exp, April 2011.
- Minto Communities Inc. Mahogany Sewage Pumping Station and Sewage Forcemain – Preliminary Design Report, J.L. Richards & Associates Ltd., February 2011.
- Final Geotechnical Investigation – Mahogany Community, Paterson Group Inc., November 2007.
- Phase 1 Environmental Site Assessment – Century Road at First Line Road, Paterson Group Inc., January 2007.
- Mahogany Community Phase 1 Stormwater Management and Fish Habitat Enhancement of the Unnamed Drain (Mahogany Creek), IBI Group, November 2010.
- Wilson Cowan Municipal Drain Report, A.J. Robinson & Associates Inc., July 1983.
- Wilson Cowan Drain Fluvial Geomorphic Existing Conditions, Minto Mahogany, Matrix Solutions Inc., May 2017.
- Groundwater Impact Assessment – Proposed Residential Development – Mahogany Community Development, Paterson Group Inc., June 9, 2017.
- North Island Link Watermain Class Environmental Assessment (Draft), Morrison Hershfield, January 30, 2017.
- City of Ottawa Sewer Design Guidelines, 2nd Ed., City of Ottawa, October 2012.
- City of Ottawa Sewer Design Guidelines, 2nd Ed., City of Ottawa, October 2012.
- City of Ottawa Design Guidelines – Water Distribution, Infrastructure Services Department, City of Ottawa, First Edition, July 2010.

3.0 POTABLE WATER ANALYSIS

3.1 BACKGROUND

The Village of Manotick is currently operating at Zone 3SW pressure within the City of Ottawa's water distribution system. Zone 3SW is serviced by the Barrhaven Pumping Station and the Fallowfield Road Pumping Station. However, following the South Urban Community (SUC) zone reconfiguration, a portion of Zone 3SW will be converted to Zone SUC in 2018. Following the zone reconfiguration, the Village of Manotick will be transferred to the SUC Pressure Zone and is expected to operate at an HGL of approximately 146m. Water to the SUC PZ will be supplied by both the Ottawa South Pumping Station and the Barrhaven Pumping Station.

3.2 PROPOSED SURROUNDING WATER INFRASTRUCTURE

The 610mm diameter Manotick Link (MWL Phases 1 and 2) and 610mm diameter North Island Link (NIL) are proposed for the near future. These watermains cross the Rideau River and are intended to provide additional transmission and redundancy to the Zone 3SW area, including the Village of Manotick. The City has indicated that timing for construction of these watermains is still uncertain.

Phase 1 of the MWL will connect to the existing main within River Road immediately south of Earl Armstrong and will extend south along River Road and cross the Rideau River immediately north of Walter Upton Collings Park. Phase 2 of the MWL will connect to the future MWL, and travel south along Long Island and Van Vilet Road, ultimately connecting to the future extended Main Street watermain north of Antochi Lane. The North Island link will connect to the future MWL Phase 1 in David Bartlett Park north of Bravar Drive and connecting to the existing 400mm diameter watermain within Rideau Valley Drive. The approximate alignments of the MWL Phases 1 and 2 and NIL are shown in **Figure 3.1**.

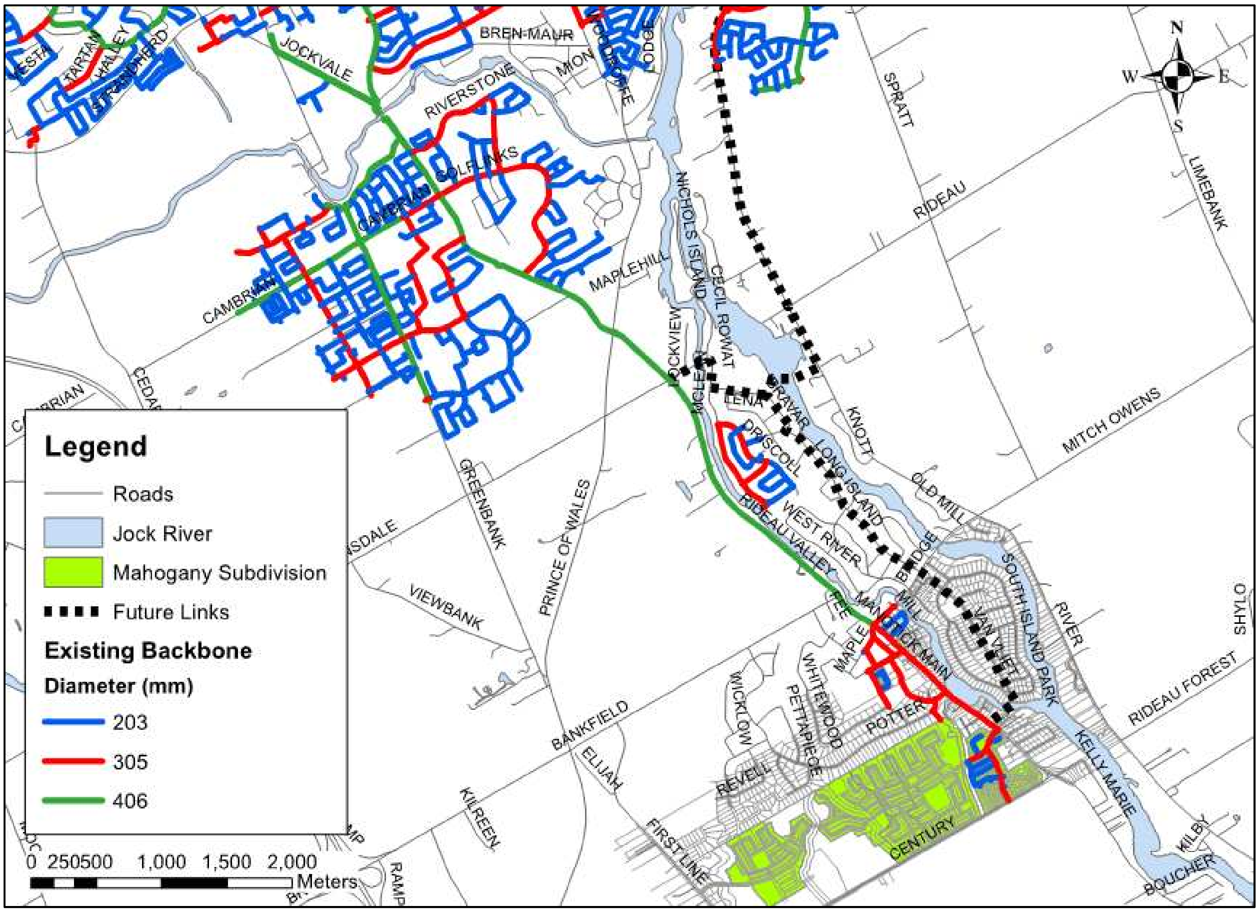


Figure 3.1: Future Manotick and North Island Watermain Link

3.3 PHASING

Figure 3.2 shows the phasing plan of the Mahogany Subdivision which is to be developed in five (5) phases. Phase 1 was allowed to proceed without a second feedermain to Manotick and has been completed.

This analysis herein has considered one interim scenario considering available fire flows and operating pressures of the existing Phase 1 and Phases 2-4 with the MWL Phase 1 and NIL in place, and without construction of MWL Phase 2. The ultimate (full build-out) scenario has also been reviewed as part of this overall servicing plan along with installation of MWL Phase 2.

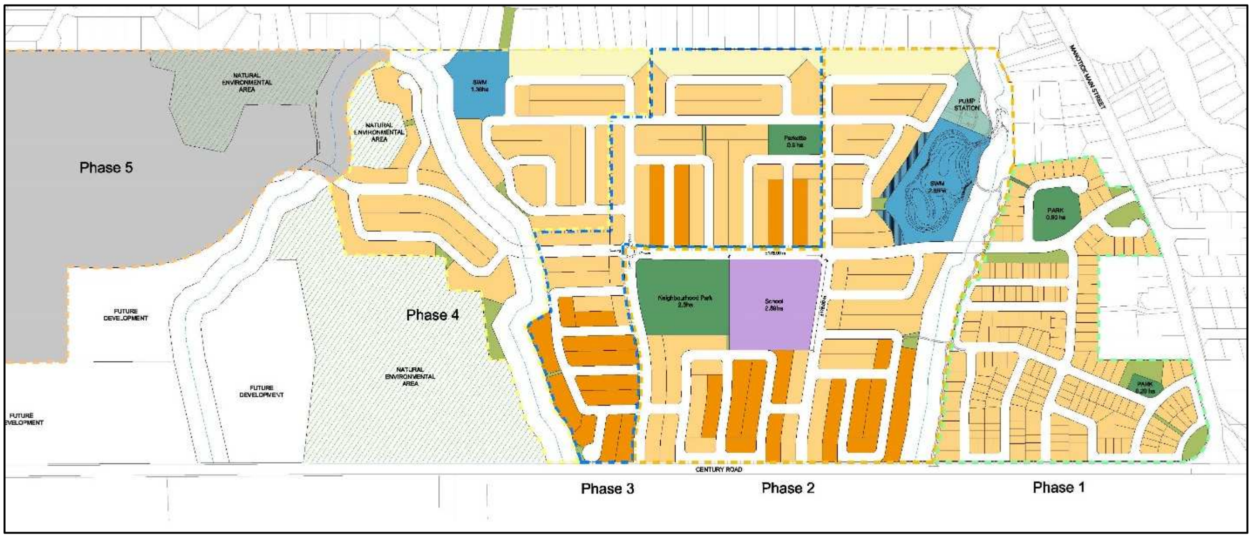


Figure 3.2: Phasing of Mahogany Subdivision

3.4 WATER DEMANDS

The latest plan for Phases 1 - 4 of the Mahogany Subdivision calls for a total of 1,108 units and a population of 3,637 (including Phase 1 which will be entirely built out in early 2019). Upon full buildout of the Mahogany subdivision, the number of units will not exceed 1400.

Water demands were estimated using the City of Ottawa's Water Distribution Design Guidelines (City of Ottawa, 2010). The population was estimated using a persons per unit (PPU) density of 2.7 for townhomes 3.4 for single homes. For residential demands, the basic day (BSDY) per capita water consumption rate is 350 L/cap/d. For maximum day (MXDY) demand, BSDY is multiplied by a factor of 2.5 and for peak hour (PKHR) demand, MXDY is multiplied by a factor of 2.2. The calculated residential water demand is represented in **Table 3.1**.

Phase 2 contains a proposed school lot and as such, non-residential demand was estimated using a consumption rate of 28,000 L/ha/d as per the City of Ottawa's Water Distribution Design Guidelines (City of Ottawa, 2010) and shown in **Table 3.2**. For MXDY demand, BSDY was multiplied by a factor of 1.5 and for PKHR demand, MXDY was multiplied by a factor of 1.8.

Table 3.1: Residential Population and Water Demands

Phase	Unit Type	Units	Population	BSDY (L/s)	MXDY (L/s)	PKHR (L/s)
1	Singles	211	717	2.91	7.27	15.98
	Towns	0				
2	Singles	256	1027	4.19	10.46	23.02
	Towns	58				

3	Singles	246	1233	5.02	12.56	27.63
	Towns	147				
4	Singles	190	646	2.62	6.54	14.39
	Towns	0				
Total		***1108***	***3637***	***14.73***	***36.83***	***81.02***

Table 3.2: Non-Residential Water Demands

Phase	Institution	Area (ha)	BSDY (L/s)	MXDY (L/s)	PKHR (L/s)
2	School	2.891	0.94	1.41	2.53

The total water demand estimation for residential and non-residential consumption in each phase is presented in **Table 3.3**.

Table 3.3: Total Water Demands

Phase	BSDY (L/s)	MXDY (L/s)	PKHR (L/s)
1	2.91	7.27	15.98
2	5.12	11.87	25.55
3	5.02	12.56	27.63
4	2.62	6.54	14.39
Total	***15.67***	***38.23***	***83.55***

3.5 PROPOSED WATERMAIN SIZING AND LAYOUT

The proposed watermain sizing for this development is comprised of 203mm diameter and 305mm diameter piping as shown in **Figure 3.3**, and demonstrates the watermain sizes and alignment proposed for Phases 2-4.

Connection locations to existing water infrastructure were identified in the MSS (DME, 2007). Two connections currently exist from the subdivision, one to the 300mm diameter watermain on Potter Drive (north) circling the proposed location of SWM Pond 2 and looping to the 300mm diameter watermain on Manotick Main via Bridgeport Avenue through Phase 1 of the development.

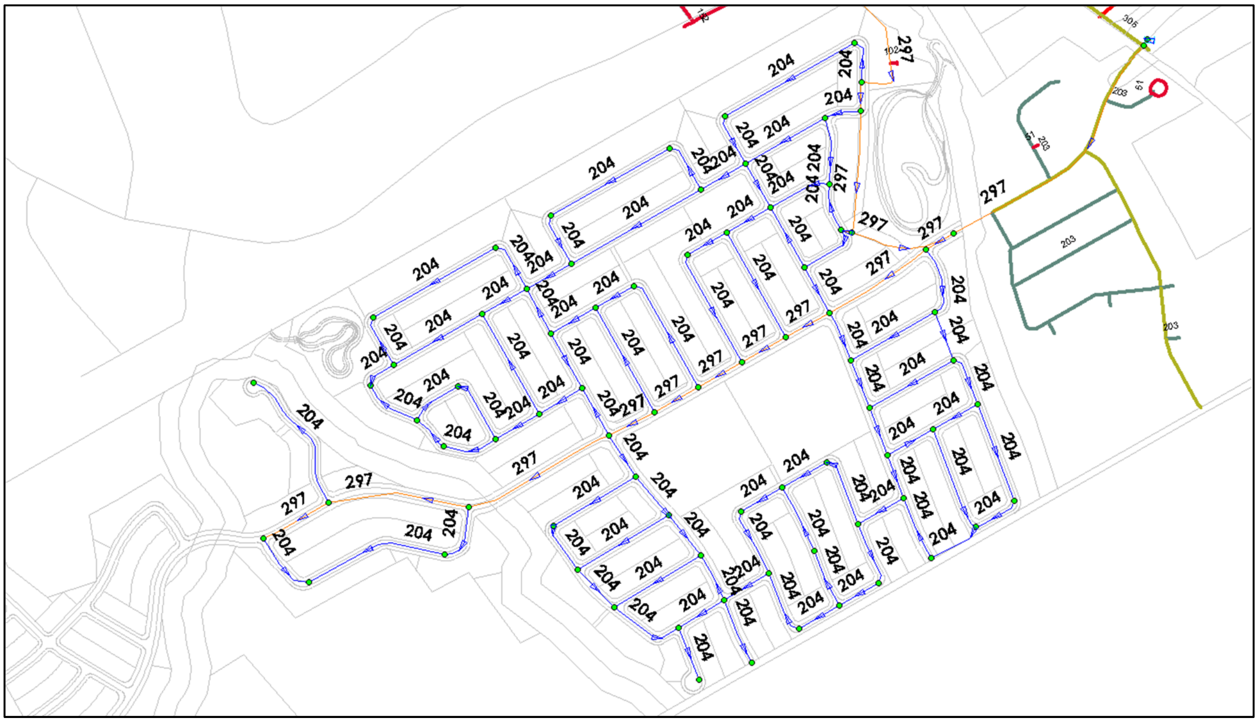


Figure 3.3: Proposed Watermain Sizing and Layout

3.6 LEVEL OF SERVICE

3.6.1 Allowable Pressures

The City of Ottawa Water Distribution Design Guidelines state that the desired range of system pressures under normal demand conditions should be in the range of 350 to 552 kPa (50 to 80 psi) and no less than 275kPa (40 psi) at the ground elevation (i.e. at hydrant level).

The maximum pressure at any point in the distribution system is 552kPa (80 psi). If pressures greater than 552kPa (80 psi) are anticipated, pressure relief measures are required. Under emergency fire flow conditions, the minimum pressure in the distribution system is allowed to drop to 138kPa (20 psi).

3.6.2 Fire Flow

A maximum fire flow for the development has been calculated based on the Fire Underwriters Survey (FUS) requirements. Townhome rows can result in a large required FUS fire flow and are typically the largest fire flow requirement in a subdivision. A typical square footage of 1,800 per townhome for a row of four (4) was used for the FUS calculations and results in a total required fire flow of 12,000L/min (200 L/s) for a duration of 2.5 hours. Additionally, a conceptual townhouse block was assessed in consideration of 2hr fire separations between each two (2) units within a townhouse block, resulting in a required fire flow of 7,000L/min (116.7L/s).

A typical single family home was also assessed at a square footage of 3,000 (1,500 sq.ft. per floor) with minimum building separation of 3m resulting in a total required fire flow of 8,000L/min (133 L/s). Refer to **Appendix A** for FUS fire flow calculations.

The City of Ottawa has recently implemented a policy that caps fire flow requirements for typical one and two storey wood frame, residential buildings with appropriate rear yard spacing to a maximum of 10,000L/min (technical bulletin ISDTB-2014-02).

Non-residential development in this area includes one school that will cover approximately 2.89ha. The area of the school building was estimated to be approximately 4,500m^2 (0.45 ha) and was determined to require a minimum fire flow of 9,000 L/min (**Appendix A**).

Local internal watermains must be assessed and verified for FUS fire flow requirements as development planning proceeds. The future Manotick links will improve overall transmission, reliability, and available fire flows to the subdivision; however, where required, mitigation measures such as fire walls, fire resistant construction materials, and/or increased spacing would need to be considered to reduce the fire areas and lower the overall fire flow requirements during the interim.

3.7 HYDRAULIC ANALYSIS

A hydraulic model was built by Stantec using the boundary conditions obtained via the latest South Urban Community (SUC) water model in consideration of future scenarios including construction of the MWL Phases 1 and 2 and NIL. Stantec assessed the anticipated pressures in this development to meet minimum servicing requirements (basic day and peak hour demands). A fire flow analysis was also performed under maximum day conditions.

3.7.1 Boundary Conditions

The boundary conditions provided were based on computer model simulations and are summarized in **Table 3.4** and **Table 3.5**.

Table 3.4: Boundary Conditions for Proposed Connection Points (Interim)

Scenario	HGL (m)		Q (L/s)	
	BC#1 (Potter)	BC#2 (Manotick)	BC#1 (Potter)	BC#2 (Manotick)
BSDY	146.0	146.0	N/A	N/A
PKHR	139.3	139.2	N/A	N/A
MXDY + FF167L/s	109.6	108.7	129.8	76.2
MXDY + FF133L/s	119.1	118.5	108.4	63.6

Table 3.5: Boundary Conditions for Proposed Connection Points (Ultimate)

Scenario	HGL (m)		Q (L/s)	
	BC#1 (Potter)	BC#2 (Manotick)	BC#1 (Potter)	BC#2 (Manotick)
BSDY	146.3	146.3	N/A	N/A
PKHR	136.8	137.3	N/A	N/A
MXDY + FF167L/s	128.9	130.4	115.4	90.5
MXDY + FF133L/s	133.2	134.3	96.2	75.7

3.7.2 Model Development

New watermains were added to the hydraulic model to simulate the proposed distribution system. Hazen-Williams coefficients ("C-Factors") were applied to the new watermain in accordance with the City of Ottawa's Water Distribution Design Guidelines (**Table 3.6**).

Table 3.6: C-Factors for Modeled Watermains

Pipe Diameter (mm)	C-Factor
150	100
200 to 300	110
350 to 600	120
> 600	130

3.7.3 Ground Elevations

The elevations shown on **Figure 3.4** below were interpolated from the overall preliminary grading plan for the development and assigned to the nodes in the hydraulic model. The ground elevations of the proposed Phases 2, 3 and 4 range from approximately 89.7m to 90.7m.

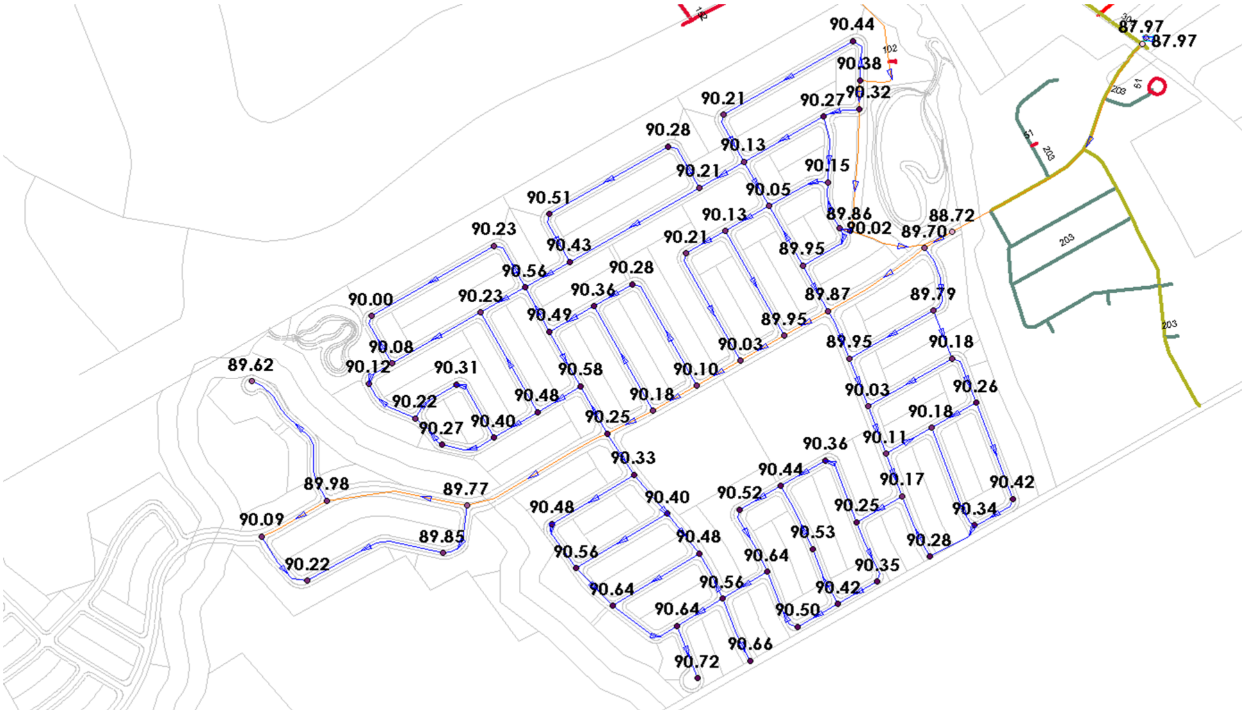


Figure 3.4: Ground Elevations (in meters)

3.8 HYDRAULIC MODELING RESULTS

The software package used to carry out the analysis was H_2OMAP Water by Innovyze. The model was tested for BSDY, PKHR and MXDY+FF demands using the boundary conditions provided.

3.8.1 Basic Day

The hydraulic model results show that during basic day conditions, the maximum pressure is anticipated to be 557kPa (81 psi) for the interim scenario, and 572kPa (83 psi) under ultimate conditions. The maximum objective pressure of 552kPa (80 psi) is approached or exceeded for several areas within Phase 2. Hydraulic modeling results are demonstrated in **Figures 3.5 and 3.6.**

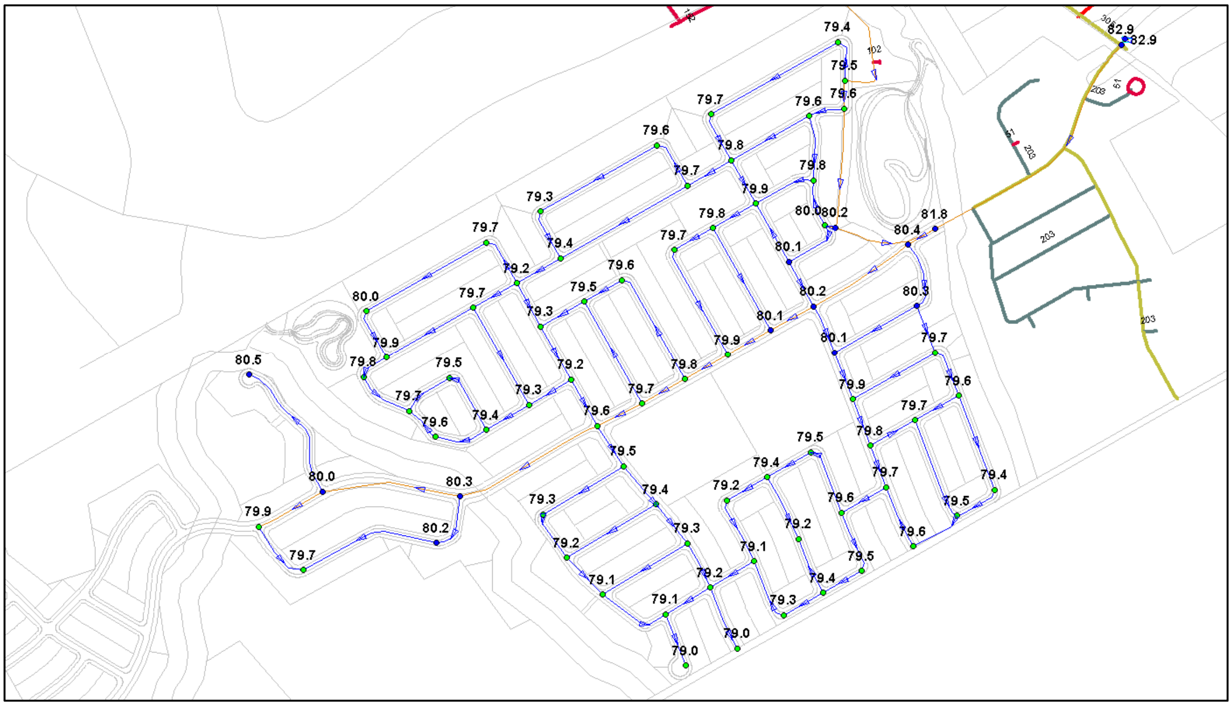


Figure 3.5: Ultimate Scenario - Maximum Pressures During BSDY Conditions

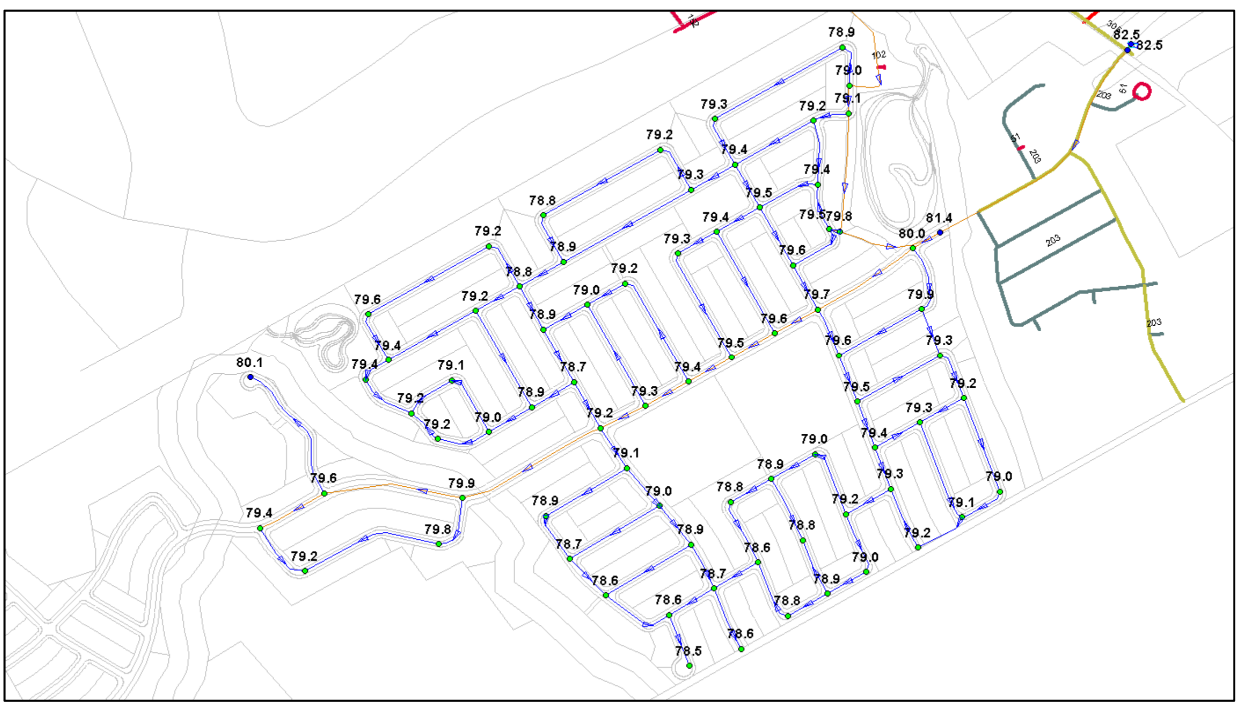


Figure 3.6: Interim Scenario - Maximum Pressures During BSDY Conditions

The Ontario Building Code (OBC) states that the static pressure shall not exceed 552 kPa (80 psi) in areas that are occupied in the water distribution system. The City's Water Distribution Design Guidelines technical bulletin (ISDTB-2014-02) further states that for areas that exceed 552 kPa (80 psi), pressure reducing valves (PRVs) should be installed immediately downstream of the isolation valve in the home/building. The requirement for PRVs will be assessed at the detailed design stage to limit maximum pressures at all units to a maximum of 80 psi.

3.8.2 Peak Hour

Minimum pressures during PKHR are anticipated to be between 495kPa (72 psi) and 488kPa (71 psi) for the interim scenario and between 468kPa (68 psi) and 445kPa (65 psi) under ultimate conditions. These pressures are well above the minimum objective pressure of 276kPa (40 psi).

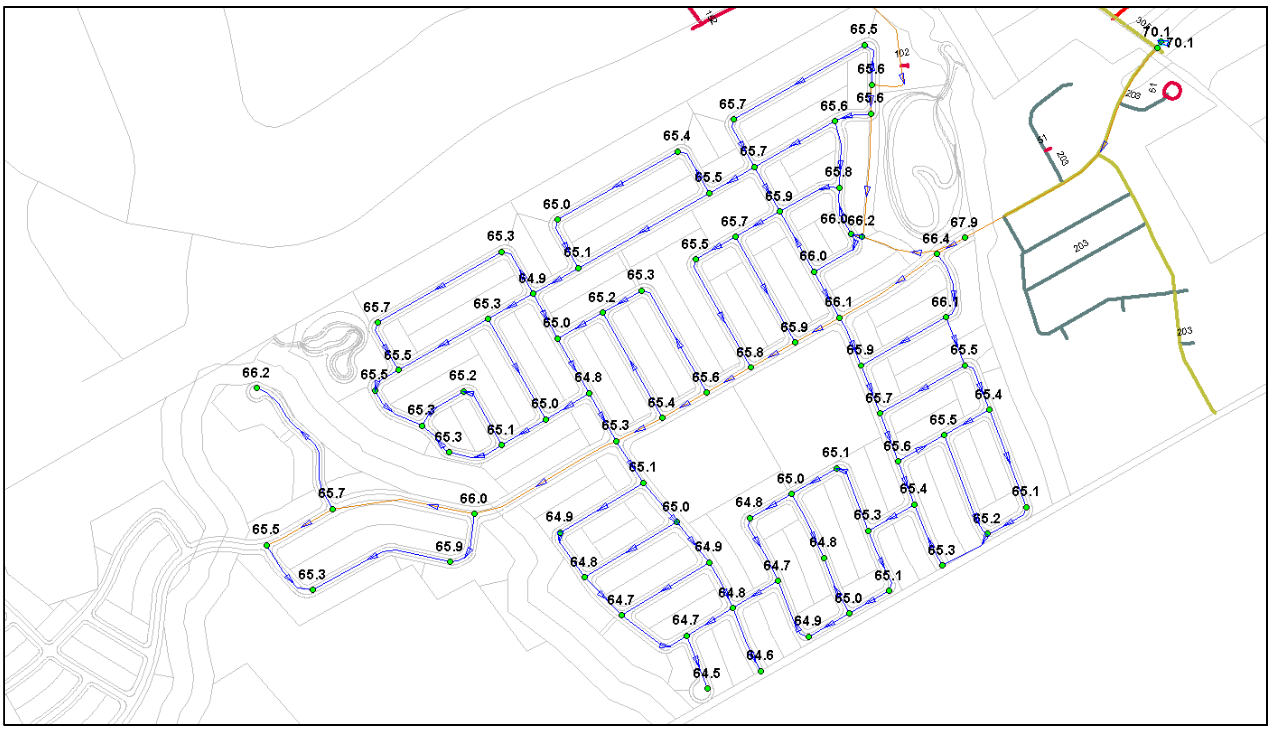


Figure 3.7: Ultimate Scenario - Minimum Pressures During PKHR Conditions

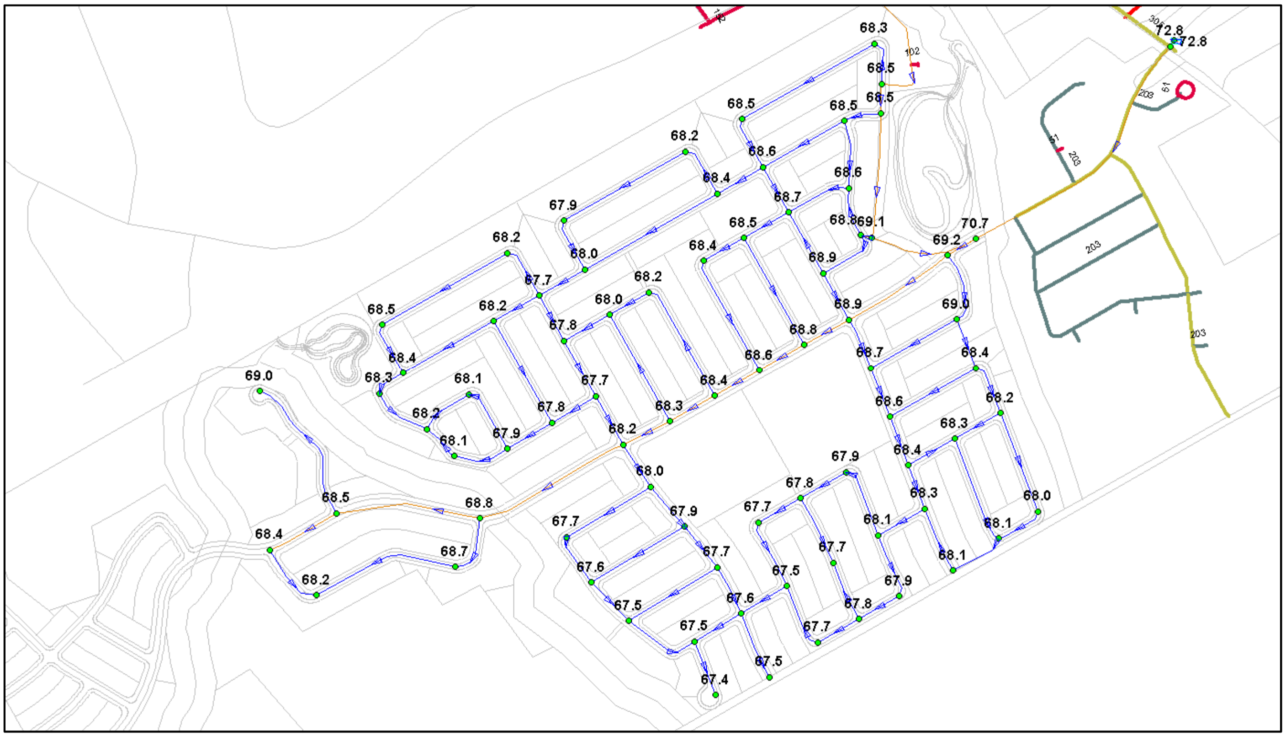


Figure 3.8: Interim Scenario - Minimum Pressures During PKHR Conditions

3.8.3 Maximum Day Plus Fire Flow

An analysis was carried out using the hydraulic model to determine if the proposed interim and ultimate buildout scenarios of the development can achieve the required FUS fire flow while maintaining a residual pressure of 138kPa (20 psi). This was accomplished using a steady-state maximum day demand scenario along with the automated fire flow simulation feature of the software. Maps of the available fire flows are presented in **Figure 3.9** and **Figure 3.10**.

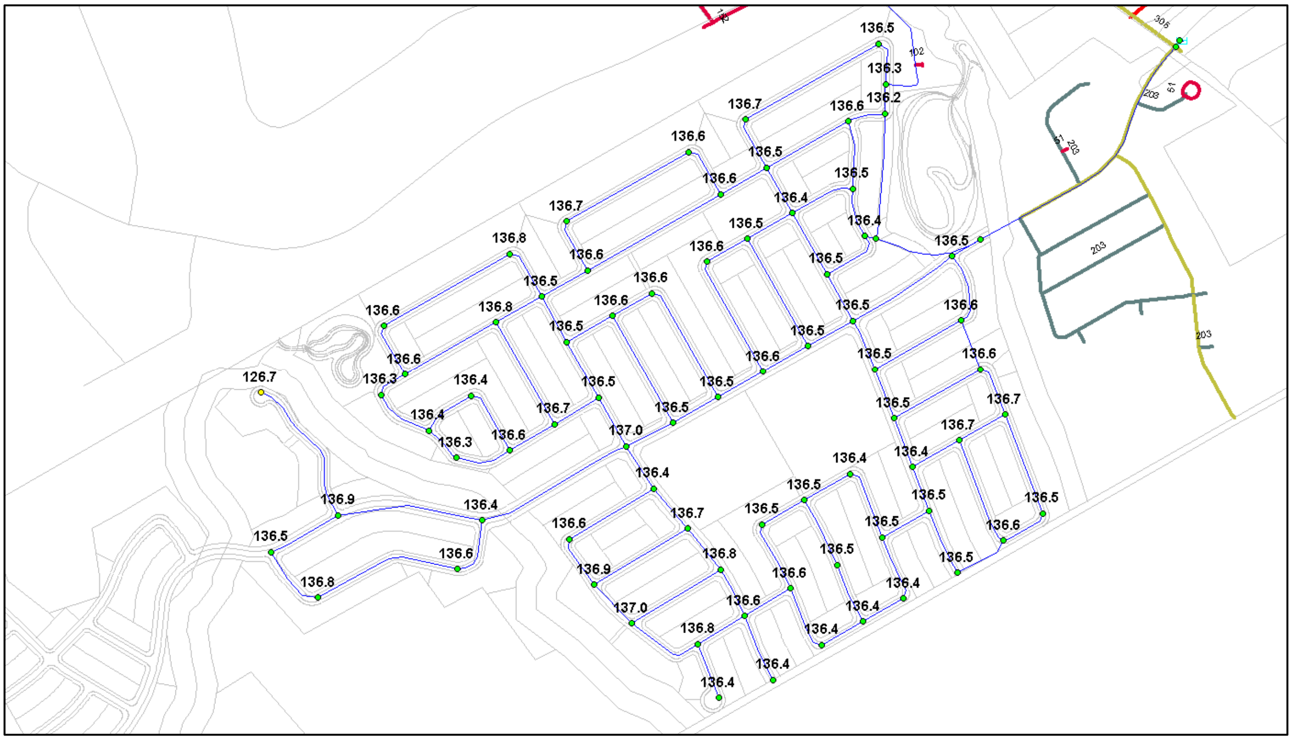


Figure 3.9: Ultimate Scenario - Available Flow (L/s) at 20 psi During MXDY+FF (8,000L/min) Conditions

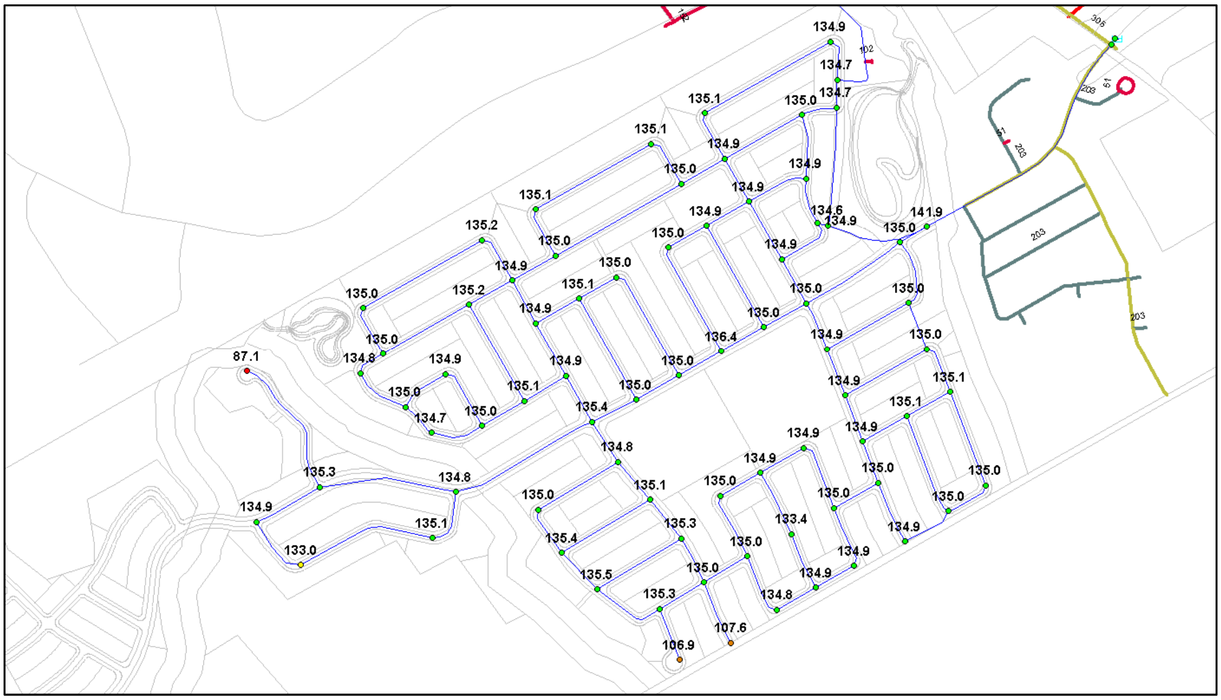


Figure 3.10: Interim Scenario - Available Flow (L/s) at 20 psi During MXDY+FF (8,000 L/min Conditions

The modeling results show the vast majority of the development will be able to achieve fire flows above 8,000 L/min under both interim and ultimate scenarios, with the exception of the extreme southwest corner of the development and the cul-de-sac within Phase 4 during the interim scenario. Construction of the MWL will allow both areas to achieve fire flows at 8,000L/min (within 5% of required value). Additional looping for these areas may be assessed at detailed design should construction in advance of the MWL Phase 2 be required to achieve the 8,000L/min target. Wood frame structures will be required to maintain a minimum 3.0m separation under this scenario to permit FUS requirements at or below the 8,000 L/min value. Additional fire separations within townhouse units will be required (to be assessed at the detailed design stage) to ensure the 8,000 L/min maximum required fire flow is not exceeded. Should townhomes or wood frame single family homes with less than 3m separation be proposed for the area, they will be required to provide mitigating measures (2h fire separations, reduced floor areas, etc.) to ensure overall fire flow requirements are below 8,000 L/min for these areas. Hydrants for fire suppression are to be located as near as possible to looped watermain sections to avoid the headloss inherent with dead end mains and provide the maximum available flow.

The ultimate condition development was further assessed under a fire flow demand of 10,000 L/min, the results of which are indicated in **Figure 3.11** below.

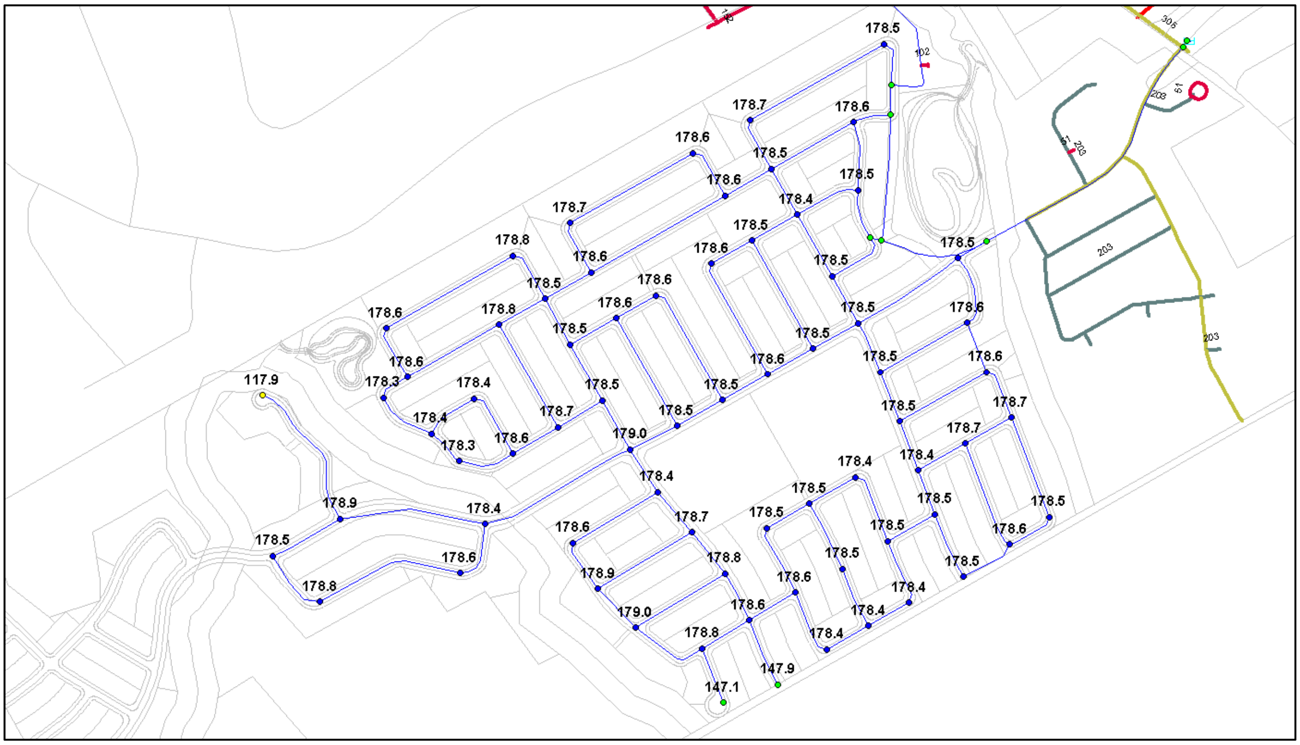


Figure 3.11: Ultimate Scenario – 10,000 L/min FF Conditions (L/s)

These results indicate that fire flows of 10,000 L/min mayl be achieved for the vast majority of the development under the ultimate development scenario.

Looping requirements within the development are achieved as there are two connections to existing watermains; however, until both of the Manotick Links are completed, the Mahogany Subdivision remains within a vulnerable service area with the Rideau Valley watermain acting as the only feed to Manotick. The system will not be able to provide basic day demand plus fire with a break in the Rideau Valley watermain.

4.0 WASTEWATER SERVICING

4.1 BACKGROUND

As indicated in the Master Servicing Study for Mahogany in Manotick (MSS), wastewater servicing for the development is conveyed to a trunk sewer within Bridgeport Avenue and through an easement within the anticipated Pond 2 block to the Eastman Pump Station constructed as part of Phase 1 of the overall development. The MSS also outlines the sanitary servicing requirements for the subject property, which identify an integrated network within Minto and further lands to the west, as well as future gravity sewer connections from existing residential developments further north as recommended by the MSS. The Sanitary Drainage Plan for the development is included in **Appendix B**. The Master Servicing Study and ECA for the pump station also include development phasing options relating to pump replacements for increasing peak inflows as the development progresses westwards. Flows are pumped via twin 200mm diameter forcemains to the existing gravity sanitary sewer at the intersection of Gladdis Court and Eastman Drive.

The report identifies an ultimate peak sanitary discharge of approximately 116L/s from the development, which includes drainage from the entirety of the existing phase 1 area, as well as future lands forming development phases 5-7.

4.2 DESIGN CRITERIA

As outlined in the City's Sewer Design Guidelines, the following design parameters were used to calculate estimated wastewater flow rates and to preliminarily size on-site sanitary sewers:

- Minimum Full Flow Velocity – 0.6 m/s
- Maximum Full Flow Velocity – 3.0 m/s
- Manning's roughness coefficient for all smooth walled pipes – 0.013
- Single Family Persons per unit – 3.4
- Townhouse Persons per unit – 2.7
- Extraneous Flow Allowance – 0.33 L/s/ha
- Residential Average Flows – 280 L/cap/day
- Commercial/Mixed Use Flows – 28,000 L/ha/day
- Minimum Cover – 2.5m

In addition, a residential peak factor based on Harmon's Equation along with a correction factor of 0.8 was used to determine the peak design flows per Ottawa's Sewer Design Guidelines. Institutional and commercial areas were assigned a peaking factor of 1.5 per Ottawa's Sewer Design Guidelines where ICI areas amount to greater than 20% of contributing land area.

Per the Master Servicing Study, future contributing areas were assessed at a residential density of approximately 50 persons/gross ha for a total population of 2,206 originating from lands west of the proposed phases 2-4, and lands under separate ownership. Contributions from the existing phase 1 of the Mahogany development have been estimated based on existing lot configuration and unit counts, as well as contribution from 88 single units over 23.2ha of existing residential development along Manotick Main Street. A projected unit count of 225 singles, 34 townhomes and 2.84 ha of institutional development was applied for future phase 5 of the Mahogany development (see drainage areas on **Drawing SAN-1**). An additional 36.70ha with a population of 330 is also directed from the existing Manotick Estates off Potter Drive to the pump station separately from flows within the Bridgeport Avenue trunk sewer.

4.3 PROPOSED SERVICING

The Mahogany development will be serviced by a network of gravity sewers which will direct wastewater flows to a proposed trunk sewer within Bridgeport Avenue, and ultimately to the Eastman Pump Station. Flows from external lands to the west will also be conveyed through the subject property as directed in the MSS. The proposed sanitary sewer design sheet and associated Sanitary Drainage Area Plan can be found in **Appendix B & Appendix E** respectively. The proposed sanitary sewer design indicates a single connection point to the recently constructed trunk sewer skirting the proposed Pond 2 block.

The connection point and associated peak internal and external flows are summarized in **Table 4.1** below and have been coordinated with the latest available plans for phase 1 of the Mahogany development. Previously allocated flows for the available connection point are noted in **Table 4.2**. Peak flows are within capacity of the downstream system per the sanitary sewer design sheet in **Appendix B.**

Table 4.1: External Contributing Areas to Bridgeport Avenue Sewer

Description	Singles	Townhomes	Population	Residential Area (ha)	Institutional Area (ha)	Total Area (ha)
Mahogany Phase 1	211	0	717	19.67	0.00	19.67
Existing Development East of Manotick Main Street	18	0	61	10.70	0.00	10.70
Existing Development West of Manotick Main Street	70	0	238	12.50	0.00	12.50
Mahogany Phase 5	225	34	857	25.63	2.84	28.47
Future Minto Development	-	-	544	-	-	10.89
Future Development (Other Owners)	-	-	819	-	-	16.38
Mahogany Phases 2-4	692	205	2906	81.45	2.89	84.34
Total			**6142**			**182.95**

Table 4.2: Calculated Peak Wastewater Flows vs. exp Mahogany Phase 1 Report (Incl. Contribution from Existing Manotick Estates)

Description	Population	Residential Area (ha)	Institutional Area (ha)	Total Area (ha)	Total Flow (L/s)	Sewer Dia. (mm)
exp (Phase 1)	5972	178.17	4.73	182.90	132.07	450
Stantec (Phases 2-4)	6142	177.22	5.73	182.95	131.20	450
Difference					**-0.67**	

4.4 MAHOGANY PUMP STATION

Pump station upgrades will be required to suit the development as construction proceeds westwards, which are to be identified during detailed design. Based on the approved ECA for the Mahogany Sewage Pumping Station, as well as the *Mahogany Sewage Pumping Station and Sewage Forcemain Design Report* (J.L. Richards & Associates, 2011), the following upgrade schedule has been identified:

Table 4.3: Mahogany Pump Station Phasing

Scenario	Pump Configuration	Pump Rated Capacity (L/s)	Forcemain Configuration	Design Peak Flow (L/s)
Initial Phase	1 x 10 HP (+Standby)	43.4	1 x 200mm	42.0
Interim Phase	1 x 34 HP (+Standby)	125.0	2 x 200mm	110.0
Ultimate Phase	2 x 34 HP (+Standby)	166.4	2 x 200mm	152.66

As both forcemains have already been installed, upgrades to the pump station will be limited to the replacement of the 10HP pump with a 34HP pump (plus an additional standby unit) at the interim stage, and addition of another 34HP pump at the ultimate stage. Based on population estimates for Phases 1-4, the Mahogany subdivision including discharge from Manotick Estates and development along Manotick Main Street is expected to have a peak design flow of approximately 96.1L/s, which may be entirely accommodated by the Interim Phase of pump upgrades. Further development west of Phase 4 will require that sanitary contributions be reassessed to verify if the 110.0L/s peak design flow trigger has been reached, requiring additional pump installation. Full buildout of Phase 2 results in a design peak flow rate above 42L/s, and will require Interim Phase upgrades to be completed per the table above.

4.4.1 Pump Station Emergency Overflow

A hydraulic analysis of the preliminary sanitary sewer layout was conducted to confirm impacts of a failure to the Mahogany Pump Station under emergency conditions. The model considers two scenarios; the first considers buildout of Phases 2-4 of the development, as well as connection to the existing properties east and west of Manotick Main Street, including a 450mm overflow from the pump station wet well to the adjacent Pond 2.

The *Mahogany Sewage Pumping Station and Sewage Forcemain Design Report* has suggested an overflow invert of 87.0, approximately 300mm above the assumed 100yr water elevation within Pond 2 per the Master Servicing Study. Per ECA 3002-8PBSB4 for the Mahogany Sewage Pumping Station, the overflow has not been constructed, although provision for the overflow pipe within the bypass control chamber immediately upstream of the pump station wet well was to be made. It is proposed to lower the 450mm outflow to elevation 86.60 to allow gravity discharge to Pond 2 at the 100-year water elevation of 86.55, which in turn will limit impacts to unit underside of footings immediately upstream.

The second scenario considers the ultimate buildout of the Mahogany subdivision, including future development lands to the west. The second scenario requires the installation of an additional emergency overflow sewer to the future Pond 4 with invert of 87.64 (approximately 300mm above Pond 4's assumed 100yr water elevation of 87.34 per the MSS) to limit gradual HGL increase across the long sanitary trunk sewer line.

Proposed unit underside of footings have been assumed at the proposed centerline of road less 1.9m. As the City of Ottawa's Sewer Design Guidelines require a minimum 0.30m separation from USF to HGL elevation within the adjacent sanitary sewer, a preliminary minimum freeboard of 2.20m is required from proposed road centerline to ensure proposed units may be serviced at the detailed design stage. Results of the analyses are demonstrated in **Table 4.4** below, as well as within **Appendix B**.

Table 4.4: Pump Station Failure HGL

Manhole	Rim Elev.	USF	HGL (Phases 2-4)	Freeboard	HGL (Full Build-out)	Freeboard
1	90.02	88.12	86.99	1.13	87.21	0.91
2	89.95	88.05	87.01	1.04	87.26	0.79
3	89.87	87.97	87.03	0.94	87.31	0.66
4	90.06	88.16	87.05	1.11	87.39	0.77
5	90.25	88.35	87.07	1.28	87.45	0.90
6	90.02	88.12	87.07	1.05	87.48	0.64
7	89.77	87.87	87.07	0.80	87.48	0.39
8	89.86	87.96	87.07	0.89	87.50	0.46
9	89.98	88.08	87.07	1.01	87.51	0.57

10	90.10	88.20	87.07	1.13	87.52	0.68
11	89.70	87.80	87.07	0.73	87.53	0.27
12	89.62	87.72	87.07	0.65	87.51	0.21
13	90.56	88.66	87.24	1.42	87.62	1.04
14	90.12	88.22	87.27	0.95	87.65	0.57
15	90.41	88.51	87.16	1.35	87.55	0.96
16	90.56	88.66	87.20	1.46	87.58	1.08
17	90.03	88.13	87.29	0.84	87.57	0.56
18	90.18	88.28	87.38	0.90	87.66	0.62
19	90.50	88.60	87.46	1.14	87.74	0.86
20	89.70	87.80	87.03	0.77	87.32	0.48
21	90.13	88.23	87.05	1.18	87.30	0.93
22	90.54	88.64	87.07	1.57	87.32	1.32

5.0 STORMWATER MANAGEMENT

The following sections describe the conceptual stormwater management (SWM) plan for the Minto Mahogany Stage 2+ Development in the context of the background documents and governing criteria.

The proposed SWM plan has been revised to reflect the latest draft plan and to address City comments to the first submission. Specifically, further explanation has been added to the text of the report to describe how the boundary conditions for the Mahogany Creek model were obtained and how the water levels in the creek compare to the permanent pool in the proposed SWM Pond 2 and to the components of the conceptual outlet structure. Additionally, the SWM Plan for the proposed and future development areas tributary to the Wilson Cowan Municipal Drain (WCMD) and its tributary has been revised. The existing conditions model for the WCMD has been calibrated based on available monitoring data obtained from June 2017 to October 2017. As requested by the City, additional servicing options for the WCMD tributary area have been assessed to provide a separate storm outlet into the drain for the future development lands and to avoid the storm sewer crossing the drain tributary.

5.1 PROPOSED CONDITIONS

The proposed concept plan consists of a mix of single family homes, 2-storey townhomes, bungalow townhomes, park areas, a school, two stormwater management (SWM) blocks, and associated transportation and servicing infrastructure. Site sewers will outlet to the proposed SWM Ponds 2 and 3. SWM Pond 2 will discharge into the Mahogany Creek east of the site and SWM Pond 3 will outlet into the Wilson Cowan Municipal Drain (WCMD) that crosses the site from south to north (see **Drawings ST-1** and **ST-2**).

Major system peak flows from the catchment areas tributary to Pond 2 will be directed to the pond. Major system peak flows from catchment areas tributary to Pond 3, east of the WC drain tributary will be directed to the pond, while major system peak flows from all other areas tributary to Pond 3 will be directed to the WCMD and its tributary directly.

5.2 BACKGROUND DOCUMENTS

5.2.1 Manotick Master Drainage Plan

Robinson Consultants prepared the Manotick Master Drainage Plan (MDP) in 1996 to address water quality and quantity requirements for future developments tributary to Mud Creek, the Baxter Drain and the Wilson Cowan Drain. The MDP concluded that water quantity control was required to mitigate the impacts of new developments on groundwater recharge and as such, infiltration techniques, BMP's are required in the proposed development to encourage groundwater recharge rates.

5.2.2 Assessment of Discharge Criteria for Stormwater Management Facilities on the Rideau River

W. F. Baird & Associates Coastal Engineers Ltd. prepared the Assessment of Discharge Criteria for SWM Facilities on the Rideau River in January 2000. The study concluded that the stormwater quality control objectives for the Rideau River in the vicinity of the village of Manotick, as determined by the Provincial Water Quality Objectives (PWQO), limit stormwater facility discharge during the recreational season (May 15-September 15) to a maximum allowable bacterial concentration of 100 counts of E. coli per dL. This target criterion is permitted to be exceeded an average of four times per recreational season.

5.2.3 Village of Manotick Environmental Management Plan Special Design Area Component

In June 2005, Marshall Macklin Monaghan & Water and Earth Science Associates prepared the Environmental Management Plan Special Design Area (SDA) Component to provide a summary of recommendations related to environmental constraints and opportunities and SWM requirements applicable to the SDA lands, which are located at the southeast quadrant of First Line Road and Bankfield Road in Manotick.

The 2005 SDA EMP identified control points in the Mahogany Creek and evaluated pre-development flows. The report recommended water quantity control measures for future developments tributary to the Mahogany Creek to meet pre-development peak flows at the control point.

5.2.4 Mahogany Community Stormwater Management Servicing Report

Minto retained IBI Group to prepare the Mahogany Community Stormwater Management Servicing Report, submitted in July 2007 (See **Appendix C.1** for report excerpts). The report provided a conceptual SWM Plan for the whole Mahogany Community which at the time, included four SWM Wet Ponds in order to meet the quality, erosion and quantity control criteria outlined for the site in background documents. The report outlined the regulatory requirements for each receiving watercourse as summarized in the following subsections.

5.2.4.1 Mahogany Creek

The proposed SWM Pond 2 will discharge into the Mahogany Creek, which is tributary to the Rideau River at the Mahogany Harbour, approximately 200 m downstream of the site’s northern limit. The following summarizes the SWM criteria for the Mahogany Creek.

- Provide Enhanced Level of Protection (80% Total Suspended Solids Removal).
- Meet Provincial Water quality objectives for bacteria concentrations.
- Provide erosion control.
- No quantity control is required for SWM facilities tributary to the Rideau River. However, as outlined in the 2005 SDA EMP, water quantity control measures are recommended for future

developments tributary to the Mahogany Creek to meet pre-development peak flows at the control point.
- Infiltration techniques, BMP's are required in the proposed development to encourage groundwater recharge rates.

5.2.4.2 Wilson Cowan Municipal Drain

The proposed SWM Pond 3 will discharge into the Wilson Cowan Municipal Drain, which is tributary to Mud Creek. The drain and its tributary extend from the southern limit to the northern limit of the site. The following summarizes the SWM criteria for the Wilson Cowan Municipal Drain.

- The 2005 SDA EMP recommends that the proposed SWM facility be designed to provide Enhanced Level of Protection (80% Total Suspended Solids Removal).
- Meet Provincial Water quality objectives for bacteria concentrations.
- Provide erosion control.
- Subject to model calibration, water quantity control measures will be included in the design of the proposed SWM Pond 3 to meet pre-development levels in the WC Drain downstream of the outlet.
- Infiltration techniques, BMP's are required in the proposed development to encourage groundwater recharge rates.

5.2.5 Mahogany Community Phase 1 Stormwater Management and Fish Habitat Enhancement of the Mahogany Creek

In January 2011, IBI prepared the Mahogany Community Phase 1 Stormwater Management and Fish Habitat Enhancement of the Mahogany Creek which determined that the Mahogany Creek, located in an actively cultivated agricultural setting, was a poorly defined, heavily intermittent watercourse that experienced prolonged absences of flow. The report provided detail on the proposed fish habitat enhancement to the Mahogany Creek and the comprehensive solution of stormwater management for Phase 1 which consisted of multiple stormwater outlets along the drain.

5.2.6 Mahogany Community Phase 1 Stormwater Management Servicing

In May 2012, IBI group was retained by Minto to complete the detailed SWM design for Phase 1 of the Mahogany Community which is located east of the Mahogany Creek and which was originally planned to be serviced by SWM Pond 1 as per the 2007 Mahogany Servicing Report. The report concluded that in order to meet pre-development levels at Point A in the Mahogany Creek which receives flow from Phase 1 and Phase 2 development, as well as two rural areas, dual drainage and on-site storage for Phase 1 and a SWM facility servicing Phase 2 would be required (see **Appendix C.1** for report excerpts). As recommended in the 2011 Mahogany Community Phase 1 Stormwater Management and Fish Habitat Enhancement of the Mahogany Creek, there are three storm sewer outlets to the Mahogany Creek from Phase 1as well as four major system outlets that discharge directly into the creek. The three storm sewer outlets have been constructed, and creek restoration works were scheduled to commence in August 2017 and have been completed.

5.2.7 Wilson Cowan Municipal Drain Fluvial Geomorphic Existing Conditions

Matrix Solutions Inc. was retained by Minto Communities Inc. in May 2017 to provide a fluvial geomorphic existing conditions assessment of the Wilson Cowan Municipal Drain and its tributary within the Minto Mahogany development area. The full report has been included in **Appendix C.1**.

The Wilson Cowan Municipal Drain (WCMD) was subdivided in three reaches (WCD-R1 to WCD-R3) based on land use, channel form, and channel function. The investigation concluded that overall, the channel is highly stable under existing conditions. WCDR-1 represents the furthest reach of channel downstream from the confluence with the tributary and it was described to have a channel bottom composed of soft but consolidated silty clay.

The tributary to the Wilson Cowan Drain was also divided into three reaches (WCDT-R1 to WCDT-R3). The tributary exhibited a good deal of stability throughout and was found to be in regime along its entire length. The downstream portion of the channel, WCDT-R1 has a bottom bed mostly composed of gravels and sands and is composed of a steep step/riffle sequence to the confluence with the main branch of the WCMD.

A detailed geomorphic survey was completed for reaches WCD-R2 and WCDT-R1, both upstream of the confluence of the tributary with the WCMD. Critical shear stresses, discharges, depths and velocities were calculated for both reaches as shown in **Table 5.1**.

Table 5.1: WCD-R2 and WCDT-R1 Erosion Threshold Result Summary

Parameter	Recommended Values	
	WCD-R2	WCDT-R1
Approximate Bankfull Discharge (m^3/s)	1.68	1.46
Critical Shear Stress (N/m^2)	1.85	22.39
Critical Discharge (m^3/s)	0.29	0.24
Critical Average Depth (m)	0.40	0.19
Critical Average Velocity (m/s)	0.30	0.76

5.3 EXISTING CONDITIONS AND TARGET PEAK OUTFLOWS

5.3.1 Wilson Cowan Drain Watershed Existing Conditions

A preliminary hydrologic analysis of the existing conditions for the WCMD watershed was conducted by IBI Group in their 2007 Mahogany Community SWM and Servicing Report (see **Appendix C.1** for report excerpts and existing conditions figure). The analysis was done using XP-SWMM for the 25 mm, 4-hour Chicago Storm and the 24 SCS Type II distribution for the 2-year, 5-year and 100-year storms. The results of the analysis are presented in **Table 5.2** below. The location of Flow Point B is shown on **Drawing ST-2**.

Table 5.2: Wilson Cowan Drain Existing Conditions Flow Rates at Point B – No Calibration

Watercourse	Flow (m^3/s)			
	25 mm	2-year	5-year	100-year
Wilson Cowan Drain	0.82	1.88	2.90	6.37

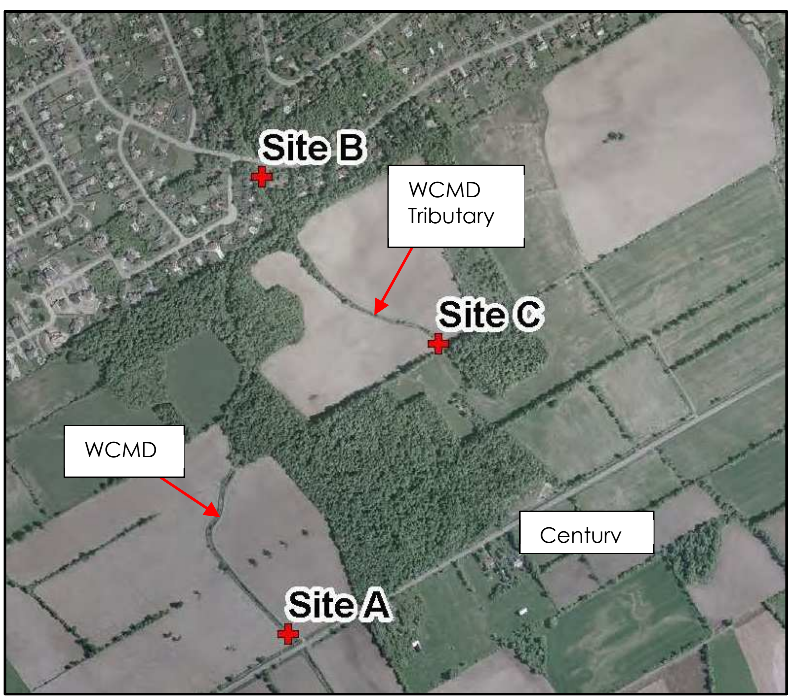
However, as mentioned by IBI in their 2007 servicing report, the existing condition model for the WCMD was subject to calibration. At the time of the 2007 servicing report, monitoring of the drain was being completed; however, the data available was deemed inadequate to calibrate the model.

5.3.1.1 Wilson Cowan Municipal Drain and Tributary Stream Flow Monitoring Program

Stantec completed a 4-month long (June to October 2017) stream flow monitoring program in the Wilson Cowan Municipal drain and its tributary. The data from the monitoring program has been used to calibrate and validate the existing conditions model of the Wilson Cowan Municipal Drain upstream of the flow control Point B. Monitoring data and calculations have been included in **Appendix C.3**.

Solinst's level loggers and a barologger were used to obtain continuous (5-min) water levels at three locations along the drain and its tributary as shown in **Figure 5.1**. A total of four (4) site visits were completed to perform stream monitoring to obtain flow rates and to develop flow curves for each site.

Figure 5.1: WCMD Water Level Monitoring Sites



At all three sites, a cross-section of the stream was selected where the stream was relatively straight and with minimal vegetation. The cross-sections were identified with wooden stakes on each bank to perform the stream measurements at the same location for the subsequent site visits.

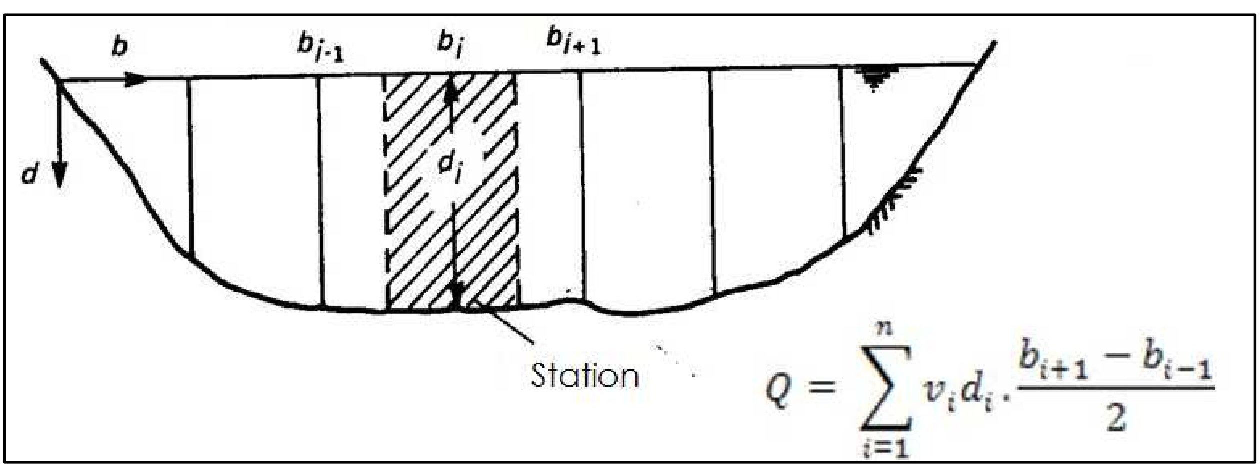
Velocity measurements were taken using a current meter across the stream, at an interval of approximately 50 cm. A measuring tape was laid out across the stream, connecting the stakes on both banks, and it was used to determine the length of the cross-section, as well as determining the distance from the velocity measurements to the bank. The water depth was measured at each velocity measurement station, and the velocity was measured as such:

- If the water depth is less than 75cm, the velocity measurement was taken at 60% of the depth (from the surface).
- If the water depth is greater than 75cm, the velocity measurement was taken at 20%, and 80% of the depth (from the surface). The average of both velocity measurements was used in the flow calculation.

5.3.1.1.1 Flow Determination

Using the velocity and water depth data gathered as part of the stream monitoring program, the flow was calculated using the Standard Mid-Section Method for each site at every site visit. For this method, the stream is divided into panels, which are defined as one half of the distance to the previous station (i.e. velocity measurement location) plus one half of the distance to the next section. The partial flow was calculated for each panel by multiplying the velocity, depth, and the width of the panel. The total flow at each site was determined by taking the sum of the partial flows. The Mid-Section Method is summarized in **Figure 5.2**. Sample calculations are provided in **Appendix C.3**.

Figure 5.2: Standard Mid-Section Method for Flow Determination



The site visit dates, recorded water depths, and corresponding calculated flows are listed in **Table 5.3.**

Table 5.3: Flow Monitoring Program Details and Findings

Site Visit	Date	Water Depth (m)			Calculated Flow (m³/s)		
		Site A	Site B	Site C	Site A	Site B	Site C
1	16-Jun-17	0.52	0.29	0.00	0.000[1]	0.019	0.000
2	30-Jun-17	0.78	0.66	0.25	0.220	0.362	0.032
3	4-Aug-17	0.42	0.34	0.05	0.050	0.052	0.002
4	5-Oct-17	0.51	0.19	0.00	0.007	0.004	0.000

[1] Standing water (i.e. no flow).

5.3.1.1.2 Rainfall Data

Attempts were made to complete the site visits following rainfall events, or after a few consecutive rainy days. Several large rain events occurred during the flow monitoring periods, as listed in

Table 5.4, and presented in **Figure 5.3**. The rainfall data was gathered from Environmental

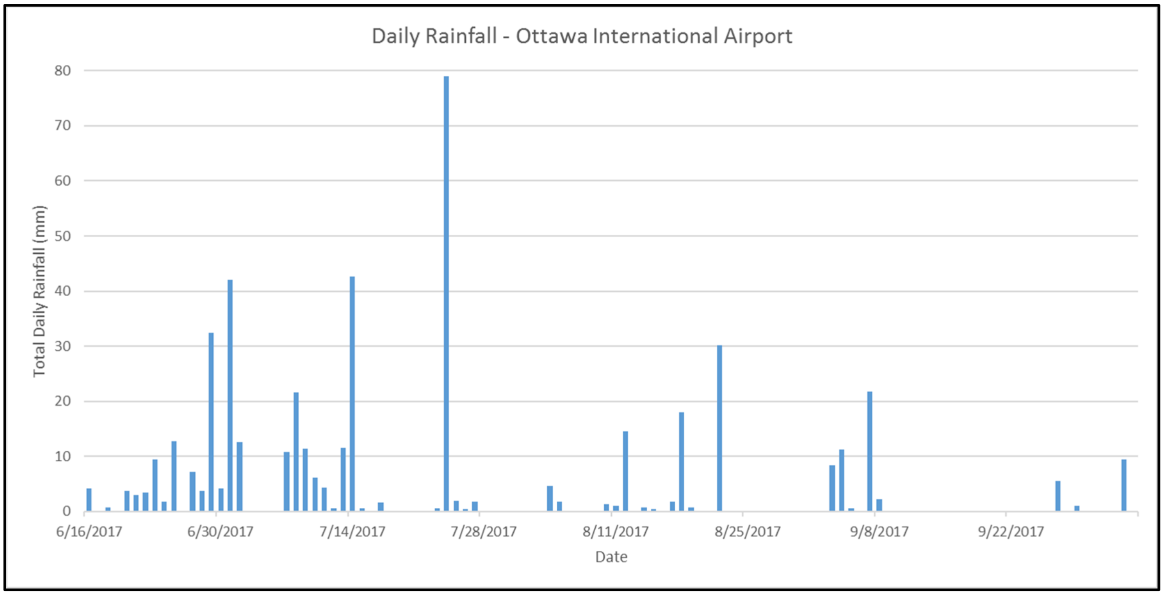
Date	Total Daily Rainfall (mm)	Date	Total Daily Rainfall (mm)
29-Jun-2017	32.4	24-Jul-2017	79.0
1-Jul-2017	42.0	22-Aug-2017	30.2
14-Jul-2017	42.6	7-Sep-2017	21.8

Canada Historical Data.

Table 5.4: Top 6 Rain Events (June 16 to October 1, 2017)

Date	Total Daily Rainfall (mm)	Date	Total Daily Rainfall (mm)
29-Jun-2017	32.4	24-Jul-2017	79.0
1-Jul-2017	42.0	22-Aug-2017	30.2
14-Jul-2017	42.6	7-Sep-2017	21.8

Figure 5.3: Daily Rainfall (June 16 – October 1, 2017)



5.3.1.1.3 Stage-Discharge Curve Determination

The recorded water level data was used with the measured velocity/flow data to establish representative stage-discharge curves for each of the sites. The recorded water depths for all three sites are shown for the entire program in **Figure 5.4**. The established stage-discharge curves based on the four (4) field measurements are shown in **Figure 5.5, Figure 5.6,** and

Figure 5.7.

Figure 5.4: Recorded Water Levels (5-min timestep)

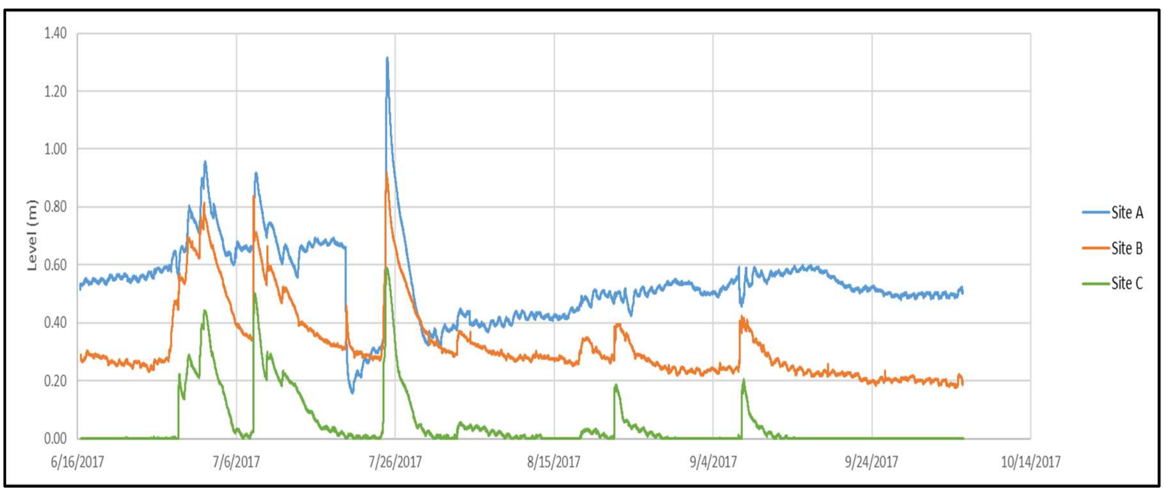


Figure 5.5: Site A Stage-Discharge Curve

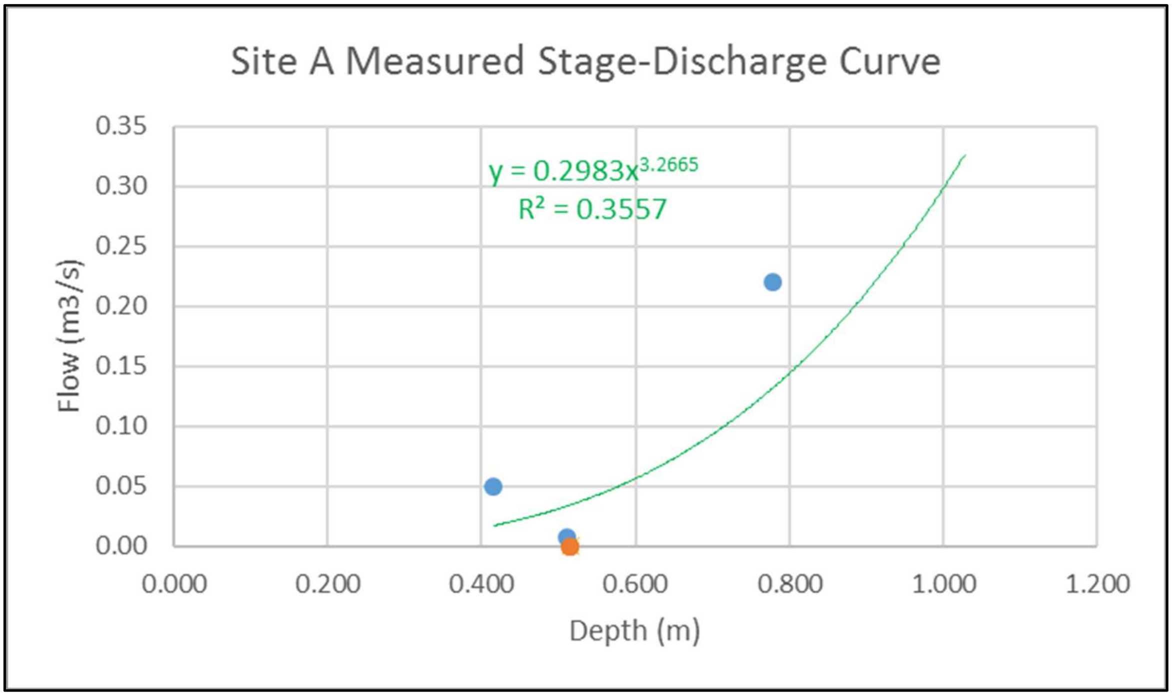


Figure 5.6: Site B Stage-Discharge Curve

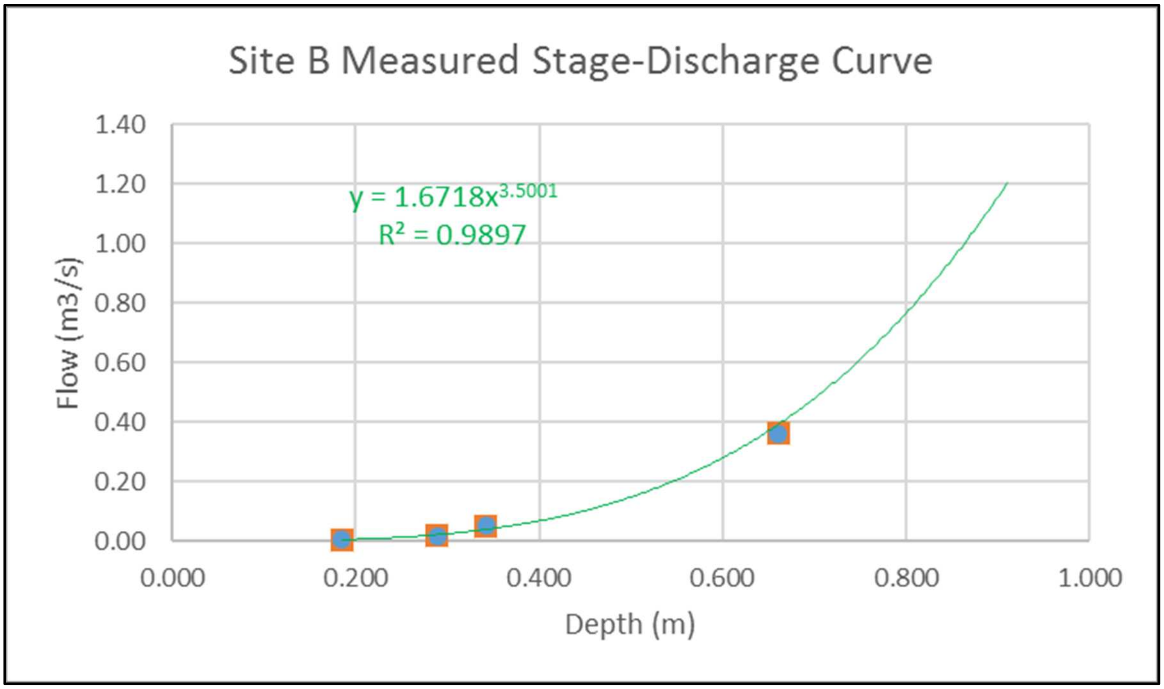
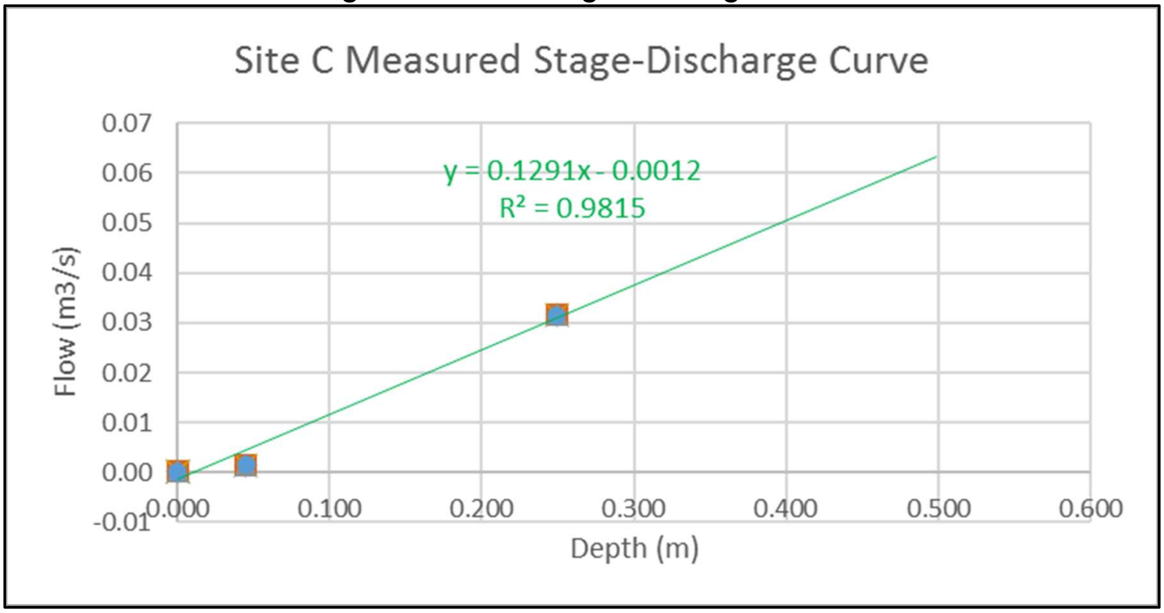


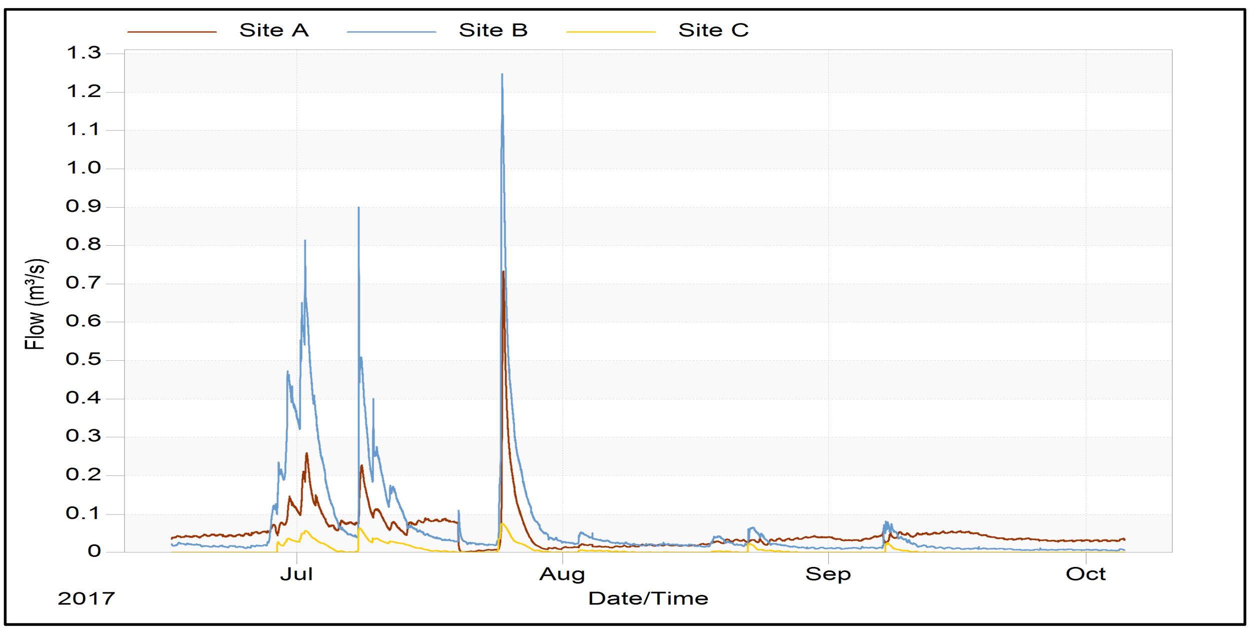
Figure 5.7: Site C Stage-Discharge Curve



5.3.1.1.4 Estimated Flow Hydrographs

Flow hydrographs were computed using the stage-discharge curves. Due to the limitation of the data quality (see **Section 5.3.1.1.5**), the flow at Site A was calculated using the Difference Basin Method (i.e. Site A = Site B – Site C). The calculated flows are shown in **Figure 5.8**.

Figure 5.8: Calculated Flows (5-min timestep)



5.3.1.1.5 Limitations

There are limitations to the validity of the established flow curves as the magnitude of the measured events was not as great as the largest event that was recorded throughout the monitoring program. The July 24th, 2017 event saw approximately 80 mm of rain over a 24-hour period, which is more than double what was measured on June 30th (~72mm over a 72-hour period).

Additionally, the depth at Site A dropped by approximately 40 cm on July 20th. It was initially suspected that an equipment malfunction was responsible, however a review of the field measurements corroborates this drop. It is suspected that some type of blockage was removed downstream and caused the water level to drop. It was also observed that there was approximately 60 cm of stagnant water during the first installation visit to Site A on June 16, 2017.

5.3.1.2 Wilson Cowan Municipal Drain Existing Condition Model Calibration

Due to the observed variability of the data at Site A as described above, the calculated flows for Site B and Site C were used to calibrate the existing conditions model for the WCMD watershed upstream of Flow Point B. Calibration parameters and a PCSWMM existing condition input file example have been provided in **Appendix C.4**.

The hydrologic parameters used in the existing condition calibrated model are presented in the tables below, while the existing condition drainage areas are shown in **Figure 5.9.**

Table 5.5: General Existing Subcatchment Parameters - Calibrated

Subcatchment Parameter	Value
Infiltration Method	Horton
Max. Infil. Rate (mm/hr)	35
Min. Infil. Rate (mm/hr)	3
Decay Constant (1/hr)	4.14
N Imperv	0.013
N Perv	0.25
Dstore Imperv (mm)	1.57
Dstore Perv (mm)	4.67

A maximum initial infiltration capacity of 35 mm/hr was used rather than the traditional default value of 76.2 mm/hr to simulate the sensitive clayey soil identified within the geotechnical report for the region (see **Section 10.0**). The 35 mm/hr value is an interpolation of commonly used Horton infiltration values for dry and moist clay soils with dense vegetation of 51 mm/hr and 18 mm/hr respectively (Akan, 1993). Suggested minimum infiltration capacities range from 0 – 1.3 mm/hr for clay loam (Type D HSG soils), and 1.3 – 3.8 mm/hr for sandy clay loam (Type C HSG soils) respectively.

Values of Maximum Initial Infiltration Capacity Suggested by Akan (1993)[1]

Soil Type	Maximum (Initial) Infiltration Capacity, *F0* (in/hr)	(mm/hr)
Dry sandy soils with little or no vegetation	5.0	127
Dry loam soils with little or no vegetation	3.0	76.2
Dry clay soils with little or no vegetation	1.0	25.4
Dry sandy soils with dense vegetation	10.0	254
Dry loam soils with dense vegetation	6.0	152
Dry clay soils with dense vegetation	2.0	51
Moist sandy soils with little or no vegetation	1.7	43
Moist loam soils with little or no vegetation	1.0	25
Moist clay soils with little or no vegetation	0.3	7.6
Moist sandy soils with dense vegetation	3.3	84
Moist loam soils with dense vegetation	2.0	5.1
Moist clay soils with dense vegetation	0.7	18

Climatology options parameters for temperature and wind speed have been applied to the calibrated model to better simulate evapotranspiration, and have been sourced from monthly averages supplied by Environment Canada data from the Ottawa Airport.

Area and subcatchment width parameters have been estimated based on background mapping for the region, and are identified in **Table 5.6** below:

[1] Akan, Osman. Urban Storm Water Hydrology: A Guide to Engineering Calculations. Lancaster. PA: Technomic Publishing Co., Inc., 1993.

Table 5.6: WCMD Existing Condition Calibrated Hydrologic Parameters

Area ID	Area (ha)	Width (m)	Slope (%)	%IMP	Subarea Routing	% Routed
3-PRE	2.86	30.0	0.100	0.0%	PERVIOUS	100
4-PRE	3.34	100.0	0.100	0.0%	PERVIOUS	100
5-PRE	10.14	450.0	0.100	0.0%	PERVIOUS	100
8-PRE1	9.43	444.0	0.100	0.0%	PERVIOUS	100
8-PRE2	5.26	213.0	0.100	0.0%	PERVIOUS	100
Area 3	2.45	150.0	0.100	0.0%	PERVIOUS	100
Area 6	1.80	66.7	0.100	0.0%	PERVIOUS	100
Area 7	9.20	450.0	0.100	0.0%	PERVIOUS	100
Area 8	13.50	444.0	0.100	0.0%	PERVIOUS	100
Area_4	0.66	100.0	0.100	0.0%	PERVIOUS	100
FUT	6.61	444.0	0.100	0.0%	PERVIOUS	100
S1	6.84	275.0	0.100	0.0%	PERVIOUS	100
S2	8.61	480.0	0.100	0.0%	PERVIOUS	100
Sub9A	263.04	2400.0	0.010	0.0%	PERVIOUS	100

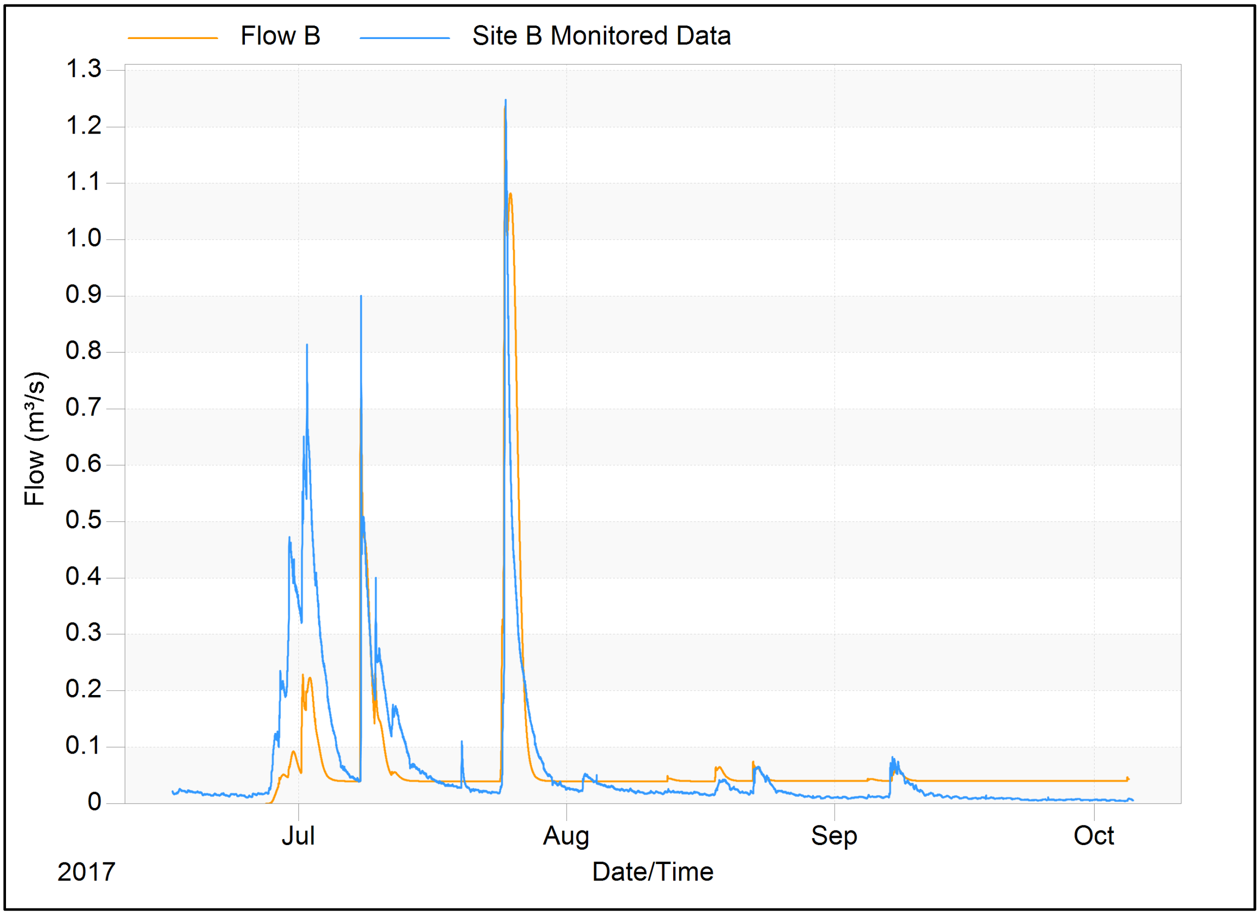
The Wilson Cowan Drain and Wilson Cowan Drain Tributary cross-sections (WCD-R2 and WCDT-R1) were developed based on average bankfull cross-section parameters as noted in Table 2 of the *Wilson Cowan Drain Fluvial Geomorphic Existing Conditions* report (Matrix Solutions Inc., May 2017). Drain cross-section downstream of the confluence of drain with its tributary stream are based on channel improvements as recommended by the *Wilson-Cowan Drain Master Drainage Plan Study*.

Figure 5.9: WCMD Existing Condition Drainage Area Plan

DO NOT PRINT – INSERT FIGURE 5.9

Figure 5.10 shows a comparison of the calibrated modeled peak flows at Point B (Flow B) versus the observed flows (Site B Monitored Data) throughout the monitoring period.

Figure 5.10: WCMD Calibrated vs Observed Peak Flow Comparison at Point B



As can be seen in the above figure, the calibrated existing conditions model generates generally lower peak flows than the observed data. However, this existing condition model is considered conservative and more representative of the existing conditions than the model previously created for the proposed site. As a result, it is recommended that the calibrated existing conditions model be used in this initial assessment, but that further monitoring be performed along the WCMD to improve the calibration for the existing conditions model during the detailed design stage.

The calibrated existing condition model was used to run the 25 mm, 4 hour Chicago storm, and the 24 hour SCS Type II distribution for the 2-year, 5-year and 100-year storms using City of Ottawa I-D-F parameters. **Table 5.7** summarizes the existing condition peak flows in the WCMD at Flow Point B.

Table 5.7: WCMD Existing Condition Peak Flow Rates at Point B

Watercourse	Flow (m^3/s)			
	25 mm	2-year	5-year	100-year
Calibrated Model for Wilson Cowan Municipal Drain	0.041	0.188	0.322	1.094
2007 Uncalibrated Model for WCMD	0.820	1.880	2.900	6.370

The calibrated existing conditions model generates significantly lower peak flows than the previous values used by IBI in their 2007 servicing report. However, the capacity of the drain and existing structures has been assessed previously based on the existing condition peak flows from the uncalibrated model and as such flooding and/or negative impacts to the downstream infrastructure are not a concern for the proposed site. <u>Based on the above, it can be concluded that erosion control (rather than drainage structure capacity) will govern the SWM criteria for the proposed and future development areas tributary to the WCMD and its tributary.</u>

As mentioned in **Section 5.2.7**, an erosion threshold analysis was undertaken for the reaches along the WCMD and its tributary upstream of their confluence, which concluded that the critical discharge for the drain and its tributary along those reaches was 0.29 m^3/s and 0.24 m^3/s respectively. It is recommended that a detailed fluvial geomorphic analysis be undertaken along the reach of the WCMD that will receive runoff from the proposed SWM Pond 3 down to the Rideau River to determine the erosion thresholds for those reaches. However, a preliminary erosion analysis to estimate the existing and proposed condition exceedances at flow Point B has been done using the most conservative critical velocity outlined in Matrix Solutions' Fluvial Geomorphic analysis (0.30 m/s). The results of this analysis are summarized in **Section 5.8.4.1**.

5.3.2 Mahogany Creek Watershed Existing Conditions

Hydrologic analysis of the existing conditions for the Mahogany Creek watershed was conducted by IBI Group in their 2012 Mahogany Community Phase 1 SWM Servicing Report (see **Appendix C.1** for report excerpts and existing conditions figure). The analysis was done in SWMYMO/XP-SWMM for the 25 mm, 4-hour Chicago Storm and the 24 SCS Type II distribution for the 2-year, 5-year and 100-year storms. The hydrologic parameters and results of the analysis are presented in the tables below. The location of Flow Point A is shown on **Drawing ST-1**.

Table 5.8: Mahogany Creek Existing Conditions Hydrologic Parameters

Drainage Area ID	Drainage Area (ha)	Time to Peak (hr)
1	81.0	0.59
2	214.0	2.70

Table 5.9: Mahogany Creek Existing Condition Flow Rates at Point A

Watercourse	Flow (m^3/s)			
	25 mm	2-year	5-year	100-year
Mahogany Creek	0.89	2.21	3.44	7.54

5.4 DESIGN METHODOLOGY AND SWM DESIGN CRITERIA

The design methodology for the SWM component of the development is as follows:

- Restrict inflows to the sewer to the 2-year rate for areas with local streets, park areas and the school block, and to the 5-year rate for areas along collector roads as per City of Ottawa Sewer Design Guidelines. (City of Ottawa)
- Produce a PCSWMM model that generates major and minor system hydrographs from the proposed development areas tributary to SWM Pond 2 and include the rehabilitated cross section of the Mahogany Creek from Century Road to the outlet of SWM Pond 2, and inflows from the Mahogany Phase 1 development and from upstream and downstream rural areas as per IBI's 2012 Phase 1 SWM design. (IBI)
- Produce a PCSWMM model that generates major and minor system hydrographs from the proposed development areas and from the future development area, and incorporates the rural runoff contribution along the Wilson Cowan Municipal Drain (WCMD) and its tributary to flow control Point B as per the calibrated existing conditions model (Stantec 2018)
- Estimate the volume requirements for each of the proposed SWM Ponds to meet the target peak outflows at Point A in the Mahogany Creek and at Point B in the Wilson Cowan Drain. (IBI 2012 / Stantec 2018)
- As requested by the City of Ottawa, assess storm servicing options to reduce the size or completely eliminate SWM Pond 3 by introducing storm sewer outlets to the drain with oil grit separators to achieve the quality control required while still meeting the quantity control target peak outflows for the WCMD (City of Ottawa)
- Develop conceptual outlet structures for the proposed SWM Ponds that meet the MOECC quality control criteria of 80% TSS removal for the facilities and that leave some room for detailed erosion analyses at the detailed design stage. (MOECC, IBI)
- Ensure that the resulting 100-year hydraulic grade line is at least 2.5 m below the proposed road grade in the proposed development condition to ensure future unit USF elevations at approximately 2.0 m in depth below proposed road grades will clear anticipated HGLs with sufficient freeboard. (City of Ottawa)
- Ensure that total flow depth on streets does not exceed 0.35 m during the 100-year storm scenario. (City of Ottawa)

5.4.1 Deviations from IBI's 2007 Servicing Report

The proposed SWM plan for the Mahogany Subdivision Phases 2-4 contains deviations from IBI's 2007 Servicing Report. Firstly, a separate storm sewer outlet into the drain has been proposed for the future development area to avoid the storm sewer crossing the woodlot area owned by the City. Additionally, the proposed storm sewer systems and conceptual sizing for SWM Pond 2 and SWM Pond 3 have been revised to reflect the latest City of Ottawa Sewer Design Guidelines minor system capture rates which correspond to the 2-year runoff for local streets and the 5-year runoff for collector roads. These capture rates are significantly higher than the 85 L/s/ha minor system inflow rates used in IBI's 2007 servicing report.

5.5 PROPOSED DEVELOPMENT CONDITIONS

The site will be designed using the "dual drainage" principle, whereby the minor (pipe) system is designed to convey the peak rate of runoff from the 2-year design storm for local streets and the 5-year design storm for collector roads, and runoff from larger events is conveyed by both minor (pipe) and major (overland) channels, such as roadways and walkways, safely to the outlet without impacting proposed downstream properties.

Drawings ST-1, and **ST-2** outline the conceptual storm sewer alignment, drainage divides and labels, and the proposed SWM Pond locations and conceptual configurations. Major system flows from the proposed SWM Pond 2 sewershed area will be conveyed to the SWM pond. Major system peak flows from the catchment areas tributary to SWM Pond 3 east of the WCMD tributary will be directed to the SWM Pond. Major system peak flows from the proposed development areas west of the WC drain tributary will discharge directly into the tributary. Major flows from the future development area east of the WC drain have been assumed to be directed to a future SWM pond to receive the required quality and quantity/erosion control.

The conceptual storm sewer design sheet is included in **Appendix C.2.**

5.6 MODELING RATIONALE

Two hydrologic/hydraulic models were completed with PCSWMM for each of the watercourse outlets, accounting for the estimated major and minor systems to evaluate the storm sewer infrastructure and to size the blocks for the SWM Ponds in order to meet the target criteria. The use of PCSWMM for modeling of the site conceptual hydrology and hydraulics allowed for an analysis of the systems response during various storm events. The following assumptions were applied to the conceptual models:

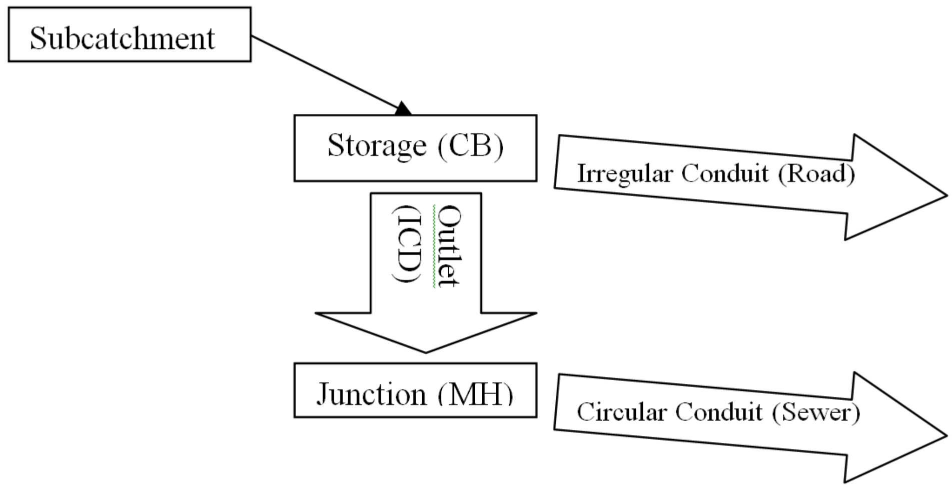
- Hydrologic parameters as per Ottawa Sewer Design Guidelines, including Horton infiltration, Manning's 'n', and depression storage values.
- 3-hour Chicago Storm distribution for 2-year, 5-year and 100-year analysis used to evaluate the urban component of the dual drainage (i.e. total overland flow depth, hydraulic grade line (HGL)).
- 25 mm, 4-hour Chicago Storm and the 24-hour SCS Type II distribution for 2-year, 5-year and 100-year analysis used to evaluate the conceptual SWM Ponds' performance and to compare post development peak flows in the receiving Mahogany Creek and Wilson Cowan Drain to pre-development levels.
- The July 1st, 1979 historical storm and a 'climate change' scenario created by adding 20% of the individual intensity values of the 100-year 3-hour Chicago storm at their specified time step were used as an analytical tool to establish the function of the system under an extreme event.

- Runoff Coefficients assumed as 0.30 for park areas, 0.20 for woodlot areas, 0.70 for the proposed school block, 0.50 for low density residential areas, and 0.60 for medium density residential and converted to percent imperviousness using the relationship %IMP = ((C - 0.2) / 0.7) x 100.
- Width parameter was taken as twice the length of the street/swale segment for two-sided catchments and as the length of the street/swale segment for one-sided catchments.
- Where detailed lot/road layout configuration was not available, subcatchment areas were defined by the limits of the future development blocks and the width of the subcatchment was defined as 225 m/ha as per the City of Ottawa Sewer Design Guidelines.
- Minor system capture rates for areas with local streets (areas named with the prefix L) restricted to the 2-year runoff and for areas along collector roads (areas named with the prefix C) restricted to the 5-year runoff.
- Park areas assumed to restrict minor system capture rates to the 2-year runoff and to discharge major system overflows uncontrolled into adjacent streets.
- Future school block (area L105B) to store major system overflows up to the 100-year storm and to restrict minor system capture rates to the 2-year runoff (448 L/s).
- SWM Pond 2 areas assumed to provide 30 m^3/ha of surface storage with the exception of areas C102A and UNC1 that were assumed to have no surface storage available.
- Major system overflows from SWM Pond 2 areas to be directed to the SWM Pond.
- SWM Pond 3 areas assumed to have no surface storage and to discharge to the SWM Pond or to the WCMD and its tributary as shown on **Drawing ST-2**.
- Hydrological parameters for areas tributary to the WCMD and its tributary that will remain rural under post development conditions were obtained from the calibrated existing condition model.

5.6.1 SWMM Dual Drainage Methodology

The proposed conceptual development is modeled in one modeling program as a dual conduit system (see **Figure 5.11**), with: 1) circular conduits representing the sewers & junction nodes representing manholes; 2) irregular conduits using street-shaped cross-sections to represent the assumed overland road network with streets at 0.5% and storage nodes representing conceptual inlets. The conceptual dual drainage systems are connected via outlet link objects from storage node (i.e. inlets) to junction (i.e. MH), and represent inlet capture rates for the lumped drainage areas. Subcatchments are linked to the storage node on the surface so that generated hydrographs are directed there firstly.

Figure 5.11: Schematic Representing Model Object Roles



Storage nodes are used in the model to represent drainage area outlets as well as major system junctions. For storage nodes representing drainage area outlets, the invert of the storage node represents the invert of the minor system inlet (1.8 m below proposed road grade for areas with no storage and 2.15 m below proposed road grade for areas with storage), and the rim of the storage node is assumed to be equal to the proposed road grade plus the allowable flow depth on the street (i.e. 0.35 m). If the available storage volume in a storage node is exceeded, flows spill above the storage node and into the downstream street segment conduit and continue routing through the system until ultimately flows either re-enter the minor system or reach the outfall of the major system. Storages representing major system junctions are assigned an invert elevation equal to the proposed road grade and a rim elevation equal to the proposed road grade plus the allowable flow depth on the street (i.e. 0.35 m). These storage nodes are assigned an area of 0 m^2 for linear volume calculations. No storage has been accounted for within storage nodes at junction points.

Storage nodes that serve as outlets for the conceptual lumped areas with surface storage mentioned in **Section 5.6**, were assigned a storage curve assuming a maximum storage of 30 m^3/ha. Storage curves in PCSWMM are required to be input as depth-area curves; as such an equivalent area was calculated for each depth along the curve. All storage was assumed to be between the top of grate and a flow depth of 0.35 m (i.e. between a depth of 1.8 m and 2.15 m).

Minor system capture rates are specified in outflow links which use a user-specified depth-discharge curve defined to restrict outlet link flows to the 2-year or the 5-year rate as described in **Section 5.6**.

Subarea routing in lumped areas has been set to route 30% of the impervious area in each subcatchment through the pervious area of the subcatchment, in order to account for directly connected imperviousness.

5.7 PROPOSED CONDITION INPUT PARAMETERS

Drawings ST-1 and **ST-2** summarize the discretized subcatchments used in the conceptual analysis of the proposed Mahogany Phases 2-4 Development and outlines the major overland flow paths. All parameters were assigned as per applicable Ottawa Sewer Design Guidelines (OSDG), Ontario Ministry of the Environment and Climate Change (MOECC) and background report requirements.

The proposed SWM Pond 3 hydrologic/hydraulic model is based on the watercourse cross sections used in IBI's 2007 existing condition XP-SWMM model of the WCMD and its tributary from Century Road to their confluence location at Point B (see **Drawing ST-2**) and the cross sections derived by Stantec in the existing condition model calibration for the section of drain upstream of Century Road. Runoff from rural areas 6, 7 and 9 enter the system at the upstream ends as per the calibrated existing conditions model and the parameters are shown in **Table 5.6**.

The proposed SWM Pond 2 hydrologic/hydraulic model is based on IBI's 2012 Mahogany Phase 1 XP-SWMM model of the rehabilitated Mahogany Creek from Century Road to Point A (see **Drawing ST-1**). Runoff from rural areas 2, and 10, as well as outflow hydrographs for the three storm sewer outlets from Phase 1 into the creek, and for the major system outfalls from Phase 1 into the creek were obtained from IBI's 2012 XP-SWMM model for the available storm events.

5.7.1 Hydrologic Parameters

Key parameters for the proposed development areas are summarized below, while example input files are provided for the 100-year, 3hr Chicago storm which indicate all other parameters for the Mahogany Creek development Areas in **Appendix C.5** and for the Wilson Cowan Drain development areas in **Appendix C.6**. For all other input files and results of storm scenarios, please examine the electronic model files located on the CD provided with this report. This analysis was performed using PCSWMM, which is a front-end GUI to the EPA-SWMM engine. Model files can be examined in any program which can read EPA-SWMM files version 5.1.012.

Table 5.10 presents the general subcatchment parameters used for proposed development areas:

Table 5.10: General Subcatchment Parameters

Subcatchment Parameter	Value
Infiltration Method	Horton
Max. Infil. Rate (mm/hr)	76.2
Min. Infil. Rate (mm/hr)	13.2
Decay Constant (1/hr)	4.14
N Imperv	0.013
N Perv	0.25
Dstore Imperv (mm)	1.57

Subcatchment Parameter	Value
Dstore Perv (mm)	4.67
Zero Imperv (%)	0

Table 5.11 and **Table 5.12** present the individual parameters that vary for each of the proposed subcatchments in the Mahogany SWM Pond 2 and WCMD SWM Pond 3 PCSWMM models respectively.

Table 5.11: Mahogany SWM Pond 2 Individual Proposed Subcatchment Parameters

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient
C102A	1.34	444.0	0.5	57.1%	0.60
C105A	0.45	173.0	0.5	57.1%	0.60
C106A	0.59	210.0	0.5	57.1%	0.60
C108A	1.14	306.0	0.5	57.1%	0.60
C109A	0.71	141.0	0.5	57.1%	0.60
C110A	1.56	348.0	0.5	57.1%	0.60
C111A	0.78	169.0	0.5	57.1%	0.60
C112A	1.58	428.0	0.5	57.1%	0.60
L101A	1.46	788.0	1.0	50.0%	0.55
L103A	7.39	2651.0	1.0	14.3%	0.55
L104A	5.30	2102.0	1.0	50.0%	0.55
L104B	0.60	135.0	1.0	14.3%	0.30
L105A	3.01	1382.0	1.0	57.1%	0.60
L105B	2.89	650.0	1.0	71.4%	0.70
L106A	3.22	1564.0	1.0	57.1%	0.60
L106B	2.49	560.0	1.0	14.3%	0.30
L107A	1.54	250.0	1.0	50.0%	0.55
L109A	2.34	1067.0	1.0	50.0%	0.55
L110A	2.94	1163.0	1.0	57.1%	0.60
L111A	2.81	1106.0	1.0	50.0%	0.55
L112A	1.85	693.0	1.0	57.1%	0.60
L113A	1.51	721.0	1.0	57.1%	0.60
L115A	3.89	1374.0	1.0	57.1%	0.60
POND2	3.28	738.0	1.0	42.9%	0.50
UNC1	0.72	162.0	1.0	28.6%	0.40

Table 5.12: WCMD SWM Pond 3 Individual Proposed Subcatchment Parameters

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient
POND3	1.36	489.0	1.0	57.14%	0.60
L207A	1.26	283.0	1.0	57.14%	0.60
L205A	3.02	967.0	1.0	57.14%	0.60
L204A	3.09	1016.0	1.0	57.14%	0.60
L203B	1.17	450.0	1.0	57.14%	0.60

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient
L202A	4.40	1237.0	1.0	57.14%	0.60
FUT	6.61	1487.0	1.0	57.14%	0.60
C206A	2.55	829.0	1.0	57.14%	0.60

Rural areas tributary to the WCMD and its tributary (see areas 3-PRE, 4-PRE, 5-PRE, 8-PRE1 and 8-PRE2 on **Drawing ST-2**) were modeled with the same hydrologic parameters used in the calibrated existing condition model shown in **Table 5.6**.

Table 5.13 and **Table 5.14** summarize the storage node parameters used in the models. Conceptual development areas with no surface storage are modeled assuming storage node depths of 2.15 m (i.e. 1.8 m below proposed road grade to the invert and 0.35 m above proposed road grade to the rim elevation). Conceptual development areas with 30 m^3/ha of surface storage are modeled assuming storage node depths of 2.50 m (i.e. 2.15 m below proposed road grade to the invert and 0.35m above proposed road grade to the rim elevation). The 30 m^3/ha surface storage is available between depths 1.8 and 2.15 m in the storage node.

Table 5.13: Mahogany SWM Pond 2 Model Storage Node Parameters

Storage Node	Invert Elevation (m)	Rim Elevation (m)	Total Depth (m)	Max. Volume (m^3)
C102A-S	87.99	90.24	2.25	0
C105A-S	87.73	90.33	2.60	13
C106A-S	87.94	90.44	2.50	18
C108A-S	88.15	90.65	2.50	34
C109A-S	88.13	90.63	2.50	21
C110A-S	88.41	90.91	2.50	41
C111A-S	87.73	90.23	2.50	23
C112A-S	88.18	90.68	2.50	45
L101A-S	87.65	90.15	2.50	41
L103A-S	87.80	90.30	2.50	216
L104A-S	87.80	90.30	2.50	161
L104B-S	88.08	90.48	2.40	0
L105A-S	87.82	90.32	2.50	90
L105B-S	88.20	90.70	2.50	430
L106A-S	88.01	90.51	2.50	94
L106B-S	88.29	90.69	2.40	0
L107A-S	88.15	90.65	2.50	46
L109A-S	88.22	90.72	2.50	68
L110A-S	88.50	91.00	2.50	87
L111A-S	88.13	90.63	2.50	85
L112A-S	88.18	90.68	2.50	55
L113A-S	88.20	90.70	2.50	44
L115A-S	88.20	90.70	2.50	117

Table 5.14: WCMD SWM Pond 3 Model Storage Node Parameters

Storage Node	Invert Elevation (m)	Rim Elevation (m)	Total Depth (m)	Max. Volume (m^3)
L207A-S	87.47	90.22	2.75	0
L205A-S	87.65	90.15	2.50	0
L204A-S	88.00	90.50	2.50	0
L203B-S	88.10	90.60	2.50	0
L202A-S	88.00	90.50	2.50	0
FUT-S	88.54	92.50	3.96	0
C206A-S	87.62	90.12	2.50	0

5.7.2 Hydraulic Parameters

As per the OSDG 2012, Manning's roughness values of 0.013 were used for sewer modeling and overland flow corridors representing roadways. Any grassed swales were modeled using Manning's roughness values of 0.025.

Conceptual storm sewers were modeled to estimate flow capacities and hydraulic grade lines (HGLs) in the proposed condition with consideration of the SWM ponds backwater acting on the sewers. The conceptual storm sewer design sheet is included in **Appendix C.2**.

Table 5.15 and **Table 5.16** below present the parameters for the outlet link objects in the models, which represent minor system inlets.

Table 5.15: Mahogany SWM Pond 2 Model Outlet Link Parameters

Outlet Name	Tributary Area ID	Inlet Storage Node	Minor System Node	100-year Capture (L/s)
C102A-IC	C102A	C102A-S	102	0.245
C105A-IC	C105A	C105A-S	105	0.082
C106A-IC	C106A	C106A-S	106	0.112
C108A-IC	C108A	C108A-S	108	0.204
C109A-IC	C109A	C109A-S	109	0.122
C110A-IC	C110A	C110A-S	110	0.286
C111A-IC	C111A	C111A-S	111	0.143
C112A-IC	C112A	C112A-S	112	0.286
L101A-IC	L101A	L101A-S	101	0.163
L103A-IC	L103A	L103A-S	103	0.184
L104A-IC	L104A	L104A-S	104	0.500
L104B-IC	L104B	L104B-S	104	0.005
L105A-IC	L105A	L105A-S	105	0.347
L105B-IC	L105B	L105B-S	105	0.448
L106A-IC	L106A	L106A-S	106	0.377
L106B-IC	L106B	L106B-S	106	0.010
L107A-IC	L107A	L107A-S	107	0.163
L109A-IC	L109A	L109A-S	109	0.224

Outlet Name	Tributary Area ID	Inlet Storage Node	Minor System Node	100-year Capture (L/s)
L110A-IC	L110A	L110A-S	110	0.326
L111A-IC	L111A	L111A-S	111	0.265
L112A-IC	L112A	L112A-S	112	0.204
L113A-IC	L113A	L113A-S	113	0.173
L115A-IC	L115A	L115A-S	115	0.418

Table 5.16: WCMD SWM Pond 3 Model Outlet Link Parameters

Outlet Name	Tributary Area ID	Inlet Storage Node	Minor System Node	100-year Capture (L/s)
C206A-IC	C206A	C206A-S	206	0.490
L202A-IC	L202A	L202A-S	202	0.459
L203B-IC	L203B	L203B-S	203	0.143
L204A-IC	L204A	L204A-S	204	0.326
L205-IC	L205A	L205A-S	205	0.377
L207A-IC	L207A	L207A-S	207	0.152

5.7.3 Boundary Conditions

The conceptual PCSWMM hydrology for the proposed development along with the proposed storm sewers were used to assess the peak inflows and hydraulic grade line (HGL) in the subdivision. Conceptual storage nodes and outlet structures for the proposed SWM Pond 2 and SWM Pond 3, as well as for the future SWM wet/dry pond within the future development area were included in the models.

The calibrated existing condition model for the Cowan Wilson Municipal Drain (CWMD) and its tributary was used along with the proposed and future development peak flows to assess the post development peak flows in the drain at Flow Point B. As per the 2007 IBI Servicing Report for the Mahogany Development, a free outfall was assumed for the 25 mm storm, a fixed backwater level equal to the normal water level in the drain of 86.42 m was used for the 2-year and 5-year, 24-hour SCS Type II simulations, and a fixed backwater level equal to 87.26 m (100-year water level in the drain as per the 2005 MMM Jock River Reach 2 & Mud Creek Subwatershed Study) was used for the 100-year storm simulations for both the 3-hr Chicago distribution and the 24-hour SCS Type II. A figure from MMM's 2005 report has been included in **Appendix C.1.**

Similarly, IBI's 2012 Mahogany Phase 1 model for the rehabilitated Mahogany Creek was used along with the proposed development peak flows from the conceptual SWM Pond 2 to assess the peak flows in the drain at Flow Point A. A free outfall was assumed for the 25 mm storm, while tidal curves were developed for all other storms at the outlet based on the Rideau River water levels at the confluence of the Mahogany Creek as obtained from the MVCA (see correspondence in **Appendix C.1**) and shown in **Table 5.17**.

Table 5.17: Rideau River Water Levels at Mahogany Creek Confluence

River Cross Section ID	Water Level (m)				
	2-year	5-year	10-year	20-year	100-year
19530	84.73	85.03	85.18	85.29	85.52

The model was initially run with a fixed backwater level equal to the water level in the Rideau River and the resulting water level time series for the most downstream node was then saved as the new tidal curve to be used in subsequent runs for that storm event. The 100-year water level at the Rideau River as a fixed water level was used to develop a tidal curve boundary condition for the July 1st, 1979 historical storm.

5.8 PROPOSED DEVELOPMENT MODELS RESULTS AND DISCUSSION

The following section summarizes the key hydrologic and hydraulic model results. For detailed model results or inputs please refer to the example input files in **Appendix C.5** and **Appendix C.6** for SWM Pond 2 and SWM Pond 3 respectively**,** and the electronic model files on the enclosed CD.

5.8.1 Wilson Cowan Municipal Drain SWM Servicing Options

As requested by the City, a separate storm sewer outlet to the WCMD has been proposed for the future development area FUT to avoid crossing the natural woodlot area owned by the City. Due to the limiting target peak flows at Point B as discussed in **Section 5.3.1.2**, it is recommended that both 'Enhanced' quality control as well as quantity/erosion control be provided for the future development area prior to discharging into the WCMD.

Similarly, the City requested Stantec assess alternative storm servicing options for the areas discharging into the WCMD to remove storm crossings under the tributary. As a result, runoff from area C108A has been diverted to go to the proposed SWM Pond 2 and thus, one of the storm crossings has been removed. However, in order to remove the second storm crossing, quality and quantity control of runoff needs to be provided for the proposed development areas within the drain and its tributary (see **Drawing ST-2**). Quality control could be provided through oil/grit separator units but discharging minor system peak flows uncontrolled into the drain from areas L207A, C206A and L205A would result in higher number of erosion threshold exceedances and negative impacts to the drain under post development conditions. Thus, an additional storage area for quantity control within the proposed development areas would be required to meet the erosion threshold targets, which has been deemed unfeasible. As a result, it is proposed to install a storm sewer under the tributary to convey minor system peak flows from areas L207A, C206A and L205A to the proposed SWM Pond 3 to receive the quality and quantity/erosion control required.

5.8.2 Overland Flow

Overland flows from the SWM Pond 2 sewershed area will be directed to the pond as shown on **Drawing ST-1**. Overland flow from SWM Pond 3 areas east of the WCMD tributary will be directed to the pond. Major flows from the proposed development areas west of the tributary will be directed to the WCMD tributary, while controlled peak flows from the future development area FUT will discharge into the WCMD on the west end of the site as shown on **Drawing ST-2**.

Table 5.18 and **Table 5.19** present the conceptual total surface water depths (static ponding depth + dynamic flow) on the proposed roads for the 100-year design storm and climate change storm assuming short streets with a 0.5% slope. The conceptual PCSWMM models do not take into account the routing effect of the streets and as such, the values shown in the tables below are considered conservative.

Table 5.18: Conceptual Dynamic Surface Water Depths – Mahogany SWM Pond 2 Model

Storage node ID	Proposed Road Grade Elevation (m)	Rim Elevation (m)	100-year, 3-hour Chicago		100-year, 3-hour Chicago+20%	
			Max Surface HGL (m)	Total Surface Ponding Depth (m)	Max Surface HGL (m)	Total Surface Ponding Depth (m)
C102A-S	89.79	90.24	90.13	0.34	90.22	0.43
C105A-S	89.88	90.33	90.21	0.33	90.32	0.44
C106A-S	90.09	90.44	90.37	0.28	90.45	0.36
C108A-S	90.30	90.65	90.54	0.24	90.57	0.27
C109A-S	90.28	90.63	90.54	0.26	90.58	0.30
C110A-S	90.56	90.91	90.70	0.14	90.73	0.17
C111A-S	89.88	90.23	90.21	0.33	90.32	0.44
C112A-S	90.33	90.68	90.46	0.13	90.48	0.15
L101A-S	89.70	90.15	90.07	0.37	90.12	0.42
L103A-S	89.95	90.30	90.16	0.21	90.20	0.25
L104A-S	89.95	90.30	90.16	0.21	90.19	0.24
L104B-S	89.88	90.48	90.11	0.23	90.16	0.28
L105A-S	89.97	90.32	90.14	0.17	90.16	0.19
L105B-S	90.35	90.70	90.32	-0.03	90.39	0.04
L106A-S	90.16	90.51	90.37	0.21	90.46	0.30
L106B-S	90.09	90.69	90.43	0.34	90.50	0.41
L107A-S	90.30	90.65	90.54	0.24	90.57	0.27
L109A-S	90.37	90.72	90.52	0.15	90.56	0.19
L110A-S	90.65	91.00	90.76	0.11	90.78	0.13
L111A-S	90.28	90.63	90.44	0.16	90.46	0.18
L112A-S	90.33	90.68	90.47	0.14	90.48	0.15
L113A-S	90.35	90.70	90.48	0.13	90.50	0.15
L115A-S	90.35	90.70	90.54	0.19	90.56	0.21

Table 5.19: Conceptual Dynamic Surface Water Depths – WCMD SWM Pond 3 Model

Storage node ID	Proposed Road Grade Elevation (m)	Rim Elevation (m)	100-year, 3-hour Chicago		100-year, 3-hour Chicago+20%	
			Max Surface HGL (m)	Total Surface Ponding Depth (m)	Max Surface HGL (m)	Total Surface Ponding Depth (m)
L207A-S	89.62	90.22	89.69	0.07	89.70	0.08
L205A-S	89.80	90.15	89.98	0.18	90.00	0.20
L204A-S	90.15	90.50	90.34	0.19	90.36	0.21
L203B-S	90.25	90.60	90.37	0.12	90.38	0.13
L202A-S	90.15	90.50	90.36	0.21	90.38	0.23
FUT-S	N/A	92.50	90.63	N/A	91.44	N/A
C206A-S	89.77	90.12	89.97	0.20	89.99	0.22

5.8.3 Hydraulics

Table 5.20 and **Table 5.21** summarize the conceptual HGL results within the subdivision for the 100-year, 3 hour Chicago storm, the 100-year, 24-hour SCS Type II storm and for the 'climate change' scenario storm required by the City of Ottawa Sewer Design Guidelines (2012), where 100-year intensities for the 3-hour Chicago storm were increased by 20%.

During the detailed design stage of the subdivision, the grading and storm design will ensure that the maximum hydraulic grade line is kept at least 0.30 m below the underside-of-footing (USF) of any adjacent units connected to the storm sewer during design storm events as required by the City of Ottawa.

Table 5.20: Mahogany SWM Pond 2 - Conceptual Hydraulic Grade Line Results

STM MH	Proposed Road Grade Elevation (m)	100-year Design Storms HGL (m)		Worst Case 100-year HGL (m)		100-year 3hr Chicago + 20%	
		3-hour Chicago	24-hour SCS Type II	HGL	Prop. Road Grade – HGL Clearance	HGL (m)	Prop. Road Grade - HGL Clearance (m)
101	89.69	86.59	86.65	86.65	3.04	86.84	2.85
102	89.88	87.13	87.16	87.16	2.72	87.36	2.52
103	90.13	87.19	87.22	87.22	2.91	87.42	2.71
104	90.54	87.39	87.40	87.40	3.14	87.54	3.00
105	90.06	87.39	87.42	87.42	2.64	87.64	2.42
106	90.25	87.49	87.52	87.52	2.73	87.77	2.48
107	90.55	87.75	87.75	87.75	2.80	87.96	2.59
108	90.05	87.55	87.57	87.57	2.48	87.81	2.24
109	90.40	87.55	87.58	87.58	2.82	87.87	2.53
110	90.56	87.61	87.63	87.63	2.93	87.95	2.61
111	90.03	87.46	87.49	87.49	2.54	87.70	2.33

STM MH	Proposed Road Grade Elevation (m)	100-year Design Storms HGL (m)		Worst Case 100-year HGL (m)		100-year 3hr Chicago + 20%	
		3-hour Chicago	24-hour SCS Type II	HGL	Prop. Road Grade – HGL Clearance	HGL (m)	Prop. Road Grade - HGL Clearance (m)
112	90.17	87.57	87.61	87.61	2.56	87.81	2.36
113	90.25	87.66	87.70	87.70	2.55	87.91	2.34
114	90.35	87.74	87.78	87.78	2.57	87.99	2.36
115	90.50	87.85	87.89	87.89	2.61	88.10	2.40

Table 5.21: WCMD SWM Pond 3 - Modeled Hydraulic Grade Line Results

STM MH	Proposed Road Grade Elevation (m)	100-year Design Storms HGL (m)		Worst Case 100-year HGL (m)		100-year 3hr Chicago + 20%	
		3-hour Chicago	24-hour SCS Type II	HGL	Prop. Road Grade – HGL Clearance	HGL (m)	Prop. Road Grade - HGL Clearance (m)
201	90.12	87.52	87.49	87.52	2.60	87.54	2.58
202	90.23	87.68	87.66	87.68	2.55	87.71	2.52
203	90.46	87.86	87.83	87.86	2.60	87.88	2.58
204	90.22	87.59	87.56	87.59	2.63	87.61	2.61
205	89.84	87.72	87.69	87.72	2.12	87.74	2.10
206	89.98	87.82	87.79	87.82	2.16	87.84	2.14
207	89.62	87.94	87.92	87.94	1.68	87.97	1.65

Based on the above results for the proposed development areas tributary to SWM Pond 3, area L207A between manholes 207 and 206 will need alternative housing design and/or sump pumps and backwater valves to prevent basement flooding. Further analysis is required during the detailed design stage to confirm if sufficient runoff can be infiltrated from the proposed development areas to incorporate an additional storm outlet into the WCMD for area L207A and still meet erosion threshold targets as confirmed by a revised geomorphic analysis for the reach of the WCMD that will serve as an outlet for the proposed subdivision.

5.8.4 Water Quantity Control

In addition to providing water quality control, it is also required that post development peak flows are at or below pre-development levels for events up to the 100-year storm at Point A in the Mahogany Creek and at Point B in the WCMD. Target peak flow rates for Point A were determined in the background documents and for Point B were determined based on a monitoring program and calibrated existing conditions model as described in **Section 5.3**.

Quantity control for the proposed and future development areas will be provided in three stormwater management facilities. The following tables show the post-to-pre development peak flow rate comparison for the receiving watercourses for the 25 mm, 4-hour Chicago Storm and the SCS Type II distribution for the 2-year, 5-year and 100-year storm events.

Table 5.22: Mahogany Creek Flow Rate Comparison at Point A

	Flow (m^3/s)			
	25 mm	**2-year**	**5-year**	**100-year**
Pre-Development	0.89	2.21	3.44	7.54
Post Development	0.78	1.71	2.54	6.43

Table 5.23: Wilson Cowan Drain Flow Rate Comparison at Point B

	Flow (m^3/s)			
	25 mm	**2-year**	**5-year**	**100-year**
Uncalibrated Pre-Development	0.820	1.880	2.900	6.370
Calibrated Pre-Development	0.041	0.188	0.322	1.094
Post Development	0.046	0.214	1.206	4.625

The above tables show that the proposed conceptual SWM Pond 2 and outlet configuration meet the quantity control criteria for the Mahogany Creek.

Post development peak flows at Flow Point B in the WCMD are significantly higher than the existing condition peak flows estimated with the calibrated existing conditions model, particularly during more infrequent storm events. Several analyses have been conducted for the WCMD and its tributary to assess the flooding capacity of the existing channels as well as the conveyance capacity of the existing infrastructure downstream of the proposed site based on the uncalibrated existing condition peak flows, which are higher than the estimated post development peak flows at Flow Point B. Based on the above and the results shown in **Table 5.23**, it can be concluded that additional quantity control to prevent negative impacts downstream due to flooding is not a requirement for the proposed development, and that quantity control for erosion control purposes will govern the detailed design of the proposed SWM Pond 3 and the SWM facility that will service the future development area.

It is recommended that a detailed geomorphic analysis be performed along the reach of the WCMD that will serve as a storm outlet for the proposed development in order to confirm the erosion thresholds for the existing channel. However, as a preliminary check, an erosion analysis has been performed to assess the number of critical velocity exceedances in both the existing and post development conditions assuming the existing channel critical velocity is 0.3 m/s as outlined in Matrix Solutions Geomorphic analysis for the WCMD reach upstream of the confluence with the tributary.

5.8.4.1 Preliminary Erosion Analysis for WCMD at Flow Point B

As mentioned above, an erosion threshold analysis was undertaken for the reaches along the WCMD and its tributary upstream of their confluence, which concluded that the critical discharge for the drain and its tributary along those reaches was 0.29 m^3/s and 0.24 m^3/s respectively. The geomorphic analysis also suggested a critical average velocity of 0.30 m/s and 0.76 m/s for the drain and its eastern tributary respectively.

Channel configuration for reach WCD-R1 (corresponding to the SWM pond 3 outlet reach) was not assessed by the geomorphology study. It is recommended that a detailed fluvial geomorphic analysis be undertaken along the reach of the WCMD that will receive runoff from the proposed SWM Pond 3 to determine the erosion thresholds for that reach.

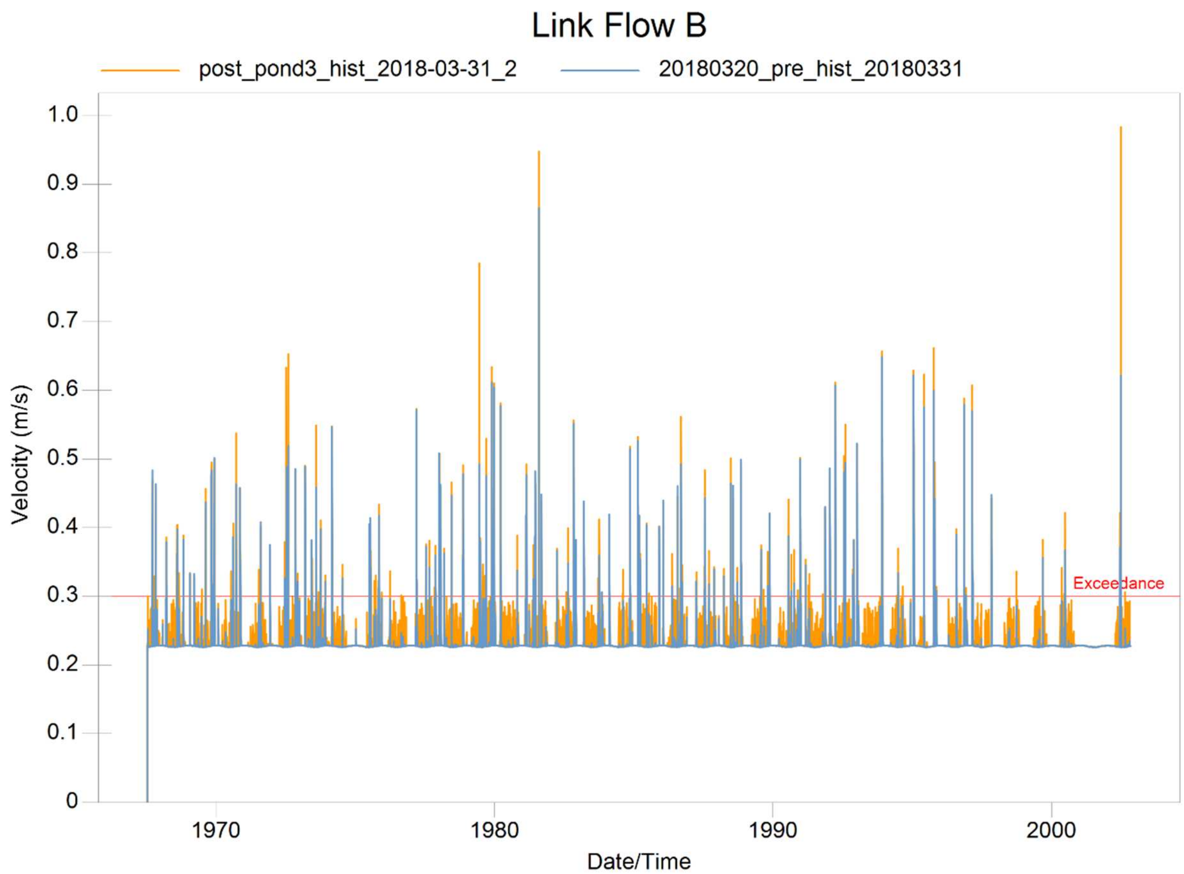
However, a preliminary erosion analysis to estimate the existing and proposed condition exceedances at flow Point B has been done using the most conservative critical velocity outlined in Matrix Solutions' Fluvial Geomorphic analysis (0.30 m/s). For the purposes of this analysis, the drain is assumed to currently maintain a channel bottom composed of soft but unconsolidated silty clay with a low channel slope similar to the main drain reach WCD-R2 upstream of the tributary confluence.

The erosion analysis considers a continuous event simulation using the pre and post development models and hourly rainfall records sourced from Environment Canada for the region for the period between 1967 and 2002.

To assess whether hourly rainfall data was of sufficient resolution for this analysis, the 5-minute rainfall data available for 2017 was statistically analyzed and used to disaggregate the historical hourly rainfall record into 15 minute increments. The 15-minute rainfall model produced results accurate to the hourly model within less than 1% difference, therefore the hourly analysis was deemed sufficient for continued modelling.

The reach downstream of Pond 3's outlet (identified as 'Flow B') was assessed for velocity threshold exceedances beyond 0.30 m/s using the erosion index feature of PCSWMM (which calculates the area under the velocity curve beyond the critical velocity). Flow B identifies 135 separate exceedances under the pre-development model, and 185 exceedances under the post-development model. Pre-development exceedances span a duration of 309,300 hours, with a total volume of exceedances of 2,127,000m. Post-development exceedances span a duration of 309,400 hours, with a total volume of exceedances of 2,281,000m, representing an increase of 7.24% above existing conditions over a similar timespan (see **Figure 5.12** below).

Figure 5.12: Velocity Threshold Exceedances for Pre and Post-Development Conditions



While the post-development conditions produce a small increase in the erosion index, the analysis is very sensitive to the value of the critical velocity (exceedance limit). If the post-development exceedance limit were to be 0.31 m/s instead of 0.30 m/s, the increase in erosion index would be effectively eliminated. This indicates that, following a more detailed fluvial geomorphic analysis of the downstream reaches from Pond 3 to the Rideau River, vulnerable erosion points could be protected to raise the average critical velocity to 0.31 m/s and the increase in erosion potential could be mitigated for the downstream reaches.

It is therefore recommended that the geomorphology of the WCMD be assessed downstream of the tributary confluence to ensure the main drain has been adequately hardened to receive the minor increase in velocity identified in the proposed condition model above. Channel improvements may have already been completed based on section 6.2.1 of the *Wilson-Cowan Drain Master Drainage Plan Study*, and it is expected that rip-rap/gabion mats have been installed previously on downstream culverts to reduce the effect of erosion on downstream flow restrictions. Should the results of such an assessment confirm the assumed channel substrate, erosion protection for the channel bottom will be required, and is to be determined concurrently with detailed design for the proposed SWM Pond 3.

5.9 CONCEPTUAL DESIGN OF STORMWATER MANAGEMENT FACILITIES

5.9.1 Facility Design Criteria

SWM Pond 2 is located in the northeast corner of the site and discharges to the Mahogany Creek. The tributary drainage area for SWM Pond 2 is 55.4 ha at 52% imperviousness consisting of low and medium density residential land use, as well as designated areas for parks, a pumping station, a school block, and a SWM block. As per IBI's 2007 Servicing Report for the Mahogany Development, the permanent pool elevation in the pond has been set at 85.50 m. It should be noted that based on the post development model, the water level in the Mahogany Creek at the outlet of SWM Pond 2 is 85.50 m during the 25 mm storm and 85.60 m during the 2-year storm. IBI's 2007 Servicing Report for the Mahogany Development outlined the HWL in SWM Pond 2 as 86.67 m, while the revised conceptual design shows a HWL of 86.55 m.

SWM Pond 3 is located along the northern boundary of the site, adjacent to the confluence of the Wilson Cowan Municipal Drain (WCMD) and its tributary. The tributary drainage area for SWM Pond 3 is 16.9 ha at 57% imperviousness consisting of low density residential land use and a SWM block. As per IBI's 2007 Servicing Report for the Mahogany Development, the permanent pool elevation in the pond has been set at 86.46 m, which is slightly higher than the normal water level in the WCMD of 86.42 m identified in IBI's report. It should be noted that based on the post development model, the water level in the WCMD at the outlet of SWM Pond 3 is 86.10 m during the 25 mm storm and 86.42 m during the 2-year storm. IBI's 2007 Servicing Report for the Mahogany Development outlined the HWL in SWM Pond 3 as 87.34 m, while the revised conceptual design shows a HWL of 87.49 m. The HWL increase in SWM Pond 3 results from further restricting the pond outflows into the WCMD.

An additional SWM Pond is recommended along the western boundary of the subject area to treat runoff from the future development area FUT prior to discharging into the WCMD. The tributary drainage area for the future SWM Pond is 6.61 ha at 50% imperviousness consisting of low density residential land use and a SWM block. Conceptual sizing and design of the additional SWM Pond is out of the scope of this report. However, target peak outflows from the future development area into the WCMD will be provided in this report.

5.9.1.1 Water Quality Control

The proposed SWM Pond 2 and SWM Pond 3 will be designed to achieve an 'enhanced' level of treatment of urban runoff according to Ministry of the Environment and Climate Change (MOECC) criteria – representing an 80% removal of total suspended solids (TSS).

The end-of-pipe facilities will be designed according to the recommendations of the Ministry of the Environment Stormwater Management Planning and Design Manual. **Table 5.24** and **Table 5.25** show the storage requirements recommended by MOECC as well as the volumes that can be provided in the conceptual SWM blocks for each facility. Detailed calculations for the conceptual SWM Ponds are included in **Appendix C.9**.

Table 5.24: SWM Pond 2 – Water Quality Requirements

Wet Pond					
Enhanced Level of Protection – Overall Removal Efficiency of TSS 80%					
Drainage Area (ha)	Imperviousness Ratio (%)	Permanent Pool (m^3)		Extended Detention Storage (m^3)	
		Req.	Provided	Req.	Provided
55.4	51.6	7,839	23,098	2,216	7,750

Table 5.25: SWM Pond 3 – Water Quality Requirements

Wet Pond					
Enhanced Level of Protection – Overall Removal Efficiency of TSS 80%					
Drainage Area (ha)	Imperviousness Ratio (%)	Permanent Pool (m^3)		Extended Detention Storage (m^3)	
		Req.	Provided	Req.	Provided
16.9	57.0	2,614	3,967	676	1,884

The extended detention of the proposed facilities has been significantly oversized for conservatism so that during the detailed design stage, the proposed facilities can be adjusted as required based on the final erosion analysis for the receiving WCMD reach.

5.9.1.2 Erosion Control

Stormwater management facilities could potentially have impacts on the amount of erosion or deposition occurring in a watercourse. As a result, it was recommended in the background documents that erosion analyses of the receiving watercourses be done during the detailed design stage to investigate potential changes in stream erosion as a result of urbanization and to evaluate the performance of the proposed SWM facilities.

As part of Phase 1 of the Mahogany Community Development, fish habitat enhancements were completed along the Mahogany Creek from Century Road to the outlet of the proposed SWM Pond 2. The drain was enhanced with a meandering low flow channel and adjacent shallow pool/wetland areas. IBI's 2012 Phase 1 SWM and Servicing Report outlined that the post development velocity at Point A during the 25 mm, 4-hour Chicago storm was 0.70 m/s compared to the 0.63 m/s velocity obtained with the current SWM Pond 2 configuration during the same storm event.

A preliminary erosion analysis for the WCMD has been undertaken as described in **Section 5.8.4.1.** The conceptual SWM Block for Pond 3 has been sized with a level of conservatism to account for future changes as a result of the final geomorphic analysis and erosion control requirements. The final SWM block size and criteria for the SWM facility should be confirmed during the detailed design stage once adequate information on the state of the watercourse is available.

5.9.1.3 Water Quantity Control

In addition to providing water quality control, it is also required that the proposed facilities provide quantity control to meet the target peak outflows at Point A in the Mahogany Creek and at Point B in the WCMD. Target rates for Point A and Point B were determined in the background documents and subsequently for the WCMD flows at Point B in an initial calibrated model as described in **Section 5.3**. Detailed calculations for the conceptual SWM Ponds are included in **Appendix C.9**.

The following tables show the pond target peak outflows, volume requirements and water levels for the proposed SWM Ponds 2 and 3 as well as for the recommended SWM facility for the future development area based on the information available to date. The information provided is based on the post development models for the Mahogany Creek and the WCMD for the 25 mm, 4-hour Chicago Storm, and the 24-hour, SCS Type II Distribution for the 2-year, 5-year and 100-year storms.

Table 5.26: SWM Pond 2 Target Peak Outflows

	Design Storm			
	25 mm	**2-year**	**5-year**	**100-year**
Pond Outflow (m^3/s)	0.129	0.143	0.768	3.563
Pond Water Level (m)	85.76	86.00	86.17	86.55
Pond Active Storage (m^3)	4,030	7,750	10,385	17,325
Creek Water Level at Pond 2 Outlet (m)	85.50	85.60	85.68	86.12

Table 5.27: SWM Pond 3 Target Peak Outflows

	Design Storm			
	25 mm	**2-year**	**5-year**	**100-year**
Pond Outflow (m^3/s)	0.029	0.055	1.103	3.874
Pond Water Level (m)	86.91	87.25	87.37	87.46
Pond Active Storage (m^3)	1,297	2,478	2,878	3,202
WCMD Water Level at Pond 3 Outlet (m)	86.10	86.42	86.44	87.26

Table 5.28: Future Development Area Target Peak Outflows

	Design Storm			
	25 mm	**2-year**	**5-year**	**100-year**
Target Peak Outflow (m^3/s)	0.013	0.029	0.395	1.770
Active Storage Required (m^3)	433	887	1,050	1,180

5.9.2 Conceptual Stormwater Management Facility Design Components

It is recommended that the two proposed SWM facilities be designed as extended detention wet ponds. The general arrangement of the facilities is described below.

The permanent pool for SWM Pond 2 has been set at 85.50 m with a single 1800 mm diameter pond inlet with an invert of 84.90 m. The inlet of SWM Pond 2 will be partially submerged, approximately 0.60 m below the permanent pool, to minimize the amount of fill required in the upstream ends of the storm system which is estimated to range between 1.0 to 1.3 m.

The conceptual outlet for SWM Pond 2 was modeled assuming an orifice for quality control and a weir for quantity control. The first 0.50 m of active storage is controlled by a 350 mm diameter orifice. The secondary pond outlet occurs via a 5m-long weir with an invert at 86.00 m. It is recommended to include an emergency spillway at the pond HWL. The emergency spillway should not be lower than the 25-year water level in the Mahogany Creek (~85.94 m) as per City guidelines. An anticipated cross section of the conceptual SWM Pond 2 is provided in **Appendix C.7**. Detailed calculations for the conceptual SWM Ponds are included in **Appendix C.9**.

The permanent pool for SWM Pond 3 has been set at 86.46 m with a single 1500 mm diameter pond inlet with an invert of 85.60 m. The inlet of SWM Pond 3 will be partially submerged, approximately 0.86 m below the permanent pool, to minimize the amount of fill required in upstream ends of the storm system which is estimated to range between 1.0 to 1.5 m.

The conceptual outlet for SWM Pond 3 was modeled assuming an orifice for quality control, and a secondary orifice and a weir for quantity control. The first 0.64 m of active storage is controlled by a 150 mm diameter orifice. The secondary pond outlet occurs via aa orifice with invert at 87.10 m and a 3m-long weir with an invert at 87.30 m. It is recommended to include an emergency spillway at the pond HWL. The emergency spillway should not be lower than the 25-year water level in the WCMD as per City guidelines. An anticipated cross section of the conceptual SWM Pond 3 is provided in **Appendix C.8**. Detailed calculations for the conceptual SWM Ponds are included in **Appendix C.9**.

5.9.3 Pond Hydraulic Modeling Results

Each PCSWMM model scenario was analysed for the peak pond inflows and discharge rates as well as for peak pond HGLs. **Table 5.29** and **Table 5.30** below summarize the peak pond outflow rates for the different storm events. Climate change and historical scenarios were not intended to provide a level of service but were modeled to stress-test the conceptual design.

Table 5.29: SWM Pond 2 Peak Pond Outflow Rates and Water Levels

Storm Type	Storm Event	Peak Pond Inflow (m^3/s)	Peak Pond Discharge (m^3/s)	Pond Water Level (m)	Active Pond Volume (m^3)
Subdivision Design Storms	100yr3hrChicago	15.65	3.67	86.55	17,325
Pond Design Storms	25mm 4hr Chicago	2.41	0.129	85.76	4,030

Storm Type	Storm Event	Peak Pond Inflow (m^3/s)	Peak Pond Discharge (m^3/s)	Pond Water Level (m)	Active Pond Volume (m^3)
	2yr24hrSCS	3.49	0.143	86.00	7,750
	5yr24hrSCS	4.68	0.768	86.17	10,385
	100yr24hrSCS	12.51	3.563	86.55	17,325
Climate Change Storms	100yr3hrChicago_20%	19.96	5.377	86.73	20,715
Historical Storms	July 1st, 1979	11.16	4.849	86.69	19,915

Table 5.30: SWM Pond 3 Peak Pond Outflow Rates and Water Levels

Storm Type	Storm Event	Peak Pond Inflow (m^3/s)	Peak Pond Discharge (m^3/s)	Pond HGL (m)	Total Pond Volume (m^3)
Subdivision Design Storms	100yr3hrChicago	4.519	4.804	87.49	3,288
Pond Design Storms	25mm 4hr Chicago	0.977	0.029	86.91	1,297
	2yr24hrSCS	1.340	0.055	87.25	2,478
	5yr24hrSCS	2.119	1.103	87.37	2,878
	100yr24hrSCS	3.565	3.874	87.46	3,202
Climate Change Storms	100yr3hrChicago_20%	5.344	5.720	87.51	3,383
Historical Storms	July 1st, 1979	3.289	3.573	87.45	3,172

5.9.4 Other Considerations

Additional key design notes to be addressed during detailed design include the following:

- A 3 m wide access road with 1.0 m shoulders for ease of inspection and maintenance of the inlet, forebay and main cell. The route should be designed with a minimum slope to facilitate maintenance equipment maneuverability.
- A 3.5 m wide bench to be provided below the permanent pool with a maximum water depth of 0.3 m.

5.10 WATER BALANCE – INFILTRATION REQUIREMENTS

As per IBI's 2007 Mahogany Community SWM Servicing Report, developments within the Mahogany Community are required to provide best management practices (BMP's) to promote infiltration. The report recommended that infiltration trenches in rear yards of the residential areas be implemented to capture overland flows as part of the storm sewer system. Infiltration trenches are designed for areas less than 2 ha and are not suitable in areas with high water table. It is therefore recommended that during the detailed design stage of the development and as confirmed by a geotechnical engineer, areas comprised of soils suitable for infiltration be considered for infiltration trenches.

The use of alternate low impact development (LID) measures will be assessed during the detailed design stage to maximize runoff infiltration across the site. Further analysis will be required to estimate the amount of runoff that can be infiltrated across the proposed site and to confirm whether based on these results, additional uncontrolled peak flows can be discharged into the drain and still meet the target erosion thresholds in the WCMD.

6.0 GRADING

The Mahogany phase 2-4 lands drain predominantly from south to north, with a large ridge running north/south acting as a dividing line for development phases 2/3 and 4. Existing drainage for the development is divided by the ridge between the Mahogany Creek to the east, and a tributary of the Wilson Cowan Municipal Drain to the west. The Master Servicing Study provided preliminary grading for the Mahogany development which has been included for reference in **Appendix E**. For the purposes of this report a conceptual grading plan has also been prepared which takes into account anticipated overland flow conveyance, cover over proposed sewers, and grade raise restrictions as identified in the geotechnical investigation (see **Section 10.0**). The conceptual grading plan has been provided for reference in **Appendix E**. A detailed grading design will be developed at the time of final design. Detailed grading will adhere to all requirements as outlined in the City of Ottawa guidelines.

The conceptual grading plan (**Drawing GP-1**) identifies a portion of the site where adherence to the permissible grade raise restriction will be possible for roadways but may be exceeded within the lots at detailed design. Areas where grades are expected near the maximum permissible grade raise will be reassessed from a geotechnical standpoint once final anticipated grades have been established, and subject to either a pre-loading/surcharge program, or lightweight fill and/or other approved means outside of proposed rights-of-way to reduce the risks of unacceptable long-term post construction differential settlements should the grade raise restriction be exceeded.

7.0 UTILITIES

7.1 HYDRO

Accessible Hydro infrastructure exists along the eastern boundary of the site via existing plant within phase 1 crossing beneath the Mahogany Creek to the existing pumping station, and along existing overhead pole lines within the Century Road right-of-way. Exact size, location and routing of hydro utilities will be finalized after design circulation. Transformer locations and positioning of required utility easements will be identified in the detailed design stage. Upgrades to the existing hydro infrastructure within adjacent rights-of-way currently servicing rural lots are anticipated due to the relatively low existing demand on plant in the area.

7.2 ENBRIDGE GAS

Similarly, to Hydro, existing gas infrastructure exists at the boundaries of the subject site. Exact size, location and routing of gas infrastructure will be finalized after design circulation.

7.3 TELECOMMUNICATIONS

Both Bell and Rogers are expected to be able to service the subdivision. Infrastructure locations and easement requirements will be identified as part of the Composite Utility Planning process, following design circulation.

8.0 APPROVALS

Ontario Ministry of Environment and Climate Change (MOECC) Environmental Compliance Approvals (ECAs, formerly Certificates of Approval (CofA)) under the Ontario Water Resources Act will be required for proposed stormwater management facilities, storm and sanitary sewers, and inlet control devices (Transfer of review) for the proposed development. The Rideau Valley Conservation Authority should be circulated on such submissions so that CA sign-off may be given and submission of the ECAs may proceed. Permits from the Conservation Authority will be required for works relating to the proposed Pond 2's outlet to the Mahogany Creek. DFO approval will also be required for any HADD for fish habitat within the Mahogany Creek. The Engineer's Report for the Wilson Cowan Municipal Drain is required to be updated to ensure the proposed development will not negatively impact the existing properties and structures downstream of the site and recommend mitigation measures as required.

A MOE Permit to Take Water (PTTW) has been previously obtained for the development at large for dewatering during construction of the proposed works below the groundwater table as identified in the geotechnical report.

9.0 EROSION CONTROL

Erosion and sediment controls must be in place during construction. The following recommendations to the contractor will be included in contract documents.

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

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

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

It is proposed to maintain a network of cutoff swales and temporary sediment control basins to provide appropriate erosion and sediment control and migrate the sediment basins and cutoff swales westwards and southwards along with construction phasing for the development.

Refer to Erosion and Sediment Control Plan drawing EC-1 included in **Appendix E** for the proposed preliminary location of silt fences, cutoff swales, temporary sediment basins and other erosion control structures.

10.0 GEOTECHNICAL INVESTIGATION

A geotechnical investigation for the development was completed by Paterson Group Inc. in June of 2017. The report summarizes the existing soil conditions within the subject area and construction recommendations. For details which are not summarized below, please see the original Paterson report.

Subsurface soil conditions within the subject area were determined through field investigations in 2007/2008. In total, 49 boreholes were drilled throughout the subject lands between 2007 and 2008. In general soil stratigraphy consisted of topsoil and/or a thin silty sand layer followed by a deep silty clay strata, followed by a till (silty sand matrix). Bedrock was encountered in boreholes with depths ranging from 2m to 17m. The thickness of the existing topsoil ranged from 10 to 25mm.

Groundwater levels were encountered between 0.34m and 4.40m in depth. It is expected that construction will occur below the existing groundwater table. A permit to take water for construction activities has been obtained for the development at large as a result of development of Phase 1.

Based on the observed soil conditions, a grade raise restriction of between 1.5m and 2.5m above existing grade was recommended (see geotechnical report excerpts in **Appendix D**). The conceptual grading plan (**Drawing GP-1**) identifies a portion of the site where adherence to the permissible grade raise restriction will be possible for roadways, but may be exceeded within the lots at detailed design. Areas where grades are expected near the maximum permissible grade raise will be reassessed from a geotechnical standpoint once final anticipated grades have been established, and subject to either a pre-loading/surcharge program, or lightweight fill and/or other approved means outside of proposed rights-of-way to reduce the risks of unacceptable long-term post construction differential settlements should the grade raise restriction be exceeded.

11.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the preceding information, the following conclusions are summarized below:

11.1 POTABLE WATER ANALYSIS

- The proposed watermain sizing and alignment in Phases 2 to 4 is recommended to include a combination of 305mm and 203mm diameter pipes;
- During basic day conditions and in existing conditions, the proposed piping results in pressures slightly above the maximum pressure objective of 552kPa (80 psi) and may require pressure reduction measures as per the OBC;
- During peak hour conditions and in existing conditions, the proposed piping is capable of operating above the minimum pressure objective of 276kPa (40psi);
- In regard to FUS fire flow requirement, local internal watermains must be assessed and verified for fire flow requirements as development planning proceeds;
- Looping requirements within the development are achieved, however; the Rideau Valley watermain is currently the only feed to Manotick and until both the Manotick Links are constructed, the Mahogany Subdivision is a vulnerable area such that in the event there is a break in the Rideau Valley watermain, the system cannot provide basic day plus fire flow.

11.2 WASTEWATER SERVICING

The Mahogany subdivision will be serviced by a network of gravity sewers which will direct wastewater flows easterly through the Bridgeport Avenue sewer, and ultimately to the Eastman Pump Station. External lands to the west, as well as existing developments within a prior phase and along Manotick Main Street will also be conveyed through the subject property as directed in the MSS. The proposed sanitary sewer design indicates one connection points to the existing trunk sewer, with a total estimated peak outflow of 135.7L/s including flow from areas outside of the current development boundary. Pump station upgrades are to be reassessed for each development phase as development proceeds west.

11.3 STORMWATER MANAGEMENT

The following summarizes the stormwater management conclusions for the proposed development:

- The proposed stormwater management plan is in compliance with the goals specified in the background reports and the 2012 City of Ottawa Sewer Guidelines
- The conceptual SWM Ponds 2 and 3 meet MOECC storage requirements to achieve an Enhanced Level of Protection.
- The conceptual SWM Pond 2 provides sufficient storage to meet the target peak outflows at control Point A in the Mahogany Creek.
- The proposed SWM Pond 2 and SWM Pond 3 will have single storm inlets that will be partially submerged.

- Two major system inlets to SWM Pond 2 will be provided to ensure total flow depths on streets remain below the allowable flow depth of 0.35 m.
- Minor system capture rates across the proposed development will be restricted to the 2-year runoff for local street, park, and school areas, and to the 5-year runoff for collector road areas.
- The proposed school area will provide on-site storage to retain major system overflows up to the 100-year event.
- Overland flows from the SWM Pond 2 sewershed area will be directed to the pond.
- Overland flow from SWM Pond 3 areas east of the WCMD tributary will be directed to the pond. Major flows from the proposed development areas west of the tributary will be directed to the WCMD tributary, while the future SWM facility servicing the future development area will discharge into the WCMD.
- The calibrated existing conditions model for the WCMD and its tributary generates generally lower peak flows than the observed data during the 2017 monitoring program.
- The initial calibration of the existing condition model for the Wilson Cowan Municipal Drain generates significantly lower peak flows than the 2007 uncalibrated existing condition model for the drain.
- The conveyance capacity of the WCMD drain and its tributary as well as the conveyance capacity of the existing structures downstream of the site have been previously assessed in several studies based on the existing condition peak flows from the uncalibrated model and as such flooding and/or negative impacts to the downstream infrastructure are not a concern for the proposed site. Based on the above, it can be concluded that erosion control will govern the SWM criteria for the proposed and future development areas tributary to the WCMD and its tributary.
- A separate storm sewer outlet to the WCMD has been proposed for the future development area FUT to avoid crossing the natural woodlot area owned by the City.
- One storm crossing of the WCMD tributary will be required to service the development area tributary to SWM Pond 3.
- A portion of the tributary area to the proposed SWM Pond 3, area L207A between manholes 207 and 206, will need alternative housing design and/or sump pumps and backwater valves to prevent basement flooding due to grade raise constraints.
- Post development peak flows at Flow Point B in the WCMD are significantly higher than the existing condition peak flows estimated with the calibrated existing conditions model, but lower than the previously established existing condition peak flows with the 2007 uncalibrated model.
- A preliminary erosion analysis for the WCMD has been undertaken**.** The conceptual SWM Block for Pond 3 has been sized with a level of conservatism to account for future changes as a result of the final geomorphic analysis and erosion control requirements.
- It is recommended that the geomorphology of the WCMD be assessed downstream of the tributary confluence to ensure the main drain has been adequately hardened to receive the minor increase in velocity identified in the proposed condition model. Should the results of such an assessment confirm the assumed unconsolidated silty clay channel substrate, erosion protection for the channel bottom will be required, and is to be determined concurrently with detailed design for the proposed SWM Pond 3.

The following summarizes the stormwater management recommendations for the proposed development:

- The two proposed SWM facilities be designed as extended detention wet ponds.
- There are limitations to the validity of the flow curves established through the WCMD 2017 monitoring program and as such, it is recommended that further monitoring be performed along the drain and that the existing condition model calibration be improved at the detailed design stage.
- A detailed fluvial geomorphic analysis be undertaken along the reach of the WCMD that will receive runoff from the proposed SWM Pond 3 to determine the erosion thresholds for that reach.
- The erosion analysis and sizing of the proposed SWM Pond 3 be finalized at the detailed design stage based on the updated fluvial geomorphic analysis and that recommendations be provided for channel improvements as required.
- Both 'Enhanced' quality control as well as quantity/erosion control be provided for the future development area prior to discharging into the WCMD.
- Further analysis be performed during the detailed design stage to confirm if sufficient runoff can be infiltrated across the proposed development through LID measures to include an additional storm outlet into the WCMD for area L207A and still meet erosion threshold targets as confirmed by a revised geomorphic analysis for the reach of the WCMD that will serve as an outlet for the proposed subdivision.
- The detailed grading and storm design for the development ensure that the maximum 100-year hydraulic grade line is kept at least 0.30 m below the underside-of-footing (USF) of any adjacent units connected to the storm sewer.
- The Engineer's Report for the Wilson Cowan Municipal Drain be updated to reflect the proposed land use and drainage area changes.

11.4 GRADING

A conceptual grading plan has been prepared taking into account required overland flow conveyance, cover over sewers, hydraulic grade line requirements, and grade raise restrictions as identified in the geotechnical investigation. A detailed grading design will be developed at the time of final design. Detailed grading will adhere to all requirements as outlined in the City of Ottawa guidelines.

11.5 UTILITIES

Utility infrastructure exists in the general area of the subject site. Exact size, location and routing of utilities will be finalized at the detailed design stage.

APPENDICES

Appendix A : WATERMAIN HYDRAULIC ANALYSIS

Appendix B : SANITARY SEWER CALCULATIONS

Appendix C : STORMWATER MANAGEMENT CALCULATIONS

C.1 Background Reports Excerpts

C.2 Conceptual Storm Sewer Design Sheet

C.3 Wilson Cowan Municipal Drain Monitoring Program Data

C.4 Wilson Cowan Municipal Drain Existing Condition Model - Calibrated

C.5 Pond 2 - PCSWMM Model Input (Mahogany Creek)

C.6 Pond 3 - PCSWMM Model Input (WCMD)

C.7 Conceptual Profile Pond 2

C.8 Conceptual Profile Pond 3

C.9 Conceptual SWM Pond Calculations

C.1 BACKGROUND REPORTS EXCERPTS

C.2 CONCEPTUAL STORM SEWER DESIGN SHEET

C.3 WILSON COWAN MUNICIPAL DRAIN MONITORING PROGRAM DATA

C.4 WILSON COWAN MUNICIPAL DRAIN EXISTING CONDITION MODEL - CALIBRATED

C.5 POND 2 PCSWMM INPUT FILE (MAHOGANY CREEK)

C.6 POND 3 PCSWMM INPUT FILE (WCMD)

C.7 CONCEPTUAL PROFILE POND 2

C.8 CONCEPTUAL PROFILE POND 3

C.9 CONCEPTUAL SWM POND CALCULATIONS

Appendix D : GEOTECHNICAL INVESTIGATION EXCERPTS

Appendix E : DRAWINGS