Mahogany Subdivision Phases 2-4 – Functional Servicing Report

Project #160410140

Prepared for:
Minto Group Inc.

Prepared by:
Stantec Consulting Ltd.

June 30, 2017

Sign-off Sheet

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Stantec

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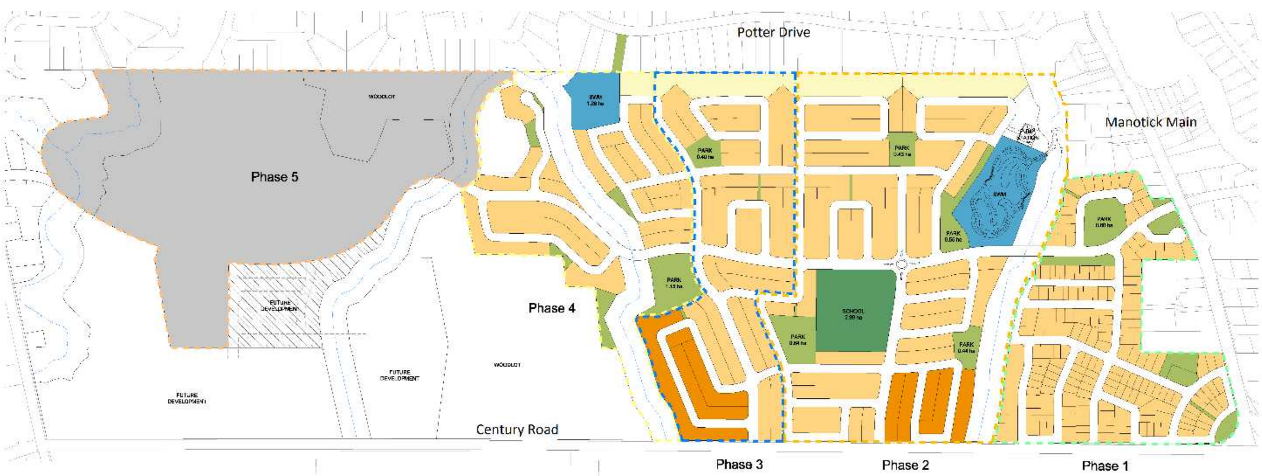
1.0 INTRODUCTION

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 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 Village of Manotick. The City has indicated that construction of these watermains are anticipated to take place in 2021 and should be completed prior to the buildout of Phase 2 of the Mahogany subdivision.

The Manotick link will connect to the future watermain along River Road at Earl Armstrong Road (east of the Jock River), travel south and connect to the future extended Main Street watermain north of Century Road (west of the Jock River). The North Island link will connect to the future Manotick link at Neilus Lane, travel northwest and connect to the 406mm diameter watermain along Rideau Valley Drive. The approximately alignments are shown in **Figure 3-1**.

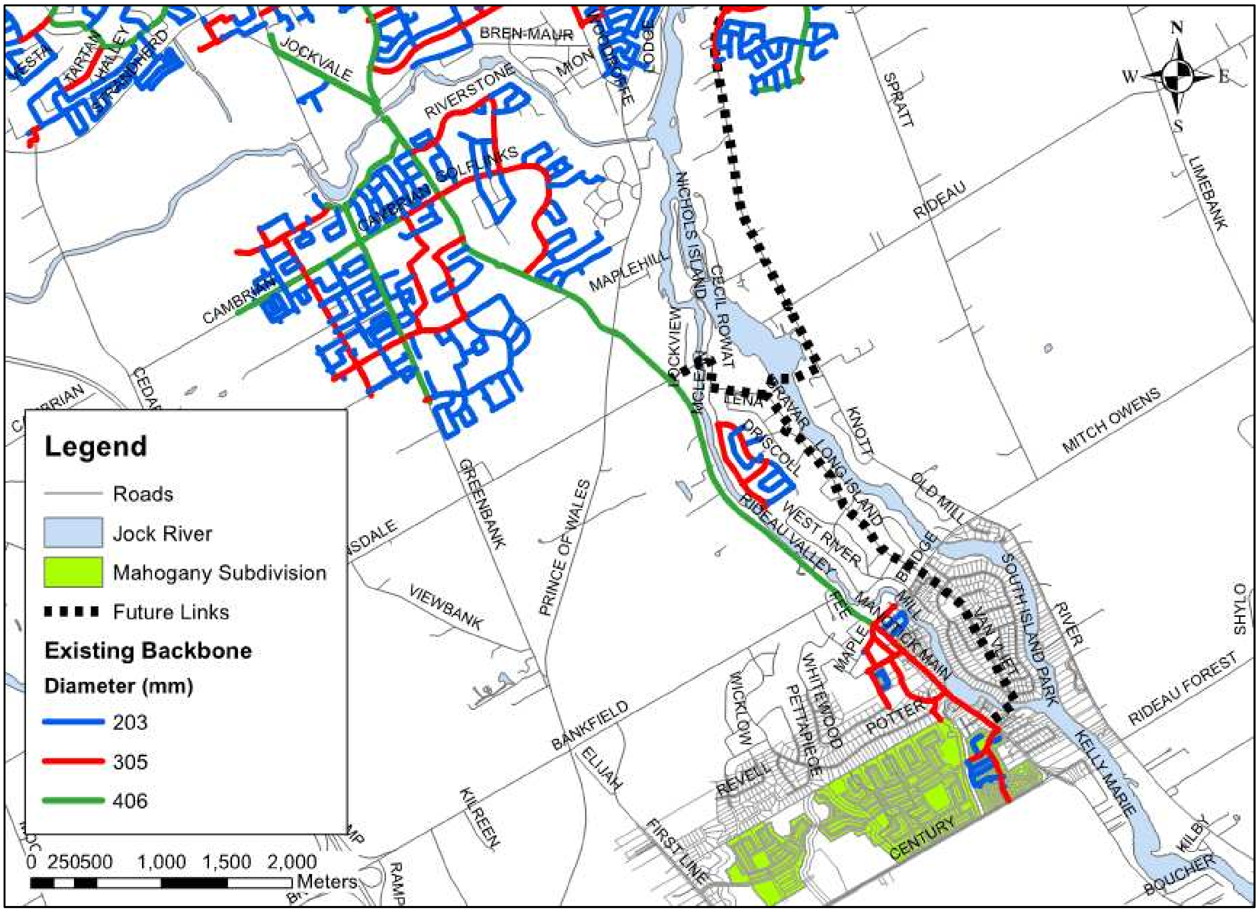


Figure 3-1: Future Manotick and North Island Watermain Link

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 has been completed and it is understood that Phase 2 and 3 are likely to proceed before the future Manotick links while Phase 4 and 5 are further into the future.

This analysis herein will review the available fire flows and operating pressures of Phase 2 and 3 under existing conditions. Phase 4 will also be reviewed as part of this overall servicing plan although it is not likely Phase 4 will be developed before the Manotick links are constructed.

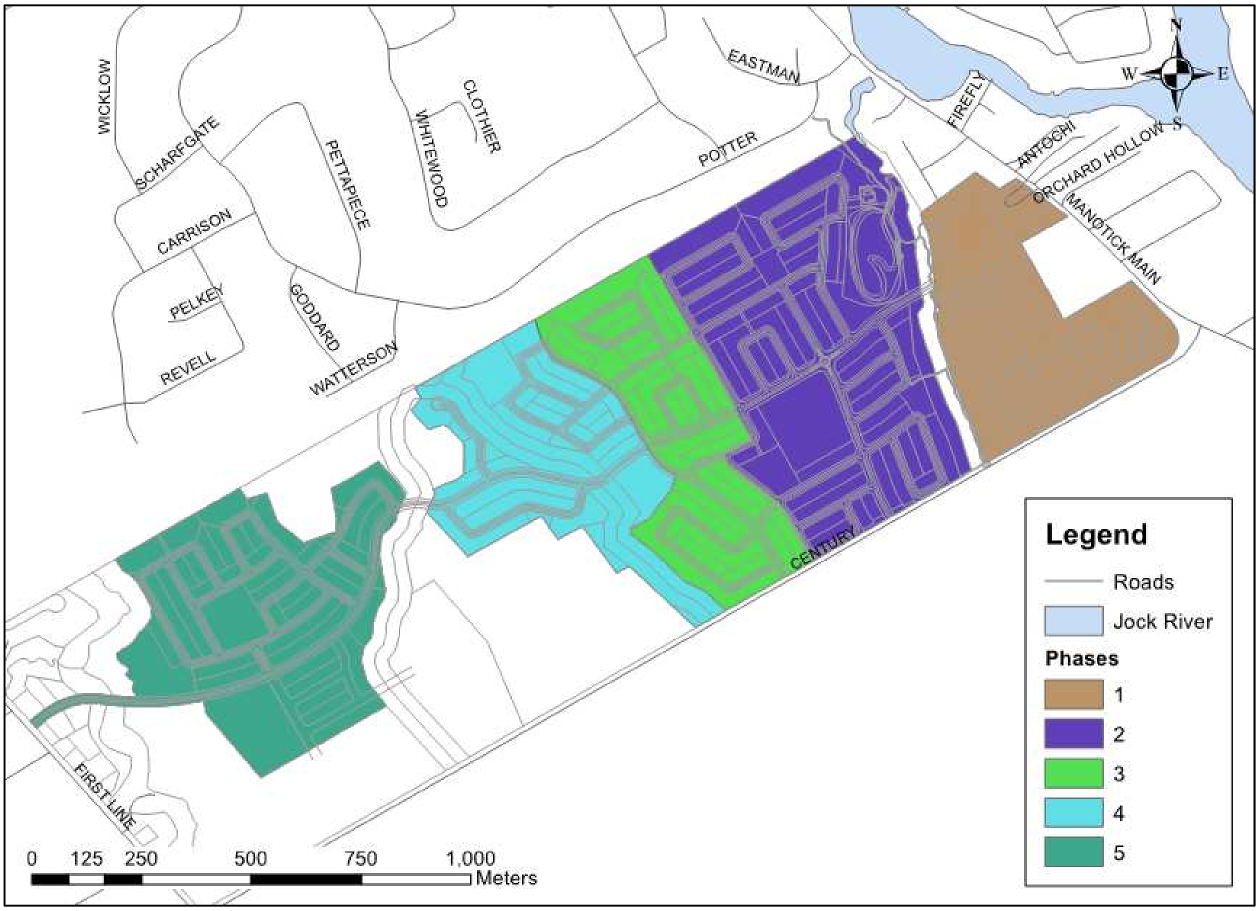


Figure 3-2: Phasing of Mahogany Subdivision

3.4 WATER DEMANDS

The latest plan for Phases 2 & 3 of the Mahogany Subdivision calls for a total of 930 units and an estimated population of 3,028. Upon build-out of Phase 4, there will be an estimated total of 1,189 units and a population of 3,907 (not including Phase 1 which will be 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)
2	Towns	99	1,447	5.86	14.66	32.24
	Singles	347				
3	Towns	93	1,013	4.10	10.26	22.56
	Singles	224				
4	Towns	0	568	2.30	5.75	12.65
	Singles	167				
	Total	***930***	***3028***	***12.26***	***30.66***	***67.46***

Table 3-2: Non-Residential Water Demands

Phase	Institution	Area (ha)	BSDY (L/s)	MXDY (L/s)	PKHR (L/s)
2	School	2.90	0.94	1.41	2.54

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)
2	6.80	16.06	34.78
3	4.10	10.26	22.56
4	2.30	39.20	86.23
Total	***13.20***	***65.52***	***143.57***

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.** shows the watermain sizes and alignment proposed for Phase 2 and 3.

The proposed connection locations to existing water infrastructure were identified in the MSS (DME, 2007). There will be two connections, one to the 300mm diameter watermain on Potter Drive (north) and one to the 300mm diameter watermain on Bridgeport Avenue that runs across Phase 1.

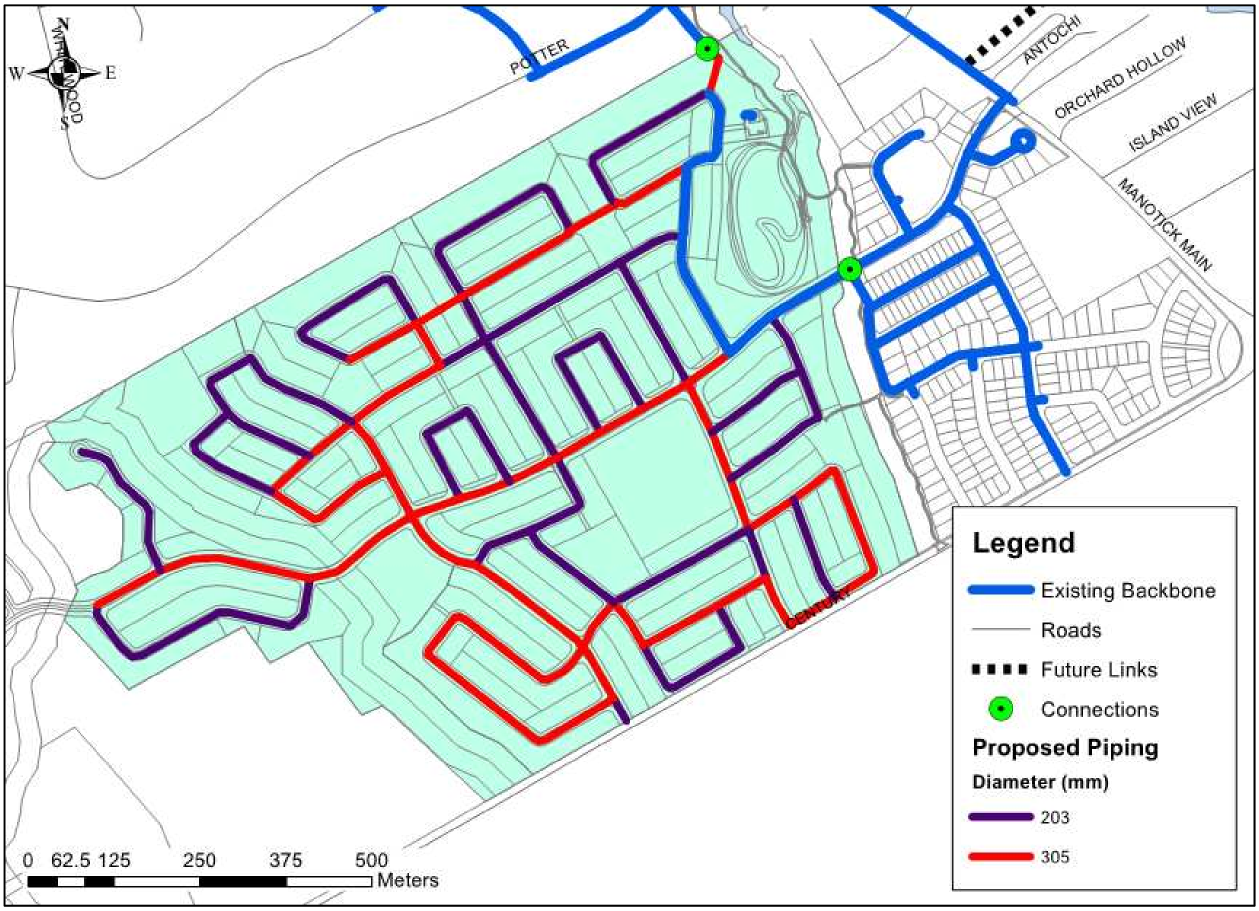


Figure 3-3: Proposed Watermain Sizing and Layout

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 13,000L/min (217 L/s) for a duration of 2 hours. Refer to **Appendix A** for FUS fire flow calculations.

In general, the FUS formula typically results in in fire flows in the range of 5,000 to 7,500 L/min for single family homes and 7,500 to 10,000L/min for townhomes or row houses with reasonable spacing and fire walls. 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 2.90ha. The area of the school building was estimated to be approximately 4,500m^2 (0.45 ha) and an FUS fire flow of 8,000 L/min required (**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 provided by the City via email on March 30, 2017. 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 by the City were based on computer model stimulations and are summarized in **Table 3-4** at two (2) locations. Boundary conditions requests and correspondence with the City are in **Appendix B**. It should be noted that the Manotick Main Street boundary conditions is upstream of the proposed connection at Bridgeport Drive

Table 3-4: Boundary Conditions for Connections Points

Location	Ground Elevation (m)	AVDY (m)	PKHR (m)	MXDY+FF* (m)
Potter Dr.	87.6	147.7	136.3	110.3
Manotick Main St.	92.5	147.7	136.4	110.0

*Fire flow = 10,000 L/min

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 3-5**).

Table 3-5: C-Factors used for applied watermain based on pipe diameter

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** 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 92.5m.

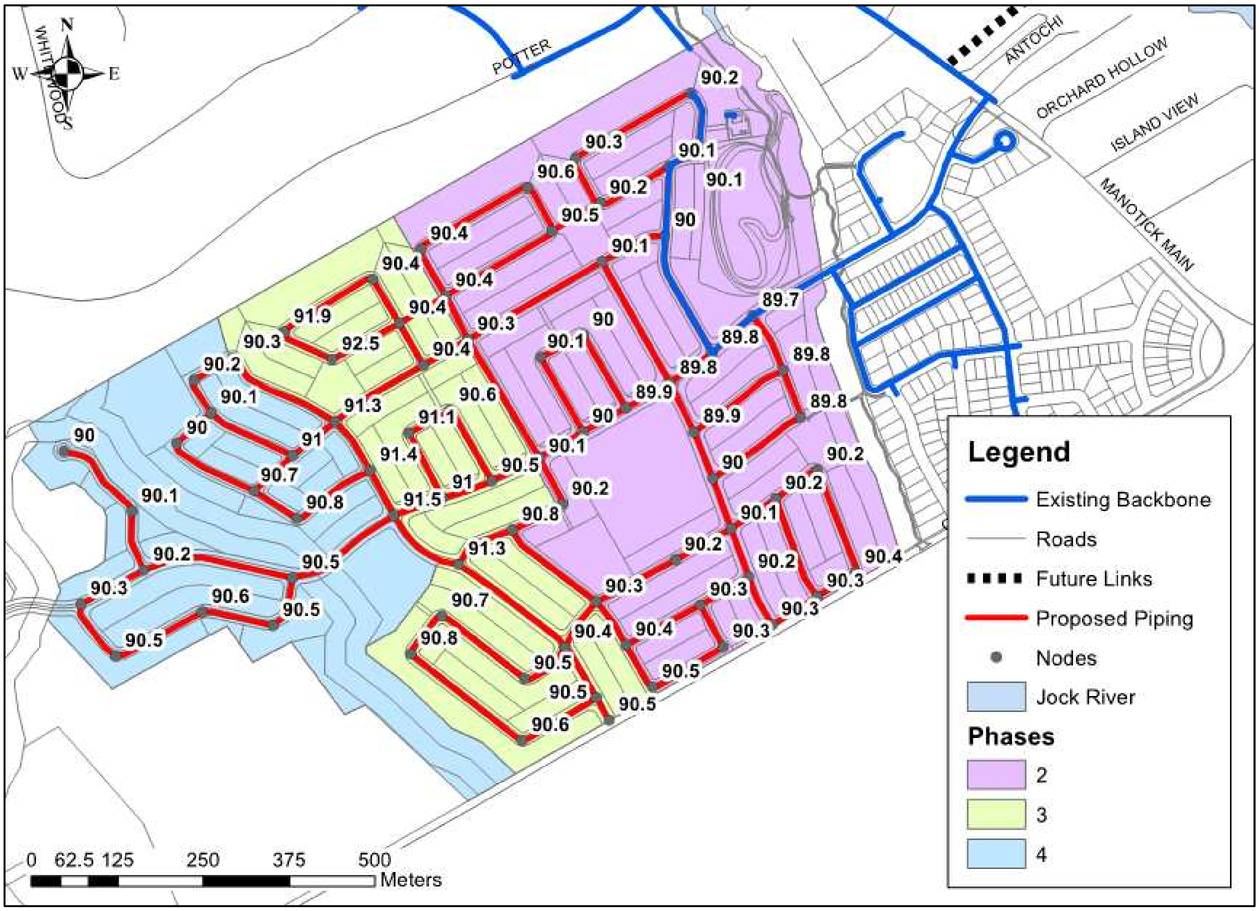


Figure 3-4: Ground Elevations (in meters)

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 by the City.

3.8.1 Basic Day

The hydraulic model results show that during basic day conditions, the maximum pressure is anticipated to be 568kPa (82 psi) for Phases 2, 3 and 4. This value exceeds the maximum objective pressure of 552kPa (80 psi). Hydraulic modeling results are demonstrated in **Figures 3-5 and 3-6.**

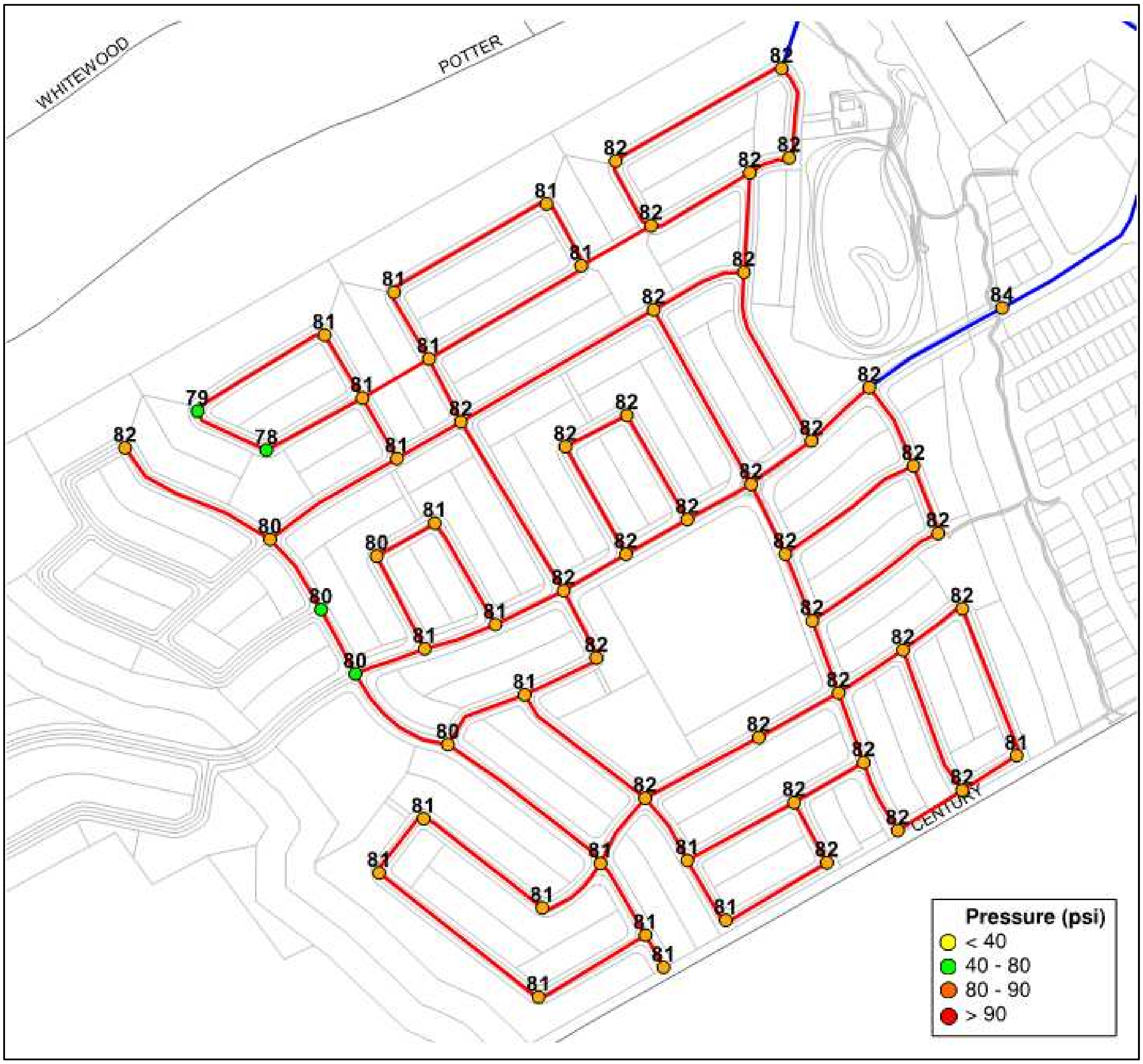


Figure 3-5: Phase 2 & 3 -Maximum Pressures During BSDY Conditions

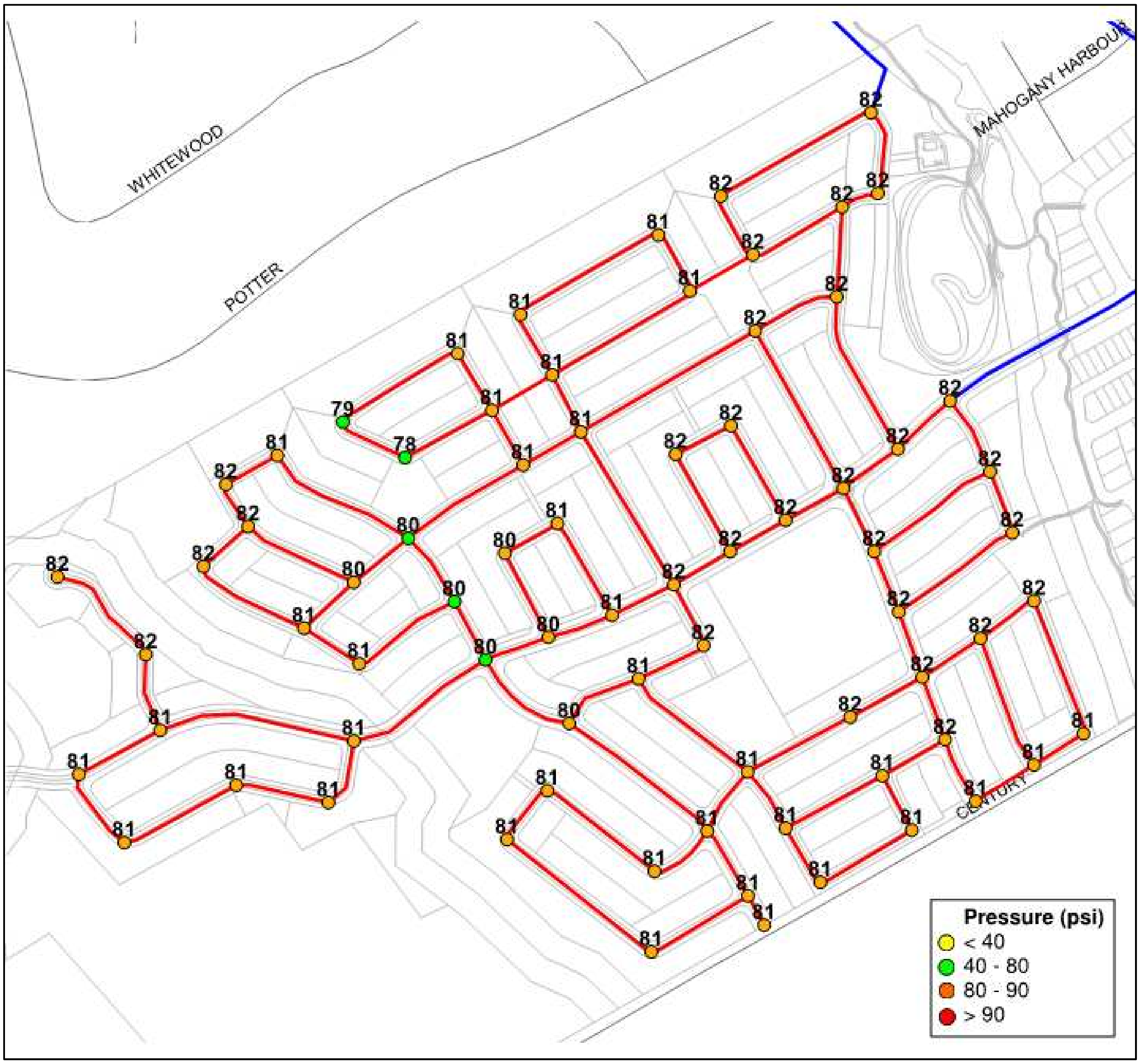


Figure 3-6: Phase 4 -Maximum Pressures During BSDY Conditions

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. Therefore, pressure reducing measures are required for services within this proposed development.

3.8.2 Peak Hour

Minimum pressures during PKHR are anticipated to be 425kPa (62 psi) and 423kPa (61 psi) for Phase 2 & 3 and Phase 4, respectively. These pressures are well above the minimum objective pressure of 276kPa (40 psi).

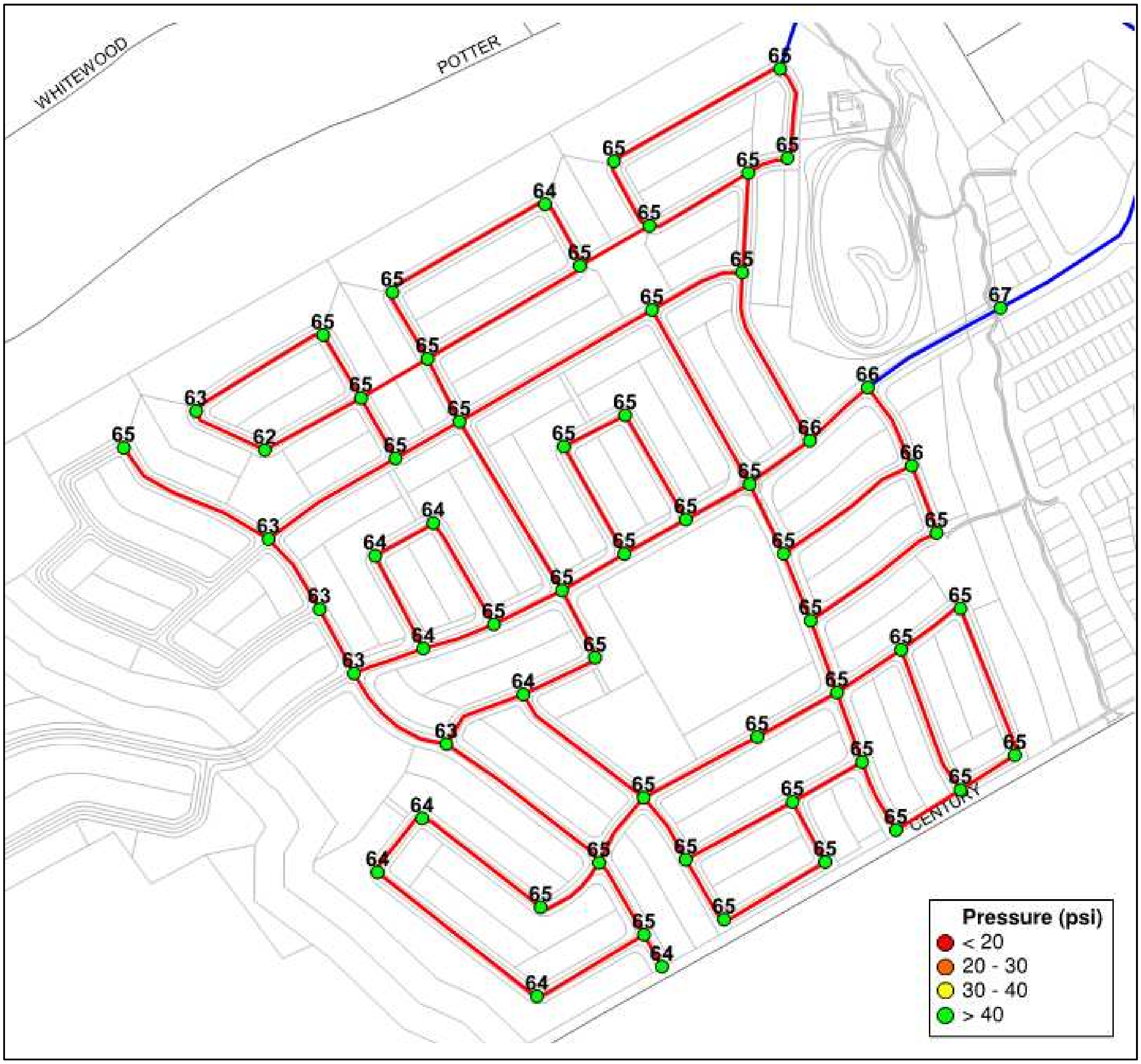


Figure 3-7: Phases 2 & 3 - Minimum Pressures During PKHR Conditions

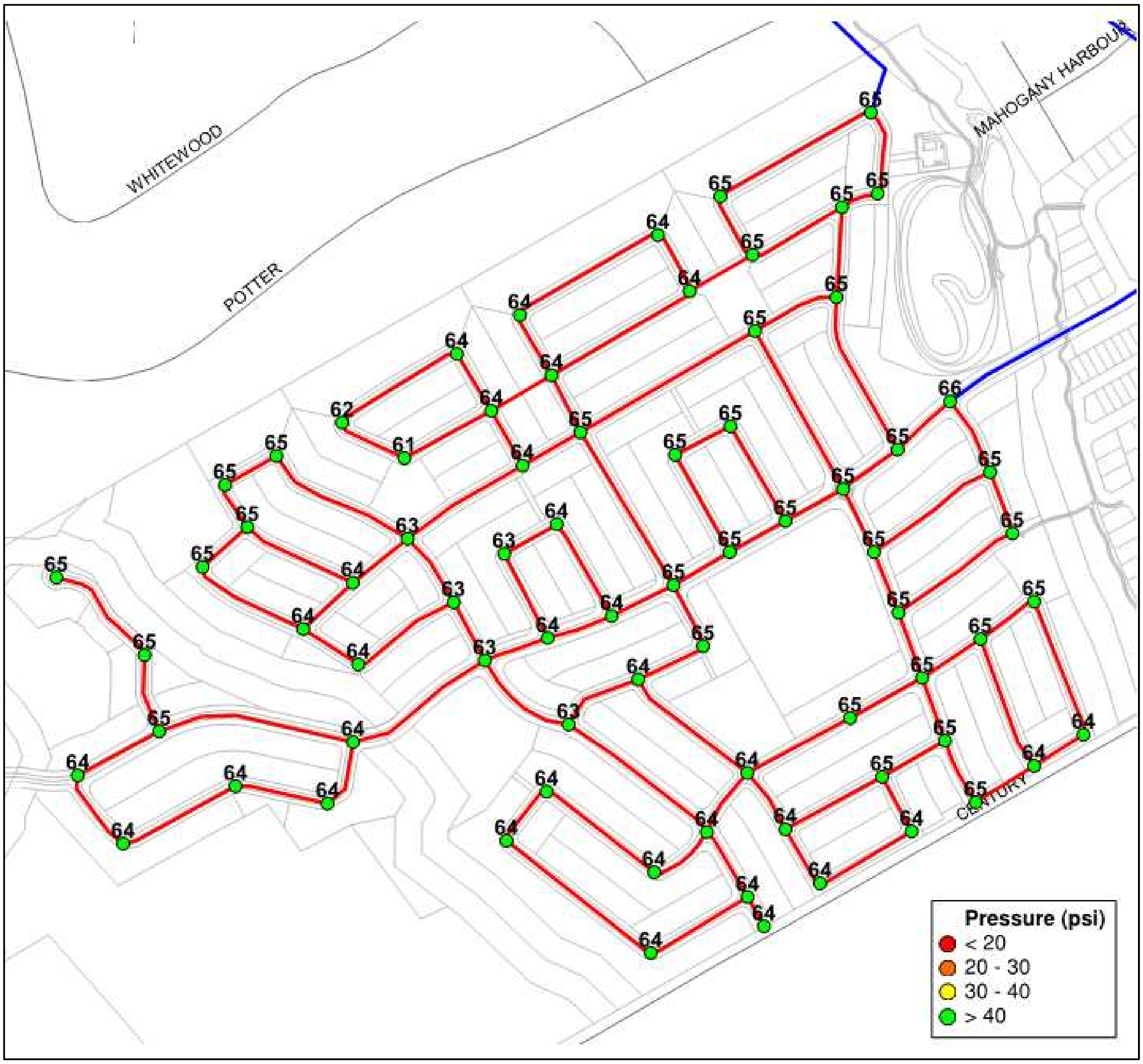


Figure 3-8: Phase 4 - Minimum Pressures During PKHR Conditions

3.8.3 Maximum Day Plus Fire Flow

An analysis was carried out using the hydraulic model to determine if the proposed Phase 2 & 3 as well as Phase 4 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**.

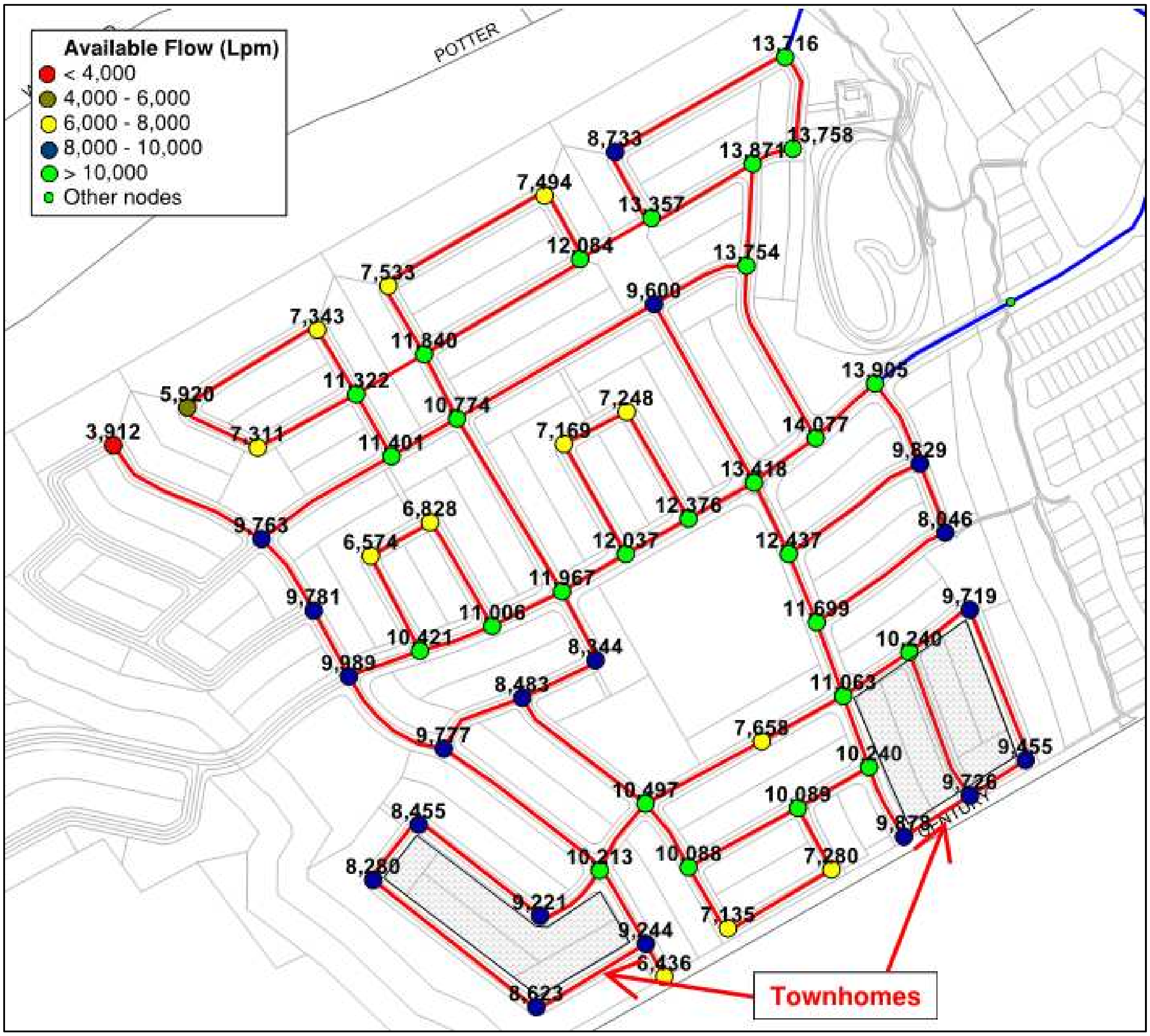


Figure 3-9: Phases 2 & 3 - Available Flow at 20 psi During MXDY+FF Conditions

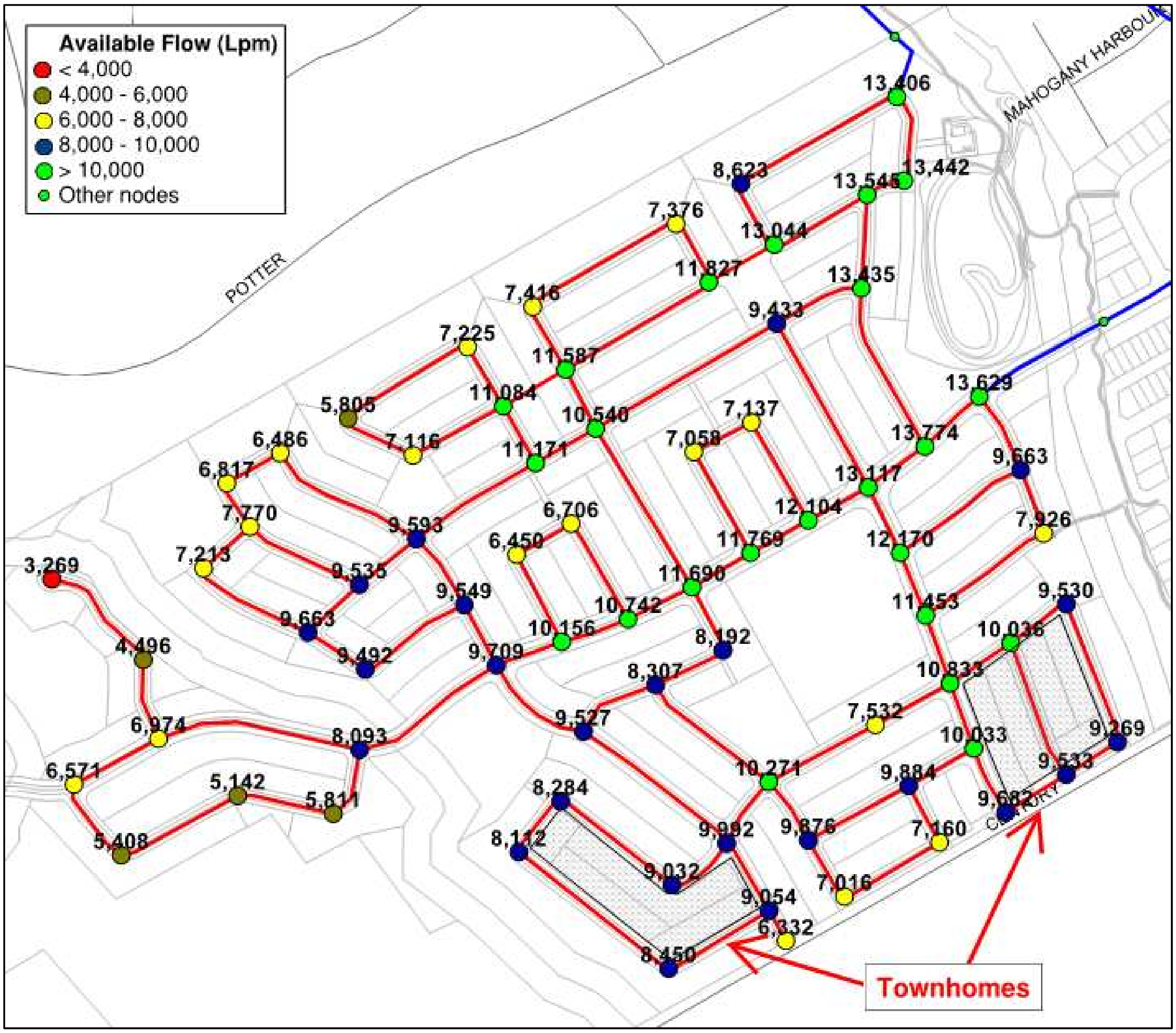


Figure 3-10: Phase 4 - Available Flow at 20 psi During MXDY+FF Conditions

It should be noted that the FUS formula rounds to the nearest thousand, therefore the values presented in **Figure 3-8** and **Figure 3-9** should be interpreted as such. Since Phase 4 is likely to be developed after the Manotick second watermain link, the available fire flows are anticipated to be higher than the values shown in **Figure 3-9**.

The majority of the development is planned for single family homes and modeling results show that these areas are anticipated to meet the required FUS fire flows which typically range from 5,000 to 7,500 L/min.

Two areas within Phase 2 and 3 are planned for townhomes and modeling results show available fire flows ranging from 8,000 to 11,000 L/min using the proposed watermain sizing. A fire flow of 10,000 L/min is not achieved at all locations within these two areas, however; it is understood that oversizing local watermains to achieve large fire flows should be avoided to prevent water quality issues.

Although the future Manotick links will provide an overall increase to available fire flows to the subdivision, mitigation measures to reduce the fire areas and lower the overall fire flow requirements may need to be considered under existing conditions once detailed information is available for Phases 2 and 3 in regards to the square footage and number of townhome rows.

Looping requirements within the development are achieved as there are two connections to existing watermains; however, until 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 – 0.28 L/s/ha
- Residential Average Flows – 350 L/cap/day
- Commercial/Mixed Use Flows – 50,000 L/ha/day
- Manhole Spacing – 120 m
- Minimum Cover – 2.5m

In addition, a residential peak factor based on Harmon's Equation was used to determine the peak design flows. Institutional and commercial areas were assigned a peaking factor of 1.5 per Ottawa's Sewer Design Guidelines.

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 205 singles, 54 townhomes and 2.84 ha of institutional development was applied for future phases of the Mahogany development (see drainage areas on **Drawing SAN-1**).

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 6** 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 7**. Although an increase in the previously determined peak inflow to the 450mm sewer has been noted, peak flows are within capacity of the downstream system per the sanitary sewer design sheet in **Appendix B.** Pump station upgrades will be required to suit the development as construction proceeds westwards, which are to be identified during detailed design.

Table 6: 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	205	54	843	25.63	2.84	28.47
Future Minto Development	-	-	545	-	-	4.28
Future Development (Other Owners)	-	-	819	-	-	16.38
Mahogany Phases 2-4	738	192	3028	88.66	2.90	91.56
Total			**6251**			**183.56**

Table 7: Calculated Peak Wastewater Flows vs. exp Mahogany Phase 1 Report

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)	6251	177.82	5.74	183.56	136.20	450
Difference					**4.13**	

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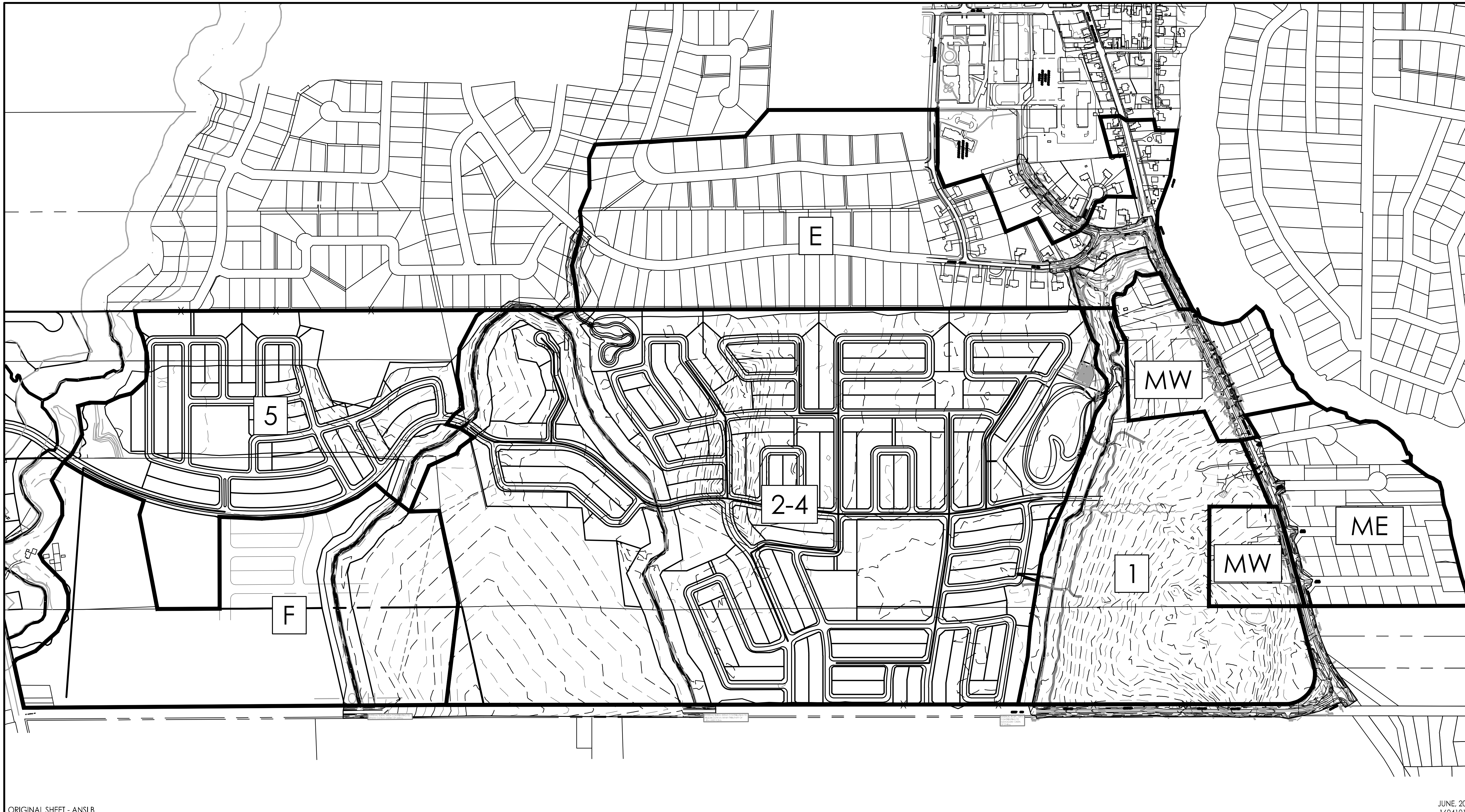

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DRAINAGE AREAS TO EASTMAN PS

ID	DESCRIPTION	AREA (ha) (RES)	AREA (ha) (INS)	UNITS	POPULATION
1	MAHOGANY PHASE 1	19.67	0.00	211	717
2-4	MAHOGANY PHASES 2-4	67.44	2.90	930	3028
5	MAHOGANY PHASE 5	25.63	2.84	259	843
F	FUTURE DEVELOPMENT	20.66	0.00	-	1364
MW	EXISTING (WEST OF MANOTICK MAIN)	10.70	0.00	18	61
ME	EXISTING (EAST OF MANOTICK MAIN)	12.50	0.00	70	238
E	EXISTING MANOTICK ESTATES	36.70	0.00	97	330

Client/Project
MINTO GROUP INC.
MAHOGANY SUBDIVISION
PHASES 2-4

Figure No.
4-1

Title
CONCEPTUAL SANITARY DRAINAGE

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.

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 (WC) Municipal Drain 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 WC Drain and its tributary directly.

5.2 BACKGROUND DOCUMENTS AND SWM CRITERIA

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 drain. The three storm sewer outlets have been constructed, and creek restoration works are scheduled to commence in August 2017.

5.3 EXISTING CONDITIONS AND TARGET PEAK OUTFLOWS

5.3.1 Wilson Cowan Drain Watershed Existing Conditions

Hydrologic analysis of the existing conditions for the Wilson Cowan Drain 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 hydrologic parameters and results of the analysis are presented in the tables below. The location of Flow Point B is shown on **Drawing ST-2**.

Table 5.1: Wilson Cowan Drain Watershed Existing Conditions Hydrologic Parameters

Watercourse	Drainage Area ID	Drainage Area (ha)	Time to Peak (hr)
Wilson Cowan Drain Tributary	3	23.0	0.67
	4	4.0	0.41
	5	7.9	0.30
	6	1.8	0.16
	7	9.2	0.58
Wilson Cowan Drain	8	34.8	0.85
	9	245.0	2.69

Table 5.2: Wilson Cowan Drain Existing Conditions Flow Rates at Point B

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

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 5.3: Mahogany Creek Watershed 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 5.4: Mahogany Creek Existing Conditions 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 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 tributary to SWM Pond 3 and include the Wilson Cowan (WC) Municipal Drain and its tributary from the southern edge of the property to Point B, as well as rural inflows as per IBI's 2007 existing condition model. (IBI)
- 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 2007/2012)
- The proposed conceptual SWM Pond 3 sizing and configuration is subject to future model calibration. (IBI)
- 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 2m 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)

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 or existing 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 WC Drain tributary, with the exception of area C203A, will be directed to the SWM Pond. Major system peak flows from the proposed development areas west of the WC drain tributary will be directed north along the proposed street and will discharge directly into the tributary. Major flows from the future development area east of the WC drain have been assumed to be discharged into the WC drain.

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

5.5 MODELING RATIONALE

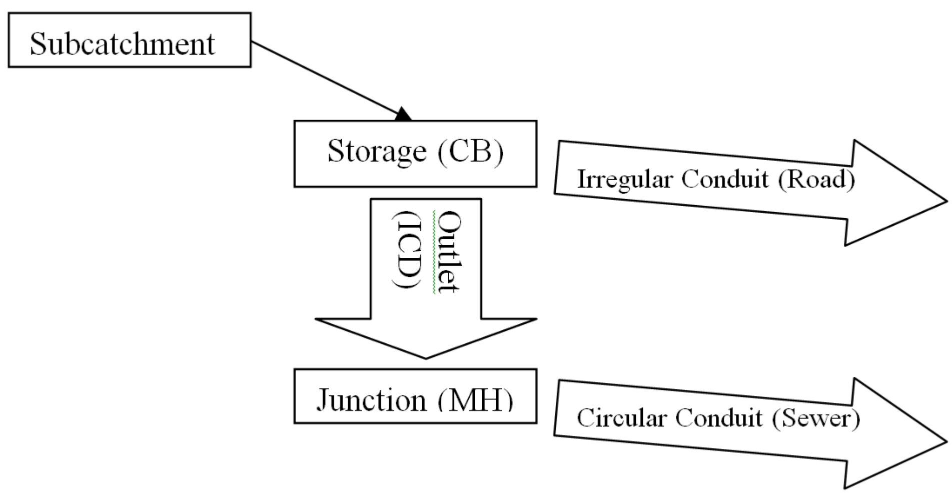
Two hydrologic/hydraulic models were completed with PCSWMM for each of the SWM facilities, accounting for the estimated major and minor systems to evaluate the storm sewer infrastructure and to size the blocks for the SWM Ponds in order to meet the target criteria. The use of PCSWMM for modeling of the 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 starting with an L) restricted to the 2-year runoff and for areas along collector roads (areas starting with a C) restricted to the 5-year runoff.
- Park areas assumed to store major system overflows up to the 100-year storm and to restrict minor system capture rates to the 2-year runoff.
- Future school block (area L111B) to store major system overflows up to the 100-year storm and to restrict minor system capture rates to the 2-year runoff.
- SWM Pond 2 areas L102A, L104A, L106A, L108A, L110A, L111A, L113A, L115A, L116A, L118A and L120A assumed to provide 20 m^3/ha of surface storage.
- Major system overflows from SWM Pond 2 areas directed to the SWM Pond.
- SWM Pond 3 areas assumed to have no surface storage and to discharge to the SWM Pond and/or to the WC Drain and/or its tributary as shown on **Drawing ST-2**.

5.5.1 SWMM Dual Drainage Methodology

The proposed conceptual development is modeled in one modeling program as a dual conduit system (see **Figure 5.1**), 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 5.1: 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 (1.8m below proposed road grade for areas with no storage and 2.15m 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 5.5**, were assigned a storage curve assuming a maximum storage of 20 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 0.35m (i.e. between a depth of 1.8m and 2.15m).

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 5.5**.

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.

5.6 PROPOSED CONDITION INPUT PARAMETERS

Drawings ST-1 and **ST-2** summarize the discretized subcatchments used in the conceptual analysis of the proposed Mahogany Stage 2+ 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 IBI's 2007 existing condition XP-SWMM model of the Wilson Cowan Drain and its tributary from Century Road to their confluence location at Point B (see **Drawing ST-2**). Runoff from rural areas 6, 7 and 9 enter the system at the upstream ends as per IBI's 2007 model and the parameters shown in **Table 5.1**.

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 drain, and for the major system outfalls from Phase 1 into the drain were obtained from IBI's 2012 XP-SWMM model.

5.6.1 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 Pond 2 development Areas in **Appendix C.4** and for Pond 3 development areas in **Appendix C.5**. 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 5.1.011.

Table 5.5 presents the general subcatchment parameters used for proposed development areas:

Table 5.5: 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
Dstore Perv (mm)	4.67
Zero Imperv (%)	0

Table 5.6 and **Table 5.7** present the individual parameters that vary for each of the proposed subcatchments in the SWM Pond 2 and SWM Pond 3 PCSWMM models respectively.

Table 5.6: SWM Pond 2 Individual Proposed Subcatchment Parameters

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient
C103A	0.82	200.0	1.0	57.1%	0.60
C105A	0.37	76.0	1.0	57.1%	0.60
C108A	0.48	228.0	0.5	57.1%	0.60
C109A	0.96	365.0	1.0	57.1%	0.60
C111A	0.60	222.0	0.5	57.1%	0.60
C116A	1.12	373.0	1.0	57.1%	0.60
C119A	0.66	95.0	1.0	57.1%	0.60
C120A	2.05	777.0	1.0	57.1%	0.60
L102A	2.65	1332.0	2.0	57.1%	0.60
L102B	0.44	99.0	2.0	14.3%	0.30
L103A	0.62	139.5	2.0	14.3%	0.30
L104A	6.58	2070.0	2.0	42.9%	0.50
L106A	2.64	800.0	1.0	50.0%	0.55
L106B	0.43	96.8	2.0	14.3%	0.30
L108A	3.02	1470.0	2.0	57.1%	0.60
L110A	4.08	1722.0	2.0	57.1%	0.60
L110B	0.84	189.0	2.0	14.3%	0.30
L111A	2.20	1126.0	2.0	57.1%	0.60
L111B	2.90	652.5	2.0	71.4%	0.70
L112A	1.13	400.0	1.0	50.0%	0.55
L113A	4.48	1550.0	2.0	42.9%	0.50
L114A	1.25	449.0	1.0	50.0%	0.55
L114B	0.40	90.0	2.0	14.3%	0.30
L115A	3.72	1330.0	2.0	42.9%	0.50
L116A	1.99	1010.0	2.0	57.1%	0.60

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient
L118A	2.03	856.0	1.0	57.1%	0.60
L120A	4.63	2029.0	2.0	57.1%	0.60
POND2	2.77	623.3	5.0	42.9%	0.50

Table 5.7: SWM Pond 3 Individual Proposed Subcatchment Parameters

Area ID	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient
C203A	0.60	115.0	1.0	57.1%	0.60
C204A	1.82	556.0	1.0	57.1%	0.60
L202A	3.34	1480.0	2.0	57.1%	0.60
L203A	3.38	1468.0	2.0	50.0%	0.55
L204B	6.61	1487.0	2.0	57.1%	0.60
L205A	1.58	436.0	1.0	50.0%	0.55
L206A	3.20	1542.0	2.0	50.0%	0.55
POND3	1.26	284.0	5.0	71.4%	0.70

Rural areas within the subject site tributary to the Wilson Cowan Municipal Drain (see areas WCD1 to WCD7 on **Drawing ST-2**) were modeled in SWMHYMO and their hydrographs were brought into the PCSWMM model. The SWMHYMO summary output file and detailed parameter calculations have been included in **Appendix C.3**. The main hydrologic parameters for the rural areas within the development are summarized below.

Table 5.8: SWM Pond 3 Hydrological Parameters – Rural Areas

Model Catchment ID	Area (ha)	Gradient (%)	Length (m)	Velocity (m/s)	Tp (hr)	Infiltration Method	CN
WCD2	14.66	1.5	410	0.09	0.85	SCS	35
WCD1	1.52	0.25	261	0.23	0.21	SCS	73
WCD5	2.89	0.20	453	0.20	0.42	SCS	76
WCD3	4.13	0.25	677	0.23	0.55	SCS	75
WCD4	1.51	1.62	122	0.10	0.23	SCS	73
WCD6	5.3	1.15	505	0.08	1.17	SCS	73
WCD7	2.23	3.19	213	0.14	0.28	SCS	73

Table 5.9 and

Table 5.10 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 20 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 20 m^3/ha surface storage is available between depths 1.8 and 2.15 m in the storage node.

Table 5.9: SWM Pond 2 Model Storage Node Parameters

Storage Node	Invert Elevation (m)	Rim Elevation (m)	Total Depth (m)	Max. Volume (m³)
103-S	87.98	90.13	2.15	0
C105A-S	88.05	90.20	2.15	0
C108A-S	88.29	90.44	2.15	0
C109A-S	88.37	90.52	2.15	0
C111A-S	88.27	90.42	2.15	0
C116A-S	89.70	91.85	2.15	0
C119A-S	89.45	91.60	2.15	0
C120A-S	89.45	91.60	2.15	0
L102A-S	87.64	90.14	2.50	53
L102B-S	88.14	90.64	2.50	115
L103A-S	89.35	89.95	0.60	0
L104A-S	87.85	90.35	2.50	127
L106A-S	87.91	90.42	2.50	53
L106B-S	88.36	90.86	2.50	110
L108A-S	88.02	90.52	2.50	60
L110A-S	88.10	90.60	2.50	82
L110B-S	88.47	90.97	2.50	198
L111A-S	87.98	90.48	2.50	43
L111B-S	88.92	91.42	2.50	424
L112A-S	88.49	90.64	2.15	0
L113A-S	88.21	90.71	2.50	90
L114A-S	88.56	90.71	2.15	0
L114B-S	88.94	91.44	2.50	102
L115A-S	88.29	90.79	2.50	74
L116A-S	88.05	90.55	2.50	40
L118A-S	88.17	90.67	2.50	41
L120A-S	88.33	90.83	2.50	93
103-S	87.98	90.14	2.15	0

Table 5.10: SWM Pond 3 Model Storage Node Parameters

Storage Node	Invert Elevation (m)	Rim Elevation (m)	Total Depth (m)	Max. Volume (m³)
C203A-S	88.97	91.12	2.15	0
C204A-S	88.65	90.80	2.15	0
L202A-S	89.62	91.77	2.15	0
L203A-S	88.82	90.97	2.15	0
L204B-S	88.85	91.00	2.15	0
L205A-S	88.20	90.35	2.15	0
L206A-S	88.34	90.49	2.15	0

5.6.2 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 5.11 and

Table 5.12 below present the parameters for the outlet link objects in the models, which represent minor system inlets.

Table 5.11: SWM Pond 2 Model Outlet Link Parameters

Outlet Name	Tributary Area ID	Inlet Storage Node	Minor System Node	100-year Capture (L/s)
C103A-IC	C103A	103-S	103	0.152
C105A-IC	C105A	C105A-S	105	0.072
C108A-IC	C108A	C108A-S	108	0.092
C109A-IC	C109A	C109A-S	109	0.181
C111A-IC	C111A	C111A-S	111	0.111
C116A-IC	C116A	C116A-S	116	0.210
C119A-IC	C119A	C119A-S	119	0.120
C120A-IC	C120A	C120A-S	120	0.391
L102A-IC	L102A	L102A-S	102	0.334
L102B-IC	L102B	L102B-S	102	0.003
L104A-IC	L104A	L104A-S	104	0.513
L106A-IC	L106A	L106A-S	106	0.283
L106B-IC	L106B	L106B-S	106	0.003
L108A-IC	L108A	L108A-S	108	0.373
L110A-IC	L110A	L110A-S	110	0.482
L110B-IC	L110B	L110B-S	110	0.010
L111A-IC	L111A	L111A-S	111	0.272
L111B-IC	L111B	L111B-S	111	0.442
L112A-IC	L112A	L112A-S	112	0.121
L113A-IC	L113A	L113A-S	113	0.352
L114A-IC	L114A	L114A-S	114	0.131
L114B-IC	L114B	L114B-S	114	0.003
L115A-IC	L115A	L115A-S	115	0.302
L116A-IC	L116A	L116A-S	116	0.252
L118A-IC	L118A	L118A-S	118	0.252
L120A-IC	L120A	L120A-S	120	0.552

Table 5.12: SWM Pond 3 Model Outlet Link Parameters

Outlet Name	Tributary Area ID	Inlet Storage Node	Minor System Node	100-year Capture (L/s)
C203A-IC	C203A	C203A-S	203	0.11
C204A-IC	C204A	C204A-S	204	0.34
L202A-IC	L202A	L202A-S	202	0.40
L203A-IC	L203A	L203A-S	203	0.34
L204B-IC	L204B	L204B-S	204B	0.69
L205A-IC	L205A	L205A-S	205	0.17
L206A-IC	L206A	L206A-S	206	0.33

5.6.3 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. The conceptual SWM Pond 2 and SWM Pond 3 (i.e. storage and outlet structures) were included in the models.

IBI's 2007 existing condition model for the Cowan Wilson Drain and its tributary was used along with the proposed development peak flows from the conceptual SWM Pond 3 to assess the peak flows in the drain at Flow Point B. A free outfall was assumed for the 25 mm, 2-year and 5-year, 24-hour SCS Type II simulations while a tidal curve with a maximum water elevation of 87.3 m was used for the 100-year, 24-hour SCS Type II simulation as per IBI's 2007 model.

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, but tidal curves as obtained from IBI's 2012 Ph1 models were used for the 2-year, 5-year and 100-year, 24-hour SCS Type II simulations, the 5-year and 100-year 3-hour Chicago simulations and for the July 1st, 1979 simulation.

5.7 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.4** and **Appendix C.5** for SWM Pond 2 and SWM Pond 3 respectively**,** and the electronic model files on the enclosed CD.

5.7.1 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 CW municipal drain tributary will be directed to the pond with the exception of overflows from area C203 A which will discharge directly into the tributary. Major flows from the proposed development areas west of the tributary will be directed to the WC drain at the northern end of the site, while overland flows from the future development area L204B will outlet directly into the WC Drain on the west end of the site as shown on **Drawing ST-2**.

Table 5.13 and **Table 5.14** present the conceptual total surface water depths (static ponding depth + dynamic flow) above the proposed road grades 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 5.13: Conceptual Dynamic Surface Water Depths – 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)
103-S	89.78	90.13	90.13	0.35	90.27	0.49
C105A-S	89.85	90.20	90.18	0.33	90.33	0.48
C108A-S	90.09	90.44	90.37	0.28	90.43	0.34
C109A-S	90.17	90.52	90.37	0.20	90.42	0.25
C111A-S	90.07	90.42	90.29	0.22	90.33	0.26
C116A-S	91.50	91.85	91.58	0.08	91.60	0.10
C119A-S	91.25	91.60	91.31	0.06	91.32	0.07
C120A-S	91.25	91.60	91.37	0.12	91.39	0.14
L102A-S	89.79	90.14	90.08	0.29	90.21	0.42
L102B-S	90.49	90.84	90.25	-0.24	90.30	-0.19
L103A-S	89.60	89.95	89.56	-0.04	89.60	0.00
L104A-S	90.00	90.35	90.38	0.38	90.48	0.48
L106A-S	90.06	90.41	90.41	0.35	90.54	0.48
L106B-S	90.71	91.06	90.45	-0.26	90.49	-0.22
L108A-S	90.17	90.52	90.36	0.19	90.42	0.25
L110A-S	90.25	90.60	90.43	0.18	90.46	0.21
L110B-S	90.82	91.17	90.62	-0.20	90.68	-0.14
L111A-S	90.13	90.48	90.29	0.16	90.34	0.21
L111B-S	91.07	91.42	91.06	-0.01	91.13	0.06
L112A-S	90.29	90.64	90.54	0.25	90.61	0.32
L113A-S	90.36	90.71	90.55	0.19	90.60	0.24
L114A-S	90.36	90.71	90.56	0.20	90.60	0.24
L114B-S	91.34	91.69	91.23	-0.11	91.34	0.00
L115A-S	90.44	90.79	90.61	0.17	90.64	0.20
L116A-S	90.20	90.55	90.34	0.14	90.35	0.15
L118A-S	90.32	90.67	90.42	0.10	90.43	0.11
L120A-S	90.48	90.83	90.69	0.21	90.71	0.23

Table 5.14: Conceptual Dynamic Surface Water Depths – 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)
C203A-S	90.77	91.12	90.84	0.07	90.85	0.08
C204A-S	90.45	90.80	90.57	0.12	90.59	0.14
L202A-S	91.42	91.77	91.60	0.18	91.62	0.20
L203A-S	90.62	90.97	90.79	0.17	90.82	0.20
L204B-S	91.00	91.35	91.17	0.17	91.19	0.19
L205A-S	N/A	90.35	90.17	-	90.16	-
L206A-S	90.14	90.49	90.31	0.17	90.33	0.19

5.7.2 Hydraulics

Table 5.15 and

Table 5.16 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 5.15: SWM Pond 2 - 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)
101	89.70	86.62	86.71	86.71	2.99	86.80	2.90
102	89.85	86.90	86.91	86.91	2.94	87.00	2.85
103	89.78	86.74	86.85	86.85	2.93	86.96	2.82
104	90.07	87.04	87.06	87.06	3.01	87.15	2.92
105	89.85	86.84	86.94	86.94	2.91	87.07	2.78
106	90.06	87.22	87.22	87.22	2.84	87.26	2.80
107	89.94	86.97	87.04	87.04	2.90	87.17	2.77
108	90.09	87.21	87.25	87.25	2.84	87.38	2.71

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)
109	90.17	87.31	87.33	87.33	2.84	87.44	2.73
110	90.37	87.50	87.51	87.51	2.86	87.57	2.80
111	90.07	87.08	87.16	87.16	2.91	87.29	2.78
112	90.29	87.21	87.28	87.28	3.01	87.38	2.91
113	90.43	87.53	87.53	87.53	2.90	87.54	2.89
114	90.36	87.38	87.40	87.40	2.96	87.45	2.91
115	90.51	87.61	87.61	87.61	2.90	87.61	2.90
116	90.97	88.25	88.25	88.25	2.72	88.25	2.72
117	90.15	87.17	87.23	87.23	2.92	87.34	2.81
118	91.08	87.38	87.40	87.40	3.68	87.45	3.63
119	91.25	87.45	87.46	87.46	3.79	87.49	3.76
120	90.40	87.76	87.76	87.76	2.64	87.77	2.63
101	89.70	86.62	86.71	86.71	2.99	86.80	2.90
102	89.85	86.90	86.91	86.91	2.94	87.00	2.85
103	89.78	86.74	86.85	86.85	2.93	86.96	2.82
104	90.07	87.04	87.06	87.06	3.01	87.15	2.92
105	89.85	86.84	86.94	86.94	2.91	87.07	2.78
106	90.06	87.22	87.22	87.22	2.84	87.26	2.80
107	89.94	86.97	87.04	87.04	2.90	87.17	2.77
108	90.09	87.21	87.25	87.25	2.84	87.38	2.71
109	90.17	87.31	87.33	87.33	2.84	87.44	2.73
110	90.37	87.50	87.51	87.51	2.86	87.57	2.80
111	90.07	87.08	87.16	87.16	2.91	87.29	2.78
112	90.29	87.21	87.28	87.28	3.01	87.38	2.91
113	90.43	87.53	87.53	87.53	2.90	87.54	2.89
114	90.36	87.38	87.40	87.40	2.96	87.45	2.91
115	90.51	87.61	87.61	87.61	2.90	87.61	2.90
116	90.97	88.25	88.25	88.25	2.72	88.25	2.72
117	90.15	87.17	87.23	87.23	2.92	87.34	2.81
118	91.08	87.38	87.40	87.40	3.68	87.45	3.63
119	91.25	87.45	87.46	87.46	3.79	87.49	3.76
120	90.40	87.76	87.76	87.76	2.64	87.77	2.63

Table 5.16: 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	90.03	87.58	87.76	87.76	2.27	87.67	2.36
202	90.69	87.71	87.91	87.91	2.78	87.82	2.87
203	90.37	87.77	87.99	87.99	2.38	87.90	2.47
204	90.23	87.79	88.01	88.01	2.22	87.92	2.31
204B	90.45	88.13	88.37	88.37	2.08	88.27	2.18
205	90.00	87.92	88.07	88.07	1.93	88.38	1.62
206	90.14	87.66	87.87	87.87	2.27	87.78	2.36

5.8 WATER QUANTITY ANALYSIS – RUNOFF VOLUME

As outlined in IBI's 2007 Mahogany Community SWM Servicing Report, BMP's to promote infiltration are required in the proposed development. 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, areas comprised of soils suitable for infiltration be considered for infiltration trenches.

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 55.9 ha consisting of low and medium density residential land use, as well as designated areas for parks and a school block.

SWM Pond 3 is located along the northern boundary of the site, adjacent to the confluence of the Wilson Cowan(WC) Municipal Drain and its tributary. The tributary drainage area for SWM Pond 3 is 21.8 ha consisting of low density residential land use, as well as designated areas for parks. A monitoring program for the WC Drain and its tributary is currently in place and as such, water quantity control requirements for the SWM facility are subject to model calibration during the detailed design stage.

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 5.17** and

Table 5.18 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.8**.

Table 5.17: 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
55.9	51	7,779	23,098	2,234	10,850

Table 5.18: 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
21.8	55	3,285	3,967	872	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 using a continuous shear stress modeling methodology to meet erosion control requirements.

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 potential 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 are currently underway 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 0.62 m/s velocity obtained with the proposed SWM Pond 2 configuration during the same storm event.

A flow monitoring program is currently in place for the Wilson Cowan Municipal Drain and its tributary to calibrate the existing conditions model and to confirm the water quantity control requirements for the proposed SWM Pond 3. In addition, a geomorphology study was completed for the WC Drain (Matrix Solutions Inc., May 2017) to confirm erosion control requirements/improvements for the drain. The conceptual SWM Block has been sized with a level of conservatism to account for future changes. However, the actual SWM block size and criteria for the SWM facility should be confirmed during the detailed design stage once adequate information from the flow monitoring program is available.

5.9.1.3 Water Quantity Control

In addition to providing water quality control, it is also required that the facilities match post-to-pre development peak flow rates for events up to the 100-year event at Point A in the Mahogany Creek and at Point B in the WC Drain. Target rates for Point A and Point B were determined in the background documents as described in **Section 5.3**. Detailed calculations for the conceptual SWM Ponds are included in **Appendix C.8**.

The following tables show the post-to-pre development peak flow rate comparison for the receiving watercourses.

Table 5.19: 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	0.75	1.66	2.50	7.15

Table 5.20: Wilson Cowan Drain Flow Rate Comparison at Point B

	Flow (m^3/s)			
	25 mm	**2-year**	**5-year**	**100-year**
Pre-Development	0.82	1.88	2.90	6.37
Post Development	0.79	1.81	2.79	6.18

The above tables show that the proposed conceptual SWM Ponds and outlet configurations meet the quantity control criteria for the receiving watercourses.

5.9.2 Conceptual Stormwater Management Facility Design Components

It is recommended that the 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 0.70 m of active storage is controlled by a 300 mm diameter orifice. The secondary pond outlet occurs via a 7m-long weir with an invert at 86.20 m. An anticipated cross section of the conceptual SWM Pond 2 is provided in **Appendix C.6**. Detailed calculations for the conceptual SWM Ponds are included in **Appendix C.8**.

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 weir for quantity control. The first 0.62 m of active storage is controlled by a 130 mm diameter orifice. The secondary pond outlet occurs via a 3m-long weir with an invert at 87.08 m. An anticipated cross section of the conceptual SWM Pond 3 is provided in **Appendix C.7**. Detailed calculations for the conceptual SWM Ponds are included in **Appendix C.8**.

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 5.21** and

Table 5.22 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 5.21: SWM Pond 2 Peak Pond Outflow Rates and HGL

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	17.56	2.95	86.58	40,978

Pond Design Storms	25mm 4hr Chicago	3.14	0.09	85.79	27,593
	2yr24hrSCS	4.58	0.11	86.06	31,778
	5yr24hrSCS	6.58	0.32	86.27	35,243
	100yr24hrSCS	14.00	3.73	86.65	42,273
Climate Change Storms	100yr3hrChicago_20%	20.63	4.66	86.73	43,813
Historical Storms	July 1st, 1979	12.55	5.04	86.75	44,223

Table 5.22: SWM Pond 3 Peak Pond Outflow Rates and HGL

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	3.87	1.78	87.57	7,678
Pond Design Storms	25mm 4hr Chicago	1.56	0.03	86.96	5,436
	2yr24hrSCS				
	5yr24hrSCS				
	100yr24hrSCS	3.37	2.64	87.74	8,583
Climate Change Storms	100yr3hrChicago_20%				
Historical Storms	July 1st, 1979				

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.

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 and 3. 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 above the maximum pressure objective of 552kPa (80 psi) and will 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;
  - The future Manotick links will improve the overall 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 under existing conditions for Phase 2 and 3.
- Looping requirements within the development are achieved, however; the Rideau Valley watermain is currently the only feed to Manotick and until 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 136.2L/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.
- Subject to model calibration, the conceptual SWM Pond 3 provides sufficient storage to meet the target peak outflows at control Point B in the Wilson Cowan Municipal Drain.
- Two storm crossings of the WC drain tributary will be required to service the development area tributary to SWM Pond 3.
- 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.
- Park and school areas 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 CW municipal drain tributary will be directed to the pond with the exception of overflows from area C203 A which will discharge directly into the tributary. Major flows from the proposed development areas west of the tributary will be directed to the WC drain at the northern end of the site, while overland flows from the future development area L204B will outlet directly into the WC Drain on the west end of the site.

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

- 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.
- During the detailed design stage of the development, areas comprised of soils suitable for infiltration be considered for infiltration trenches.
- The water quantity control requirements for SWM Pond 3 be confirmed once adequate information from the flow monitoring program is available to calibrate the existing conditions model.
- A geomorphology study be done for the WC Drain to confirm erosion control requirements/improvements for the drain.
- The Engineer's Report for the Wilson Cowan Municipal Drain be updated to ensure the proposed development will not negatively impact the existing properties and structures downstream of the site.

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.