MCNEELY LANDING (FORMERLY RSSR AND LAING LANDS)

CONCEPTUAL SITE SERVICING AND STORMWATER MANAGEMENT REPORT

Prepared for:

UNIFORM URBAN DEVELOPMENTS LTD.
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Prepared By:

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June 10, 2021
(May 13, 2022 Rev. 1)

Novatech File: 119221
Ref: R-2021-085

Engineers, Planners & Landscape Architects

May 13, 2022

Lanark County
99 Christie Lake Road
Perth, ON K7H 3C6

Town of Carleton Place
175 Bridge Street
Carleton Place, ON K7C 1T8

Attention: **Julie Stewart, MCIP, RPP**
Planner (County of Lanark)

Niki Dwyer, MCIP, RPP
Director of Development Services (Town of Carleton Place)

Reference: **McNeely Landing (Formerly RSSR and Laing Lands)**
Conceptual Site Servicing and Stormwater Management Report
Novatech File No.: 119221

In support of the Draft Plan of Subdivision application for the above-noted site, you will find enclosed the Conceptual Site Servicing and Stormwater Management Report for the McNeely Landing (Formerly RSSR and Laing Lands) development.

This report addresses the approach to site servicing and stormwater management for the Subject Site, which been developed based on the requirements of the Town of Carleton Place and Mississippi Valley Conservation Authority.

Should you have any questions, or require additional information, please contact me.

Yours truly,

NOVATECH

Bassam Bahia, M.Eng., P. Eng.
Senior Project Manager | Land Development

/bs

cc: Steve Pentz / Jordan Jackson, Novatech
Annibale Ferro / Ryan MacDougall, Uniform Urban Developments Ltd.
Diane Reid, Mississippi Valley Conservation Authority

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1.0 INTRODUCTION

1.1 Background

This report addresses the approach to site servicing for the McNeely Landing development (Subject Site), formerly known as RSSR and Laing Lands, which is being proposed by Uniform Urban Developments Ltd. (Developer).

The Subject Site is located at the south-west corner of the McNeely Avenue and Captain A. Roy Brown Boulevard intersection, as shown on **Figure 1.1** – Site Plan. The site is bound to the north by the future Captain A. Roy Brown Boulevard extension, to the east by McNeely Avenue, to the south by open space / agricultural lands, and to the west by Highway 15.

The existing land usage is currently undeveloped consisting of grass, brush, trees, and agricultural lands as shown on **Figure 1.2** – Existing Conditions Plan. The grade of the Subject Site generally slopes from west to east towards the McNeely Avenue and Captain A. Roy Brown Boulevard intersection with a grade difference of 7.0 metres.

The existing residential subdivision to the west, Miller's Crossing is currently serviced with public services (i.e. sanitary and storm sewers, and watermain).

1.2 Development Intent

The Subject Site has an area of 25.20 ha, and the proposed subdivision will comprise of residential housing, local roads, pathways, a road widening block (along Highway 15), an institutional block, a stormwater facility, and parkland, as shown in **Table 1.1**. The development will contain municipal road allowances of 18.0 metres wide.

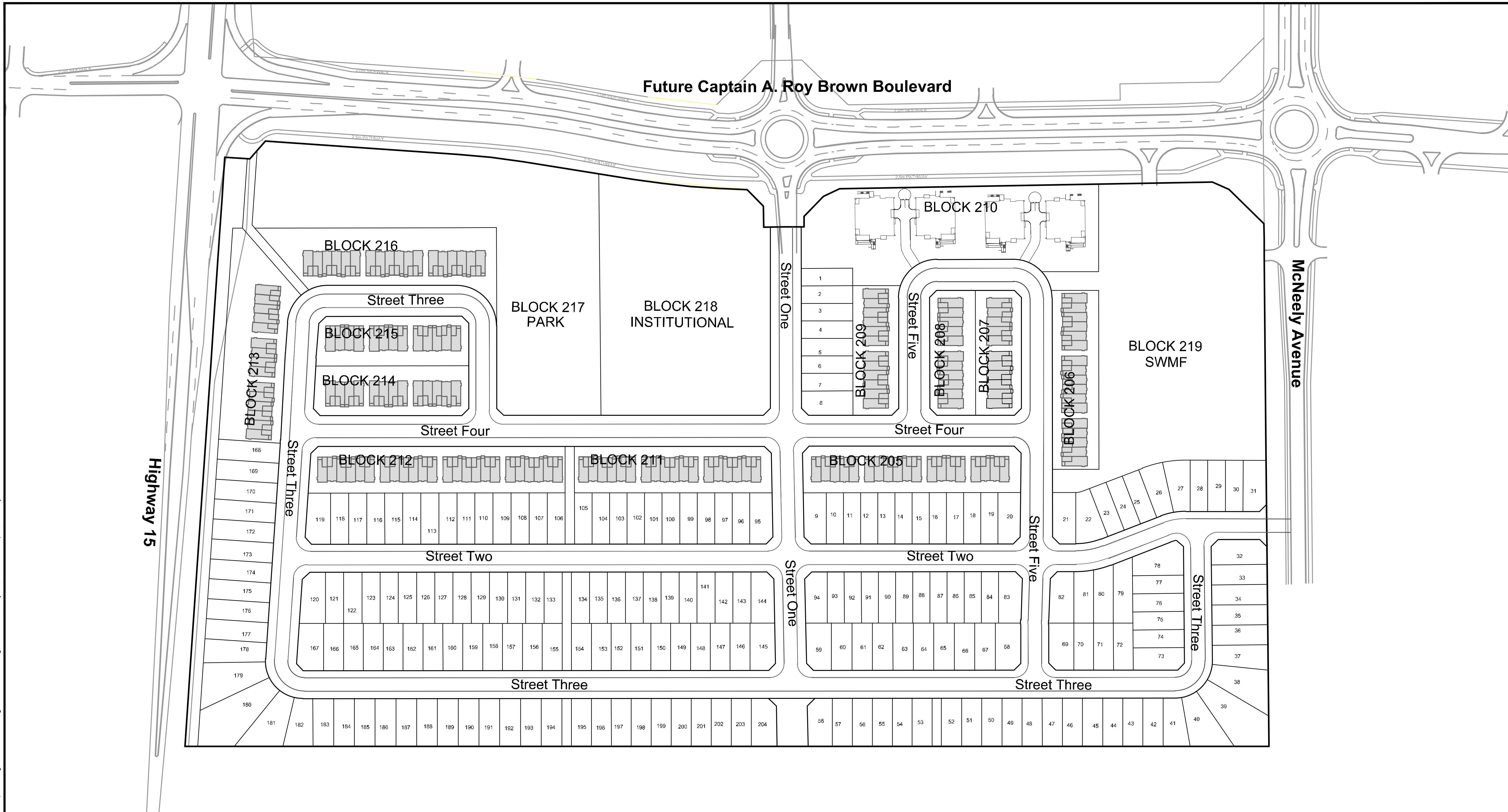
Table 1.1: Land Use, Development Potential, and Yield

Unit Type	Number of Units	Area
Singles	204	8.72 ha
Townhouses	171	4.02 ha
High Density	56	1.00 ha
Local Roads / Pathways	-	5.63 ha
Road Widening (Highway 15)	-	0.34 ha
Institutional	-	1.62 ha
Stormwater Facility	-	2.00 ha
Parkland	-	1.87 ha
TOTAL	**431**	**25.20 ha**

The Subject Site is located within the serviced area in the Town of Carleton Place Official Plan and was included in the Highway 7 South, Town of Carleton Place, Conceptual Design Plan (CDP) and Master Servicing and Stormwater Management Report (MSSMR); therefore, the site has been designed with municipal water and sanitary sewage collection.

1.3 Report Objective

This report assesses the adequacy of existing and proposed services to support the proposed development. This report will be provided to the various agencies for approval and to obtain any applicable permits.



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McNEELY LANDING

SITE PLAN

SCALE		
NOT TO SCALE		
DATE: MAY 2022	JOB: 119221	FIGURE: 1.1

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McNEELY LANDING

EXISTING CONDITIONS PLAN

SCALE NOT TO SCALE

DATE MAY 2022 | JOB 119221 | FIGURE 1.2

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The County of Lanark's Applicant Study and Plan Identification List along with proof of a pre-consultation meeting is provided in **Appendix A**.

A Servicing Study Guidelines for Development Applications checklist has been completed and is provided in **Appendix B**.

2.0 REFERENCES AND SUPPORTING DOCUMENTS

2.1 Guidelines and Supporting Studies

The following guidelines and supporting documents were utilized in the preparation of this report:

- **Highway 7 South, Town of Carleton Place, Conceptual Design Plan** (CDP)
  Novatech, August 2013.
- **Highway 7 South, Town of Carleton Place, Master Servicing and Stormwater Management Report** (MSSMR)
  Novatech, July 2013.
- **Town of Carleton Place Southeast Carleton Place Sanitary Pumping Station & Twin Forcemain/Sanitary Sewer, Phase 1, Design Brief and Detailed Engineering Drawings [Contract No. PW4-2015]**
  Ainley & Associated Ltd., July 2015 and December 2015.
- **McNeely Avenue Extension & Captain A. Roy Brown Boulevard Construction, Detailed Engineering Drawings [Contract No. PW1-2016]**
  BT Engineering, September 2016.
- **Design Brief for Cardel Homes, Miller's Crossing Subdivision** (Miller's Crossing DB)
  DSEL, April 2016.
- **City of Ottawa Water Distribution Guidelines** (OWDG)
  City of Ottawa, October 2012.
- **Revisions to OWDG** (ISTB-2010-01, ISTB-2014-02, ISTB-2018-02, ISTB-2018-04, ISTB-2021-03)
  City of Ottawa, December 2010, May 2014, March 2018, June 2018, and August 2021.
- **City of Ottawa Sewer Design Guidelines** (OSDG)
  City of Ottawa, October 2012.
- **Revisions to OSDG** (ISTB-2016-01, ISTB-2018-01, & ISTB-2018-03)
  City of Ottawa, September 2016, and March 2018.
- **Design Guidelines for Sewage Works and Drinking Water System** (MECP Guidelines)
  Ontario's Ministry of the Environment, 2008.
- **Stormwater Management Planning and Design Manual** (MECP SWM Guidelines)
  Ontario's Ministry of the Environment, 2003.

2.2 Geotechnical Investigation

Paterson Group Inc. (Paterson) conducted a geotechnical investigation (**Appendix F**) in support of the proposed residential development:

Geotechnical Investigation – Proposed Residential Development Highway 7 at Highway 15, Carleton Place, Ottawa, Ontario; Report No. PG5212-1, Paterson Group Inc., May 1, 2021.

Based on the geotechnical study, it is not anticipated that there will be any significant geotechnical concerns with respect to servicing and developing the site. Although, further review will be required at the north-east quadrant of the site, where there are some grade raise restrictions and additional recommendations would be required for the stormwater management facility. The test hole locations are provided as **Figure 2.1**. A summary of the geotechnical report findings is provided in **Table 2.1** below.

Table 2.1: Summary of Geotechnical Servicing and Grading Considerations

Parameter	Summary
Sub-Soil Conditions	Silty Clay / Glacier Till / Bedrock
Grade Raise Restriction	Up to 2.0m (area specific / NE Quadrant, refer to geotechnical report)
OHSA Soil Type	Type 2 and 3
Groundwater Considerations	Low to Moderate groundwater flow
Pipe Bedding / Backfill	Pipe Bedding 150 mm to 300mm Granular A Pipe Cover 300 mm Granular A Backfill Native Material
Pavement Structure (Driveways)	50mm Wear Course (SuperPave 12.5) 150mm Base (Granular A) 300mm Subbase (Granular B Type II)
Pavement Structure (Local Roadways)	40mm Wear Course (SuperPave 12.5) 50mm Binder Course (SuperPave 19.0) 150mm Base (Granular A) 400mm Subbase (Granular B Type II)
Pavement Structure (Collector Roads)	40mm Wear Course (SuperPave 12.5) 50mm Upper Binder Course (SuperPave 19.0) 50mm Lower Binder Course (SuperPave 19.0) 150mm Base (Granular A) 550mm Subbase (Granular B Type II)
SWMF Consideration	TBD as part of the detailed design stage. A liner may be required where bedrock excavation is required within the permanent pool.
Landscape Consideration	TBD as part of the detailed design stage

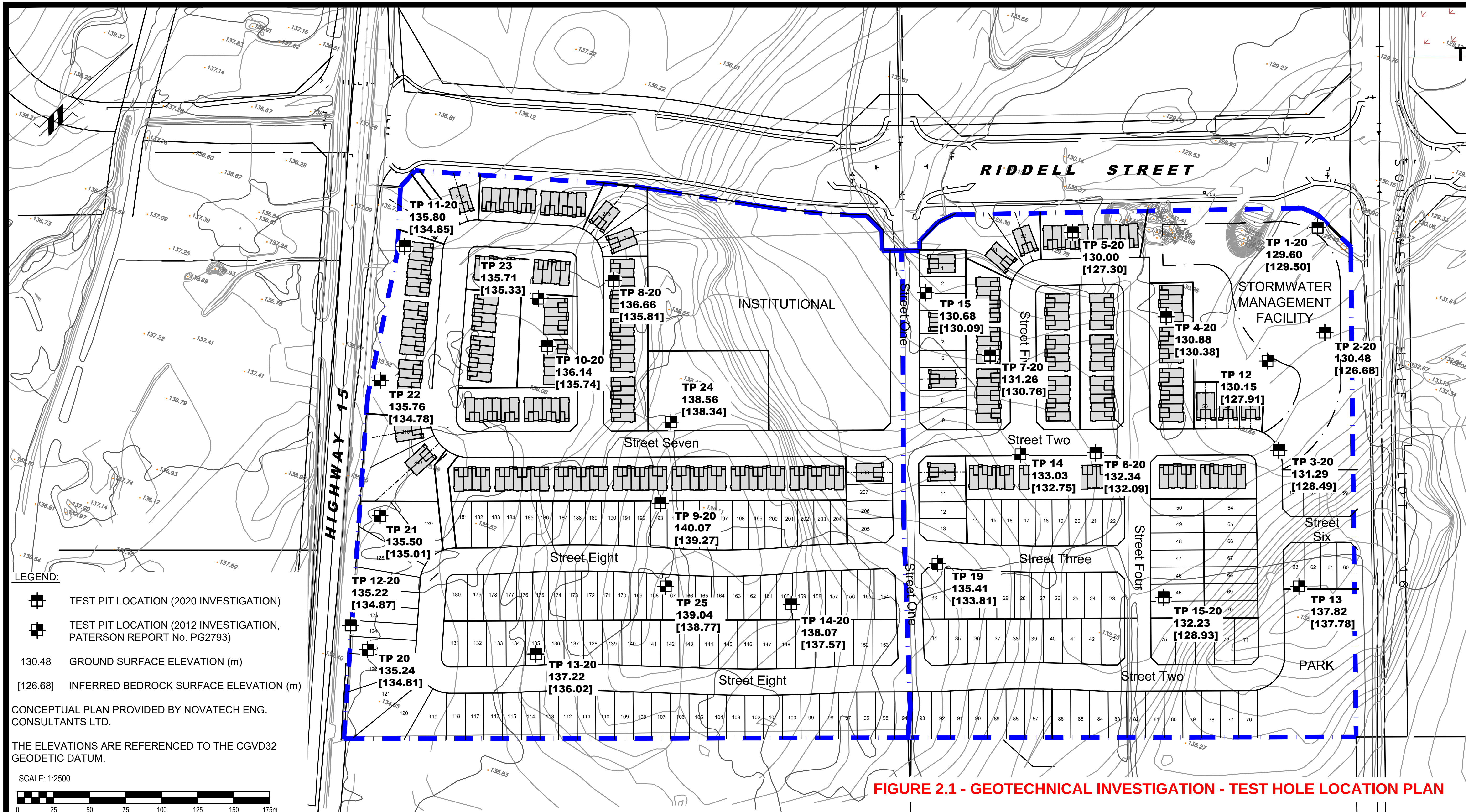

FIGURE 2.1 - GEOTECHNICAL INVESTIGATION - TEST HOLE LOCATION PLAN

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO NEW CONCEPTUAL PLAN	05/19/2021	FAS

REGIONAL GROUP c/o NOVATECH ENGINEERING CONSULTANTS LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - HIGHWAY 7
CARLETON PLACE, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:2500	Date: 01/2020
Drawn by: NFRV	Report No.: PG5212-1
Checked by: DP	Dwg. No.: PG5212-1
Approved by: FAS	Revision No.: 1

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3.0 SERVICING AND GRADING

3.1 Servicing Connections

Sanitary servicing for the Subject Site will connect to the existing sanitary sewer stub located at the McNeely Avenue and Captain A. Roy Brown Boulevard intersection.

Storm servicing for the Subject Site will outlet into the proposed stormwater management (SWM) facility, discharging into the re-aligned Beckwith Drain along Captain A. Roy Brown Boulevard.

Water service for the Subject Site will connect to the existing watermain stub located at the McNeely Avenue and Captain A. Roy Brown Boulevard intersection, and the existing watermain stub located at the McNeely Avenue and Flegg Way intersection.

3.2 General Servicing

The Subject Site will be serviced using local storm and sanitary sewers, and watermain. The storm / stormwater management, sanitary, and water servicing strategy is discussed in further detail in the following sections.

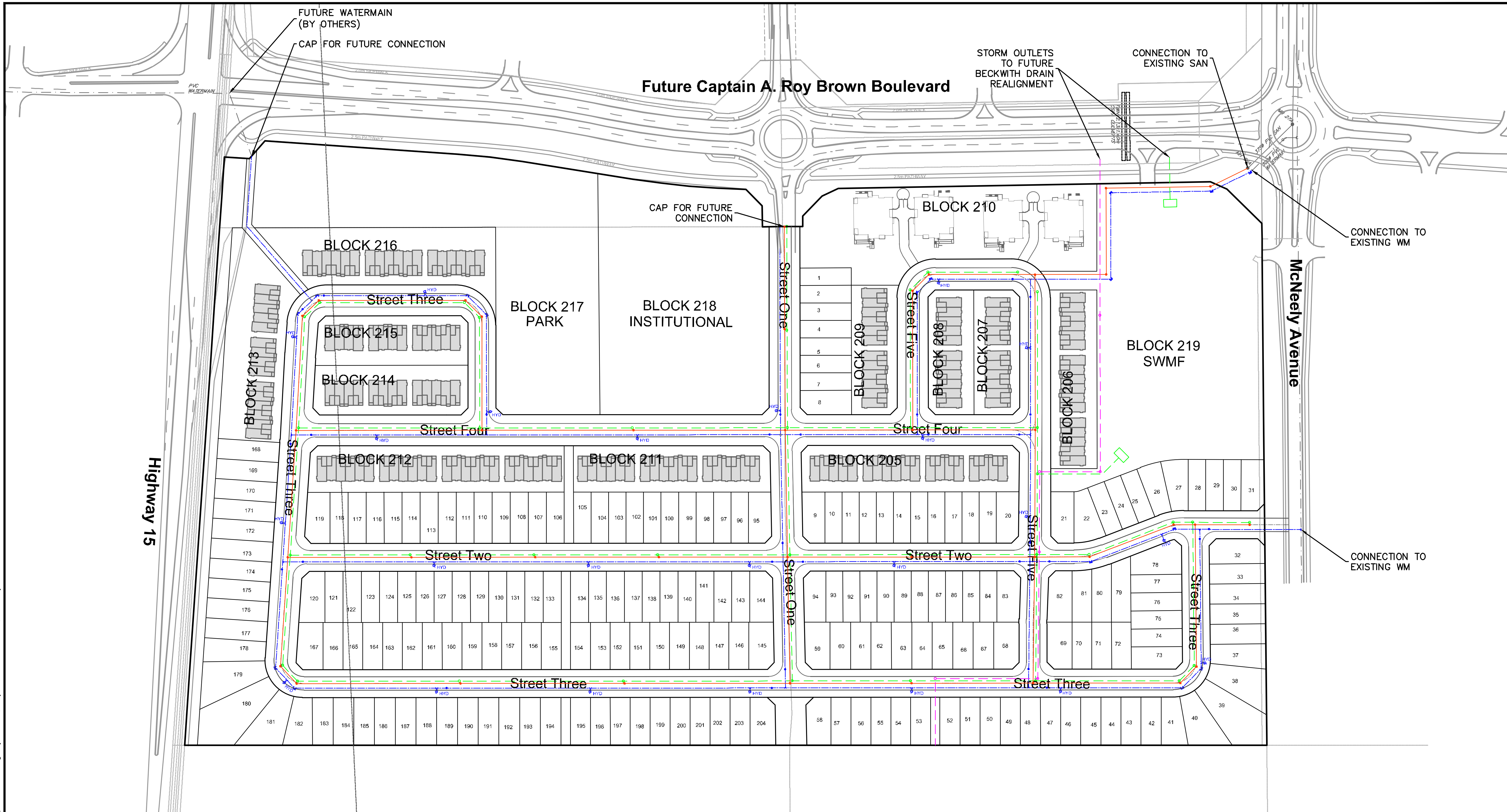
Refer to **Figure 3.1** – Proposed Conceptual Servicing Layout Plan.

3.3 General Grading

The local roadway within the Subject Site will be graded in a saw-toothed pattern to promote surface storage of stormwater. The grading will direct major overland flows from the local roads to the proposed SWM facility, except for a section of Street One where the flows will outlet to Captain A. Roy Brown Boulevard due to grading constraints.

The lots will be graded from front to back to direct surface drainage to the rearyard areas.

Refer to the **Figure 3.2** – Proposed Conceptual Grading Plan.



LEGEND

PROPOSED SANITARY SEWER
PROPOSED STORM SEWER
PROPOSED BYPASS STORM SEWER
PROPOSED WATERMAIN
HYD FIRE HYDRANT
EXISTING SANITARY SEWER
EXISTING WATERMAIN

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PROPOSED CONCEPTUAL SERVICING LAYOUT PLAN

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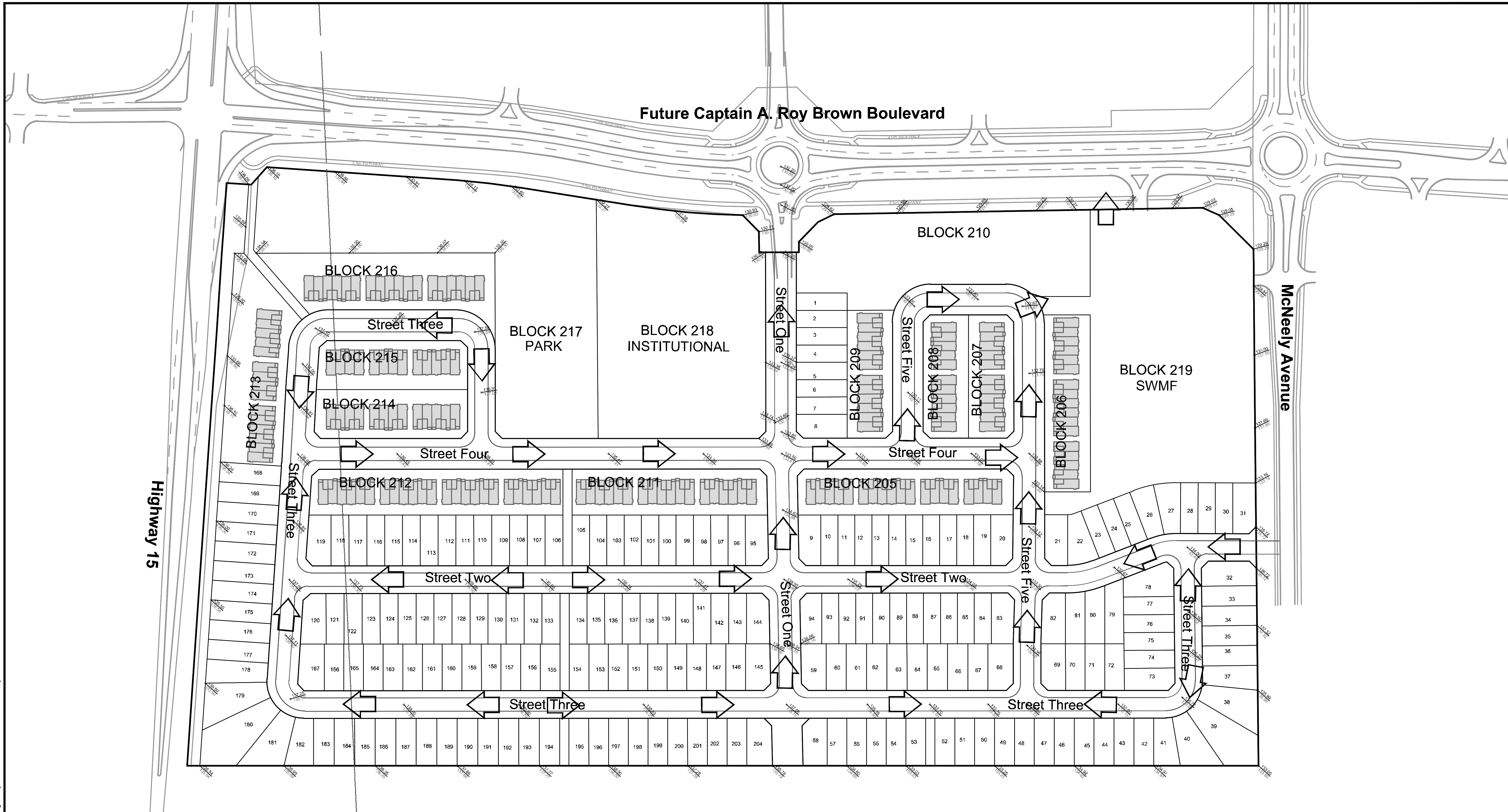
FIGURE 3.1



LEGEND

133.92 PROPOSED FINISHED GRADE
136.93 ORIGINAL GRADE

EMERGENCY OVERLAND FLOW

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PROPOSED CONCEPTUAL GRADING PLAN

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4.0 STORM SEWER SYSTEM AND STORMWATER MANAGEMENT

The post-development storm sewer and stormwater management system will adhere to the criteria outlined as a part of the MSSMR. Storm runoff from the Subject Site will outlet to the SWM facility at the north-east quadrant of the site. The following sections outline the preliminary stormwater management design and analysis.

4.1 Stormwater Management Criteria

The Subject Site is located within the Mississippi River Subwatershed, which is tributary to the Ottawa River, which falls under the jurisdiction of the Mississippi Valley Conservation Authority (MVCA). The Subject Site discharges into the future re-aligned Beckwith Drain, which outlets into Lavelle Creek, and ultimately into the Mississippi River.

The following SWM criteria have been developed based on the criteria in the MSSMR, and requirements of the MECP SWM Guidelines, MVCA and the OSDG.

<u>*Minor System (Storm Sewers)*</u>

- Storm sewers are to be designed using the Rational Method as follows:
  - 1:5-year return period for local and collector roads.
- Inlet control devices (ICDs) are to be installed in road and rearyard catchbasins to control inflows to the storm sewers;
- Ensure that the 100-year hydraulic grade line in the storm sewer is at least 0.3 m below the underside of footing (USF) elevations for the proposed development; or 0.3 m below sump pump goosenecks, where sump pumps are required.

<u>*Major System (Overland Flow)*</u>

- Overland flows are to be confined within the right-of-ways and/ or a defined drainage easements for all storms up to and including the 1:100-year event;
- Storm runoff that exceeds the capacity of the minor system will be stored within road sags;
  - Runoff that exceeds the capacity of the road sags will be conveyed overland along defined major system flow routes towards the proposed major system outlet to the SWM facility;
  - Runoff from a section of Street One (between Street Four and Captain A. Roy Brown Boulevard) will outlet to Captain A. Roy Brown Boulevard due to grading constraints. For this specific drainage area, an increased inlet capture rate may be contemplated as part of the detailed design to capture flows greater than the 1:5-year return period.
- Major system storage in backyards is not to be included/ accounted for in design computations;
- Maximum depth of flow (static + dynamic) on local and collector streets shall not exceed 0.35 m and shall be confined to the road right-of-way, as well as not touch any part of the building envelope and must remain below the lowest building opening during the stress test event;
- The product of the 100-year flow depth (m) on street and flow velocity (m/s) shall not exceed 0.60.

Water Quality & Quantity Control

- An *Enhanced* (80% TSS removal) level of quality control will be provided by the proposed SWM facility, which outlets to the re-aligned Beckwith Drain along Captain A. Roy Brown Boulevard;
- Quantity control is to be provided to control post-development peak flows to pre-development levels;
- Implement lot level and conveyance Best Management Practices to promote infiltration and treatment of storm runoff;
- Inflows to the storm sewer are to be controlled by inlet control devices installed in all catchbasins to limit inflows during larger storm events.

4.2 Proposed Storm Drainage System

Storm servicing for the Subject Site will be provided using a dual drainage system: Runoff from frequent events will be conveyed by storm sewers (minor system), while runoff from larger storm events which exceed the capacity of the minor system will be conveyed overland along defined overland flow routes (major system). The proposed SWM facility is the outlet for both the major and minor systems, except for a section of Street One where the major system flows will outlet to Captain A. Roy Brown Boulevard as mentioned above.

4.2.1 *Storm Sewer Design (Minor System)*

The proposed on-site works will require approximately 3,000 m of on-site storm sewer to collect stormwater flows and to direct flows to the proposed SWM facility.

Refer to **Figure 3.1** – Proposed Conceptual Servicing Layout Plan for an illustration of the proposed SWM facility and layout details.

Refer to **Figure 4.1** – Storm Drainage Area Plan for an illustration of the proposed drainage areas.

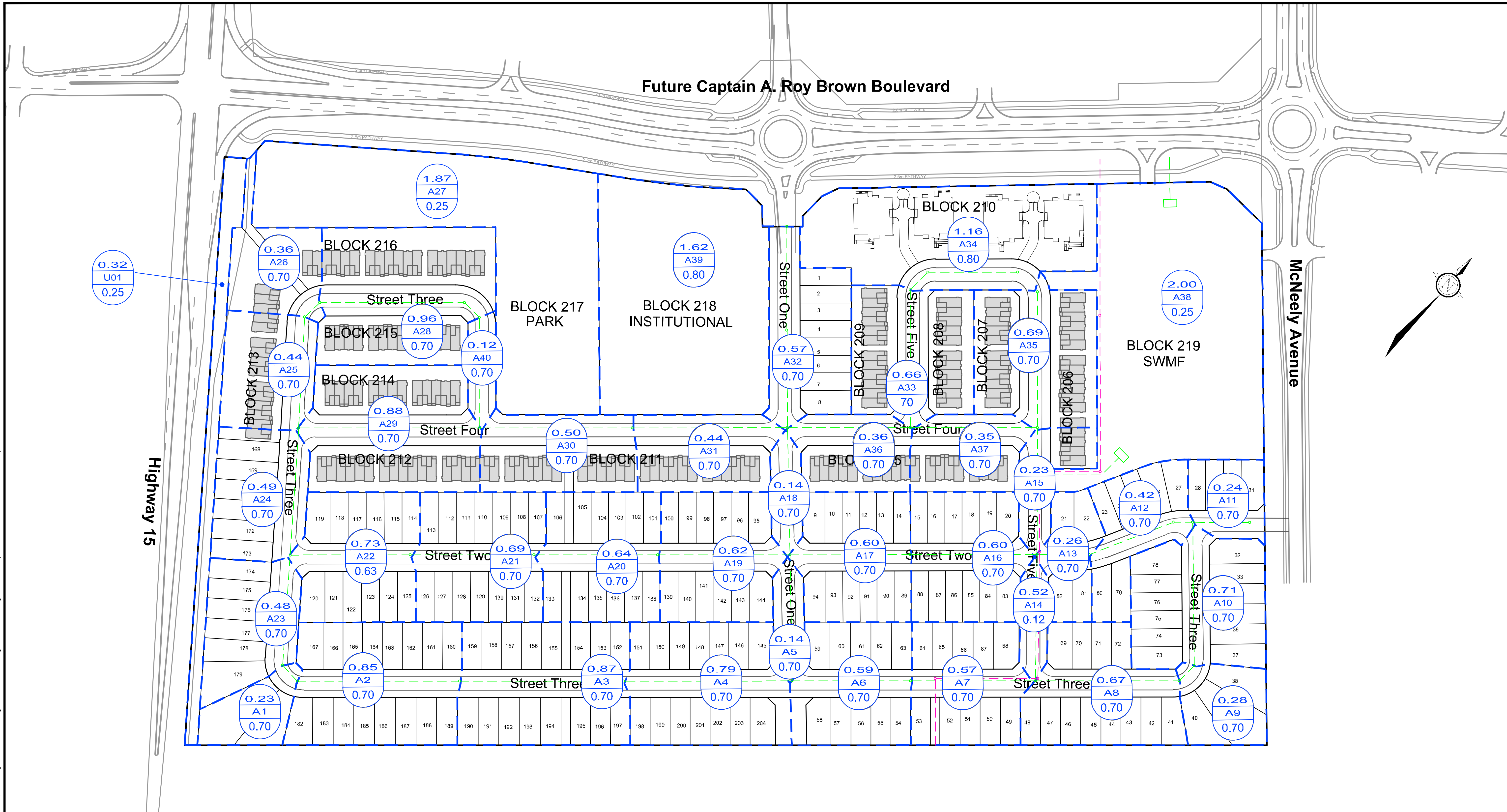
The storm sewer design parameters in **Table 4.1** will be used in the sewer capacity analysis.

Table 4.1: Storm Sewer Design Parameters

Parameter	Design Criteria
Local and Collector Roads	5-year Return Period
Storm Sewer Design	Rational Method/Modeling
IDF Rainfall Data	OSDG (refer to excerpts in **Appendix C**)
Initial Time of Concentration (T_c)	10 minutes
Minimum Velocity [1]	0.8 m/s
Maximum Velocity	3.0 m/s
Minimum Pipe Diameter	250 mm

[1]A minimum gradient of 0.65% is required for any initial sewer run with less than 10 residential connections.

As part of the detailed design a storm sewer design sheet will be prepared to ensure pipe sizes and slopes are in accordance to the minimum requirements set out by the MECP Guidelines and OSDG.



LEGEND

DRAINAGE AREA BOUNDARY

0.27 — CATCHMENT AREA (ha)
A1 — AREA ID
0.55 — RUNOFF COEFFICIENT

PROPOSED STORM SEWER

PROPOSED BYPASS STORM SEWER

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STORM DRAINAGE AREA

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Inlet Control Devices

Inlet control devices (ICDs) are to be installed in all catchbasins to limit inflows to the minor system during larger storm events. ICDs will be sized as part of the detailed design.

Storm Sewer Hydraulic Grade Line

A review of the storm sewer HGL will be completed as part of the detailed design to ensure the HGL will not pose a risk to the proposed dwellings.

Sump pumps may be required in areas impacted by grade raise restrictions if the storm sewer HGL does not provide sufficient vertical clearance to the proposed dwellings underside of footing. The use of sump pumps effectively disconnects the foundation drain from the 100-year HGL in the storm sewer. The use of sump pumps will be reviewed further as part of the detailed design, if required.

4.2.2 ***Overland Flow Path (Major System)***

As part of the detailed design, the Subject Site will be graded to provide an engineered overland flow route (major system) for large, infrequent storms or in the event that the storm sewer system becomes obstructed. Major system flows will be directed to the proposed SWM facility, except for a section of Street One where the flows will outlet to Captain A. Roy Brown Boulevard as mentioned above.

4.2.3 ***Best Management Practices and Low Impact Development***

The proposed development will explore the following stormwater best management practices (BMPs) and low impact development (LID) techniques to mitigate the reduction in groundwater infiltration / recharge resulting from the proposed development:

- Perforated pipes, and clear stone pipe trenches in rear yard areas of low density and medium density residential uses will be used to promote infiltration;
- Roof leaders should be directed to grassed rear yard areas.

By implementing stormwater management BMPs and LIDs as part of the storm drainage design, the impacts of development on the hydrologic cycle can be reduced. The use and implementation of BMPs and LIDs will be reviewed again during the detailed design process.

4.2.4 ***Stormwater Management Facility***

The proposed SWM facility will be sized to provide water quality and quantity control for the Subject Site.

Additional details for the proposed SWM facility will be provided as part of the detailed design.

Design Criteria

The proposed SWM facility will be designed to meet the following criteria:

- An *Enhanced* level of water quality control (80% long-term TSS removal);
- Quantity control storage to limit post-development peak flows to pre-development levels;
- The SWM facility is to have side slopes of 5:1 (H:V) or shallower;
- Guardrails will be installed at the inlet and outlet structures, as required.

As part of the detailed design two options will be reviewed in order to adhere to the design criteria and quality and quantity control requirements:

- **Option 1** consists of an Oil Grit Separator (OGS) installed upstream of a dry-pond;
- **Option 2** consists of a wet-pond, including a sediment forebay, permanent pool, and active storage volume.

It should be noted that Option 1 is the Town's preference.

SWM Facility Access

Access to the inlet and outlet structures will be provided by a 5.0m wide reinforced grass service road and forebay access.

Geotechnical Considerations / Pond Liner

The base and the sidewalls of the proposed SWM facility will be inspected by a geotechnical consultant to confirm the requirement for a geotechnical liner. The thickness of the pond liner (if required) would be designed to be outside the limits of the design grades of the SWM facility and would have no impact on the storage volume of the pond.

Inlet Structure

The inlet to the proposed SWM facility will be designed for a 5-year storm event. The SWM facility inlet structure will consist of the following:

- Pipe outletting to the dry-pond (Option 1) or forebay (Option 2), sized for the flows from the 5-year storm event;
- Headwall for the pipe outlet (both options), including an adjustable stop log restrictor intended for dewatering of the sediment forebay (Option 2 only).
- Bypass pipe outletting directly to the pond permanent pool (Option 2 only). This pipe will be utilized for maintenance and cleanout of the forebay.

Sediment Forebay / Permanent Pool (Option 2 only)

The sediment forebay and permanent pool will be designed in accordance with the MECP SWMF Guidelines. The sediment forebay and permanent pool will consist of the following:

- Sediment forebay, sized to allow for a minimum of 10 years of sediment accumulation, with a submerged riprap berm set 0.05 m below the normal water level, to separate the forebay from the main cell of the pond;
- Permanent pool, sized for an *Enhanced* level of protection (80% long-term TSS removal). The normal water level for the permanent pool will be set at the 2-year water level or 0.1m above the normal water level of the recipient. Refer to Section 4.2.2 for further information of the recipient, the re-aligned Beckwith Drain.

Outlet Structure (both options)

Outflows from the proposed SWM facility will be routed through an outlet control structure before discharging to a pipe outletting to the future re-aligned Beckwith Drain.

Overflow Spillway (both options)

The proposed SWM facility will be sized to provide sufficient storage for storms up to and including the 100-year event. An overflow spillway will be provided in case the outlet storm sewer is obstructed or an extreme event (greater than the 100-year event) generates runoff exceeding the maximum available storage in the SWM facility.

Extended Detention (Option 2 only)

Extended detention will be provided for active storage to allow for settling of suspended sediment in the pond. Extended detention outflows will be controlled by the outlet structure. A steel hood will protect the outlet control from clogging due to floating debris. The extended detention volume will be released over a period of approximately 24 hours.

Stage-Storage-Discharge Table (both options)

Based on a preliminary analysis, a storage volume of approximately 12,140 m^3 will be required for quantity, to ensure the post-development conditions match pre-development rates. The draft plan of subdivision proposes a SWM facility block of 2.00 ha to accommodate the above requirements. The SWM facility block layout and configuration will be detailed as part of detailed design. The allowable release rate to the outlet will be approximately 674 L/s. As part of detailed design, stage-storage curves will be provided. A summary of the SWM facility operating characteristics has been provided in **Appendix C**.

Preliminary SWM Facility Layout

In order to achieve the above requirements, **Figure 4.2** – Proposed Conceptual SWM Facility (Option 1 & 2) has been provided to demonstrate the functional design of the storm servicing, SWM facility layout, and the operating levels. The top of pond has been offset 20m from the property limits to account for access, grading tie-in, landscaping, and sediment storage (if required). It should be noted that applying an offset of 20m from the property limits is conservative and will likely be reduced as part of the detailed design.

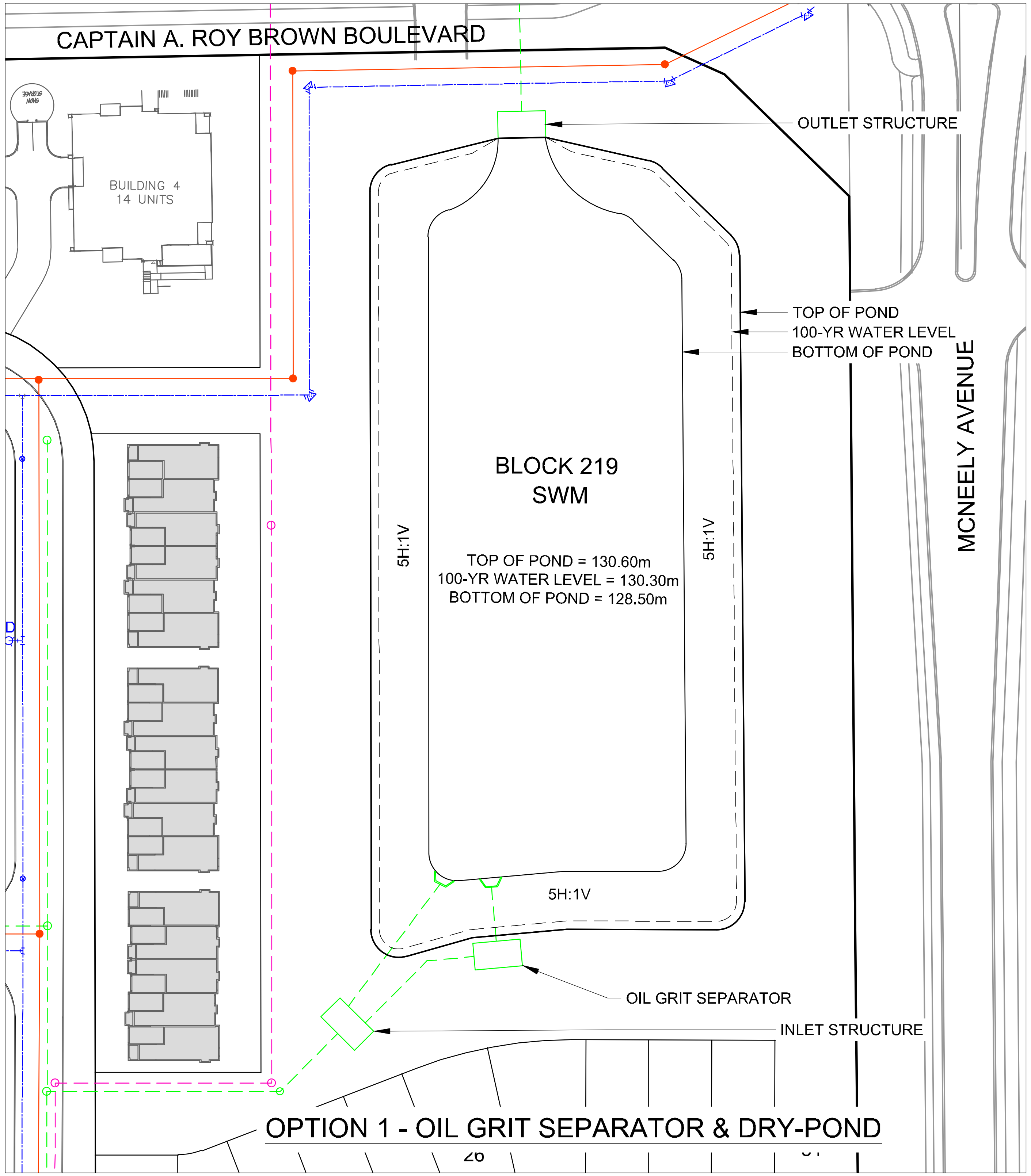
Option 2 is being provided as an alternative should other approval authorities require additional water quality treatment measures to meet the *Enhanced* level of protection.

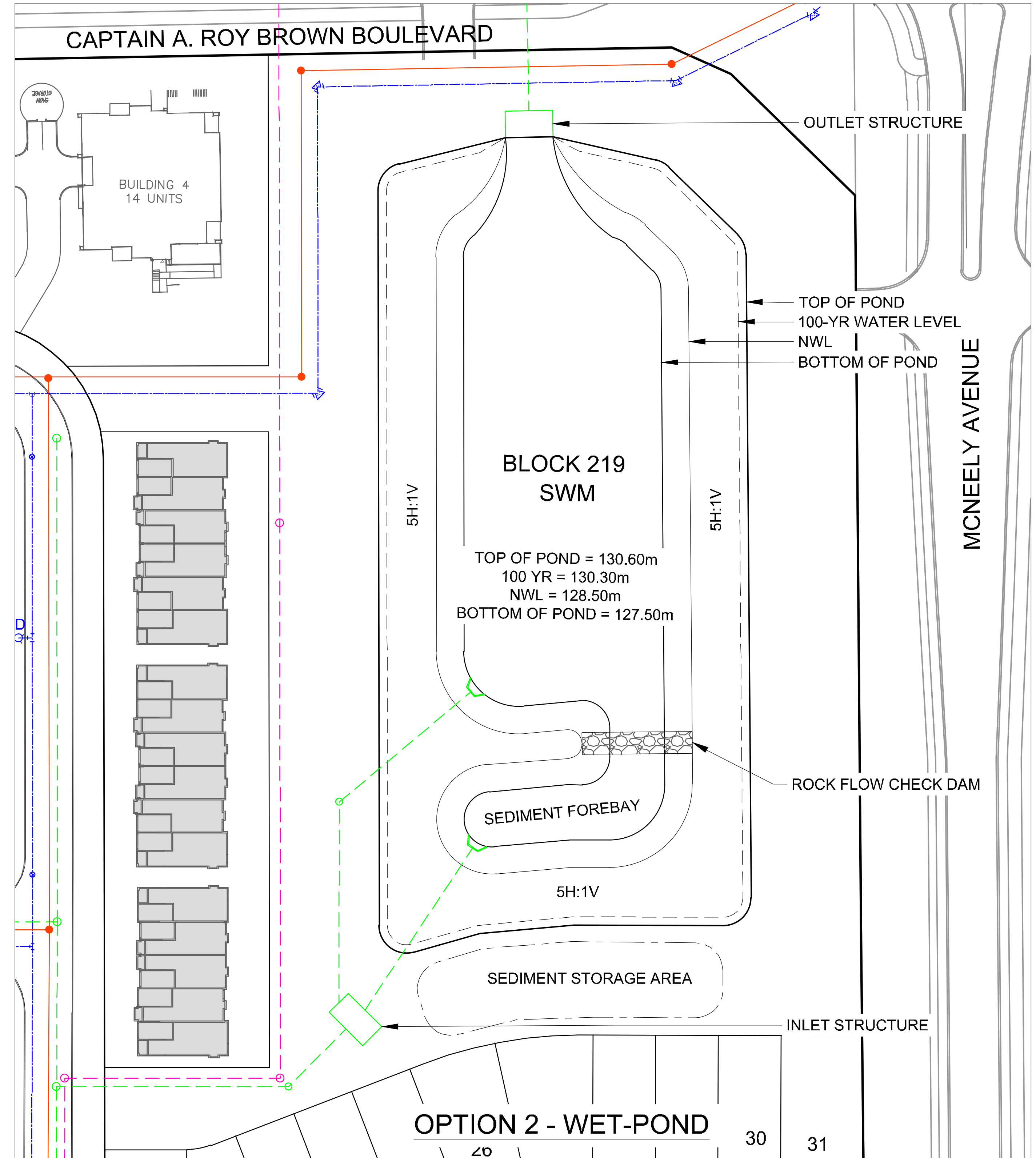
4.2.5 *Re-Aligned Beckwith Drain*

Under existing conditions, storm runoff from the area west of Highway 15 (312.16 ha) are conveyed within an existing ditch, referred to as the Beckwith Drain. In order for the Town to implement the construction of the future Captain A. Roy Brown Boulevard, the Beckwith Drain will need to be re-aligned, along the south side boulevard, and new culverts will need to be proposed. The Town of Carleton Place has engaged By-town Engineering to oversee the design of the Captain A. Roy Brown Boulevard, and Beckwith Drain re-alignment.

Prior to and during detailed design of the subdivision, Novatech will actively coordinate with the Town and By-town Engineering to obtain boundary conditions of the re-aligned drain, as it relates to the proposed SWM facility operating levels.

Refer to **Figure 4.3** – Pre-Development Storm Drainage Area Plan for an illustration of the pre-development drainage areas and **Figure 4.4** – Post-Development Storm Drainage for an illustration of the pre- and post-development drainage areas and the section of re-aligned Beckwith Drain.





LEGEND

PROPOSED SANITARY SEWER
PROPOSED STORM SEWER
PROPOSED BYPASS STORM SEWER
PROPOSED WATERMAIN
HYD FIRE HYDRANT
EXISTING SANITARY SEWER
EXISTING WATERMAIN

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McNEELY LANDING

PROPOSED CONCEPTUAL SWM FACILITY (OPTION 1 & 2)

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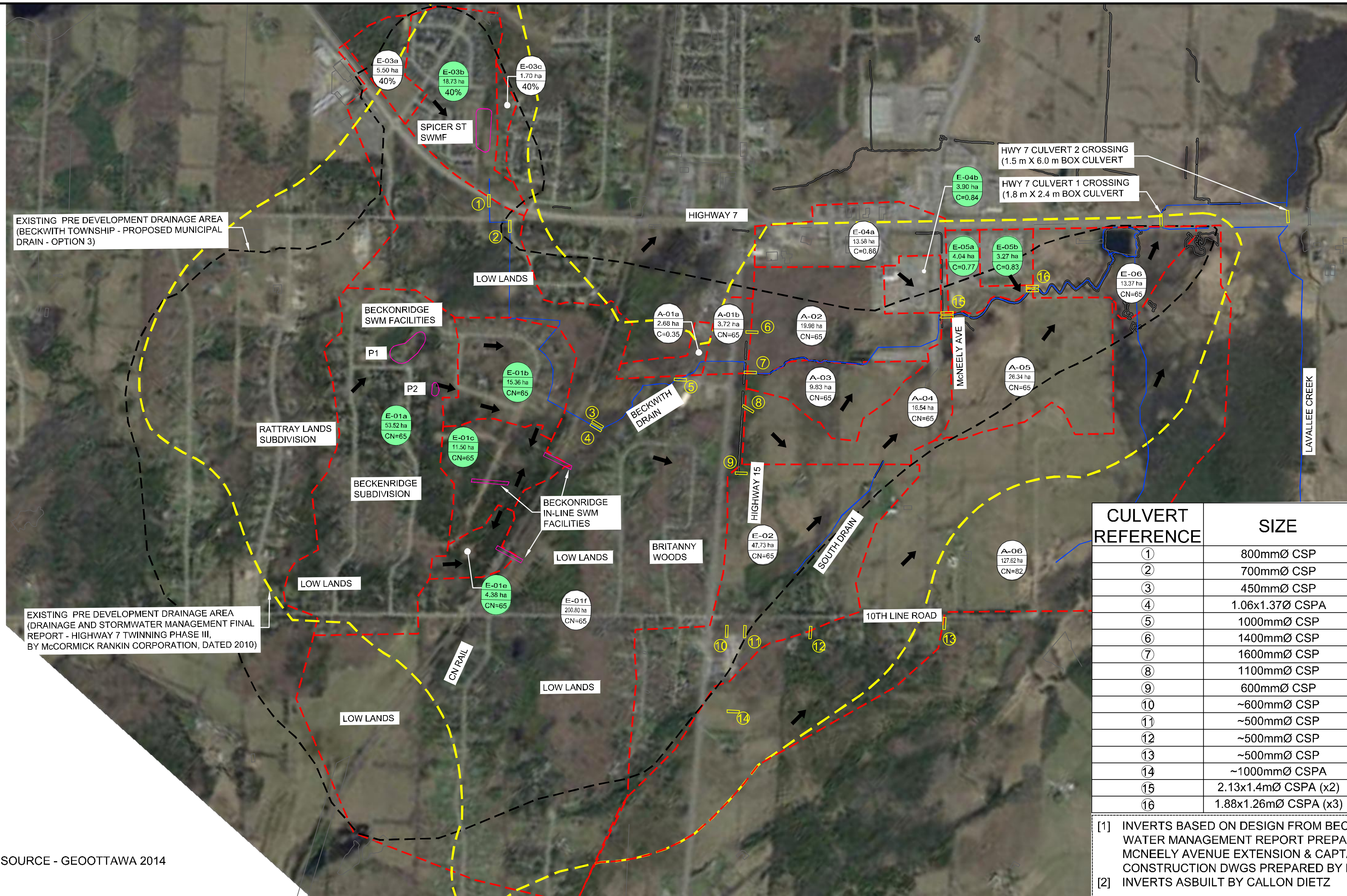


CULVERT REFERENCE	SIZE	INVERTS
1	800mmØ CSP	N=139.33, S=139.48 [1]
2	700mmØ CSP	N=139.21, S=139.07 [1]
3	450mmØ CSP	E=137.02, W=136.63 [1]
4	1.06x1.37Ø CSPA	W=135.68, E=135.61 [1]
5	1000mmØ CSP	W=135.70, E=135.65 [1]
6	1400mmØ CSP	SW=135.25, NE=135.30 [2]
7	1600mmØ CSP	SW=134.78, NE=134.96 [2]
8	1100mmØ CSP	W=134.91, E=135.01 [2]
9	600mmØ CSP	W=135.08, E=135.14 [2]
10	~600mmØ CSP	
11	~500mmØ CSP	
12	~500mmØ CSP	
13	~500mmØ CSP	
14	~1000mmØ CSPA	
15	2.13x1.4mØ CSPA (x2)	W=126.86, E=126.78 [2]
16	1.88x1.26mØ CSPA (x3)	W=127.11, E=127.10 [2]

[1] INVERTS BASED ON DESIGN FROM BECKENRIDGE PH3 FINAL STORM WATER MANAGEMENT REPORT PREPARED BY MCINTOSH PERRY AND MCNEELY AVENUE EXTENSION & CAPTAIN ROY BROWN BOULEVARD CONSTRUCTION DWGS PREPARED BY BT ENGINEERING
[2] INVERTS ASBUILT BY CALLON DIETZ

AERIAL IMAGE SOURCE - GEOOTTAWA 2014

LEGEND

ID — DRAINAGE AREA ID
AREA — DRAINAGE AREA (HECTARES)
%/CN — TOTAL IMPERVIOUS/ CURVE NUMBER

EXISTING CULVERT

STORMWATER MANAGEMENT CONTROL

SUBCATCHMENT BOUNDARY

FLOWPATH

EXISTING WATERWAY

ONSITE CONTROL AND ATTENUATION

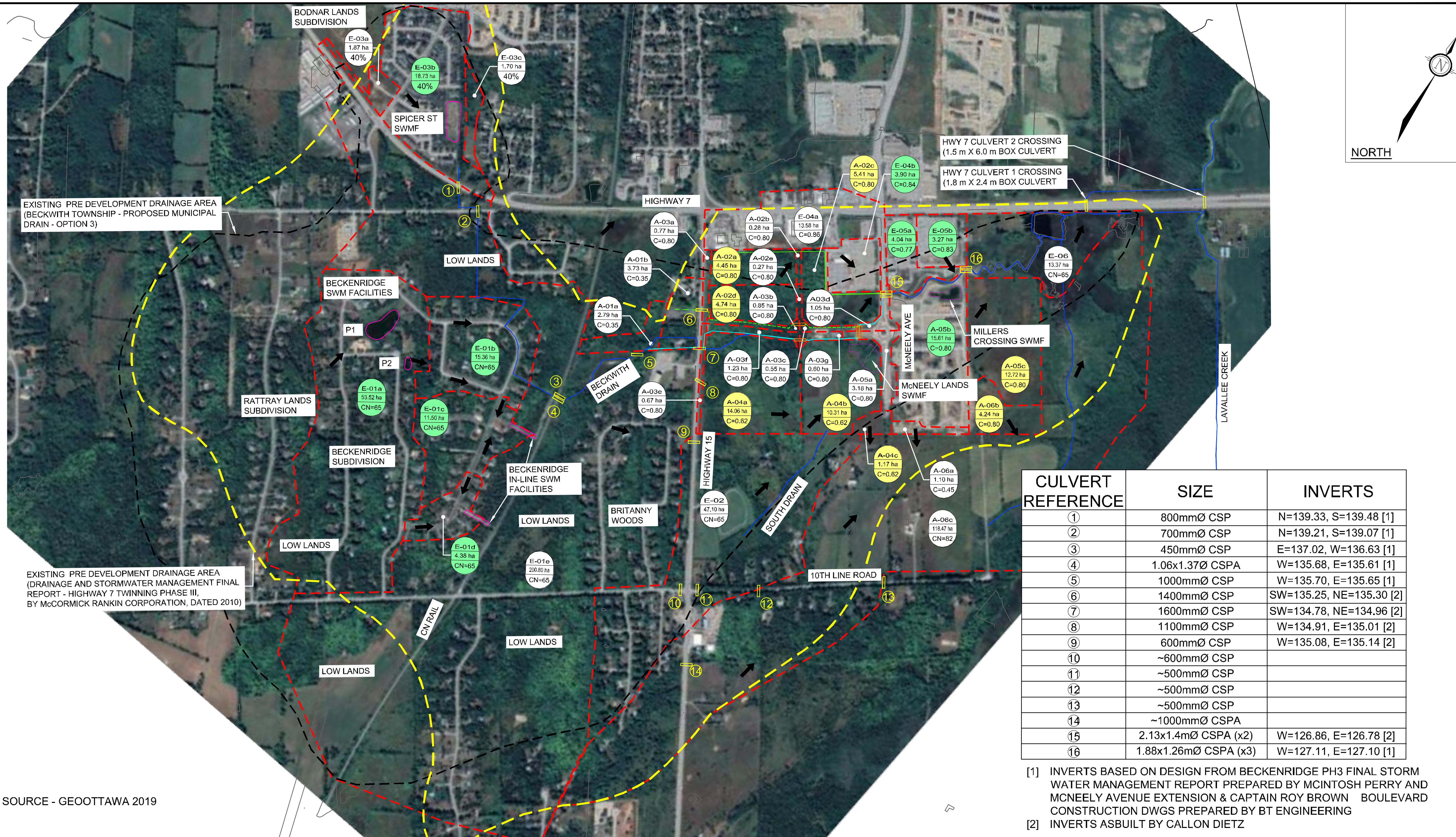
NOVATECH
ENGINEERING CONSULTANTS LTD.
ENGINEERS & PLANNERS
Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada
K2M 1P6
Telephone (613) 254-9643
Facsimile (613) 254-5867
Email: novainfo@novatech-eng.com

CARLETON PLACE HIGHWAY 7 SOUTH DEVELOPMENT AREA

PRE-DEVELOPMENT DRAINAGE

SCALE: NTS

DATE: MAY 2022 | JOB: 119221 | FIGURE: 4.3



CULVERT REFERENCE	SIZE	INVERTS
1	800mmØ CSP	N=139.33, S=139.48 [1]
2	700mmØ CSP	N=139.21, S=139.07 [1]
3	450mmØ CSP	E=137.02, W=136.63 [1]
4	1.06x1.37Ø CSPA	W=135.68, E=135.61 [1]
5	1000mmØ CSP	W=135.70, E=135.65 [1]
6	1400mmØ CSP	SW=135.25, NE=135.30 [2]
7	1600mmØ CSP	SW=134.78, NE=134.96 [2]
8	1100mmØ CSP	W=134.91, E=135.01 [2]
9	600mmØ CSP	W=135.08, E=135.14 [2]
10	~600mmØ CSP	
11	~500mmØ CSP	
12	~500mmØ CSP	
13	~500mmØ CSP	
14	~1000mmØ CSPA	
15	2.13x1.4mØ CSPA (x2)	W=126.86, E=126.78 [2]
16	1.88x1.26mØ CSPA (x3)	W=127.11, E=127.10 [1]

[1] INVERTS BASED ON DESIGN FROM BECKENRIDGE PH3 FINAL STORM WATER MANAGEMENT REPORT PREPARED BY MCINTOSH PERRY AND MCNEELY AVENUE EXTENSION & CAPTAIN ROY BROWN BOULEVARD CONSTRUCTION DWGS PREPARED BY BT ENGINEERING

[2] INVERTS ASBUILT BY CALLON DIETZ

LEGEND

ID — DRAINAGE AREA ID
AREA — DRAINAGE AREA (HECTARES)
%/CN — TOTAL IMPERVIOUS/ CURVE NUMBER

EXISTING CULVERT
PROPOSED CULVERT (BY OTHERS)
STORMWATER MANAGEMENT CONTROL
SUBCATCHMENT BOUNDARY
FLOWPATH
EXISTING WATERWAY
PROPOSED CUTOFF DITCH (BY OTHERS)
PROPOSED DITCH (BY OTHERS)
PROPOSED REALIGNMENT OF BECKWITH DRAIN (BY OTHERS)
ONSITE CONTROL AND ATTENUATION
CONTROL POST TO PRE (CN=65)

NOVATECH
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CARLETON PLACE HIGHWAY 7
SOUTH DEVELOPMENT AREA

POST-DEVELOPMENT DRAINAGE

SCALE		
NTS		
DATE MAY 2022	JOB 119221	FIGURE 4.4

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SHT11X17.DWG - 279mmX432mm

4.2.6 *External Drainage Areas*

Under existing conditions, storm runoff from Area E-02, the area to the south of the Subject Site (47.73 ha) is directed through the Subject Site towards future Captain A. Roy Brown Boulevard and the Beckwith Drain.

Under post-development conditions, runoff from the southern external area will be captured by a proposed ditch inlet catch basin (DICB) between Lots 86 and 87 and conveyed within a bypass storm sewer to the re-aligned Beckwith Drain. Based on a preliminary analysis, it was determined that the maximum 100-year peak flow that the bypass sewer will need to be sized for will be 0.951 m^3/s based on the 100-year snowmelt and rain, 10-day event. The design storm peak flow and runoff volume summary has been provided in **Appendix C**.

Refer to **Figure 3.1** – Proposed Conceptual Servicing Layout Plan for an illustration of the proposed bypass storm sewer and **Figure 4.3** – Pre-Development Storm Drainage Area Plan for an illustration of the pre-development drainage areas. The proposed bypass storm sewer will be approximately 480 m in length.

4.3 Hydrologic & Hydraulic Modeling

The OSDG requires hydrologic modeling for all dual drainage systems. The performance of the proposed storm drainage system for the Subject Site will be evaluated using a PCSWMM hydrologic / hydraulic model. This will be completed as part of the detailed design.

5.0 SANITARY SEWER SYSTEM

5.1 Existing Sanitary Infrastructure

The sanitary outlet for the Subject Site is an existing 375 mm sanitary sewer located at the McNeely Avenue and Captain A. Roy Brown Boulevard intersection.

Excerpts of the sanitary sewer design sheets from the Miller's Crossing DB, demonstrating that the Subject Site was accounted for in the downstream sewers, can be found in **Appendix D**. The External Sanitary Drainage Area Plan that was prepared in support of the design sheets is also included in **Appendix D**.

5.2 Proposed Sanitary Infrastructure

On-site works

The proposed on-site works will require approximately 3,140 m of on-site sanitary sewer to collect wastewater flows and to direct flows to the sanitary outlet.

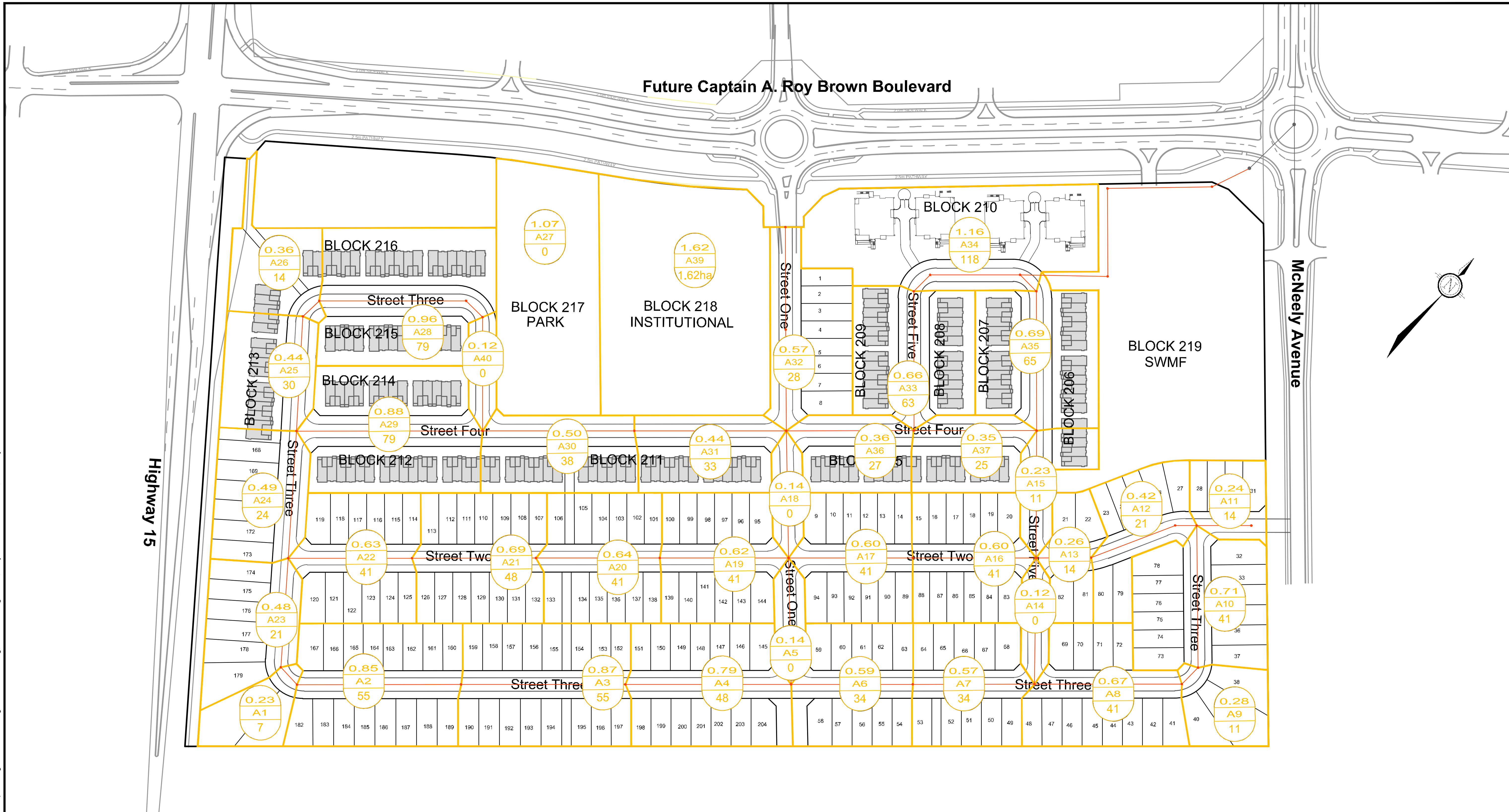
The majority of the roadway elevations within the Subject Site are at an elevation of 132m or greater. As the sanitary outlet invert elevation at MH 26A is 125.78m, there should not be any issues for pipe cover and the site can be serviced by gravity.

Refer to **Figure 3.1** – Proposed Conceptual Servicing Layout Plan for an illustration of the proposed sanitary connection and layout details.

Refer to **Figure 5.1** – Sanitary Drainage Area Plan for an illustration of the proposed drainage areas.

5.3 Sanitary Demand and Design Parameters

The peak design flow parameters in **Table 5.1** will be used in the sewer capacity analysis.



LEGEND

SANITARY DRAINAGE AREA

0.27 — AREA (ha)
A1 — AREA ID
10 — POPULATION

PROPOSED SANITARY SEWER

EXISTING SANITARY SEWER

NOVATECH

Engineers, Planners & Landscape Architects

Suite 200, 240 Michael Cowpland Drive
Ottawa, Ontario, Canada K2M 1P6

Telephone (613) 254-9643
Facsimile (613) 254-5867
Website www.novatech-eng.com

McNEELY LANDING

SANITARY DRAINAGE AREA

SCALE: NOT TO SCALE

DATE: MAY 2022 | JOB: 119221 | FIGURE: 5.1

M:\2019\119221\CAD\Design\Figures\Design Brief\Conceptual Figures\119221-Figures revised.dwg, SAN Drainage Area, May 16, 2022 - 10:00am, anikitchyn

SHT11X17.DWG - 279mmX432mm

Table 5.1: Sanitary Sewer Design Parameters

Design Component	Design Parameter
Unit Population: Single Detached Homes Semis-Detached /Townhomes Medium Density Units	 3.4 people/unit 2.7 people/unit 1.8 people/unit
Residential Flow Rate, Average Daily	350 L/cap/day
Residential Peaking Factor	Harmon Equation (min=2.0, max=4.0) Harmon Correction Factor = 1.0
Institutional Flow Rate	50,000 L/day/ha
Institutional Peaking Factor	1.5
Extraneous Flow Rate	0.23 L/s/ha
Minimum Pipe Size	200mm (Res)
Minimum Velocity[1]	0.6 m/s
Maximum Velocity	3.0 m/s
Minimum Pipe Cover	2.5 m (Unless frost protection provided)

[1]A minimum gradient of 0.65% is required for any initial sewer run with less than 10 residential connections.

Based on the sanitary sewer design parameters outlined above, and the development intent of the Subject Site, the peaked sanitary flows to the receiving sanitary sewer will be 25.43 L/s.

For comparison, the previous submission used sanitary sewer design parameters based on the OSDG (residential flow rate, average daily of 280 L/cap/day, harmon correction factor of 0.8, institutional flow rate of 28,000 L/day/ha, institutional peaking factor of 1.0, and extraneous flow rate of 0.33 L/s/ha). The flows generated from these parameters are in the range of 20.05 L/s. The 25.43 L/s is based on exclusions of the SWM facility, road widening (highway 15), and passive portions of the parkland blocks from the extraneous flows.

As part of the detailed design, a sanitary sewer design sheet will be prepared to ensure pipe sizes and slopes are in accordance to the minimum requirements set out by the MECP Guidelines and OSDG. It is anticipated that pipe sizes of 200 mm and 250 mm diameter will be used to service the Subject Site based on the peaked sanitary flows to the receiving sanitary sewer (25.43 L/s) and the available capacity of these pipe sizes at the minimum cleansing velocity (200 mm diameter at 0.32% = 19.36 L/s; 250 mm diameter at 0.24% = 30.39 L/s).

The receiving sanitary sewers, which were sized as part of the sanitary sewer design sheets prepared as part of the Miller's Crossing DB, allocated a flow of 25.26 L/s for the Subject Site. Although the flows from the Subject Site are 0.17 L/s above the allocated flows previously contemplated, the increase in flows are negligible.

<u>Sanitary Sewer Hydraulic Grade Line</u>

A review of the sanitary sewer HGL will be completed as part of the detailed design to ensure the HGL will not pose a risk to the proposed dwellings.

6.0 WATER SUPPLY SYSTEM

6.1 Existing Water Infrastructure

The watermain connection points for the Subject Site are an existing 300 mm watermain stub located at the McNeely Avenue and Captain A. Roy Brown Boulevard intersection (Connection 1); and an existing 200 mm watermain stub located at the McNeely Avenue and Flegg Way intersection (Connection 2).

6.2 Proposed Water Infrastructure

The Subject Site will be serviced with approximately 3,340 m of on-site watermain for domestic water supply and fire fighting purposes.

A 300 mm trunk watermain will be extended within the Subject Site, from the existing 300 mm watermain stub to Highway 15, to accommodate a future connection (by others – employment lands). This is in line with the recommendations of the MSSMR.

Additionally, a 200 mm watermain stub will be left at Street One and Captain A. Roy Brown Boulevard, to accommodate a potential future connection (by others – Muturra and Scowcroft commercial lands).

Refer to **Figure 3.1** – Proposed Conceptual Servicing Layout Plan for an illustration of the proposed watermain connections and layout details.

The location of hydrants will be confirmed during detailed design.

6.3 Watermain Design Parameters

The domestic demand design parameters, fire fighting demand design scenarios and system pressure criteria design parameters are outlined in **Table 6.1** below. The system pressure design criteria used to determine the size of the watermains, required within the Subject Site, and are based on a conservative approach that considers three possible scenarios.

Table 6.1: Watermain Design Parameters and Criteria

Domestic Demand Design Parameters	**Design Parameters**
Unit Population: Single Detached Home Semis-Detached /Townhomes 2-BR Apartments	 3.4 people/unit 2.7 people/unit 2.1 people/unit
Average Day Residential Demand (AVDY)	350 L/c/d
Maximum Day Residential Demand (MXDY)	2.5 x Average Day
Peak Hour Residential Demand (PKHR)	2.2 x Maximum Day
Average Day Institutional Demand (AVDY)	28,000 L/day/ha
Maximum Day Institutional Demand (MXDY)	1.8 x Average Day
Peak Hour ICI Demand (PKHR)	1.8 x Maximum Day
Fire Demand (FF) Design	**Design Flows**
Conventional single/town units, unless otherwise noted. Hydrant spacing and coding	10,000L/min per FUS / OWDG TB-2014 90 to 120m spacing per OWDG
System Pressure Criteria Design Parameters	**Criteria**
Maximum Pressure (AVDY) Condition	< 80 psi occupied areas < 100 psi unoccupied areas
Minimum Pressure (PKHR) Condition	> 40 psi
Minimum Pressure (MXDY+FF) Condition	> 20 psi

The firefighting water demands for the Subject Site have been estimated per OWDG which refers to the Fire Underwriters Survey (CGI, 1999) document, abbreviated as FUS.

In accordance with the FUS and based on the proposed zoning, there is potential for less than 3m of separation between the single family, semi-detached, and row townhome wood-framed buildings, which would require the fire area in the FUS estimate for multiple buildings to be treated as a contiguous block area. This results in a high fire flow demand which is difficult to attain from the existing system; moreover, it would trigger larger diameter watermain size within the Subject Site, creating system vulnerabilities such as water age issues. As per the ISTB-2014-02, fire flows may be capped at 167 L/s (10,000 L/min) for single family, semi-detached, and row townhome, provided certain site criteria are met. The criteria are:

- For singles: a min separation of 10m between the backs of adjacent units.
- Traditional side-by-side semi-detached or row townhomes:
  a. firewalls with a min two-hour rating to separate the block into fire areas of no more than the lesser of 7 dwelling units, or 600 m^2 of building area; and
  b. Min separation of 10 m between the backs of adjacent units.

In general, the proposed layout of the Subject Site in conjunction with the established zoning setbacks ensures that the minimum separation of 10 meters between the backs of adjacent units is achieved.

Areas where the minimum separations are not achieved will require additional analysis. These areas will be highlighted as part of detailed design.

Notwithstanding the above, the Subject Site's layout shall meet the foregoing criteria allowing the capped fire flow of 167 L/s to be used for these particular unit types of residential units.

6.4 System Pressure Modeling and Results

System pressures for the Subject Site were estimated using the EPANET engine within PCSWMM.

The Miller's Crossing boundary conditions and hydraulic model were used as a base, and expanded on to model the system pressures for the Subject Site. It should be noted that the watermain design parameters outlined in **Table 6.1** above are different then those outlined within the Miller's Crossing DB. Notwithstanding, the hydraulic model that was developed is reflective of the watermain design parameters for each respective development. Excerpts of the watermain design parameters and watermain hydraulic analysis are included in **Appendix E**.

The PCSWMM model layout is demonstrated in **Figure 6.1** – Proposed Watermain Sizing, Layout and Junction IDs and **Figure 6.2** – Ground Elevations (m).

Domestic Demand

The water demand summary for the complete build out of the Subject Site for the basic daily and peak hour demands has been provided in **Table 6.2** below. For detailed results refer to the tables provided in **Appendix E.** The detailed results are also demonstrated in **Figure 6.3** – Maximum Pressures During BSDY Condition and **Figure 6.4** – Minimum Pressures During PKHR Condition.

Table 6.2: System Pressure (EPANET)

Condition	Demand (L/s)	Allowable Pressure (psi)	Max/Min Pressure (psi)
Average Day Demand	5.681	80 (Max)	71
Peak Hour Demand	30.06	40 (Min)	54

Fire Demand

Furthermore, an analysis was carried out to determine the available fire flow under maximum day demand while maintaining a residual pressure of 20psi. This was completed using the EPANET fire flow analysis feature within PCSWMM. For detailed results refer to the tables provided in **Appendix E.** The detailed results are also demonstrated in **Figure 6.5** – Available Flow at 20psi During MXDY+FF Condition.

To achieve the required fire flow, the OWDG and its subsequent revisions (specifically ISTB-2018-02) allow for multiple hydrants to be drawn from, as opposed to drawing from a single hydrant to meet the required demand. As part of detailed design, the location and spacing of hydrants will be reviewed to ensure the required fire flows can be achieved by utilizing multiple hydrants. An excerpt from ISTB-2018-02 of Appendix I: Guideline on Coordination of Hydrant Placement with Required Fire Flow has been included in **Appendix E** for reference on the maximum flow that can be considered from a given hydrant.

The hydraulic analysis demonstrates that the proposed watermain sizing meets the design criteria.

As part of the detailed design, boundary conditions will be confirmed with the Town to ensure that interim and ultimate conditions using the existing 300 mm watermain feed and future looping through the employment lands, respectively, will provide acceptable levels of service as the development advances.

7.0 UTILITIES

The development will be serviced by Hydro One, Bell Canada, Rogers Communications, and Enbridge Gas Distribution Inc. Furthermore, streetlighting will be provided within the proposed road allowances, and will be designed in accordance with the Town's lighting policy. The works will be coordinated with local utility companies during detailed design.

A Composite Utility Plan will be prepared as part of detailed design.

8.0 EROSION AND SEDIMENT CONTROL AND DEWATERING MEASURES

Temporary erosion and sediment control measures will be implemented during construction in accordance with the "Guidelines on Erosion and Sediment Control for Urban Construction Sites" (Government of Ontario, May 1987). Details will be provided on an Erosion and Sediment Control Plan, prepared as part of detailed design. Erosion and sediment control measures may include:

- Placement of silt sacs under all catch basins and maintenance holes;
- Tree protection fence around the trees to be maintained;
- Silt fence around the area under construction placed as per OPSS 577 / OPSD 219.110;
- Light duty straw bale check dam per OPSD 219.180.

The erosion and sediment control measures will need to be installed to the satisfaction of the engineer, the Town, the Ontario Ministry of Environment, Conservation, and Parks (MECP), and the MVCA, prior to construction and will remain in place during construction until vegetation is established. The erosion and sediment control measure will also be subject to regular inspection to ensure that measures are operational.

Furthermore, due to the dewatering activities required during construction of the proposed infrastructure, a Permit-To-Take-Water (PTTW) application or activity registry will be submitted to the MECP. The permit will outline the water taking quantity, and location / quality of the discharge.

9.0 NEXT STEPS, COORDINATION, AND APPROVALS

The proposed municipal infrastructure may be subject, but not limited to the following approvals:

- MECP PTTW. Submitted to: MECP. Proponent: Uniform Urban Developments Ltd.
- MECP Environmental Certificate of Approval (ECA) for the storm / sanitary sewers, and proposed SWM facility as a “Direct Submission”. Submitted to: MECP. Proponent: Uniform Urban Developments Ltd.
- MECP Pre-authorized watermain alteration and extension program granted as part of Town of Carleton Place Drinking Water Works Permit (F-1 Form). Submitted to: Town of Carleton Place. Proponent: Uniform Urban Developments Ltd.
- Tree Cutting Permit. Submitted to: Town of Carleton Place. Proponent: Uniform Urban Developments Ltd., or its contractor/agent.
- Road Cut Permit. Submitted to: Town of Carleton Place. Proponent: Uniform Urban Developments Ltd., or its contractor/agent.

10.0 SUMMARY AND CONCLUSIONS

This report demonstrates that the proposed development can be adequately serviced with storm and sanitary sewers and watermain. The report is summarized below:

Stormwater Management:

- The Subject Site will be serviced with approximately 3,000 m of on-site storm sewers. The on-site storm sewers will outlet to the proposed SWM facility located in the northeast corner of the Subject Site.
- Storm servicing for the Subject Site will be provided using a dual drainage system: Runoff from frequent events will be conveyed by storm sewers (minor system), while runoff from larger storm events which exceed the capacity of the minor system will be conveyed overland along defined overland flow routes (major system). The proposed SWM facility is the outlet for both the major and minor systems, except for a section of Street One where the major system will outlet to Captain A. Roy Brown Boulevard.
- A review of the storm sewer HGL will be completed as part of the detailed design to ensure the HGL will not pose a risk to the proposed dwellings. Sump pumps may be required in areas impacted by grade raise restrictions if the storm sewer HGL does not provide sufficient vertical clearance to the proposed dwellings underside of footing.
- An *Enhanced* level of water quality control (minimum 80% long-term TSS removal) will be achieved prior to releasing flows from the Subject Site;
- Quantity control is to be provided to control post-development peak flows to pre-development levels.
- Based on a preliminary analysis, a storage volume of approximately 12,140 m^3 will be required for quantity, to ensure the post-development conditions match pre-development rates. The draft plan of subdivision proposes a SWM facility block of 2.00 ha to accommodate the above requirements.
- The Beckwith Drain will need to be re-aligned, along the south side boulevard, and new culverts will need to be proposed. The Town of Carleton Place has engaged By-town Engineering to oversee the design of the Captain A. Roy Brown Boulevard, and Beckwith Drain Re-alignment. Further coordination with By-town Engineering is required.
- Runoff from the southern external area will be captured by a proposed ditch inlet catch basin (DICB) and conveyed within a bypass storm sewer to the re-aligned Beckwith Drain.

Sanitary and Wastewater Collection System:

- The Subject Site will be serviced with approximately 3,140 m of on-site sanitary sewers. The onsite sanitary sewers will direct flows to an existing 375 mm sanitary sewer located at the McNeely Avenue and Captain A. Roy Brown Boulevard intersection.
- The downstream existing sanitary sewer system have been designed for the flows of the Subject Site and have adequate capacity.

Water Supply System

- The Subject Site will be serviced with approximately 3,340 m of on-site watermain. The watermain connection points for the Subject Site are an existing 300 mm watermain stub located at the McNeely Avenue and Captain A. Roy Brown Boulevard intersection (Connection 1); and an existing 200 mm watermain stub located at the McNeely Avenue and Flegg Way intersection (Connection 2).
- As part of the detailed design, boundary conditions will be confirmed with the Town to ensure that interim and ultimate conditions using the existing 300 mm watermain feed and future looping through the employment lands, respectively, will provide acceptable levels of service as the development advances.

Erosion and Sediment Control

- Temporary erosion and sediment control measures will be implemented both prior to commencement and during construction in accordance with the “Guidelines on Erosion and Sediment Control for Urban Construction Sites” (Government of Ontario, May 1987).

Next Steps, Coordination, and Approvals

- MECP PTTW.
- MECP Environmental Certificate of Approval (ECA) for the storm / sanitary sewers, and proposed SWM facility as a “Direct Submission”.
- MECP Pre-authorized watermain alteration and extension program granted as part of Town of Carleton Place Drinking Water Works Permit (F-1 Form).
- Tree Cutting Permit.
- Road Cut Permit.

11.0 CLOSURE

This report is respectfully submitted for review and subsequent approval. Please contact the undersigned should you have questions or require additional information.

NOVATECH

Prepared by:

Ben Sweet, P.Eng.
Project Coordinator I Land Development

Reviewed by:

Bassam Bahia, M.Eng., P.Eng.
Senior Project Manager | Land Development

Appendix A
Correspondence

Pre-Consultation Meeting Notes
Virtual zoom meeting – May 27th, 2020
Prepared By: Julie Stewart

In Attendance
Steve Pentz – Senior Project Manager, Novatech
John Ridell – Regional Group
Sam Bahia – P.Eng., Regional Group
Annibale Ferro – Uniform Developments
Ryan Robert – Uniform Developments
Joanna Bowes – Manager of Development Services, Town of Carleton Place
Robin Daigle – Engineering Manager, Town of Carleton Place
Diane Reid – Environmental Planner, MVCA
Julie Stewart – County Planner, County of Lanark
Stephen Kapusta – Planner, MTO – provided comments to Julie via e-mail as he was not available for the zoom meeting.

Steve Pentz provided a brief background.
The subject lands are the RSSR lands and the Laing Lands. The lands are south of Highway 7 and to the west of McNeely, behind the Home Depot.
Official Plan designates the subject lands as Residential District.
The lands are Residential District in the Development Permit By-law as well.
A concept plan was provided on May 27th and is attached to these meeting notes as reference.
The access to the site will be by Captain A. Roy Brown Blvd., and McNeely Avenue South.
The site is proposed to be developed by a mix of singles and medium density units of semi-detached and townhouse units.

John Riddell advised that the focus of the proposed development at this time is for the RSSR lands only.

The Lanark County Pre-Consultation Checklist is also attached, this includes reports / studies provided by the Town during the meeting. The reports / studies / plans as noted on the attached checklist are required to be submitted at the time of application. The following provides some additional comments:

Planning Report – Development Permit also to be addressed within.

Environmental Impact Study
– MVCA noted there is a tributary of Lavallee Creek in the northwest corner of the property. In an e-mail provided to Julie, MVCA advised that Lavallee Creek originates on the west side of Hwy 15 and would have to be considered (EIS with fish habitat assessment and setback recommendations) as part of any development proposal in this area, including streets (i.e. extension of the Blvd to Hwy 15).

Servicing Options Statement
- As the site is will be on public services, a Conceptual Servicing Report shall be submitted with the application.

Stormwater Drainage Plan
- MVCA advised that Stormwater Management, Quality and Quantity control would be required, with Quality to an enhanced level of treatment.

Noise Attenuation Study
- As requested by the Town for the lots immediately adjacent to Captain Roy Brown Blvd.
- For the future phase – Laing lands – adjacent to Highway 15 – a noise study will be required at that time. Details to be determined by future pre-consultation for those lands.

Traffic Study
- The Town advised that a scoped traffic study will be required
- Lands are part of the Conceptual Design Plan (CDP)

Ministry of Transportation
- Provided comments in an e-mail to Julie Stewart.
- The Ministry will have all of its standard requirements for development. There should be some discussion relative to the timing of construction of Captain Roy Brown and whether this development pattern / density is following along with the plans previously provided.
- In regards to Highway 15 – MTO may require a land conveyance along Highway 15. This is to put the developer on notice that above and beyond our typical requirements, we are working to protect for a future widening of Highway 15 that will likely require a land conveyance and all setbacks should be form a distance of 8 metres form the Highway 15 right of way. Meaning, nothing critical can be located within the conveyance area and setback area. Also, the required yard calculations in bylaws as any required yard must be beyond the setback area.
- Noise – the ministry will have in our permit applications that we are not responsible for noise. The developer and the municipality are responsible parties for any and all noise abatement measures.
- Further discussion should be had with MTO in regards to Hwy 15.

McNeely Avenue South

- The Town advised that this is a Town project but will eventually become a County road.
- Therefore, Julie will circulate the proposal to the Lanark County Public Works Department for review and comment.

Captain Roy Brown Blvd.

- The Town advised that the extension of Captain Roy Brown Blvd. is a Town project.
- The EA has been done.

OTHER
Environmental Site Assessment

- The Town indicated that an ESA is required for submission.

The Town noted that the following reports / plans will be required as a draft condition:

- Coloured perspective drawings of models
- Illumination and Traffic Signal Plan
- Landscape Plan
- Utilities Plan

For the future development of the Laing lands, a pre-consultation meeting will be required.

LANARK COUNTY

PLANS OF SUBDIVISION

PRE-CONSULTATION - checklist
UNIFORM DEVELOPMENTS
MAY 27, 2020

Report	Comments	Required Yes/No
Planning Rationale ✓	Include justification Must have regard for PPS Lanark County Official Plan compatibility Local Official Plan compatibility	Yes
Hydrogeological Study, Terrain Analysis	Availability and suitability of water and waste water MOE – D-5-4 Guidelines MOE – D-5-5 Guidelines ODWSOG Checklist Summary & Sign-off	N/A Municipal Services
Environment Impact Study ✓	SAR & Significant Habitat Wetlands Organic Soils Natural Heritage Features & Systems Significant Wetlands Significant Woodlands Significant Valleylands Significant Wildlife ANSI Fish Habitat	Yes
Servicing Options Statement ✓	Guidelines – MOE D-5-3 Conceptual Servicing Options	Report Yes
Stormwater Drainage Plan ✓	Guidelines - MOE-2003 / MNR-2001 Checklist Summary & Sign-off x MVCA comments	Yes
Grading Plan	Sloping land within lot to direct flow of surface water away from foundations & abutting properties.	Yes

Geotechnical Investigation
-blasting -will be rqd. @ some point.

LANARK COUNTY

PLANS OF SUBDIVISION

PRE-CONSULTATION - checklist

Report	Comments	Required Yes/No
Sediment and Erosion Control Hazardous Sites	Flooding, erosion hazard Slope and Soil Stability Organic Soils - near creek - ? Karst Topography	If identified on site
Archeological Investigation ✓	Standards & Guidelines 2011 - creek - in area - Known sites	Yes
Tree Preservation Plan or Tree Conservation Plan / ~~Landscape Plan~~	Check with local municipality Tree Inventory / Tree Preservation Plan (Town)	Yes
Other Noise Attenuation Study Noise Study - Traffic Study	- lots immediately adjacent to CRB. Blvd. (Town) - Laing Lands - Hwy. 15	Yes
Draft Plan	To include: Planning Act 50(17) Ont. Reg. 544/06 Lot and block configuration Compatibility with adjacent uses Road access, street layout & Pedestrian amenities Parks & Open Space amenities Easement and right-of-way requirements	Yes

Traffic Study - as part of the CDP
- fairly scoped

MTO - land conveyance Hwy. 15
- noise
- timing of construction - CRB Blvd &
comparison w previous plans

MVCA - SWM - quality & quantity. Enhanced.

Mississippi Valley Conservation Authority

Ontario Regulation 153/06

Property Map

Legend

Property - Parcels

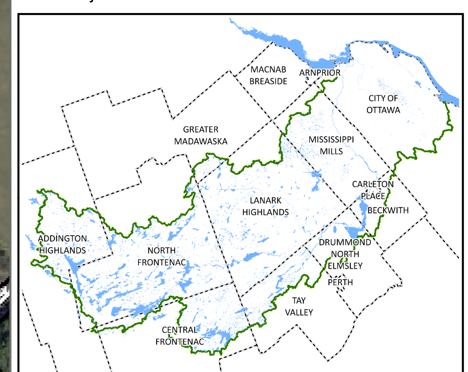
MVCA Streams

0 0.125 0.25

km

1:6,000

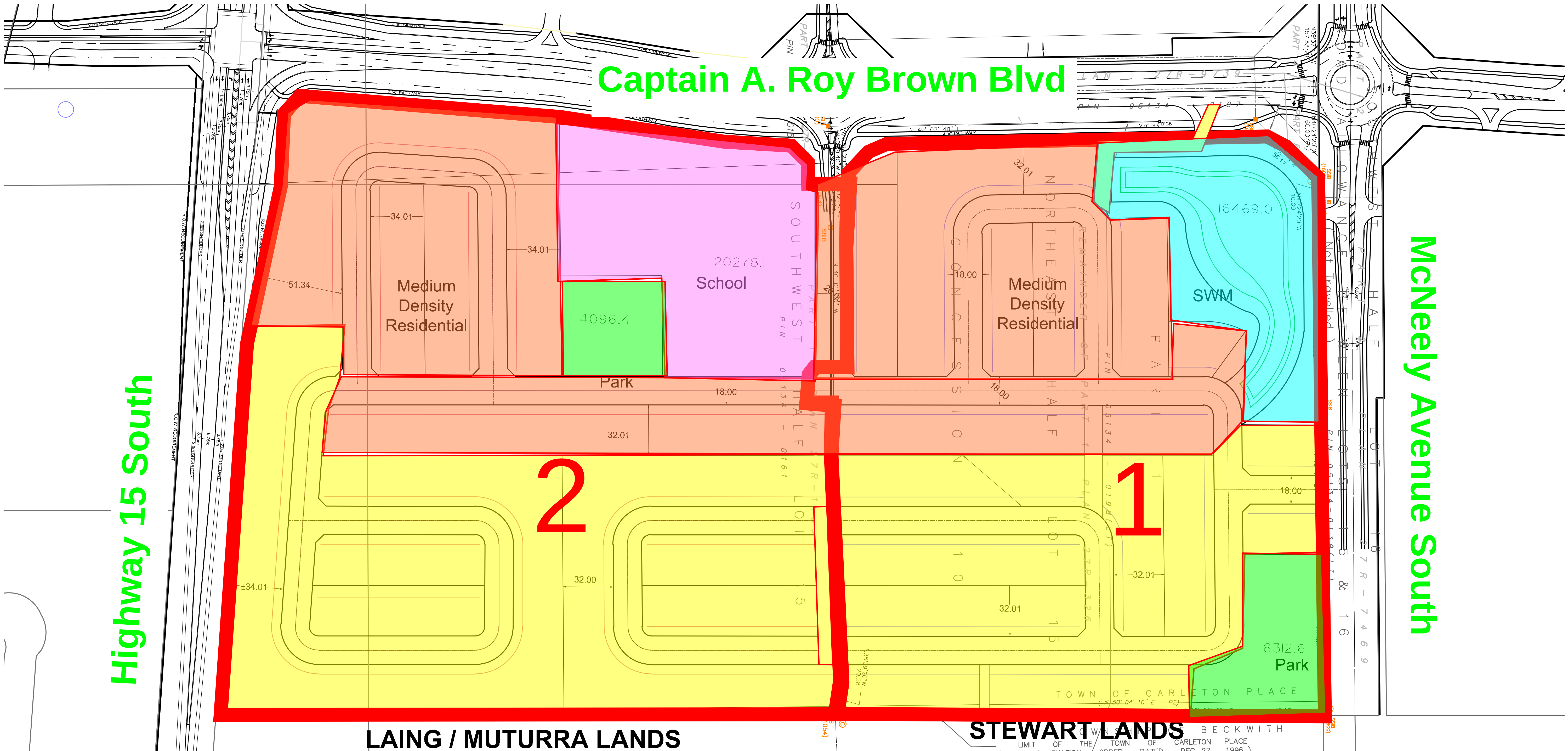
Projection: UTM Zone 18- NAD 83 Datum



This map is produced in part with data provided by the Ontario Geographic Data Exchange under License with the Ontario Ministry of Natural Resources and the Queen's Printer for Ontario, 2019

Imagery @ Fugro Geospatial, May 2014

Date: 5/27/2020



Laing/Muturra		
* Setback Assumptions: 6.0m FY, 7.5m RY, 4.5m Ext Side, 1.2m Int	length (m)	length (ft)
Saleable Frontage		
Low Density Residential	2282.91	7489.84
Total frontage	2282.91	7489.84
Road Lengths		
Local Roads	1473.09	4832.96
Total roads	1473.09	4832.96

Stewart		
* Setback Assumptions: 6.0m FY, 7.5m RY, 4.5m Ext Side, 1.2m Int	length (m)	length (ft)
Saleable Frontage		
Low Density Residential	1823.66	5983.12
Total frontage	1823.66	5983.12
Road Lengths		
Local Roads	1251.11	4104.68
Total roads	1251.11	4104.68

LEGEND

- Singles
- Semis / Towns
- Institutional
- Parks
- SWM Pond

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STEWART & LAING LANDS

CONCEPT PLAN

SCALE 1 : 2500

DATE JAN 2020

JOB

FIGURE Concept6

M:\2019\119221\CAD\Design\119221-BP CP5.dwg, 11x17 landscape, Jan 27, 2020 - 5:08pm, kcrossman

LANARK COUNTY

Pre-Consultation Meeting Notes
Virtual zoom meeting – September 10, 2020
Prepared By: Julie Stewart

In Attendance
Steve Pentz – Senior Project Manager, Novatech
Sam Bahia – P.Eng., Regional Group
Ryan MacDougall – Uniform Developments
Robin Daigle – Engineering Manager, Town of Carleton Place
Julie Stewart – County Planner, County of Lanark
Stephen Kapusta – Planner, MTO

This is the second pre-consultation meeting which focussed on Phase 2 which is the lands referred to as the Laing Lands. The first pre-consultation meeting was held virtually on May 27th and meeting notes and a checklist were circulated.

Steve Pentz provided a brief background. A concept plan was previously circulated indicating the Phase 2 lands. The proposal is for one draft plan of subdivision submission, with phased registration.

The majority of the discussion was related to the lands adjacent to Highway 15.

The following comments were provided verbally at the virtual meeting and summarized by Stephen Kapusta, MCIP, RPP, Senior Project Manager, Highway Corridor Management , **Ministry of Transportation** - Eastern Region :

1) There are some yet unknowns relative to the required setbacks along Highway 15. To help mitigate that, the Ministry will require a 9 metre gratuitous land conveyance. However, since there is an Enbridge gas line sandwiched between our highway and the plan of subdivision, there will need to be some investigation as to how to address any future widening so that we are reducing the chances of having to expand into the subdivision.

2) Intermingled with all of that is both Hydro One and a possible noise barrier.

3) So therefore, at a minimum we are looking at 9 metres of land conveyance and 14 metres of setback beyond that 9 metres for structures within the subdivision.

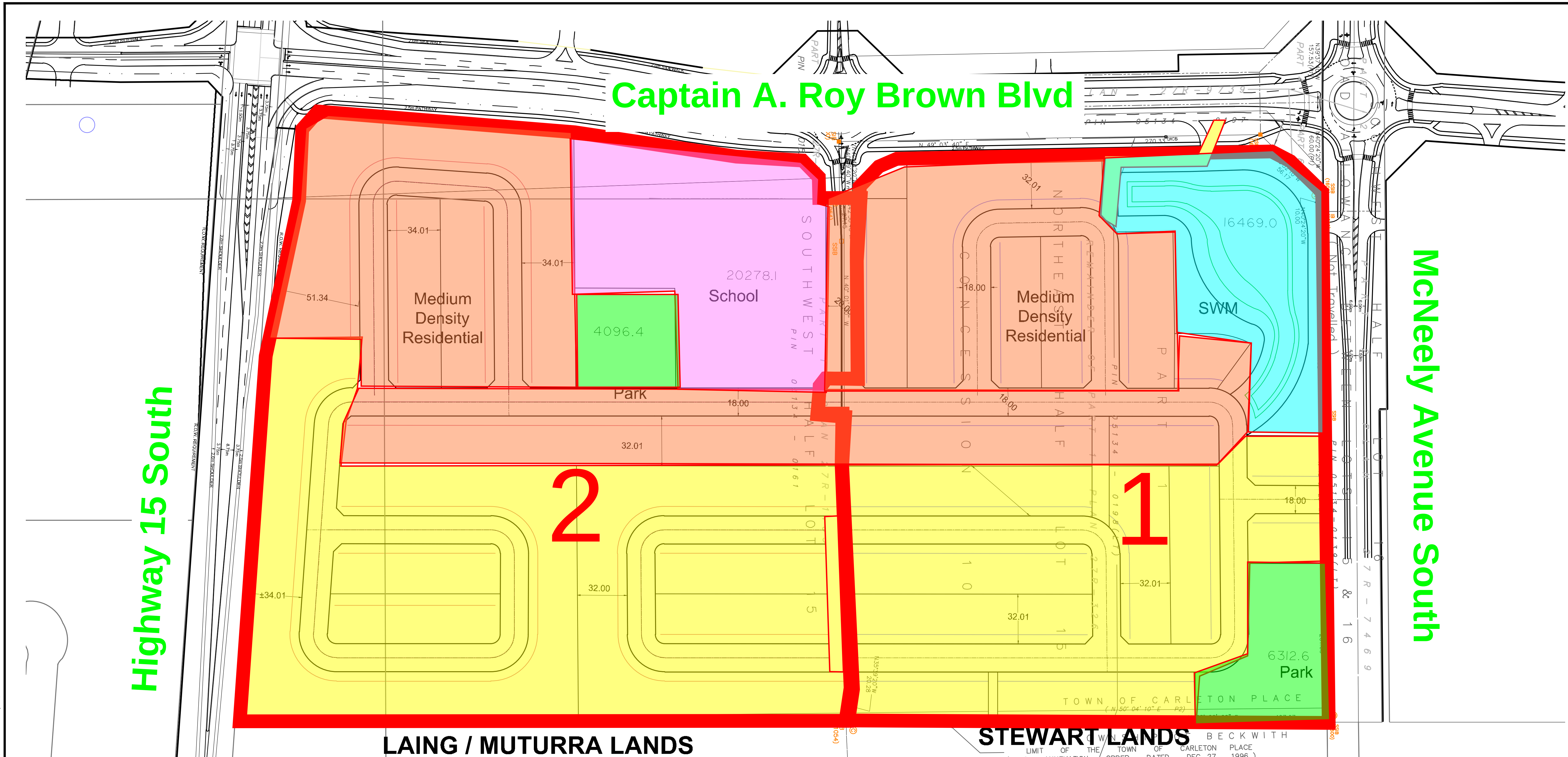
4) I have reached out to my Project Manager for the Highway improvements to get both updated plans, traffic volumes and a contact at Enbridge to loop into the discussion. I will forward that information when it comes available.

5) Beyond the above, our typical Stormwater requirements apply. I see no reason for a traffic impact study for any of the development in this plan of subdivision.

6) The ministry would like to see a connection or two from this development area to the south. It appears that McNeely will be the main connection. However , as development continues to expand south, it would be advantageous for there to be further connections. This is however just a suggestion and it is up to the County as to whether this is a requirement or not.

The owner / agents will follow up with Enbridge once the information is available,

The link to the most recent Environmental Assessment information was also requested to be provided by MTO.



LEGEND

- Singles
- Semis / Towns
- Institutional
- Parks
- SWM Pond

Laing/Muturra		
* Setback Assumptions: 6.0m FY, 7.5m RY, 4.5m Ext Side, 1.2m Int	length (m)	length (ft)
Saleable Frontage		
Low Density Residential	2282.91	7489.84
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FIGURE Concept6

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Appendix B
Servicing Report Checklist

4.1 General Content	**Addressed (Y/N/NA)**	**Section**	**Comments**
Executive Summary (for larger reports only).	NA		
Date and revision number of the report.	Y	Cover	
Location map and plan showing municipal address, boundary, and layout of proposed development.	Y	Fig 1.1	
Plan showing the site and location of all existing services.	Y	Fig 1.1	
Development statistics, land use, density, adherence to zoning and official plan, and reference to applicable subwatershed and watershed plans that provide context to which individual developments must adhere.	NA		
Summary of Pre-consultation Meetings with City and other approval agencies.	Y	1	
Reference and confirm conformance to higher level studies and reports (Master Servicing Studies, Environmental Assessments, Community Design Plans), or in the case where it is not in conformance, the proponent must provide justification and develop a defendable design criteria.	Y	2	
Statement of objectives and servicing criteria.	Y	1	
Identification of existing and proposed infrastructure available in the immediate area.	Y	4,5,6	
Identification of Environmentally Significant Areas, watercourses and Municipal Drains potentially impacted by the proposed development (Reference can be made to the Natural Heritage Studies, if available).	NA		
Concept level master grading plan to confirm existing and proposed grades in the development. This is required to confirm the feasibility of proposed stormwater management and drainage, soil removal and fill constraints, and potential impacts to neighboring properties. This is also required to confirm that the proposed grading will not impede existing major system flow paths.	Y	Fig 3.2	

Development Servicing Study Checklist

4.1 General Content	**Addressed (Y/N/NA)**	**Section**	**Comments**
Identification of potential impacts of proposed piped services on private services (such as wells and septic fields on adjacent lands) and mitigation required to address potential impacts.	NA		
Proposed phasing of the development, if applicable.	NA		
Reference to geotechnical studies and recommendations concerning servicing.	Y	2.2	
All preliminary and formal site plan submissions should have the following information:			
Metric scale	NA		
North arrow (including construction North)	NA		
Key plan	NA		
Name and contact information of applicant and property owner	NA		
Property limits including bearings and dimensions	NA		
Existing and proposed structures and parking areas	NA		
Easements, road widening and rights-of-way	NA		
Adjacent street names	NA		

4.2 Water	**Addressed (Y/N/NA)**	**Section**	**Comments**
Confirm consistency with Master Servicing Study, if available.	Y	6	
Availability of public infrastructure to service proposed development.	Y	6	
Identification of system constraints.	Y	6	
Identify boundary conditions.	Y	6	
Confirmation of adequate domestic supply and pressure.	Y	6	
Confirmation of adequate fire flow protection and confirmation that fire flow is calculated as per the Fire Underwriter's Survey. Output should show available fire flow at locations throughout the development.	Y	6	
Provide a check of high pressures. If pressure is found to be high, an assessment is required to confirm the application of pressure reducing valves.	Y	6	
Definition of phasing constraints. Hydraulic modeling is required to confirm servicing for all defined phases of the project including the ultimate design.	N		TBD as part of detailed design
Address reliability requirements such as appropriate location of shut-off valves.	N		TBD as part of detailed design
Check on the necessity of a pressure zone boundary modification.	NA		
Reference to water supply analysis to show that major infrastructure is capable of delivering sufficient water for the proposed land use. This includes data that shows that the expected demands under average day, peak hour and fire flow conditions provide water within the required pressure range.	Y	6	
Description of the proposed water distribution network, including locations of proposed connections to the existing system, provisions for necessary looping, and appurtenances (valves, pressure reducing valves, valve chambers, and fire hydrants) including special metering provisions.	Y	6, Fig 3.1	
Description of off-site required feedermains, booster pumping stations, and other water infrastructure that will be ultimately required to service proposed development, including financing, interim facilities, and timing of implementation.	NA		
Confirmation that water demands are calculated based on the City of Ottawa Design Guidelines.	Y	6	
Provision of a model schematic showing the boundary conditions locations, streets, parcels, and building locations for reference.	Y	Fig 6.1	

Development Servicing Study Checklist

4.3 Wastewater	Addressed (Y/N/NA)	Section	Comments
Summary of proposed design criteria (Note: Wet-weather flow criteria should not deviate from the City of Ottawa Sewer Design Guidelines. Monitored flow data from relatively new infrastructure cannot be used to justify capacity requirements for proposed infrastructure).	Y	5	
Confirm consistency with Master Servicing Study and/or justifications for deviations.	Y	5	
Consideration of local conditions that may contribute to extraneous flows that are higher than the recommended flows in the guidelines. This includes groundwater and soil conditions, and age and condition of sewers.	NA		
Description of existing sanitary sewer available for discharge of wastewater from proposed development.	Y	5	
Verify available capacity in downstream sanitary sewer and/or identification of upgrades necessary to service the proposed development. (Reference can be made to previously completed Master Servicing Study if applicable)	Y	5	
Calculations related to dry-weather and wet-weather flow rates from the development in standard MOE sanitary sewer design table (Appendix 'C') format.	NA		
Description of proposed sewer network including sewers, pumping stations, and forcemains.	Y	5	
Discussion of previously identified environmental constraints and impact on servicing (environmental constraints are related to limitations imposed on the development in order to preserve the physical condition of watercourses, vegetation, soil cover, as well as protecting against water quantity and quality).	NA		
Pumping stations: impacts of proposed development on existing pumping stations or requirements for new pumping station to service development.	NA		
Forcemain capacity in terms of operational redundancy, surge pressure and maximum flow velocity.	NA		
Identification and implementation of the emergency overflow from sanitary pumping stations in relation to the hydraulic grade line to protect against basement flooding.	NA		
Special considerations such as contamination, corrosive environment etc.	NA		

4.4 Stormwater	Addressed (Y/N/NA)	Section	Comments
Description of drainage outlets and downstream constraints including legality of outlet (i.e. municipal drain, right-of-way, watercourse, or private property).	Y	4	
Analysis of the available capacity in existing public infrastructure.	NA		
A drawing showing the subject lands, its surroundings, the receiving watercourse, existing drainage patterns and proposed drainage patterns.	Y	Fig 4.1	
Water quantity control objective (e.g. controlling post-development peak flows to pre-development level for storm events ranging from the 2 or 5 year event (dependent on the receiving sewer design) to 100 year return period); if other objectives are being applied, a rationale must be included with reference to hydrologic analyses of the potentially affected subwatersheds, taking into account long-term cumulative effects.	Y	4	
Water Quality control objective (basic, normal or enhanced level of protection based on the sensitivities of the receiving watercourse) and storage requirements.	Y	4	
Description of stormwater management concept with facility locations and descriptions with references and supporting information.	Y	4	
Set-back from private sewage disposal systems.	NA		
Watercourse and hazard lands setbacks.	NA		
Record of pre-consultation with the Ontario Ministry of Environment and the Conservation Authority that has jurisdiction on the affected watershed.	NA		
Confirm consistency with sub-watershed and Master Servicing Study, if applicable study exists.	Y	4	
Storage requirements (complete with calcs) and conveyance capacity for 5 yr and 100 yr events.	Y	4	
Identification of watercourse within the proposed development and how watercourses will be protected, or, if necessary, altered by the proposed development with applicable approvals.	Y	4	
Calculate pre and post development peak flow rates including a description of existing site conditions and proposed impervious areas and drainage catchments in comparison to existing conditions.	Y	4	
Any proposed diversion of drainage catchment areas from one outlet to another.	Y	4	
Proposed minor and major systems including locations and sizes of stormwater trunk sewers, and SWM facilities.	Y	4	
If quantity control is not proposed, demonstration that downstream system has adequate capacity for the post-development flows up to and including the 100-year return period storm event.	NA		

4.4 Stormwater	**Addressed (Y/N/NA)**	**Section**	**Comments**
Identification of municipal drains and related approval requirements.	NA		
Description of how the conveyance and storage capacity will be achieved for the development.	Y	4	
100 year flood levels and major flow routing to protect proposed development from flooding for establishing minimum building elevations (MBE) and overall grading.	N		TBD as part of detailed design
Inclusion of hydraulic analysis including HGL elevations.	N		TBD as part of detailed design
Description of approach to erosion and sediment control during construction for the protection of receiving watercourse or drainage corridors.	Y	8	
Identification of floodplains – proponent to obtain relevant floodplain information from the appropriate Conservation Authority. The proponent may be required to delineate floodplain elevations to the satisfaction of the Conservation Authority if such information is not available or if information does not match current conditions.	Y	4	
Identification of fill constrains related to floodplain and geotechnical investigation.	NA		

4.5 Approval and Permit Requirements	**Addressed (Y/N/NA)**	**Section**	**Comments**
Conservation Authority as the designated approval agency for modification of floodplain, potential impact on fish habitat, proposed works in or adjacent to a watercourse, cut/fill permits and Approval under Lakes and Rivers Improvement Act. The Conservation Authority is not the approval authority for the Lakes and Rivers Improvement Act. Where there are Conservation Authority regulations in place, approval under the Lakes and Rivers Improvement Act is not required, except in cases of dams as defined in the Act.	Y	9	
Application for Certificate of Approval (CofA) under the Ontario Water Resources Act.	Y	9	
Changes to Municipal Drains.	NA		
Other permits (National Capital Commission, Parks Canada, Public Works and Government Services Canada, Ministry of Transportation etc.)	Y	9	

4.6 Conclusion	**Addressed (Y/N/NA)**	**Section**	**Comments**
Clearly stated conclusions and recommendations.	Y	10	
Comments received from review agencies including the City of Ottawa and information on how the comments were addressed. Final sign-off from the responsible reviewing agency.	NA		
All draft and final reports shall be signed and stamped by a professional Engineer registered in Ontario.	Y	11	

Appendix C
Stormwater Management Calculations

Table 1: External Lands E-02 (47.73 ha)
Design Storm Peak Flow and Runoff Volume Summary

Event	E-02 (47.10 ha) Peak Flow (m^3/s)	E-02 (47.10 ha) Runoff Volume (m^3)
25mm CHI 3Hr	0.070	692
2-Year CHI 3Hr	0.123	1,212
5-Year CHI 3Hr	0.231	2,262
10-Year CHI 3Hr	0.318	3,102
25-Year CHI 3Hr	0.444	4,305
50-Year CHI 3Hr	0.550	5,317
100-Year CHI 3Hr	0.669	6,463
2-Year SCS 24 Hr	0.198	2,969
5-Year SCS 24 Hr	0.351	5,207
10-Year SCS 24 Hr	0.471	6,940
25-Year SCS 24 Hr	0.636	9,307
50-Year SCS 24 Hr	0.776	11,307
100-Year SCS 24 Hr	0.935	13,570
2Yr Snow Melt + Rain - 10 Day	0.471	47,296
5Yr Snow Melt + Rain - 10 Day	0.602	66,497
10Yr Snow Melt + Rain - 10 Day	0.687	79,284
25Yr Snow Melt + Rain - 10 Day	0.794	95,427
50Yr Snow Melt + Rain - 10 Day	0.873	107,416
100Yr Snow Melt + Rain - 10 Day	0.951	119,344

External Lands E-02 were assessed using the 3 Hour Chicago, 24 hour SCS and a 10 Day snowmelt + Rainfall design events.

From this analysis it was determined that the maximum 100-year peak flow is 0.951 m^3/s based on the 100 Year Snowmelt + Rain 10 day event.

Table 2: Summary of SWM Pond Operating Characteristics

Pond Component	Pond Inflow [1] (m^3/s)	Allowable Release Rate [2] (m^3/s)	Pond Release Rate (m^3/s)	Volume Used (m^3)
Quality Control[3]	-	-	0.012	1022
25mm CHI 3Hr	1.386	0.050	0.050	2928
2-Year CHI 3Hr	1.985	0.089	0.089	3988
5-Year CHI 3Hr	3.169	0.170	0.170	5742
10-Year CHI 3Hr	4.105	0.235	0.235	6993
25-Year CHI 3Hr	5.219	0.329	0.329	8537
50-Year CHI 3Hr	6.183	0.409	0.409	9760
100-Year CHI 3Hr	7.233	0.500	0.500	10970
2-Year SCS 24 Hr	2.007	0.149	0.149	5073
5-Year SCS 24 Hr	3.110	0.266	0.266	7043
10-Year SCS 24 Hr	3.834	0.357	0.320	8324
25-Year SCS 24 Hr	4.735	0.483	0.483	9888
50-Year SCS 24 Hr	5.456	0.590	0.590	11030
100-Year SCS 24 Hr	6.148	0.712	0.712	12140

(1) Pond inflow as calculated by pre development SWMHYMO model

(2) Allowable release rate set to match pre development peak flows from A-04 & A-03

(3) Required quality control volume based on 40 m3/ha released over 48 hours

Table 2 below outlines the required storage volume for the proposed SWM facility to ensure runoff from areas A-04 & A-03 (26.5 ha) under post-development conditions match pre-development rates.

Note that this analysis includes the volume required to provide quality control (80% TSS), and also matches the pre-development 25mm event rates to provide erosion control.

Based on this analysis, a storage volume of approximately 12,140 m^3 will be required to ensure the site matches pre development rates up to the 100-year event.

5.4.2 IDF Curves and Equations

An IDF (Intensity Duration Frequency) curve is a statistical description of the expected rainfall intensity for a given duration and storm frequency. In Ottawa, the IDF curve is derived from Meteorological Services of Canada (MSC) rainfall data taken from the Macdonald-Cartier airport. Rainfall collected from 1967 to 1997 was analyzed using the Gumbel Distribution. The following Table 5.1 shows the analysis results provided by MSC. The IDF equations have been derived on the basis of a regression equation of the form:

$$Intensity = \left[\frac{A}{(Td + C)^B} \right]$$

where:

Intensity = mm/hr

Td = time of duration (min)

A,B,C = regression constants for each return period

Table 5.1 Ottawa IDF Table: 1967 to 1997

Time (min)	**2 year (mm/hr)**	**5 year (mm/hr)**	**10 year (mm/hr)**	**25 year (mm/hr)**	**50 year (mm/hr)**	**100 year (mm/hr)**
5	102.80	140.20	165.00	196.00	219.00	242.60
10	77.10	104.40	122.50	145.30	162.20	179.00
15	63.30	85.60	100.40	119.10	133.00	146.80
30	39.90	53.90	63.10	74.70	83.40	91.90
60	24.20	32.00	37.10	43.60	48.50	53.20
120	14.30	18.90	22.00	25.80	28.70	31.50
360	6.20	8.40	9.90	11.70	13.10	14.50
720	3.60	4.80	5.60	6.60	7.30	8.00
1440	2.00	2.60	3.00	3.50	3.90	4.30

IDF curve equations (Intensity in mm/hr)

100 year Intensity = 1735.688 / (Time in min + 6.014) $^{0.820}$
50 year Intensity = 1569.580 / (Time in min + 6.014) $^{0.820}$
25 year Intensity = 1402.884 / (Time in min + 6.018) $^{0.819}$
10 year Intensity = 1174.184 / (Time in min + 6.014) $^{0.816}$
5 year Intensity = 998.071 / (Time in min + 6.053) $^{0.814}$
2 year Intensity = 732.951 / (Time in min + 6.199) $^{0.810}$

The IDF curves based on the above equations can be found in Appendix 5-A

5.4.3 Design Storms

Computer modeling requires the input of a design storm. The design storm is then used to generate a runoff hydrograph to determine how an area will respond and perform. Numerous types of design storms can be used ranging from historical storms to IDF curve-derived storms. This section briefly discusses the various types of design storms.

5.4.3.1 Application to Hydrologic Models

The design storms presented herein are meant to be used in hydrologic models to simulate runoff from events of various return frequencies. When choosing a design storm, the designer should perform a sensitivity analysis using various storms and use the one that is most conservative.

As noted below, the Chicago distribution is one of the most used storms for urban runoff applications. When dealing with rural areas, the SCS Type II storm is preferred. The AES storm can also be used for urban applications; however, care must be taken when choosing the type of distribution. As a rule of thumb, the 30% distribution should be used unless historical data proves otherwise.

When using a design storm, the designer must be careful in choosing the right storm time step. The storm's duration should be greater than twice the basin's time of concentration. A time step that is too small may overestimate peak flows. Should it be required to maintain a storm time step less than 10 minutes, consideration should be given to averaging the peak intensities to a 10-minute or greater average.

Some historical storms are also presented below and are to be used as a check of how various systems function during extreme events. It is not the intent of these guidelines to require that these storms be used for design purposes.

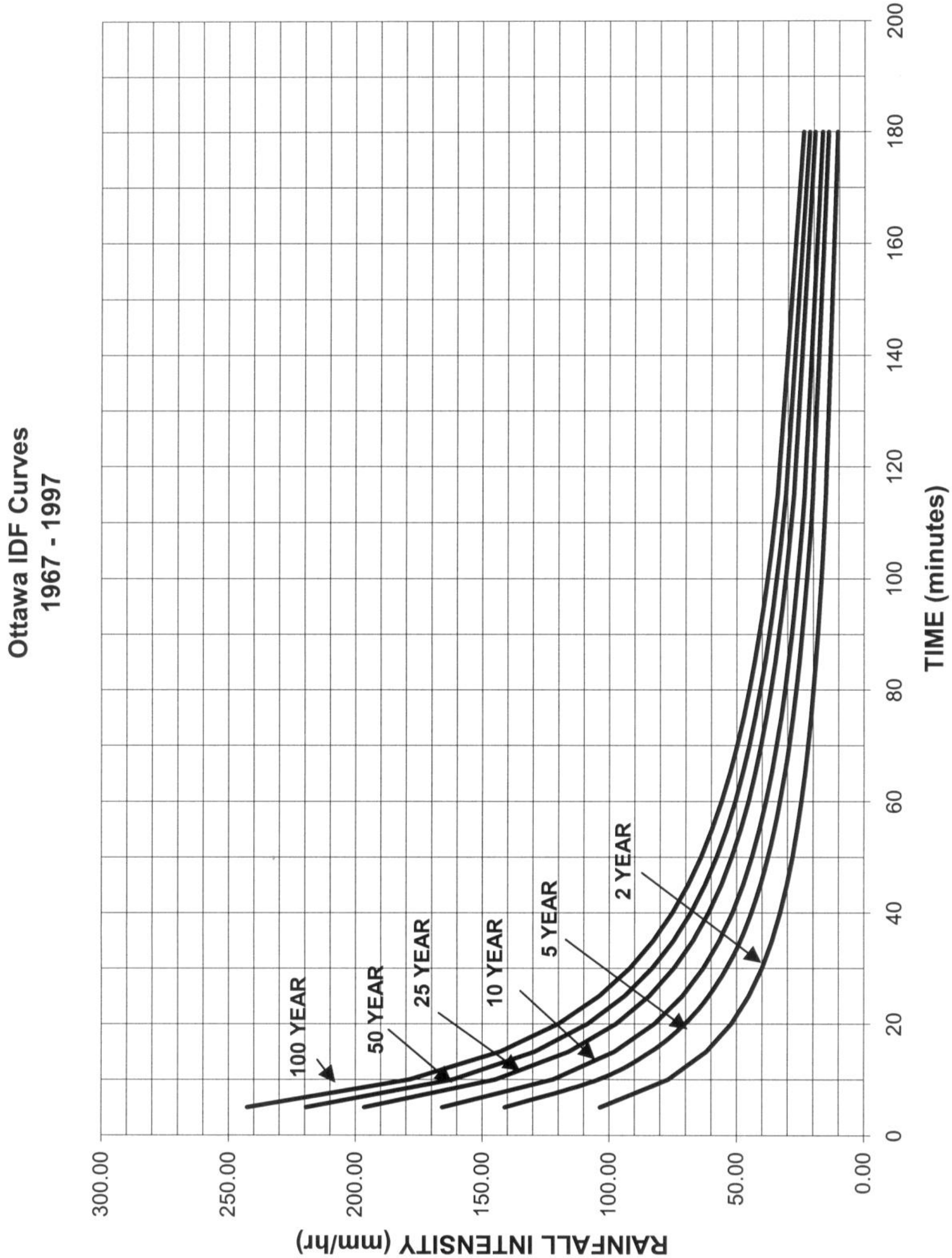
5.4.3.2 Chicago Design Storm

The Chicago storm distribution was developed by C.J. Keifer and H. Chu and is based on 25 years of rainfall record in the city of Chicago. This storm distribution, which is derived with IDF curves, is generally applied to urban basins where peak runoff rates are largely influenced by peak rainfall intensities.

APPENDIX 5-A

OTTAWA INTENSITY DURATION FREQUENCY (IDF) CURVE

APPENDIX 5-A OTTAWA INTENSITY DURATION FREQUENCY (IDF) CURVE



Appendix D
Sanitary Calculations

SANITARY SEWER CALCULATION SHEET

CARLETON PLACE
Meet me on the Mississippi

LOCATION			RESIDENTIAL AREA AND POPULATION							COMMERCIAL		INSTITUTIONAL		PARK		C+I+P	INFILTRATION				PIPE						
STREET	FROM M.H.	TO M.H.	AREA (ha)	UNITS	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	TOTAL FLOW (l/s)	DIST (m)	DIA Nominal (mm)	SLOPE (%)	CAP. (FULL) (l/s)	RATIO Qact/Qcap	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
BLOCK 108 (SERVICING & WALKWAY BLOCK)																											
	CTRL 1A	2A	0.04			0.04								0.84	0.84	0.09	0.88	0.88	0.202	0.29	44.0	200	0.70	27.44	0.01	0.87	0.28
To FANNING STREET , Pipe 2A - 3A						0.04								0.84													
FLEGG WAY																											
	12A	13A	0.60	8	27.2	0.60	27.2	4.00	0.44								0.60	0.60	0.138	0.58	120.0	200	2.20	48.65	0.01	1.55	0.50
To FANNING STREET , Pipe 13A - 14A						0.60	27.2																				
STANZEL DRIVE																											
	4A	5A	0.28	4	13.6	0.28	13.6	4.00	0.22								0.28	0.28	0.064	0.28	53.0	200	0.70	27.44	0.01	0.87	0.28
To FANNING STREET , Pipe 5A - 6A						0.28	13.6																				
	4A	8A	0.44	7	23.8	0.44	23.8	4.00	0.39								0.44	0.44	0.101	0.49	67.0	200	0.70	27.44	0.02	0.87	0.34
	8A	9A	0.20	2	6.8	0.64	30.6	4.00	0.50								0.20	0.64	0.147	0.65	12.0	200	1.00	32.80	0.02	1.04	0.40
	9A	10A	0.56	9	30.6	1.20	61.2	4.00	0.99								0.56	1.20	0.276	1.27	73.5	200	0.80	29.34	0.04	0.93	0.46
To RATHWELL STREET, Pipe 10A - 11A						1.20	61.2																				
FANNING STREET																											
	1A	2A	0.29	5	17.0	0.29	17.0	4.00	0.28								0.29	0.29	0.067	0.35	33.0	200	1.50	40.17	0.01	1.28	0.39
Contribution From BLOCK 108, Pipe CTRL 1A - 2A			0.04											0.84			0.88	1.17									
	2A	3A	0.09	1	3.4	0.42	20.4	4.00	0.33						0.84	0.09	0.09	1.26	0.290	0.71	14.5	200	0.40	20.74	0.03	0.66	0.30
	3A	5A	0.52	7	23.8	0.94	44.2	4.00	0.72						0.84	0.09	0.52	1.78	0.409	1.22	78.0	200	0.40	20.74	0.06	0.66	0.35
Contribution From STANZEL DRIVE, Pipe 4A - 5A			0.28		13.6												0.28	2.06									
	5A	6A	0.65	10	34.0	1.87	91.8	4.00	1.49						0.84	0.09	0.65	2.71	0.623	2.20	82.0	200	0.40	20.74	0.11	0.66	0.42
To RATHWELL STREET, Pipe 6A - 7A						1.87	91.8							0.84													
	1A	13A	0.26	4	13.6	0.26	13.6	4.00	0.22								0.26	0.26	0.060	0.28	52.0	200	0.90	31.12	0.01	0.99	0.30
Contribution From FLEGG WAY, Pipe 12A - 13A			0.60		27.2												0.60	0.86									
	13A	14A	0.73	12	40.8	1.59	81.6	4.00	1.32								0.73	1.59	0.366	1.69	102.0	200	0.50	23.19	0.07	0.74	0.43
	14A	15A	0.19	2	6.8	1.78	88.4	4.00	1.43								0.19	1.78	0.409	1.84	11.0	200	0.50	23.19	0.08	0.74	0.44
	15A	16A	0.62	10	34.0	2.40	122.4	4.00	1.98								0.62	2.40	0.552	2.53	66.0	200	0.50	23.19	0.11	0.74	0.48
	16A	17A	0.20	2	6.8	2.60	129.2	4.00	2.09								0.20	2.60	0.598	2.69	11.0	200	0.50	23.19	0.12	0.74	0.49
	17A	23A	0.21	2	6.8	2.81	136.0	4.00	2.20								0.21	2.81	0.646	2.85	76.0	200	0.50	23.19	0.12	0.74	0.50
To RIDELL STREET , Pipe 23A - 24A						2.81	136.0																				
RIDELL STREET																											
	18A	19A	0.71	24	64.8	0.71	64.8	4.00	1.05								0.71	0.71	0.163	1.21	78.5	200	1.20	35.93	0.03	1.14	0.50
	19A	20A	0.52	17	45.9	1.23	110.7	4.00	1.79								0.52	1.23	0.283	2.07	83.0	200	0.50	23.19	0.09	0.74	0.46
Contribution From BLOCK 111 (HIGH DENSITY)																											
	CTRL 2A	20A	0.38	24	43.2	0.38	43.2	4.00	0.70								0.38	1.61	0.370	1.07	10.0	200	0.70	27.44	0.04	0.87	0.41
	20A	21A	0.08	1	2.7	1.69	156.6	4.00	2.54								0.08	1.69	0.389	2.93	20.0	200	0.50	23.19	0.13	0.74	0.51
			0.05	1	3.4	1.74	160.0	4.00	2.59								0.05	1.74									
	21A	22A	0.20	6	16.2	1.94	176.2	4.00	2.86								0.20	1.94	0.446	3.31	44.0	200	0.80	29.34	0.11	0.93	0.61
			0.44	7	23.8	2.38	200.0	4.00	3.24								0.44	2.38									
	22A	23A	0.46	17	45.9	2.84	245.9	4.00	3.98								0.46	2.84	0.653	4.63	115.0	200	0.70	27.44	0.17	0.87	0.64
Contribution From FANNING STREET, Pipe 17A - 23A			2.81		136.0												2.81	5.65									
			0.29	4	13.6	5.94	395.5	4.00	6.41								0.29	5.94									
	23A	24A	0.29	10	27.0	6.23	422.5	4.00	6.85								0.29	6.23	1.433	8.28	82.0	200	0.60	25.41	0.33	0.81	0.72
To RATHWELL STREET, Pipe 24A - 25A						6.23	422.5																				

LICENSED PROFESSIONAL ENGINEER
K. MITIC
100122349
MAR 22, 2016
PROVINCE OF ONTARIO

DESIGN PARAMETERS

Average Daily Domestic Flow =	350 l/p/day	Industrial Peak Factor = as per MOE Graph	
Commercial Flow =	50000 L/ha/day	Infiltration Rate =	0.23 L/s/Ha
Institutional Flow =	50000 L/ha/day	Minimum Velocity =	0.6 m/s
Light Industry Flow =	35000 L/ha/day	Maximum Velocity =	3.0 m/s
Park Flow =	9300 L/ha/day	Manning's n =	0.013
Max Res. Peak Factor =	4.00	Single Unit coeff=	3.4
Commercial/School Peak Factor =	1.50	Townhouse Unit coeff=	2.7
		Average Condo Unit coeff=	1.8

Designed: P.P	PROJECT: Miller's Crossing		
Checked: K.M.	LOCATION: Carleton Place		
Dwg. Reference: Sanitary Drainage Plan, Dwgs. No. 12 and 13	File Ref: 13-693	March, 2016	Sheet No. 1 of 3

693_San2.xls

SANITARY SEWER CALCULATION SHEET

CARLETON PLACE
Meet me on the Mississippi

LOCATION			RESIDENTIAL AREA AND POPULATION							COMMERCIAL		INSTITUTIONAL		PARK		C+I+P	INFILTRATION				PIPE						
STREET	FROM M.H.	TO M.H.	AREA (ha)	UNITS	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	FLOW (l/s)	TOTAL AREA (ha)	ACCU AREA (ha)	INFILT. FLOW (l/s)	TOTAL FLOW (l/s)	DIST (m)	DIA Nominal (mm)	SLOPE (%)	CAP. (FULL) (l/s)	RATIO Qact/Qcap	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
CAPTAIN A. ROY BROWN BOULEVARD																											
(TRUNK PIPE)																											
Contribution From EXTERNAL FUTURE RESIDENTIAL			19.34		1387.0	As per MSS report by Novatech dated July 2013											19.34	19.34									
Contribution From EXTERNAL FUTURE COMMERCIAL										19.46	As per MSS report by Novatech dated July2013					12.23	19.46	38.80									
From External			1.81													18.35	1.81	40.61									
	26A	27A	0.37			21.52	1387.0	3.70	20.79		19.46					18.35	0.37	40.98	9.425	48.57	40.0	375	0.16	70.13	0.69	0.63	0.68
To MCNEELY AVENUE (TRUNK) , Pipe 27A - 28A						21.52	1387.0				19.46																
MCNEELY AVENUE																											
(TRUNK PIPE)																											
Contribution From CAPTAIN A. ROY BROWN BOULEVARD (TRUNK), Pipe 26A - 27A																											
			21.52		1387.0					19.46							40.98	40.98									
	27A	28A	0.25			21.77	1387.0	3.70	20.79		19.46					18.35	0.25	41.23	9.483	48.62	51.0	375	0.16	70.13	0.69	0.63	0.68
	28A	29A	0.82			22.59	1387.0	3.70	20.79		19.46					18.35	0.82	42.05	9.672	48.81	28.5	375	0.16	70.13	0.70	0.63	0.68
To STOKES DRIVE, Pipe 29A- 31A						22.59	1387.0				19.46																
STOKES DRIVE																											
Contribution From MCNEELY AVENUE (TRUNK), Pipe 28A - 29A																											
			22.59		1387.0					19.46							42.05	42.05									
	29A	31A	0.34	9	24.3	22.93	1411.3	3.70	21.15		19.46					18.35	0.34	42.39	9.750	49.25	72.5	375	0.16	70.13	0.70	0.63	0.68
Contribution From BLOCK 116 (HIGH DENSITY)																											
	CTRL 3A	31A	0.39	24	43.2	0.39	43.2	4.00	0.70								0.39	42.78	9.839	10.54	11.0	200	0.70	27.44	0.38	0.87	0.81
	31A	32A	0.24	7	18.9	23.56	1473.4	3.69	22.02		19.46					18.35	0.24	43.02	9.895	50.27	47.5	375	0.16	70.13	0.72	0.63	0.68
			0.65			24.21	1473.4	2.00	11.94		19.46					18.35	0.65	43.67	10.044	40.33							
	32A	33A	0.69	22	59.4	24.90	1532.8	3.67	22.79		19.46					18.35	0.69	44.36	10.203	51.34	88.0	375	0.16	70.13	0.73	0.63	0.69
	33A	34A	0.65	21	56.7	25.55	1589.5	3.66	23.57		19.46					18.35	0.65	45.01	10.352	52.27	80.0	375	0.16	70.13	0.75	0.63	0.69
To SANITARY TRUNK , Pipe 34A - 35A						25.55	1589.5				19.46																
RATHWELL STREET																											
Contribution From FANNING STREET, Pipe 5A - 6A			1.87		91.8									0.84			2.71	2.71									
Contribution From EXTERNAL FUTURE RESIDENTIAL			2.73		233.0												2.73	5.44									
From External			0.20	3	10.2	4.80	335.0	4.00	5.43						0.84	0.09	0.20	5.64									
	6A	7A	0.21	3	10.2	5.01	345.2	4.00	5.59						0.84	0.09	0.21	5.85	1.346	7.03	82.5	200	0.40	20.74	0.34	0.66	0.60
From External			0.19	3	10.2	5.20	355.4	4.00	5.76						0.84	0.09	0.19	6.04									
	7A	10A	0.03			5.23	355.4	4.00	5.76						0.84	0.09	0.03	6.07	1.396	7.25	45.0	200	0.40	20.74	0.35	0.66	0.60
Contribution From STANZEL DRIVE, Pipe 9A - 10A			1.20		61.2												1.20	7.27									
From External			0.39	7	23.8	6.82	440.4	4.00	7.14						0.84	0.09	0.39	7.66									
	10A	11A	0.36	6	20.4	7.18	460.8	3.99	7.45						0.84	0.09	0.36	8.02	1.845	9.30	118.5	200	0.40	20.74	0.45	0.66	0.64
From External			0.04			7.22	460.8	3.99	7.45						0.84	0.09	0.04	8.06									
	11A	24A	0.03			7.25	460.8	3.99	7.45						0.84	0.09	0.03	8.09	1.861	9.45	47.5	200	0.40	20.74	0.46	0.66	0.64
Contribution From RIDELL STRET, Pipe 23A - 24A			6.23		422.5												6.23	14.32									
Contribution From EXTERNAL FUTURE RESIDENTIAL			7.37		663.0									0.42			7.79	22.11									
From External			0.05			20.90	1546.3	3.67	22.99						1.26	0.14	0.05	22.16									
	24A	25A	0.22			21.12	1546.3	3.67	22.99						1.26	0.14	0.22	22.38	5.147	28.28	61.0	300	0.40	61.16	0.46	0.87	0.85
Contribution From EXTERNAL FUTURE RESIDENTIAL			5.51		469.0												5.51	27.89									
Contribution From EXTERNAL FUTURE RESIDENTIAL			39.39		2047.0	As per MSS report by Novatech dated July 2013											39.39	67.28									
Contribution From EXTERNAL FUTURE COMMERCIAL										2.47	As per MSS report by Novatech dated July 2013					1.5	2.47	69.75									
From External			0.06			66.08	4062.3	3.33	54.80		2.47				1.26	2.39	0.06	69.81									
	25A	34A	0.22			66.30	4062.3	3.33	54.80		2.47				1.26	2.39	0.22	70.03	16.107	73.30	66.5	375	0.40	110.89	0.66	1.00	1.07
To SANITARY TRUNK , Pipe 34A - 35A						66.30	4062.3				2.47				1.26												

DESIGN PARAMETERS

Average Daily Domestic Flow =	350 l/p/day		Light Industrial Peak Factor = as per MOE Graph	
Commercial Flow =	50000 L/ha/day		Infiltration Rate =	0.23 L/s/Ha
School Flow =	50000 L/ha/day		Minimum Velocity =	0.6 m/s
Light Industry Flow =	35000 L/ha/day		Maximum Velocity =	3.0 m/s
Park Flow =	9300 L/ha/day		Manning's n =	0.013
Max Res. Peak Factor =	4.00		Single Unit coeff=	3.4
Commercial/School Peak Factor =	1.50		Townhouse Unit coeff=	2.7
			Average Condo Unit coeff=	1.8

LICENSED PROFESSIONAL ENGINEER
K. MITIC
100122340
MAR 22, 2016
PROVINCE OF ONTARIO

Designed:	P.P	PROJECT:	Miller's Crossing	
Checked:	K.M.	LOCATION:	Carleton Place	
Dwg. Reference: Sanitary Drainage Plan, Dwgs. No. 12 and 13		File Ref: 13-693	March, 2016	Sheet No. 2 of 3

693_San2.xls

SANITARY SEWER CALCULATION SHEET

CARLETON PLACE
Meet me on the Mississippi

LOCATION			RESIDENTIAL AREA AND POPULATION							COMMERCIAL		INSTITUTIONAL		PARK		C+I+P	INFILTRATION				PIPE						
STREET	FROM M.H.	TO M.H.	AREA (ha)	UNITS	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	TOTAL FLOW (l/s)	DIST (m)	DIA Nominal (mm)	SLOPE (%)	CAP. (FULL) (l/s)	RATIO Qact/Qcap	VEL (FULL) (m/s)	VEL (ACT.) (m/s)
SANITARY TRUNK																											
Contribution From RATHWELL STREET, Pipe 25A - 34A			66.30		4062.3					2.47				1.26			70.03	70.03									
Contribution From STOKES DRIVE, Pipe 33A- 34A			25.55		1589.5					19.46							45.01	115.04									
From External			0.05			91.90	5651.8	3.20	73.26		21.93				1.26	20.73	0.05	115.09									
	34A	35A	0.01			91.91	5651.8	3.20	73.26		21.93				1.26	20.73	0.01	115.10	26.473	120.46	36.0	525	0.15	166.56	0.72	0.77	0.84
	35A	36A	0.23			92.14	5651.8	3.20	73.26		21.93				1.26	20.73	0.23	115.33	26.526	120.52	127.5	525	0.15	166.56	0.72	0.77	0.84
	36A	37A	0.21			92.35	5651.8	3.20	73.26		21.93				1.26	20.73	0.21	115.54	26.574	120.56	121.0	525	0.15	166.56	0.72	0.77	0.84
	37A	MH-D (B.O.)	0.07			92.42	5651.8	3.20	73.26		21.93				1.26	20.73	0.07	115.61	26.590	120.58	32.5	525	0.15	166.56	0.72	0.77	0.84
	MH-D (B.O.)	MH-BC (B.O.)				92.42	5651.8	3.20	73.26		21.93				1.26	20.73	0.00	115.61	26.590	120.58	30.0	525	0.30	235.55	0.51	1.09	1.09
To PUMPING STATION						92.42	5651.8				21.93				1.26												

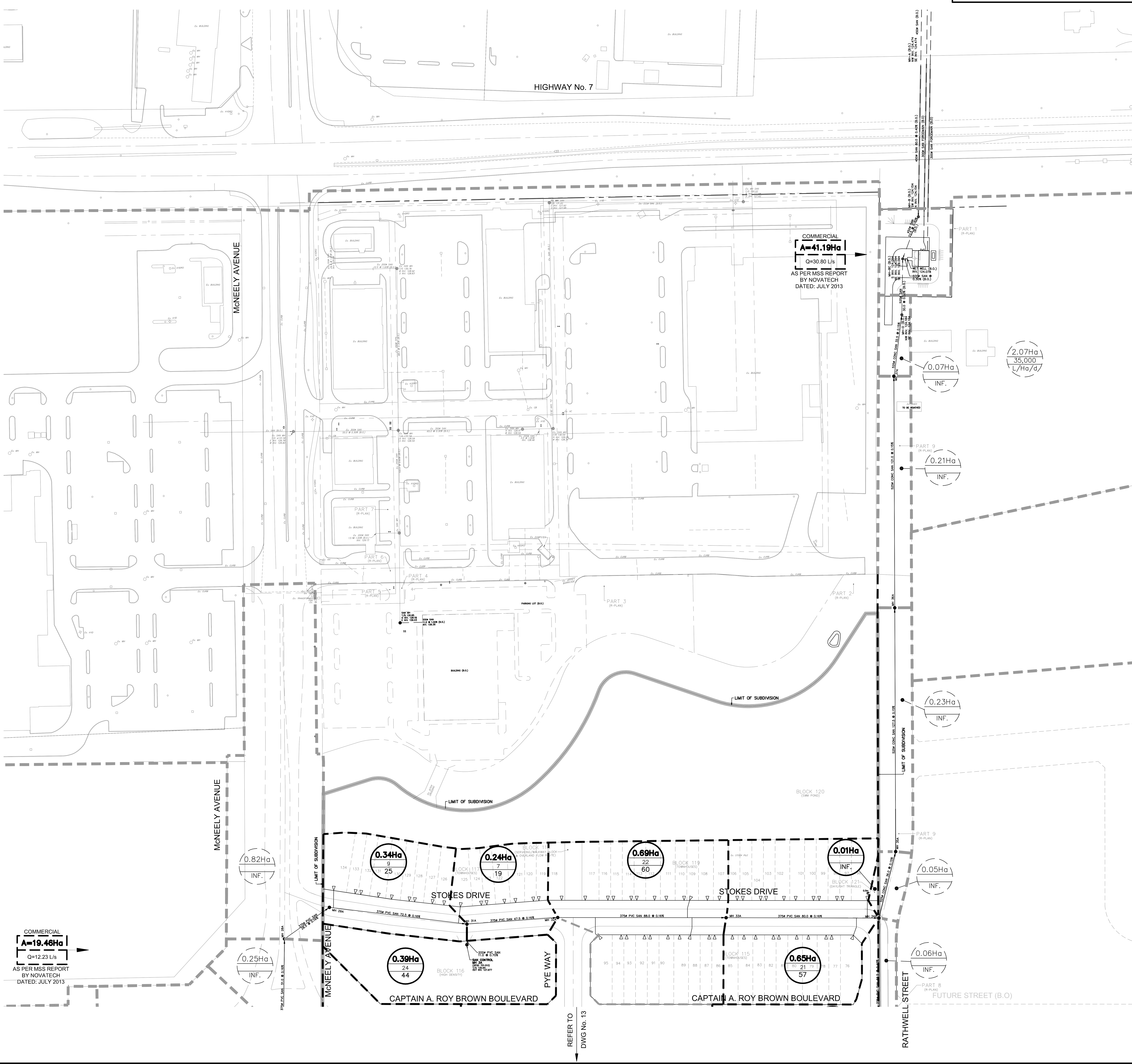
LICENSED PROFESSIONAL ENGINEER
K. MITIC
100122349
MAR 22, 2016
PROVINCE OF ONTARIO

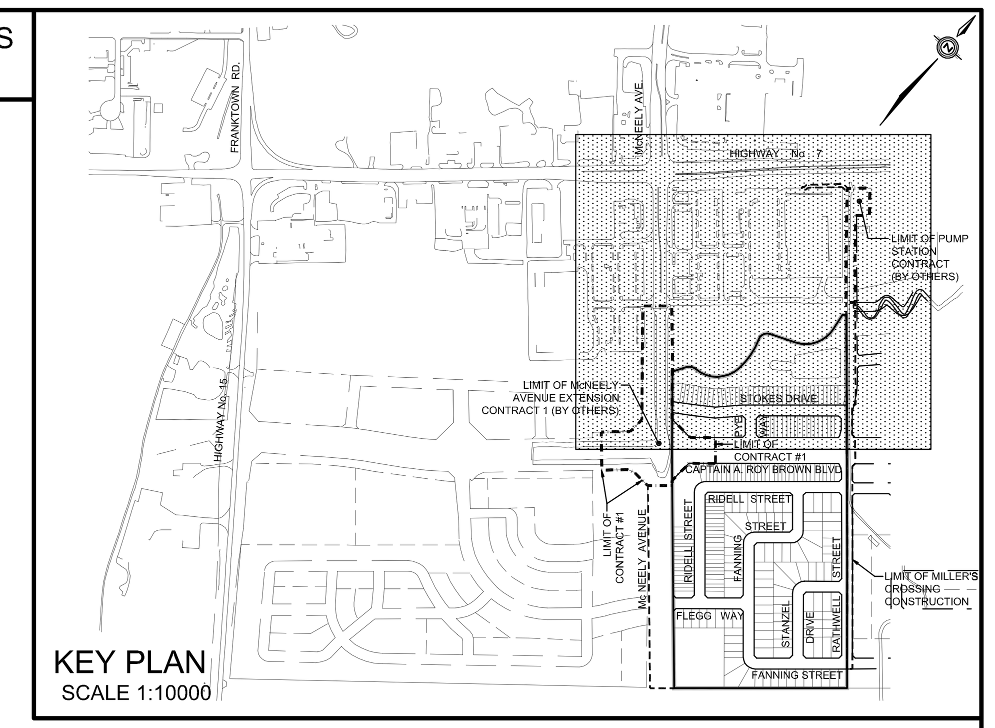
DESIGN PARAMETERS

Average Daily Domestic Flow =	350 l/p/day	Light Industrial Peak Factor = as per MOE Graph	
Commercial Flow =	50000 L/ha/day	Infiltration Rate =	0.23 L/s/Ha
School Flow =	50000 L/ha/day	Minimum Velocity =	0.6 m/s
Light Industry Flow =	35000 L/ha/day	Maximum Velocity =	3.0 m/s
Park Flow =	9300 L/ha/day	Manning's n =	0.013
Max Res. Peak Factor =	4.00	Single Unit coeff=	3.4
Commercial/School Peak Factor =	1.50	Townhouse Unit coeff=	2.7
		Average Condo Unit coeff=	1.8

Designed: P.P	PROJECT: Miller's Crossing			
Checked: K.M.	LOCATION: Carleton Place			
Dwg. Reference: Sanitary Drainage Plan, Dwgs. No. 12 and 13	File Ref:	13-693	March, 2016	Sheet No. 3 of 3

ALL DWELLINGS ARE TO BE PROVIDED WITH SUMP PUMPS UNLESS OTHERWISE NOTED. SEE DETAIL ON DWG. 44.





KEY PLAN
SCALE 1:10000

LEGEND

0.82Ha / 23 / 63	DRANAGE AREA IN HECTARES NUMBER OF UNITS POPULATION (2.5 PERSON PER UNIT FOR SINGLE HOUSE)
8.19Ha	DRAINAGE AREA IN HECTARES
18.1Ha / 1493	EXTERNAL DRANAGE AREA POPULATION
●	SANITARY MANHOLE
∇	SANITARY SINGLE HOUSE CONNECTION
—	SANITARY SINGLE HOUSE CONNECTION
	SANITARY SEWER TRIBUTARY BOUNDARY
	SANITARY SEWER SUB TRIBUTARY BOUNDARY
	EXTERNAL SANITARY SEWER TRIBUTARY BOUNDARY
	LIMIT OF BTE CONTRACT 1 AND PUMP STATION CONTRACT LIMIT
	PHASE LINE
	LIMIT OF UNDERGROUND CONSTRUCTION

TOPOGRAPHIC INFORMATION

TOPOGRAPHIC INFORMATION PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD., JOB No. 15742–15, SURVEY DATED MARCH 19, 2015.

LEGAL INFORMATION

CALCULATED M–PLAN PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD., JOB No. 16079–15, RECEIVED ON FEBRUARY 23, 2016.
CALCULATED R–PLAN PROVIDED BY G. A. SMITH SURVEYING LTD. PROJECT No. 15–4011, RECEIVED JULY 21, 2015

2nd SUBMISSION 16–03–22

NOT FOR CONSTRUCTION

JOB BENCHMARK 1 — ELEVATION = 132.230 m
CUT CROSS ON LIGHT STANDARD

JOB BENCHMARK 2 — ELEVATION = 132.060 m
TOP OF SPINDLE ON FIRE HYDRANT

No.	DATE	BY	DESCRIPTION
2.	16–03–22	K.M.	2nd SUBMISSION
1.	15–12–14	K.M.	1st SUBMISSION

CARLETON PLACE — Meet me on the Mississippi

LANARK COUNTY

PROJECT No. 13–693

LICENSED PROFESSIONAL ENGINEER
K. MITIC
100122349
16-03-22
PROVINCE OF ONTARIO

SANITARY DRAINAGE PLAN

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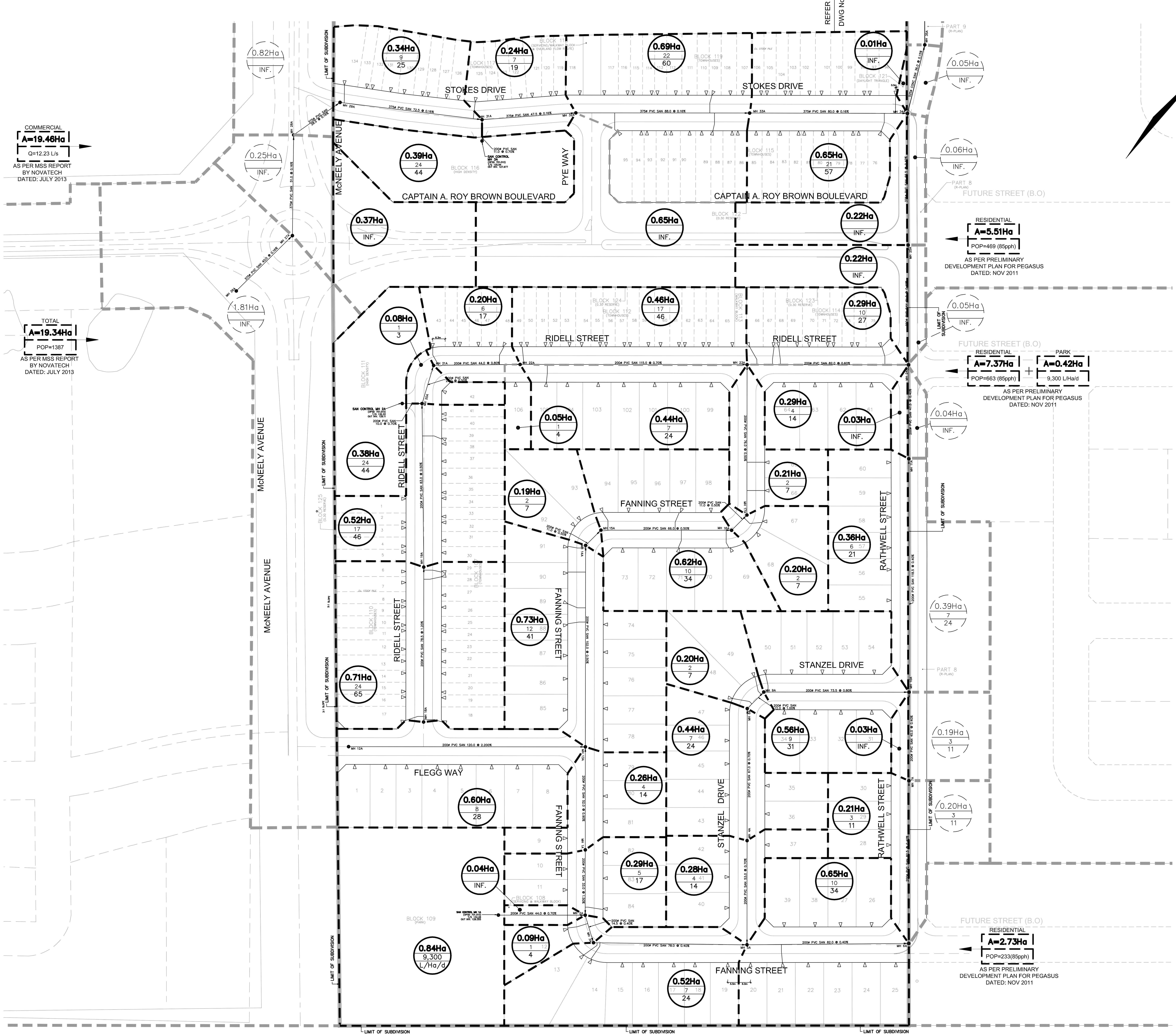
CARDEL HOMES

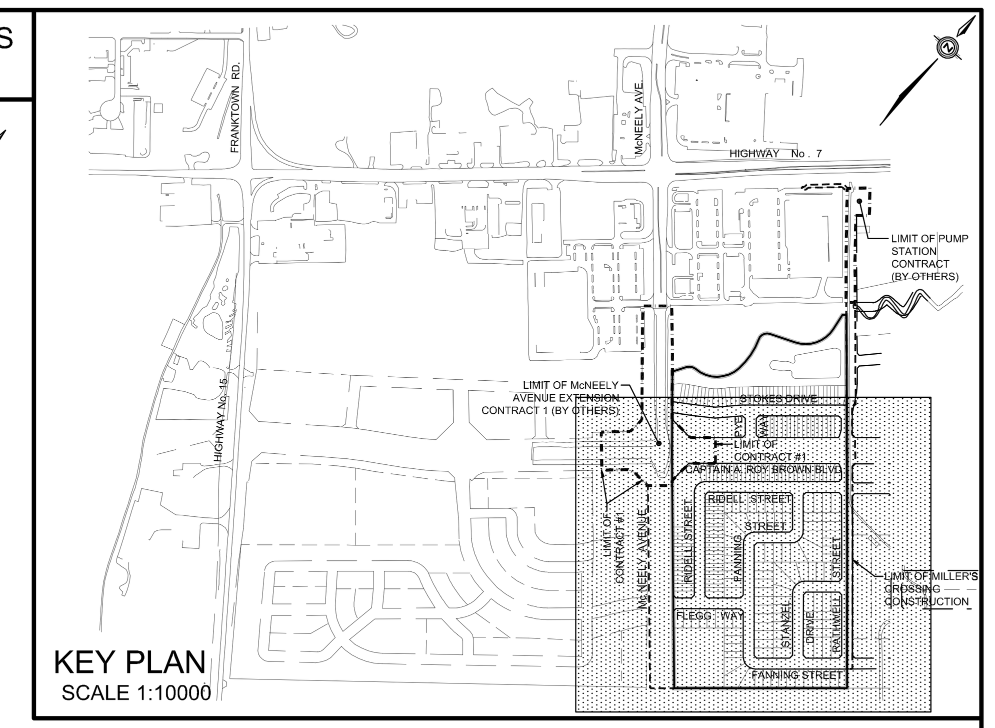
MILLER'S CROSSING

DSEL
david schaeffer engineering ltd
120 Iber Road, Unit 103
Stittsville, ON K2S 1E9
Tel. (613) 836-0856
Fax. (613) 836-7183
www.DSEL.ca

DRAWN BY:	E.G./M.Z.	CHECKED BY:	K.M.	DRAWING NO.	SHEET NO.
DESIGNED BY:	P.P.	CHECKED BY:	K.M.		12
SCALE:	1:1000	DATE:	DECEMBER 2015		

ALL DWELLINGS ARE TO BE PROVIDED WITH SUMP PUMPS UNLESS OTHERWISE NOTED. SEE DETAIL ON DWG. 44.





KEY PLAN
SCALE 1:10000

LEGEND

0.82Ha / 23 / 63	DRANAGE AREA IN HECTARES NUMBER OF UNITS POPULATION (2.5 PERSON PER UNIT FOR SINGLE HOUSE)
8.19Ha	DRAINAGE AREA IN HECTARES
18.1Ha / 1493	EXTERNAL DRANAGE AREA POPULATION
●	SANITARY MANHOLE
∇	SANITARY SINGLE HOUSE CONNECTION
—	SANITARY SINGLE HOUSE CONNECTION
	SANITARY SEWER TRIBUTARY BOUNDARY
	SANITARY SEWER SUB TRIBUTARY BOUNDARY
	EXTERNAL SANITARY SEWER TRIBUTARY BOUNDARY
	LIMIT OF BTE CONTRACT 1 AND PUMP STATION CONTRACT LIMIT
	PHASE LINE
	LIMIT OF UNDERGROUND CONSTRUCTION

TOPOGRAPHIC INFORMATION

TOPOGRAPHIC INFORMATION PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD., JOB No. 15742–15, SURVEY DATED MARCH 19, 2015.

LEGAL INFORMATION

CALCULATED M–PLAN PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD., JOB No. 16079–15, RECEIVED ON FEBRUARY 23, 2016.
CALCULATED R–PLAN PROVIDED BY G. A. SMITH SURVEYING LTD. PROJECT No. 15–4011, RECEIVED JULY 21, 2015

2nd SUBMISSION 16–03–22

NOT FOR CONSTRUCTION

JOB BENCHMARK 1 CUT CROSS ON LIGHT STANDARD	ELEVATION = 132.230 m
JOB BENCHMARK 2 TOP OF SPINDLE ON FIRE HYDRANT	ELEVATION = 132.060 m

No.	DATE	BY	DESCRIPTION
2.	16–03–22	K.M.	2nd SUBMISSION
1.	15–12–14	K.M.	1st SUBMISSION

CARLETON PLACE — Meet me on the Mississippi
LANARK COUNTY

K. MITIC
100122349
16-03-22
LICENSED PROFESSIONAL ENGINEER
PROVINCE OF ONTARIO

PROJECT No. 13–693

SANITARY DRAINAGE PLAN

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CARDEL HOMES

MILLER'S CROSSING

DSEL
david schaeffer engineering ltd
120 Iber Road, Unit 103
Stittsville, ON K2S 1E9
Tel. (613) 836-0856
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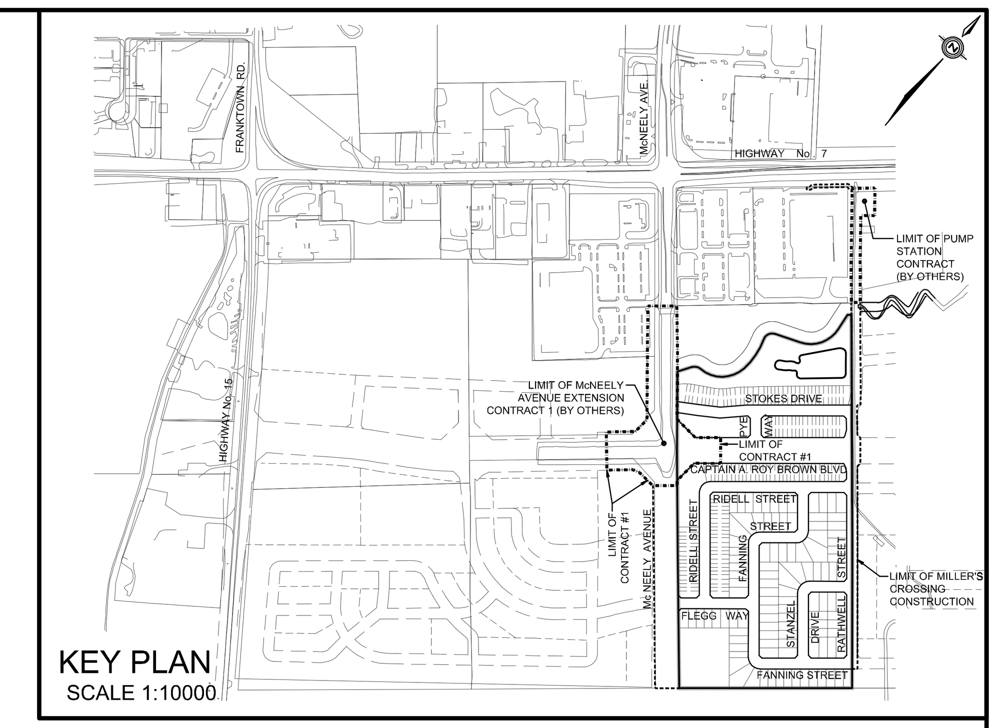
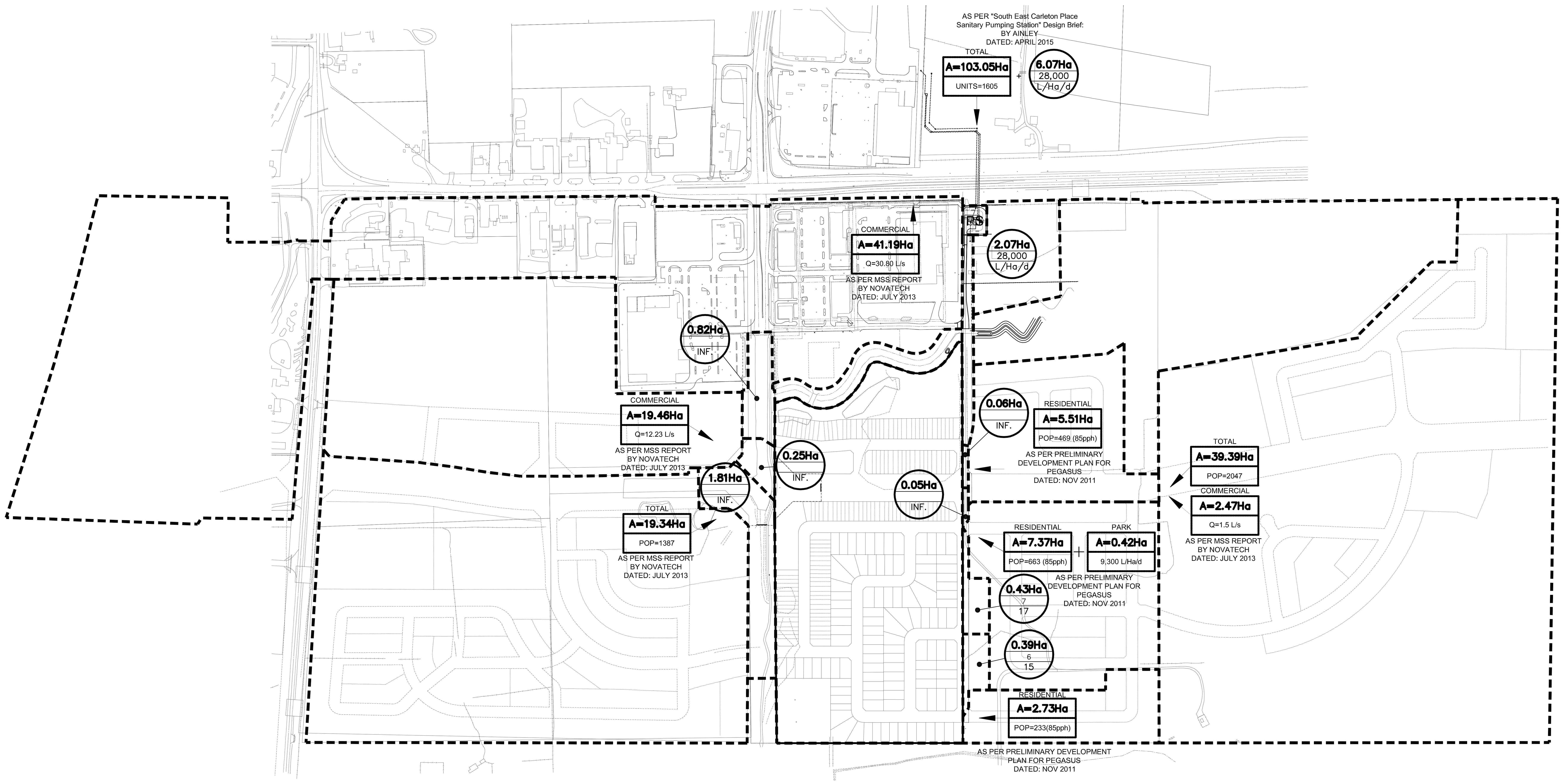
DRAWN BY: E.G./M.Z.	CHECKED BY: K.M.	DRAWING NO.		SHEET NO.
DESIGNED BY: P.P.	CHECKED BY: K.M.			13
SCALE: 1:1000	DATE: DECEMBER 2015			



LEGEND

0.82Ha / 135 / 111	AREA IN HECTARES POPULATION DENSITY (PERSONS PER HECTARE) POPULATION
- - - -	EXTERNAL SANITARY TRIBUTARY BOUNDARY



EXTERNAL SANITARY
DRAINAGE PLAN

TOPOGRAPHIC INFORMATION
TOPOGRAPHIC INFORMATION PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD., JOB No. 15742-15, SURVEY DATED MARCH 19, 2015.

LEGAL INFORMATION
CALCULATED M-PLAN PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD., JOB No. 16079-15, RECEIVED ON FEBRUARY 23, 2016.
CALCULATED R-PLAN PROVIDED BY G. A. SMITH SURVEYING LTD. PROJECT No. 15-4011, RECEIVED JULY 21, 2015

2nd SUBMISSION 16-03-22

NOT FOR CONSTRUCTION

JOB BENCHMARK 1 CUT CROSS ON LIGHT STANDARD	ELEVATION = 132.230 m
JOB BENCHMARK 2 TOP OF SPINDLE ON FIRE HYDRANT	ELEVATION = 132.060 m

No.	DATE	BY	DESCRIPTION
2.	16-03-22	K.M.	2nd SUBMISSION
1.	15-12-14	K.M.	1st SUBMISSION

CARLETON PLACE — Meet me on the Mississippi
LANARK COUNTY
LANARK COUNTY

PROJECT No. 13-693

LICENSED PROFESSIONAL ENGINEER
K. MITIC
100122349
16-03-22
PROVINCE OF ONTARIO

EXTERNAL SANITARY DRAINAGE PLAN
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DRAWN BY:	E.G./M.Z.	CHECKED BY:	K.M.	DRAWING NO.	SHEET NO.
DESIGNED BY:	P.P.	CHECKED BY:	K.M.		14
SCALE:	1:4000	DATE:	DECEMBER 2015		

Appendix E
Water Demand Calculations and Hydraulic Modeling

The proposed watermains for the development can meet the minimum required flushing velocity of 0.8 m/s (for cleaning and disinfection procedures) using conventional flushing methods with one 2.5 inch port open. Hydrants which will be installed on Stanzel Drive should have two 2.5 inch ports open using conventional flushing, while hydrants near McNeely Avenue/Captain A Roy Brown and McNeely Avenue/ Flegg Way require unidirectional flushing with two 2.5 inch ports open to satisfy the required minimum flushing velocity (WSP, October 2015).

Because of the location of the subdivision at the limit of the Town's network, and due to the incremental construction phasing plan that is anticipated and the unknown timing of the future trunk watermain connections defined in the MSSMR, water quality should be confirmed prior to occupancy. A water circulation program may be required to be developed until enough homes are drawing from the network to prevent stagnation within the Miller's Crossing watermain network.

Table 4: Water Supply Design Criteria (MSSMR & MOE Guidelines)

Design Parameter	Value
Residential - Single Family	3.4p/unit
Residential – Townhome/ Semi	2.7p/unit
Residential – Apartment	1.8p/unit
Residential – Average Daily Demand	350L/p/day
Residential - Maximum Daily Demand	2.0 x Average Daily Demand
Residential - Maximum Hourly Demand	3.0 x Average Daily Demand
Residential – Minimum Daily Demand	Average Daily Demand
Commercial/Institutional – Average Day	28,000 L/ha/day
Commercial/Institutional – Maximum Daily Demand	1.5 x Average Daily Demand
Commercial/Institutional – Maximum Hourly Demand	2.7 x Average Daily Demand
Commercial/Institutional – Minimum Daily Demand	Average Daily Demand
Fire Flow	Calculated as per the Fire Underwriter's Survey, 1999
Minimum Watermain Size	150mm diameter
Service Lateral Size	19mm dia. Soft Copper Type 'K' or 25mm dia. PEX
Minimum Depth of Cover	2.4m from top of watermain to finished grade
Peak hourly demand operating pressure	275.8 kPa and 551.6 kPa
Fire flow operating pressure minimum	137.9 kPa

Note that although the individual high density and community centre blocks were incorporated into the hydraulic model, it is expected that these sites will be further evaluated as detailed plans for these sites are developed.

151-05603-00

October 28th, 2015

Ms. Laura Maxwell
David Schaeffer Engineering Ltd.
600 Alden Rd, no. 500
Markham, ON, L3R 0E7

Re: Miller’s Crossing Development
Watermain Hydraulic Analysis

Dear Ms. Maxwell:

WSP Canada Inc. (WSP) is pleased to present the results of its watermain analysis for the proposed Miller’s Crossing Development in the Town of Carleton Place.

The analysis in this report includes individual hydraulic examination of the Average Day demand, Maximum Day plus fire flow and the Maximum (Peak) Hour demand of the proposed development for present development conditions. The hydraulic analysis was completed using a WaterCAD model of the proposed development with boundary conditions provided by J. L. Richards and Associates Limited.

If you have any questions, please do not hesitate to call.

Yours truly,

WSP Canada Inc.

Jean-Luc Daviau, M.A.Sc., P.Eng.
Sr. Hydraulic Specialist,
Manager, Hydraulics

Gian Carlo Manigbas, E.I.T.
Hydraulic Modeller

WSP Canada Inc.
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FIGURES

APPENDICES

1 INTRODUCTION

The proposed Miller's Crossing Development is located south of Trans-Canada Hwy (Hwy 7) and east of the future extension of McNeely Avenue in the Town of Carleton Place, Ontario. The purpose of this report is to examine the water servicing capacity of the proposed development, which consists of single-family dwellings, townhome units, high density residential uses and an associated park. A plan showing the development location is provided in Figure 1-1

The proposed development will be serviced by connection to the existing 250 mm diameter watermain on a commercial area located south-east of the intersection of Hwy 7 and McNeely Avenue. The proposed sizes of the pipes and water distribution system layout for the development are labeled on Figure 1-2.

Figure 1-1 Miller's Crossing Development Location Plan

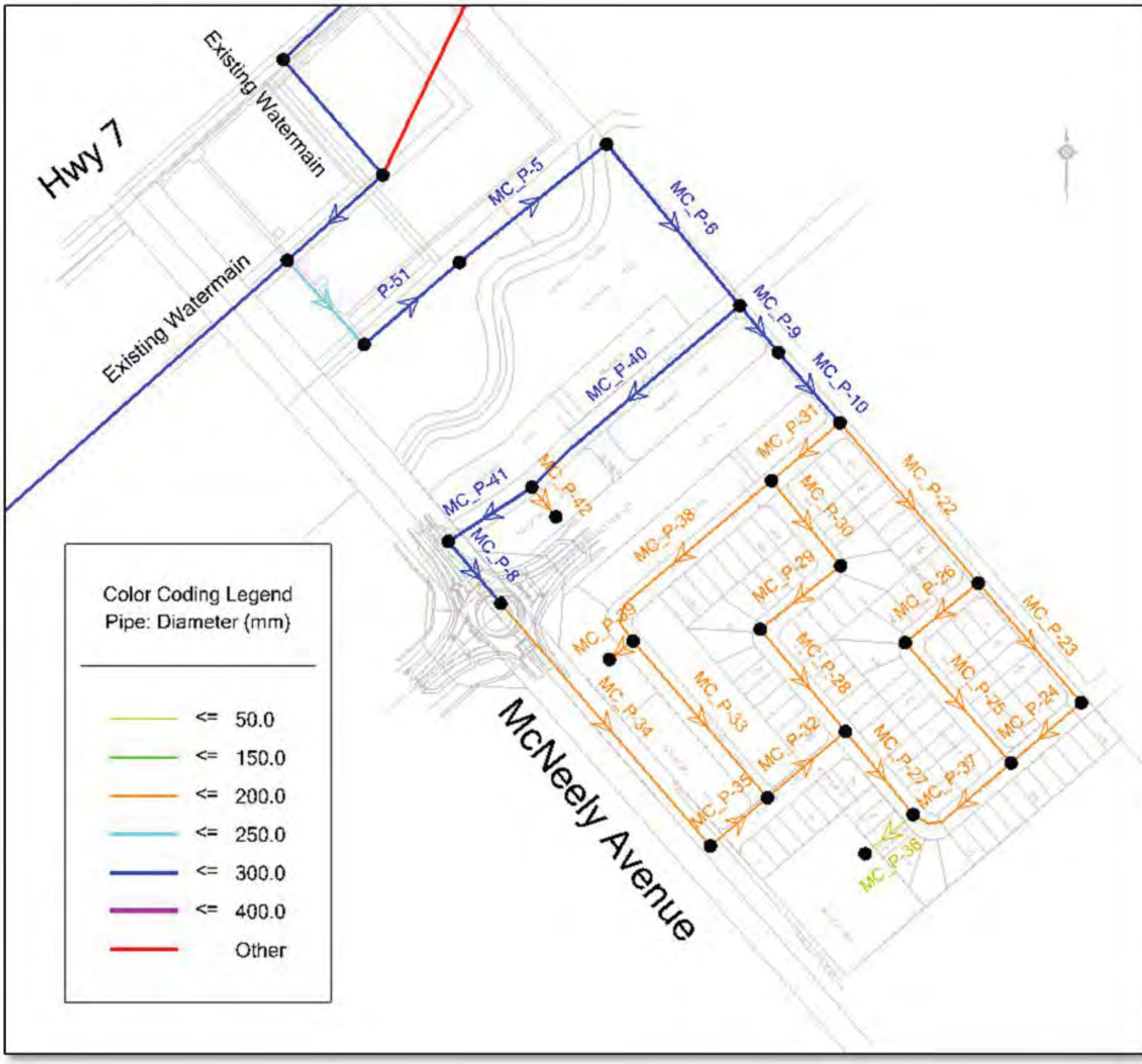


Figure 1-2 Proposed Water Distribution System Layout

2 CRITERIA

2.1 DOMESTIC DEMAND

Demands for the Miller's Crossing Development were calculated using the design criteria used from the Highway 7 South, Town of Carleton Place Master Servicing and Stormwater Management Report in determining the size of the watermains. Table 2-1 lists the factors used to determine the demands for the development.

Table 2-1 Demand Factors and Inputs

DEMAND FACTORS AND INPUTS	VALUE
Single-Family	3.4 ppu
Semi-Detached/Townhomes	2.7 ppu
Apartment	1.8 ppu
Average Day Demand	350 L/Person/day
Maximum Day Peaking Factor	2.0
Peak Hour Factor	3.0

Detailed calculations of domestic demands are shown in Appendix A. Residential demands were calculated by counting the number of units on the site plan shown in Figure A-1 and allocating the demands of each unit to the closest node in the water model.

2.2 MAXIMUM AND MINIMUM OPERATING PRESSURES

According to the Ministry of Environment (MOE) Design Guidelines for Drinking Water Systems, 2008; the normal operating pressure in the distribution system should be approximately 350 kPa (50 psi) to 480 kPa (70 psi) and not less than 275 kPa (40 psi). Pressures outside this range may be dictated by distribution system size and/or topography.

The maximum pressures in the distribution system should not exceed 690 kPa (100 psi) to avoid damage to household plumbing and unnecessary water and energy consumption. When static pressure exceeds 690 kPa (100 psi), pressure reducing devices should be provided on mains or service connections in the distribution system.

2.3 FIRE FLOW DEMAND

The fire flows used in the model were calculated in accordance with the 2012 Ontario Building Code (OBC) and with reference to "Water Supply for Public Fire Protection" 1999 by the Fire Underwriters Survey (FUS).

According to Section A-3.2.5.7 of the 2012 OBC, except if sprinklered, buildings should have a supply of water available for firefighting purposed not less than the quantity derived from the following formula:

$$Q = K \bullet V \bullet S_{tot}$$

where

Q = minimum supply of water in liters

K = water supply coefficient (see appendix A)

V = total building volume in cubic meters

S_{tot} = total of spatial coefficient values from property line exposures on all sides as obtained from the formula:

$$S_{tot} = 1.0 + [\, S_{side1} + S_{side2} + S_{side3} + \ldots \text{ etc. }]$$

The required fire flow for the largest building within the proposed development was calculated to be 150 L/s (9,000 L/min), which is the worst case scenario. This is higher than the minimum required fire flow of 80 L/s (4,800 L/min) for residential areas under the FUS.

Detailed calculations of fire flow requirements are contained in Appendix C. In accordance with the OBC and with the FUS, a minimum residual pressure of 140 kPa must be maintained within the distribution system under Maximum Day Demand plus fire flow conditions.

3 HYDRAULIC MODELING

3.1 BOUNDARY CONDITIONS

The boundary conditions used in the model were provided by J. L. Richards and Associates Limited. The proposed residential development was simulated in the Town of Carleton Place's existing hydraulic water model based on the calculated theoretical water demands by WSP. Hydraulic boundary conditions have been determined at the proposed connection location to the existing 300 mm diameter watermain for a single feed The connection location correspond to node J-859 as shown in Figure 2-1.

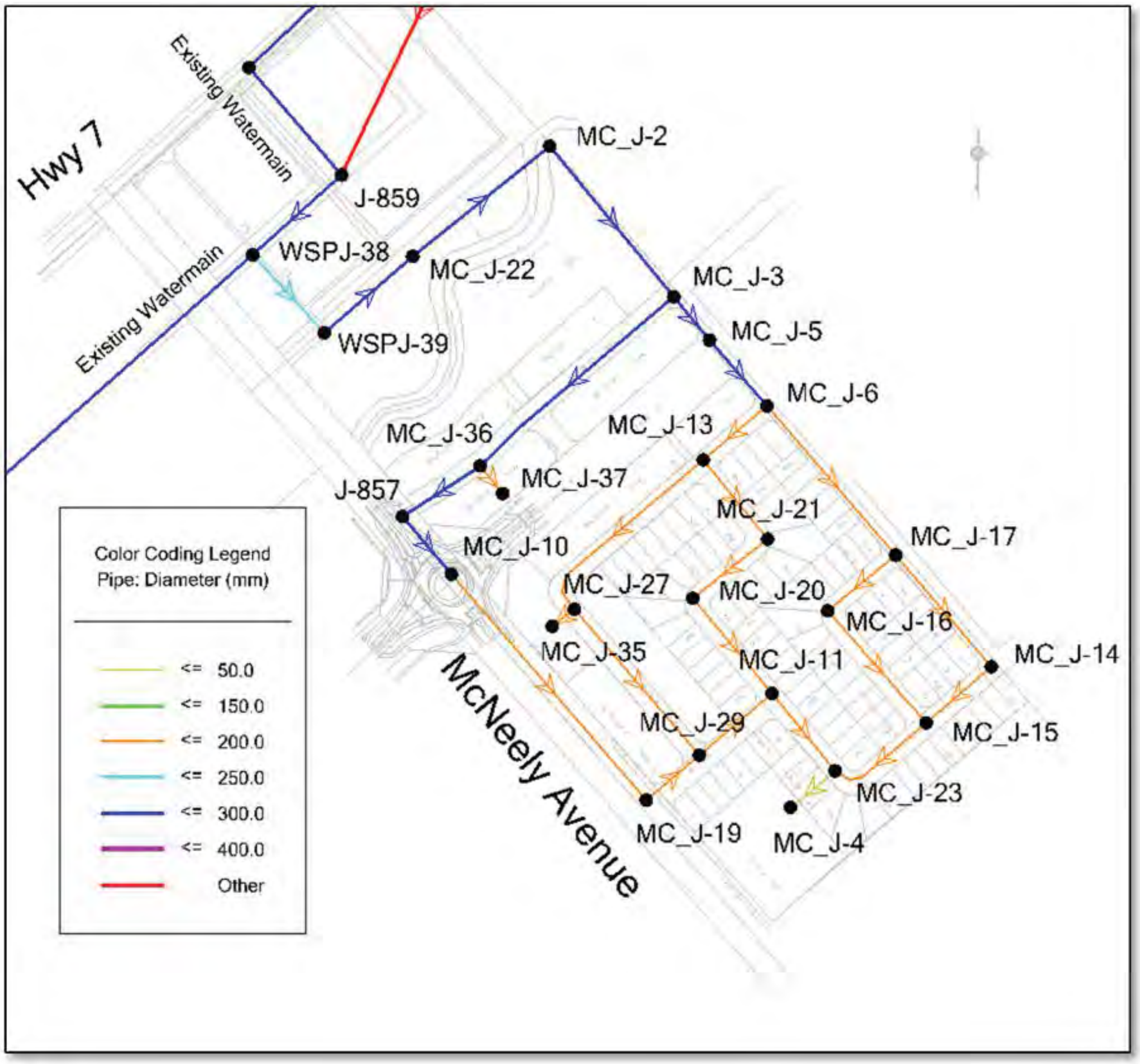


Figure 2-1 Proposed System Layout – Nodes

Water demands associated with the Miller's Crossing Development were applied at the junction at an assumed ground elevation of 130 m per background information sent by WSP. WaterCAD results of the hydraulic simulation of the existing model are shown in Table 3-1.

Table 3-1 Boundary Conditions

SCENARIOS	CONNECTION LOCATION (J-859) PRESSURE (KPA)	HGL (M)	GROUND ELEVATION (M)	ELEVATED TANK WATER LEVEL (M)
Existing Average Day + 3.21 L/s	498	181.23	130.30	181.10
Maximum Day + 6.42 L/s	497	181.08	130.30	181.10
Maximum Day + 150 L/s Fire Flow + 6.42 L/s	415	172.73	130.30	181.10
Peak Hour + 9.63 L/s	494	180.83	130.30	181.10

3.2 MILLER'S CROSSING SUBDIVISION MODEL

A model of the proposed development and surrounding watermains was created using WaterGEMS V8i software. Elevation information of the proposed development was taken from a grading plan prepared by David Schaeffer Engineering Limited. Junction elevations were taken as the finished grade elevation at the centerline of road or applicable adjacent proposed grade point. Service elevations of the development range from approximately 130.25 m to 135 m.

Friction Factors for all new pipes added to the model were assigned according to the Ministry of the Environment (MOE) watermain Design Criteria as listed in Table 3-2.

Table 3-2 Hazen-Williams C-Factors

DIAMETER - NOMINAL	C-FACTOR
150mm	100
200mm	110
300mm to 600mm	120

4 ANALYSIS

The proposed watermains within the proposed development were sized to satisfy the greater of either Peak Hour or Maximum Day plus Fire Flow demands. Modeling was carried out for Average Day, Maximum Day plus Fire Flow and Peak Hour conditions under the present demand scenarios using a WaterGEMS V8i model of the proposed development.

4.1 SYSTEM PRESSURES AND AVAILABLE FIRE FLOW

As outlined in the Ministry of Environment (MOE), the acceptable pressures under normal conditions are between 275 kPa (40 psi) and 690 kPa (100 psi). The minimum allowable pressure under maximum day demand plus fire flow is 140 kPa (20 psi) at the location of the fire.

Modeled service pressures at steady state for the proposed development are shown in the tables below.

Table 4-1 Model Results Summary

SCENARIOS	PRESSURE (KPA)
Average Day	443 - 502
Maximum Day	442 - 501
Peak Hour	439 - 490

The minimum modeled available fire flow obtained under the Maximum Day plus Fire scenario is approximately 165 L/s (node MC J-4), which is greater than the required fire flow of 150 L/s as shown in Table 4-2. Required fire flows will be met at hydrants near the junctions shown in Figure 3-1.

Table 4-4 Modeled Available Fire Flows under Max Day + Fire Flow while maintaining a minimum pressure of 140 kPa at all points in the distribution system.

NODE ID	REQUIRED FIRE FLOW (L/S)	AVAILABLE FIRE FLOW (L/S)
J-857	150	284
MC_J-3	150	326
MC_J-4	150	165
MC_J-5	150	312
MC_J-6	150	296
MC_J-10	150	274
MC_J-11	150	223
MC_J-13	150	248
MC_J-14	150	184
MC_J-15	150	197
MC_J-16	150	184
MC_J-17	150	203
MC_J-19	150	196
MC_J-20	150	204
MC_J-21	150	209
MC_J-23	150	199

NODE ID	REQUIRED FIRE FLOW (L/S)	AVAILABLE FIRE FLOW (L/S)
MC_J-27	150	194
MC_J-29	150	214
MC_J-35	150	176
MC_J-36	150	295
MC_J-37	150	191

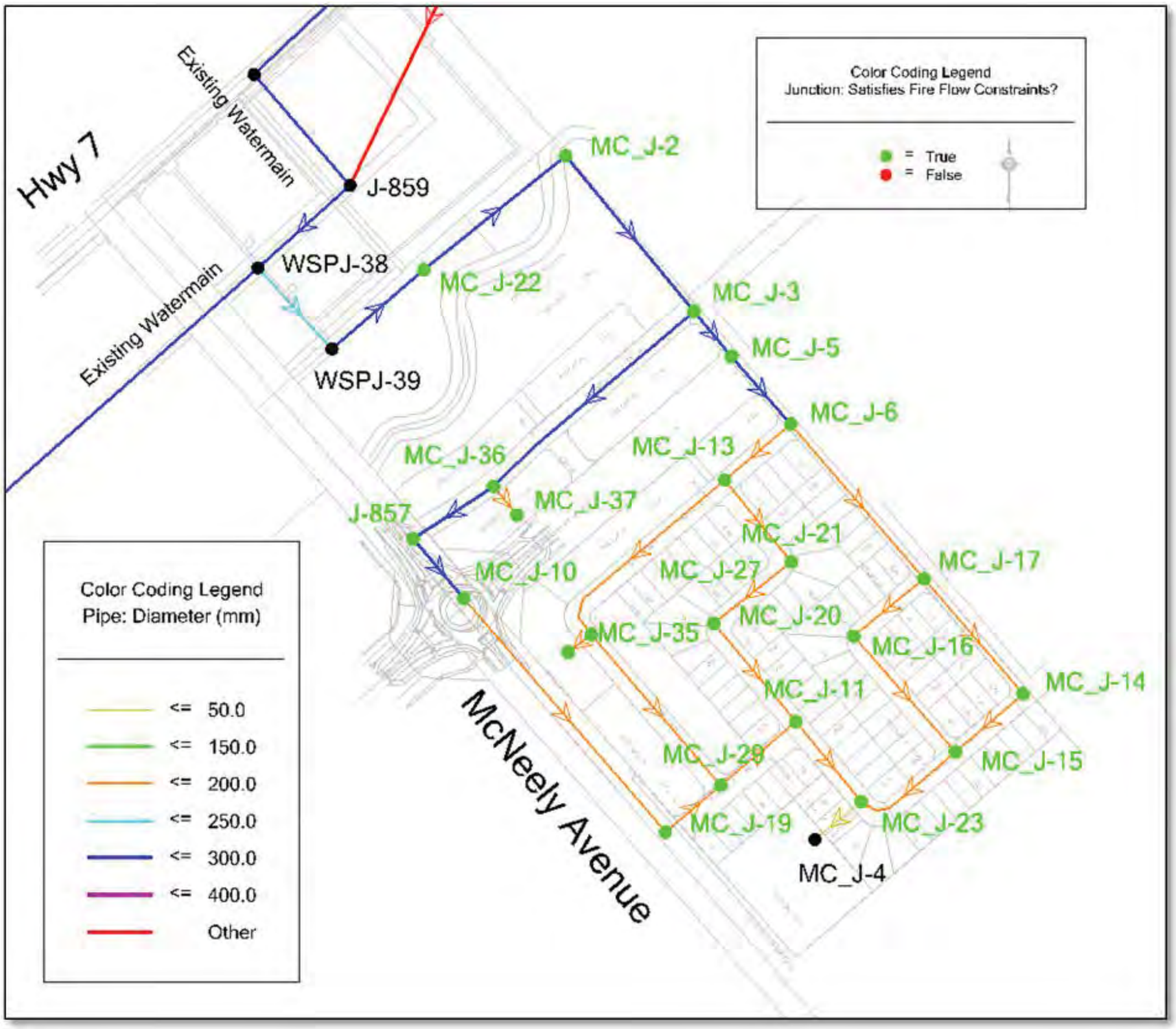


Figure 4-1 Pass/Fail under Maximum Day Demand + Fire Flow Scenario

These results indicate that the development can be adequately serviced by the proposed water distribution system layout.

Based on the modeled results shown in Appendix C, the proposed watermain addition does not adversely affect the distribution system's ability to maintain a minimum pressure of 140 kPa at ground level at all points in the in the distribution system under Maximum Day plus Fire Flow condition.

4.2 SYSTEM FLUSHING

A modeled flushing test was performed for the proposed water distribution network to determine the achievable flushing velocities of the system. The MOE watermain design criterion requires a minimum flushing velocity of 0.8 m/s for cleaning and disinfection procedures.

WaterGEMS software allows for testing of flushing by representing a modeled hydrant as a flow emitter with an emitter coefficient K equivalent to the components of the hydrant including the lateral, valve, bends and outlet. Hydrants were added to the model with a K value taken as 11.2 $l/s/m^{0.5}$ (150 $gpm/psi^{0.5}$) which is the minimum value prescribed by the American Water Works Association (AWWA) standard for flow calculations through a single 60 mm (2.5") outlet.

Flushing velocities using conventional flushing with one 2.5 inch port open were simulated between 0.80 and 5.23 m/s and therefore meet the required minimum flushing velocity of 0.8 m/s. However, hydrant/s which will be installed near junction MC_J-16 should have two 2.5 inch ports open, while hydrants near junctions MC_J-10 and MC_J-19 need unidirectional flushing with two 2.5 inch ports open to satisfy the required minimum flushing velocity. Figure 4-2 shows that the proposed system for the development can satisfy the minimum required flushing velocity. Appendix D includes a Flushing report showing each pipe and its conventional and unidirectional flushing velocities.

Figure 4-2 Pass/Fail under Average Day Demand Scenario

5 CONCLUSIONS

The proposed system layout for the Miller's Crossing Development can meet the expected hydraulic demands while remaining in compliance with the Ministry of Environment and the Town of Carleton Place's watermain design criteria, as summarised below:

1. Service pressures are expected to be a minimum of 439 kPa (Peak Hour) and a maximum of 502 kPa (Average Day). These pressures are within the MOE guidelines for water distribution systems;
2. The Required Fire Flow can be achieved under Maximum Day plus Fire Flow Scenarios. Under Maximum Day plus Required Fire Flows for the present demand condition within the proposed development, the distribution system is able to maintain pressures above 140 kPa at ground level at all points within the proposed development; and,
3. The proposed watermains for the development can meet the minimum required flushing velocity of 0.8 m/s using conventional flushing methods with one 2.5 inch port open. Hydrant/s which will be installed near junction MC_J-16 should have two 2.5 inch ports open using conventional flushing, while hydrants near junctions MC_J-10 and MC_J-19 need unidirectional flushing with two 2.5 inch ports open to satisfy the required minimum flushing velocity.
4. The results of hydraulic simulations of the expected pressures under all demand conditions show that the installation of pressure reducing valves in the residential buildings will not be required.

These conclusions remain valid as long as the proposed water distribution system and the Town's network configuration remain as described herein. If significant changes are contemplated, this analysis should be updated.

Appendix A

SITE PLAN, DEMANDS AND PROPOSED SYSTEM LAYOUT

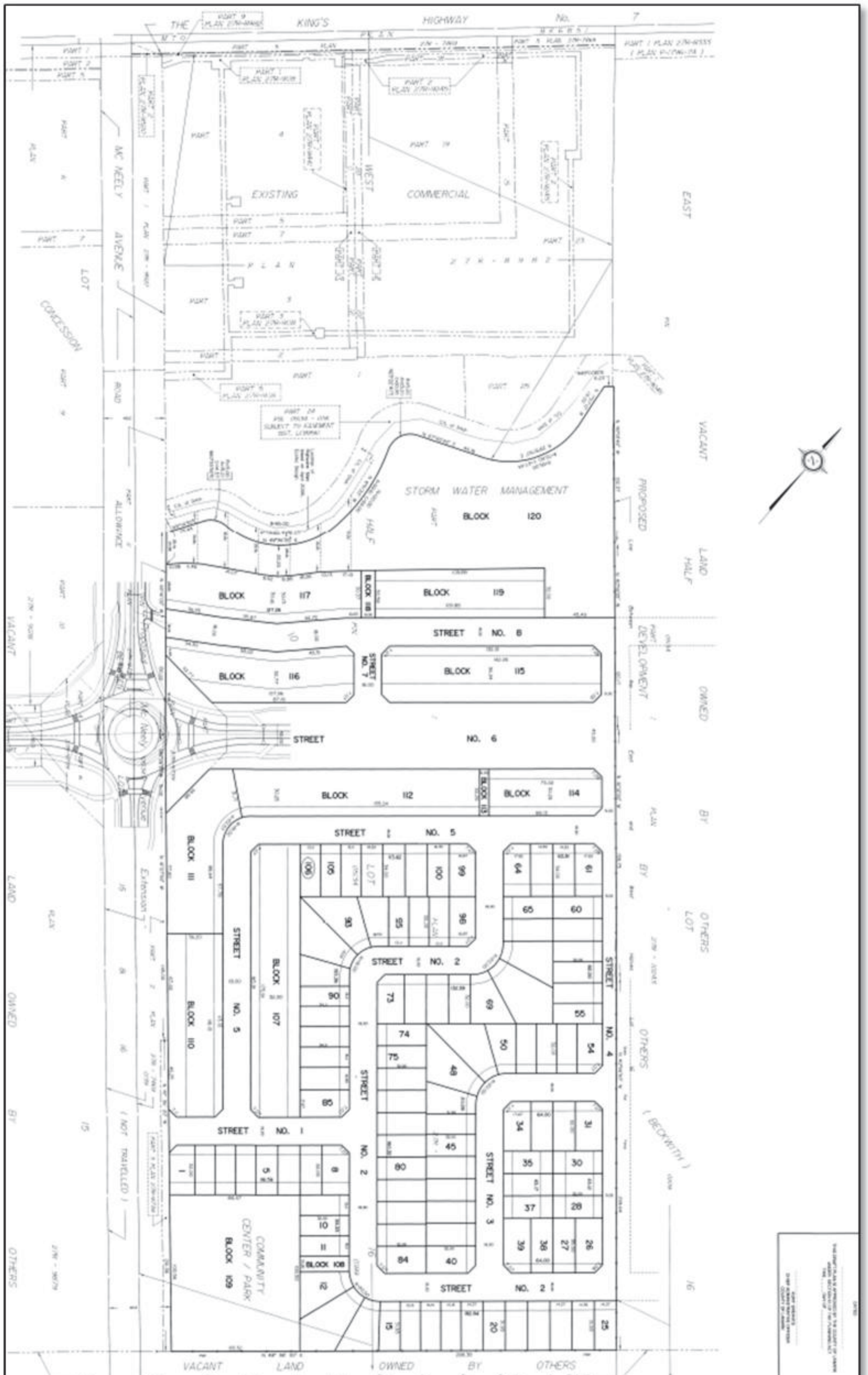


Figure A-1 Site Plan

Water Demands

Population			Average Demand			Peaking Factors	
Single Detached	3.4	persons/unit	Residential	350	L/ capita/ day	Maximum Day Residential	2
Townhome/Semi	2.7	persons/unit				Peak Hour Residential	3
Apartments	1.8	persons/unit					

Note: Persons per unit from Highway 7 South, Town of Carleton Place Master Servicing and Stormwater Management Report Section 5.2.1

Residential:

Node	Single Family Units	Townhomes	Apartments	Number of People	Average Day (L/day)	Average Day (L/s)	Maximum Day (L/s)	Peak Hour (L/s)
PHASE 1								
Singles	106			360.4	126140	1.46	2.92	4.38
Townhomes		128		345.6	120960	1.40	2.80	4.20
Apartments			48	86.4	30240	0.35	0.70	1.05
Total:	**106**	**128**	**48**	**792.4**	**277340**	**3.21**	**6.42**	**9.63**

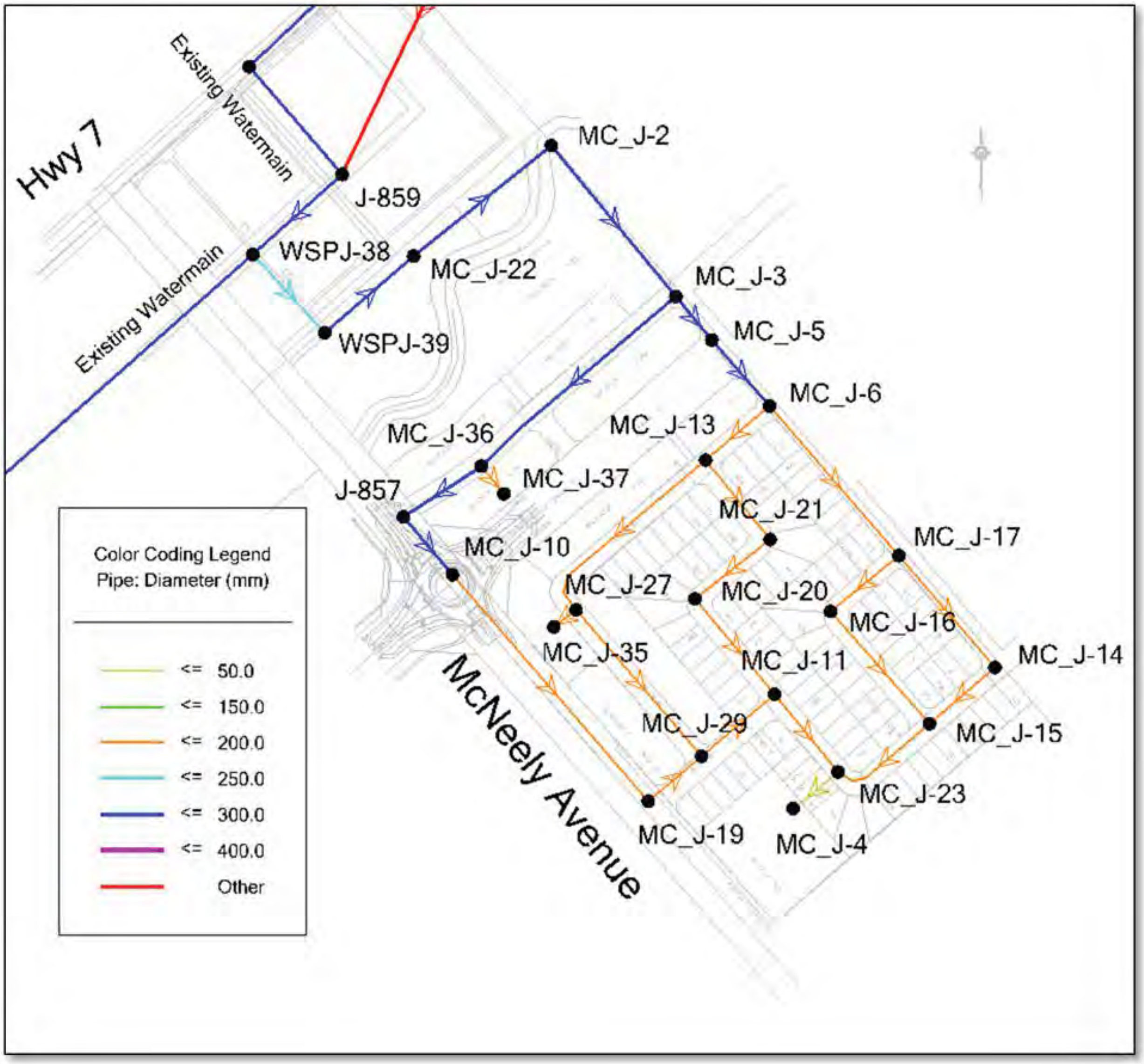


Figure A-2 Proposed Water Distribution System Layout – Nodes

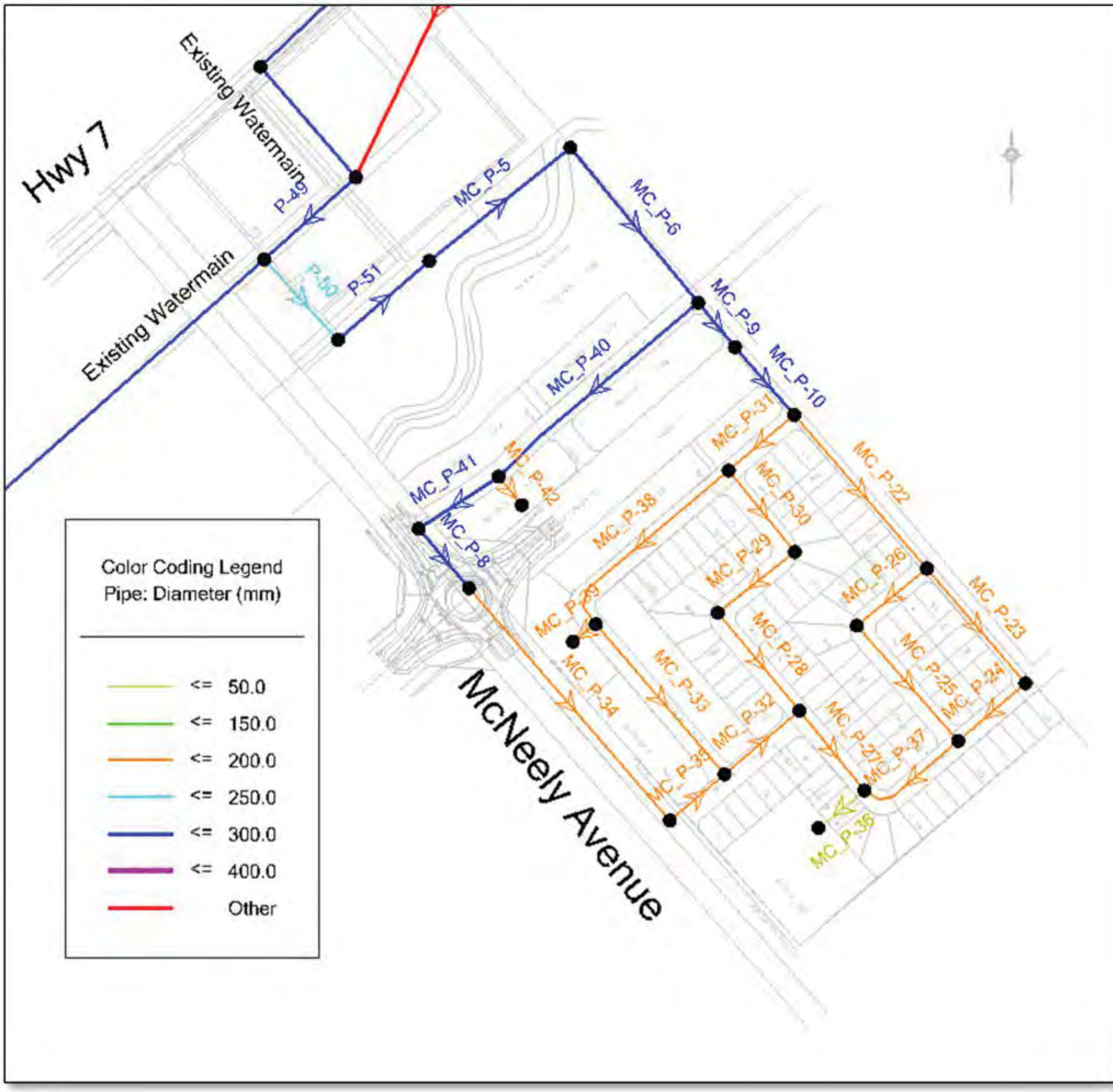


Figure A-3 **Proposed Water Distribution System Layout – Pipes**

Appendix B

PIPE AND JUNCTION TABLES

Pipe Parameter Table
Average Day

ID	Label	Start Node	Stop Node	Length (Scaled) (m)	Diameter (mm)	Hazen-Williams C	Velocity (m/s)	Headloss (m)	Flow (L/s)
649	MC_P-5	MC_J-22	MC_J-2	150.00	300.00	120.00	0.05	0.00	3.21
652	MC_P-6	MC_J-2	MC_J-3	166.00	300.00	120.00	0.05	0.00	3.21
632	MC_P-8	J-857	MC_J-10	64.00	300.00	120.00	0.01	0.00	0.78
637	MC_P-9	MC_J-3	MC_J-5	48.00	300.00	120.00	0.03	0.00	2.09
630	MC_P-10	MC_J-5	MC_J-6	74.00	300.00	120.00	0.03	0.00	1.92
654	MC_P-22	MC_J-6	MC_J-17	168.00	200.00	110.00	0.02	0.00	0.75
646	MC_P-23	MC_J-17	MC_J-14	125.00	200.00	110.00	0.01	0.00	0.27
635	MC_P-24	MC_J-14	MC_J-15	73.00	200.00	110.00	0.00	0.00	0.10
647	MC_P-25	MC_J-15	MC_J-16	127.00	200.00	110.00	0.00	0.00	-0.14
636	MC_P-26	MC_J-16	MC_J-17	75.00	200.00	110.00	0.01	0.00	-0.31
641	MC_P-27	MC_J-23	MC_J-11	85.00	200.00	110.00	0.01	0.00	-0.26
645	MC_P-28	MC_J-11	MC_J-20	106.00	200.00	110.00	0.00	0.00	-0.14
639	MC_P-29	MC_J-20	MC_J-21	81.00	200.00	110.00	0.01	0.00	-0.31
644	MC_P-30	MC_J-21	MC_J-13	87.00	200.00	110.00	0.02	0.00	-0.48
634	MC_P-31	MC_J-13	MC_J-6	71.00	200.00	110.00	0.03	0.00	-1.00
633	MC_P-32	MC_J-11	MC_J-29	81.00	200.00	110.00	0.01	0.00	-0.29
655	MC_P-33	MC_J-29	MC_J-27	164.00	200.00	110.00	0.00	0.00	-0.01
660	MC_P-34	MC_J-10	MC_J-19	255.00	200.00	110.00	0.02	0.00	0.61
638	MC_P-35	MC_J-19	MC_J-29	59.00	200.00	110.00	0.01	0.00	0.44
629	MC_P-36	MC_J-23	MC_J-4	49.00	50.00	100.00	0.09	0.02	0.17
640	MC_P-37	MC_J-15	MC_J-23	97.00	200.00	110.00	0.00	0.00	0.08
653	MC_P-38	MC_J-27	MC_J-13	186.00	200.00	110.00	0.01	0.00	-0.35
628	MC_P-39	MC_J-27	MC_J-35	24.00	200.00	110.00	0.01	0.00	0.17
678	MC_P-40	MC_J-3	MC_J-36	219.00	300.00	120.00	0.01	0.00	0.95
679	MC_P-41	MC_J-36	J-857	79.00	300.00	120.00	0.01	0.00	0.78
681	MC_P-42	MC_J-36	MC_J-37	30.00	200.00	110.00	0.01	0.00	0.17
715	P-51	WSPJ-39	MC_J-22	100.00	300.00	120.00	0.05	0.00	3.21

Pipe Parameter Table
Maximum Day

Miller's Crossing WDM
151-05603-00

ID	Label	Start Node	Stop Node	Length (Scaled) (m)	Diameter (mm)	Hazen-Williams C	Velocity (m/s)	Headloss (m)	Flow (L/s)
649	MC_P-5	MC_J-22	MC_J-2	150.00	300.00	120.00	0.09	0.01	6.42
652	MC_P-6	MC_J-2	MC_J-3	166.00	300.00	120.00	0.09	0.01	6.42
632	MC_P-8	J-857	MC_J-10	64.00	300.00	120.00	0.02	0.00	1.56
637	MC_P-9	MC_J-3	MC_J-5	48.00	300.00	120.00	0.06	0.00	4.19
630	MC_P-10	MC_J-5	MC_J-6	74.00	300.00	120.00	0.05	0.00	3.85
654	MC_P-22	MC_J-6	MC_J-17	168.00	200.00	110.00	0.05	0.00	1.51
646	MC_P-23	MC_J-17	MC_J-14	125.00	200.00	110.00	0.02	0.00	0.54
635	MC_P-24	MC_J-14	MC_J-15	73.00	200.00	110.00	0.01	0.00	0.21
647	MC_P-25	MC_J-15	MC_J-16	127.00	200.00	110.00	0.01	0.00	-0.29
636	MC_P-26	MC_J-16	MC_J-17	75.00	200.00	110.00	0.02	0.00	-0.63
641	MC_P-27	MC_J-23	MC_J-11	85.00	200.00	110.00	0.02	0.00	-0.52
645	MC_P-28	MC_J-11	MC_J-20	106.00	200.00	110.00	0.01	0.00	-0.29
639	MC_P-29	MC_J-20	MC_J-21	81.00	200.00	110.00	0.02	0.00	-0.62
644	MC_P-30	MC_J-21	MC_J-13	87.00	200.00	110.00	0.03	0.00	-0.96
634	MC_P-31	MC_J-13	MC_J-6	71.00	200.00	110.00	0.06	0.00	-2.00
633	MC_P-32	MC_J-11	MC_J-29	81.00	200.00	110.00	0.02	0.00	-0.57
655	MC_P-33	MC_J-29	MC_J-27	164.00	200.00	110.00	0.00	0.00	-0.02
660	MC_P-34	MC_J-10	MC_J-19	255.00	200.00	110.00	0.04	0.00	1.22
638	MC_P-35	MC_J-19	MC_J-29	59.00	200.00	110.00	0.03	0.00	0.88
629	MC_P-36	MC_J-23	MC_J-4	49.00	50.00	100.00	0.17	0.08	0.34
640	MC_P-37	MC_J-15	MC_J-23	97.00	200.00	110.00	0.00	0.00	0.16
653	MC_P-38	MC_J-27	MC_J-13	186.00	200.00	110.00	0.02	0.00	-0.70
628	MC_P-39	MC_J-27	MC_J-35	24.00	200.00	110.00	0.01	0.00	0.34
678	MC_P-40	MC_J-3	MC_J-36	219.00	300.00	120.00	0.03	0.00	1.90
679	MC_P-41	MC_J-36	J-857	79.00	300.00	120.00	0.02	0.00	1.56
681	MC_P-42	MC_J-36	MC_J-37	30.00	200.00	110.00	0.01	0.00	0.34
715	P-51	WSPJ-39	MC_J-22	100.00	300.00	120.00	0.09	0.00	6.42

Pipe Parameter Table
Peak Hour

ID	Label	Start Node	Stop Node	Length (Scaled) (m)	Diameter (mm)	Hazen-Williams C	Velocity (m/s)	Headloss (m)	Flow (L/s)
649	MC_P-5	MC_J-22	MC_J-2	150.00	300.00	120.00	0.14	0.01	9.63
652	MC_P-6	MC_J-2	MC_J-3	166.00	300.00	120.00	0.14	0.02	9.63
632	MC_P-8	J-857	MC_J-10	64.00	300.00	120.00	0.03	0.00	2.34
637	MC_P-9	MC_J-3	MC_J-5	48.00	300.00	120.00	0.09	0.00	6.28
630	MC_P-10	MC_J-5	MC_J-6	74.00	300.00	120.00	0.08	0.00	5.77
654	MC_P-22	MC_J-6	MC_J-17	168.00	200.00	110.00	0.07	0.01	2.26
646	MC_P-23	MC_J-17	MC_J-14	125.00	200.00	110.00	0.03	0.00	0.81
635	MC_P-24	MC_J-14	MC_J-15	73.00	200.00	110.00	0.01	0.00	0.31
647	MC_P-25	MC_J-15	MC_J-16	127.00	200.00	110.00	0.01	0.00	-0.43
636	MC_P-26	MC_J-16	MC_J-17	75.00	200.00	110.00	0.03	0.00	-0.94
641	MC_P-27	MC_J-23	MC_J-11	85.00	200.00	110.00	0.02	0.00	-0.78
645	MC_P-28	MC_J-11	MC_J-20	106.00	200.00	110.00	0.01	0.00	-0.43
639	MC_P-29	MC_J-20	MC_J-21	81.00	200.00	110.00	0.03	0.00	-0.94
644	MC_P-30	MC_J-21	MC_J-13	87.00	200.00	110.00	0.05	0.00	-1.44
634	MC_P-31	MC_J-13	MC_J-6	71.00	200.00	110.00	0.10	0.01	-3.00
633	MC_P-32	MC_J-11	MC_J-29	81.00	200.00	110.00	0.03	0.00	-0.86
655	MC_P-33	MC_J-29	MC_J-27	164.00	200.00	110.00	0.00	0.00	-0.04
660	MC_P-34	MC_J-10	MC_J-19	255.00	200.00	110.00	0.06	0.01	1.83
638	MC_P-35	MC_J-19	MC_J-29	59.00	200.00	110.00	0.04	0.00	1.33
629	MC_P-36	MC_J-23	MC_J-4	49.00	50.00	100.00	0.26	0.18	0.51
640	MC_P-37	MC_J-15	MC_J-23	97.00	200.00	110.00	0.01	0.00	0.24
653	MC_P-38	MC_J-27	MC_J-13	186.00	200.00	110.00	0.03	0.00	-1.05
628	MC_P-39	MC_J-27	MC_J-35	24.00	200.00	110.00	0.02	0.00	0.51
678	MC_P-40	MC_J-3	MC_J-36	219.00	300.00	120.00	0.04	0.00	2.85
679	MC_P-41	MC_J-36	J-857	79.00	300.00	120.00	0.03	0.00	2.34
681	MC_P-42	MC_J-36	MC_J-37	30.00	200.00	110.00	0.02	0.00	0.51
715	P-51	WSPJ-39	MC_J-22	100.00	300.00	120.00	0.14	0.01	9.63

ID	Label	Demand (L/s)	Elevation (m)	Hydraulic Grade (m)	Pressure Head (m)	Pressure (kPa)
627	J-857	0.00	130.80	181.23	50.43	494.00
620	MC_J-3	0.17	130.00	181.23	51.23	502.00
622	MC_J-4	0.17	132.55	181.22	48.67	477.00
623	MC_J-5	0.17	131.38	181.23	49.85	489.00
624	MC_J-6	0.17	131.51	181.23	49.72	487.00
593	MC_J-10	0.17	131.03	181.23	50.20	492.00
594	MC_J-11	0.17	132.96	181.22	48.26	473.00
596	MC_J-13	0.17	131.67	181.22	49.55	486.00
597	MC_J-14	0.17	132.09	181.22	49.13	482.00
598	MC_J-15	0.17	132.24	181.22	48.98	480.00
599	MC_J-16	0.17	132.04	181.22	49.18	482.00
600	MC_J-17	0.17	131.85	181.22	49.37	484.00
602	MC_J-19	0.17	136.00	181.22	45.22	443.00
604	MC_J-20	0.17	131.97	181.22	49.25	483.00
605	MC_J-21	0.17	131.87	181.22	49.35	484.00
614	MC_J-23	0.17	133.16	181.22	48.06	471.00
592	MC_J-27	0.17	132.00	181.22	49.22	482.00
613	MC_J-29	0.17	135.01	181.22	46.21	453.00
603	MC_J-35	0.17	132.00	181.22	49.22	482.00
677	MC_J-36	0.00	131.20	181.23	50.03	490.00
680	MC_J-37	0.17	131.20	181.23	50.03	490.00

ID	Label	Demand (L/s)	Elevation (m)	Hydraulic Grade (m)	Pressure Head (m)	Pressure (kPa)
627	J-857	0.00	130.80	181.06	50.26	493.00
620	MC_J-3	0.34	130.00	181.06	51.06	501.00
622	MC_J-4	0.34	132.55	181.06	48.51	475.00
623	MC_J-5	0.34	131.38	181.06	49.68	487.00
624	MC_J-6	0.34	131.51	181.06	49.55	486.00
593	MC_J-10	0.34	131.03	181.06	50.03	490.00
594	MC_J-11	0.34	132.96	181.06	48.10	471.00
596	MC_J-13	0.34	131.67	181.06	49.39	484.00
597	MC_J-14	0.34	132.09	181.06	48.97	480.00
598	MC_J-15	0.34	132.24	181.06	48.82	478.00
599	MC_J-16	0.34	132.04	181.06	49.02	480.00
600	MC_J-17	0.34	131.85	181.06	49.21	482.00
602	MC_J-19	0.34	136.00	181.06	45.06	442.00
604	MC_J-20	0.34	131.97	181.06	49.09	481.00
605	MC_J-21	0.34	131.87	181.06	49.19	482.00
614	MC_J-23	0.34	133.16	181.06	47.90	469.00
592	MC_J-27	0.34	132.00	181.06	49.06	481.00
613	MC_J-29	0.34	135.01	181.06	46.05	451.00
603	MC_J-35	0.34	132.00	181.06	49.06	481.00
677	MC_J-36	0.00	131.20	181.06	49.86	489.00
680	MC_J-37	0.34	131.20	181.06	49.86	489.00

ID	Label	Demand (L/s)	Elevation (m)	Hydraulic Grade (m)	Pressure Head (m)	Pressure (kPa)
627	J-857	0.00	130.80	180.77	49.97	490.00
620	MC_J-3	0.51	131.38	180.77	49.39	484.00
622	MC_J-4	0.51	132.55	180.75	48.20	472.00
623	MC_J-5	0.51	131.38	180.77	49.39	484.00
624	MC_J-6	0.51	131.51	180.76	49.25	483.00
593	MC_J-10	0.51	131.03	180.77	49.74	487.00
594	MC_J-11	0.51	132.96	180.75	47.79	468.00
596	MC_J-13	0.51	131.67	180.76	49.09	481.00
597	MC_J-14	0.51	132.09	180.75	48.66	477.00
598	MC_J-15	0.51	132.24	180.75	48.51	476.00
599	MC_J-16	0.51	132.04	180.75	48.71	477.00
600	MC_J-17	0.51	131.85	180.75	48.90	479.00
602	MC_J-19	0.51	136.00	180.76	44.76	439.00
604	MC_J-20	0.51	131.97	180.75	48.78	478.00
605	MC_J-21	0.51	131.87	180.76	48.89	479.00
614	MC_J-23	0.51	133.16	180.75	47.59	466.00
592	MC_J-27	0.51	132.00	180.75	48.75	478.00
613	MC_J-29	0.51	135.01	180.75	45.74	448.00
603	MC_J-35	0.51	132.00	180.75	48.75	478.00
677	MC_J-36	0.00	131.20	180.77	49.57	486.00
680	MC_J-37	0.51	131.20	180.77	49.57	486.00

Appendix C

FIRE FLOW REPORT

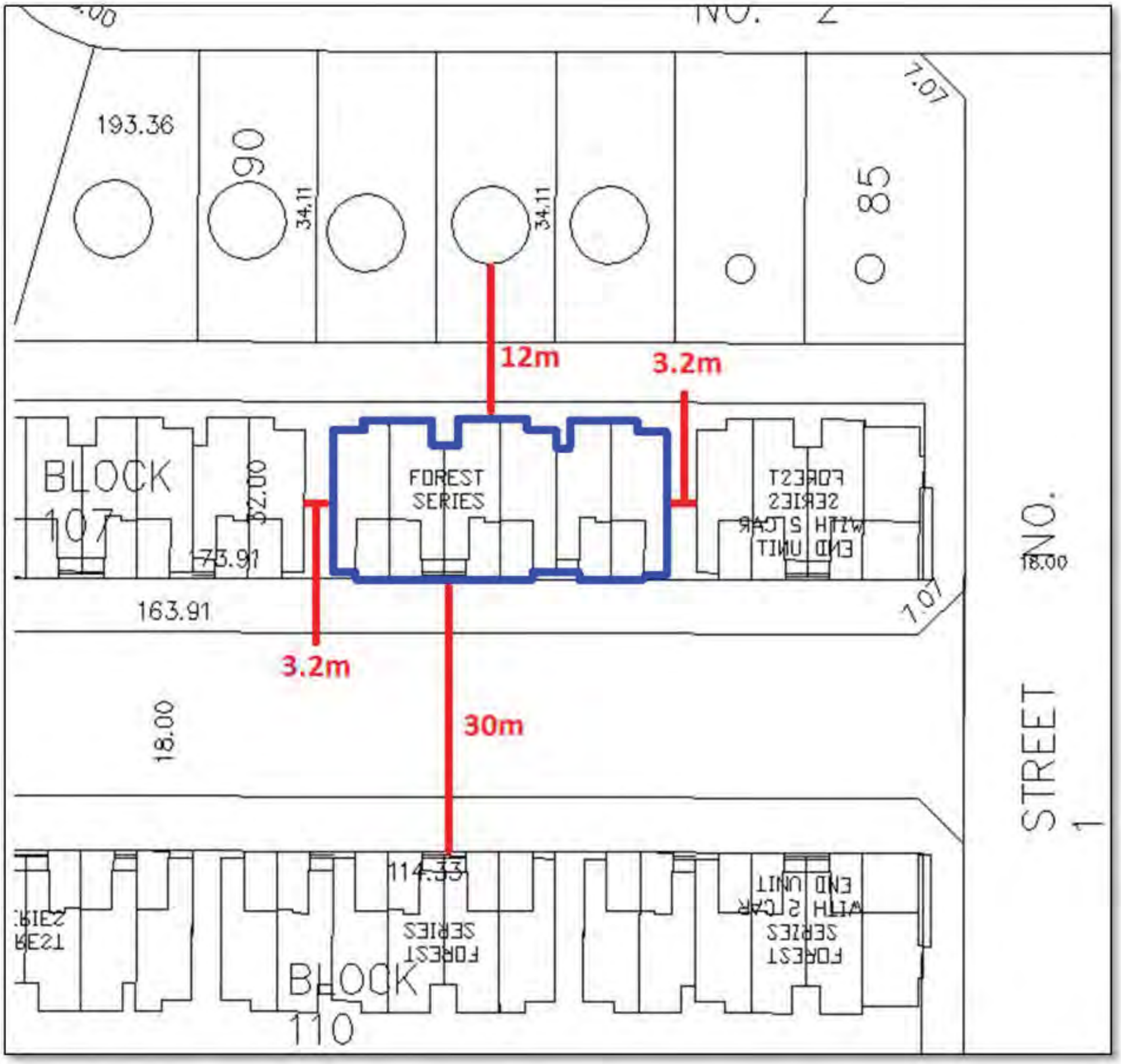


Figure A-4 **Exposure Distances (Townhouse)**

Required Fire Flow Worksheet

Ontario Building Code (OBC) – 2012, Section A-3.2.5.7.3

$$Q - K \cdot V \cdot S_{tot}$$

where

Q – minimum supply of water in litres

K = water supply coefficient from Table 1

V = total building volume in cubic metres

S_{tot} = total of spatial coefficient values from property line exposures on all sides as obtained from the formula:

$$S_{tot} = 1.0 + [S_{side1} + S_{side2} + S_{side3} + \dots \text{etc.})]$$

where

S_{side} values are established from Figure 1, as modified by Items 3(d) and 3(f), and

S_{tot} need not exceed 2.0.

K = 18 (see Table 1 attached)

V = Largest Building Area (Townhouse) • Height of the Building
= 945 sq. m. • 11 m.
V = 10,395 sq. m.

S_{tot} = 1.0 + [S_{side1} + S_{side2} + S_{side3} + … etc.] ; ≤ 2.0
= 1.0 + [0.1 + 0.5 + 0.5 + 0.0] ; ≤ 2.0
= 2.1 ; ≤ 2.0
S_{tot} = 2.0

Q = K • V • S_{tot}
= (18) • (10,395) • (2.0)
Q = 374,220 Liters

Therefore, according to Table 2 of the 2012 OBC (see attached):

Required Minimum Water Supply Flow Rate = 9000 L/min or 150 L/sec

Table 1					
Water Supply Coefficient - K					
Type of Construction	Classification by Group or Division in Accordance with Table 3.1.2.1. of the Building Code				
	A-2 B-1 B-2 B-3 C D	A-4 F-3	A-1 A-3	E F-2	F-1
Building is of noncombustible construction with fire separations and fire-resistance ratings provided in accordance with Subsection 3.2.2., including loadbearing walls, columns and arches.	10	12	14	17	23
Building is of noncombustible construction or of heavy timber construction conforming to Article 3.1.4.6. Floor assemblies are fire separations but with no fire-resistance rating. Roof assemblies, mezzanines, loadbearing walls, columns and arches do not have a fire-resistance rating.	16	19	22	27	37
Building is of combustible construction with fire separations and fire-resistance ratings provided in accordance with Subsection 3.2.2., including loadbearing walls, columns and arches. Noncombustible construction may be used in lieu of fire-resistance rating where permitted in Subsection 3.2.2.	18	22	25	31	41
Building is of combustible construction. Floor assemblies are fire separations but with no fire-resistance rating. Roof assemblies, mezzanines, loadbearing walls, columns and arches do not have a fire-resistance rating.	23	28	32	39	53
Column 1	2	3	4	5	6

Table 2	
Part 3 Buildings under the Building Code	Required Minimum Water Supply Flow Rate, L/min
One-storey building with building area not exceeding 600 m²	1 800
All other buildings	2 700 (if Q ≤ 108 000 L)(1) 3 600 (if Q > 108 000 L and ≤ 135 000 L)(1) 4 500 (if Q > 135 000 L and ≤ 162 000 L)(1) 5 400 (if Q > 162 000 L and ≤ 190 000 L)(1) 6 300 (if Q > 190 000 L and ≤ 270 000 L)(1) 9 000 (if Q > 270 000 L)(1)

Notes to Table 2:

(1) $Q = KVS_{tot}$ as referenced in Paragraph 3(a)

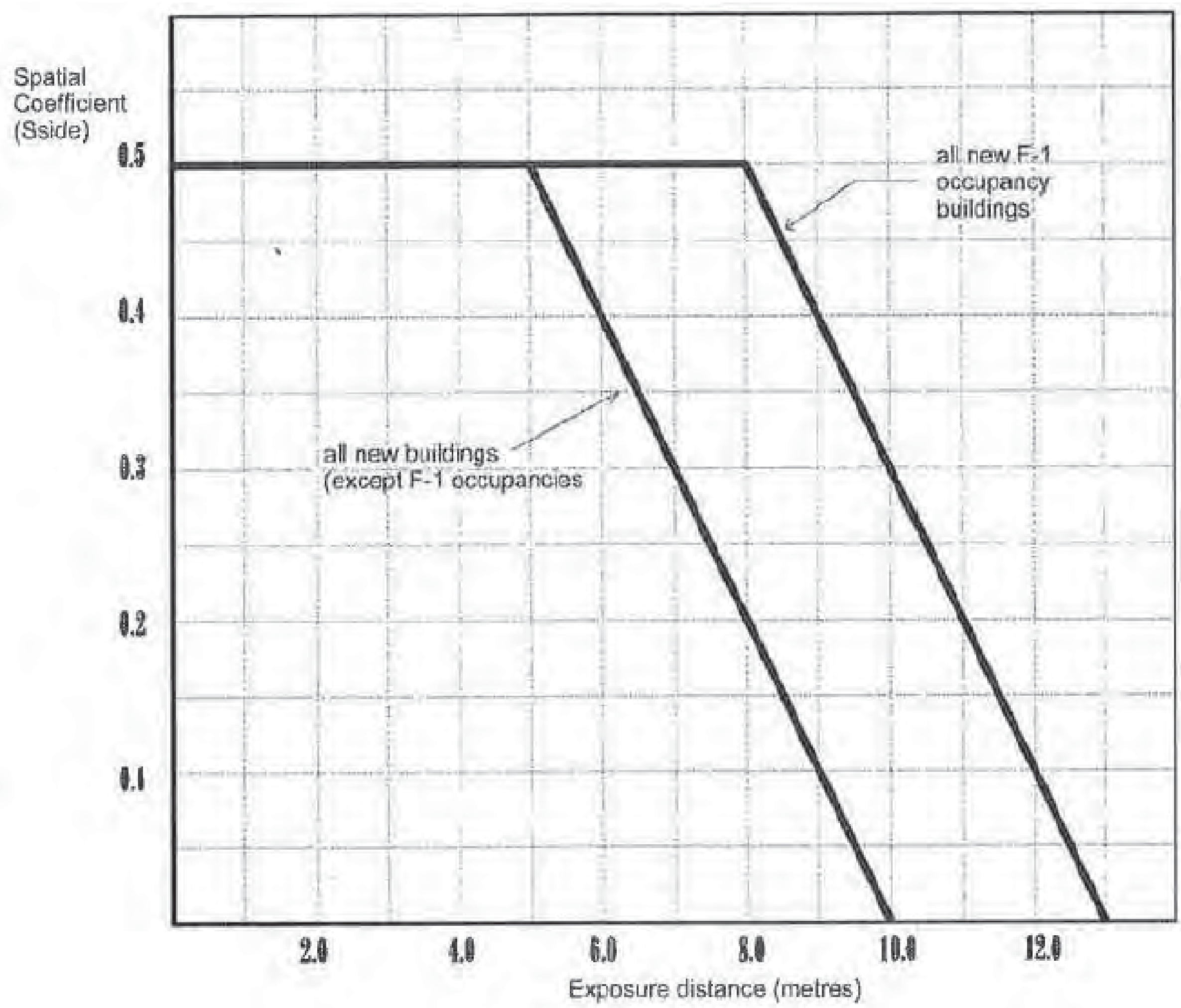


Figure 1
Spatial Coefficient vs Exposure Distance

Fire Flow Report
Maximum Day + Fire

Miller's Crossing WDM
151-05603-00

Label	Elevation (m)	Demand (L/s)	Pressure (kPa)	Pressure (Residual Lower Limit) (kPa)	Pressure (Zone Lower Limit) (kPa)	Pressure (Calculated Residual) (kPa)	Pressure (Calculated Zone Lower Limit) (kPa)	Junction w/ Minimum Pressure (Zone)	Fire Flow (Needed) (L/s)	Fire Flow (Available) (L/s)	Satisfies Fire Flow Constraints?
J-857	130.80	0.00	410.00	140.00	140.00	141.00	140.00	MC_J-10	150.00	283.87	TRUE
MC_J-3	130.00	0.34	418.00	140.00	140.00	199.00	140.00	MC_J-19	150.00	325.85	TRUE
MC_J-4	132.55	0.34	393.00	140.00	140.00	140.00	211.00	MC_J-23	150.00	164.78	TRUE
MC_J-5	131.38	0.34	405.00	140.00	140.00	177.00	140.00	MC_J-19	150.00	311.68	TRUE
MC_J-6	131.51	0.34	403.00	140.00	140.00	167.00	140.00	MC_J-19	150.00	295.57	TRUE
MC_J-10	131.03	0.34	408.00	140.00	140.00	140.00	155.00	MC_J-19	150.00	274.15	TRUE
MC_J-11	132.96	0.34	389.00	140.00	140.00	140.00	162.00	MC_J-23	150.00	222.92	TRUE
MC_J-13	131.67	0.34	402.00	140.00	140.00	140.00	153.00	MC_J-21	150.00	248.01	TRUE
MC_J-14	132.09	0.34	398.00	140.00	140.00	140.00	186.00	MC_J-15	150.00	183.63	TRUE
MC_J-15	132.24	0.34	396.00	140.00	140.00	140.00	155.00	MC_J-14	150.00	197.23	TRUE
MC_J-16	132.04	0.34	398.00	140.00	140.00	140.00	189.00	MC_J-15	150.00	183.81	TRUE
MC_J-17	131.85	0.34	400.00	140.00	140.00	140.00	147.00	MC_J-16	150.00	203.07	TRUE
MC_J-19	136.00	0.34	359.00	140.00	140.00	140.00	196.00	MC_J-29	150.00	196.36	TRUE
MC_J-20	131.97	0.34	399.00	140.00	140.00	140.00	188.00	MC_J-21	150.00	203.99	TRUE
MC_J-21	131.87	0.34	400.00	140.00	140.00	140.00	174.00	MC_J-20	150.00	209.47	TRUE
MC_J-23	133.16	0.34	387.00	140.00	140.00	140.00	146.00	MC_J-4	150.00	198.65	TRUE
MC_J-27	132.00	0.34	398.00	140.00	140.00	140.00	140.00	MC_J-35	150.00	194.48	TRUE
MC_J-29	135.01	0.34	369.00	140.00	140.00	140.00	151.00	MC_J-19	150.00	214.30	TRUE
MC_J-35	132.00	0.34	398.00	140.00	140.00	140.00	182.00	MC_J-27	150.00	176.36	TRUE
MC_J-36	131.20	0.00	406.00	140.00	140.00	140.00	140.00	MC_J-37	150.00	295.06	TRUE
MC_J-37	131.20	0.34	406.00	140.00	140.00	140.00	260.00	MC_J-19	150.00	191.36	TRUE

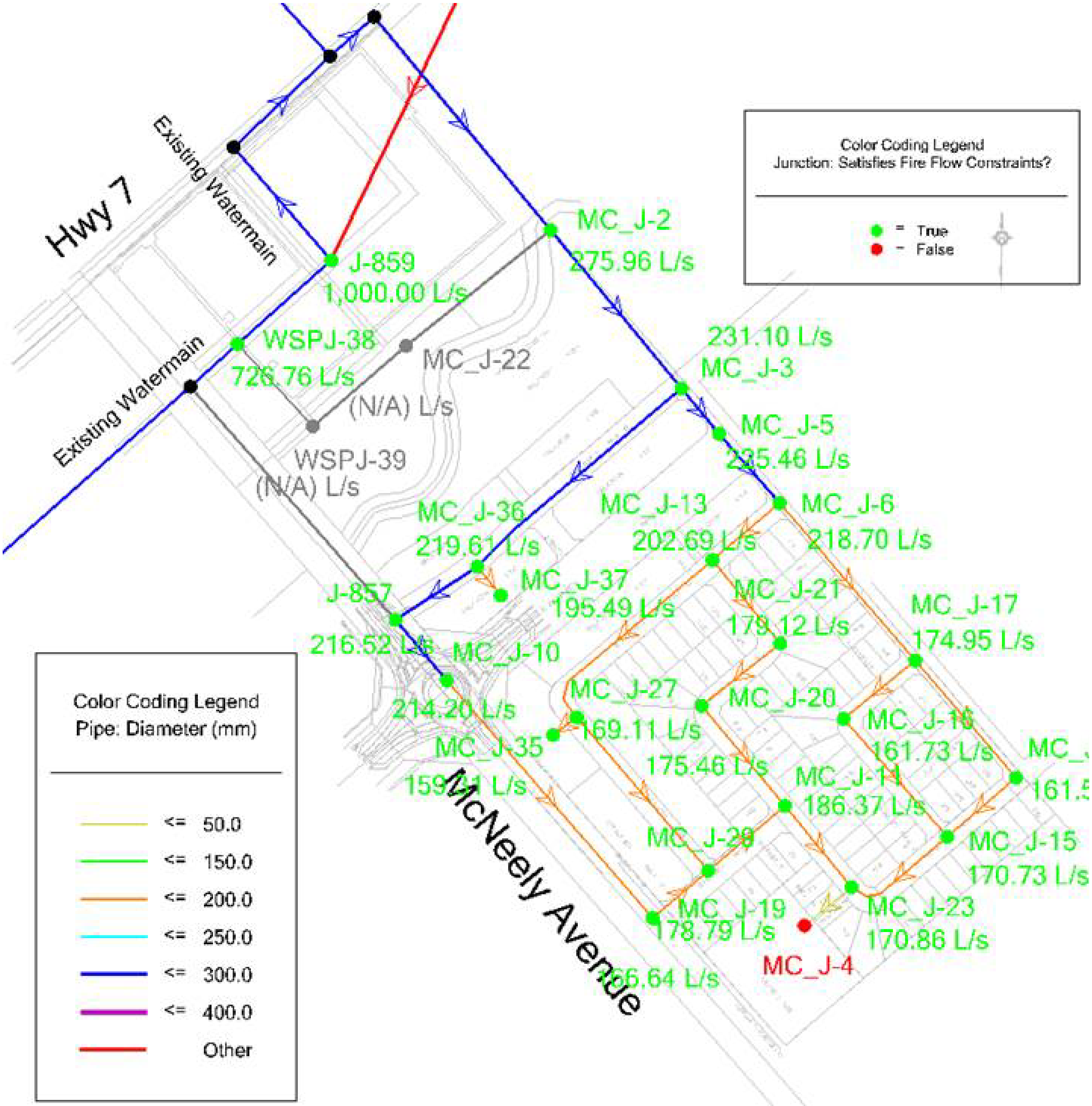
Laura Maxwell

From: Manigbas, Gian Carlo <GianCarlo.Manigbas@wspgroup.com>
Sent: April-12-16 2:20 PM
To: Laura Maxwell
Subject: RE: Miller's Crossing Watermain Analysis

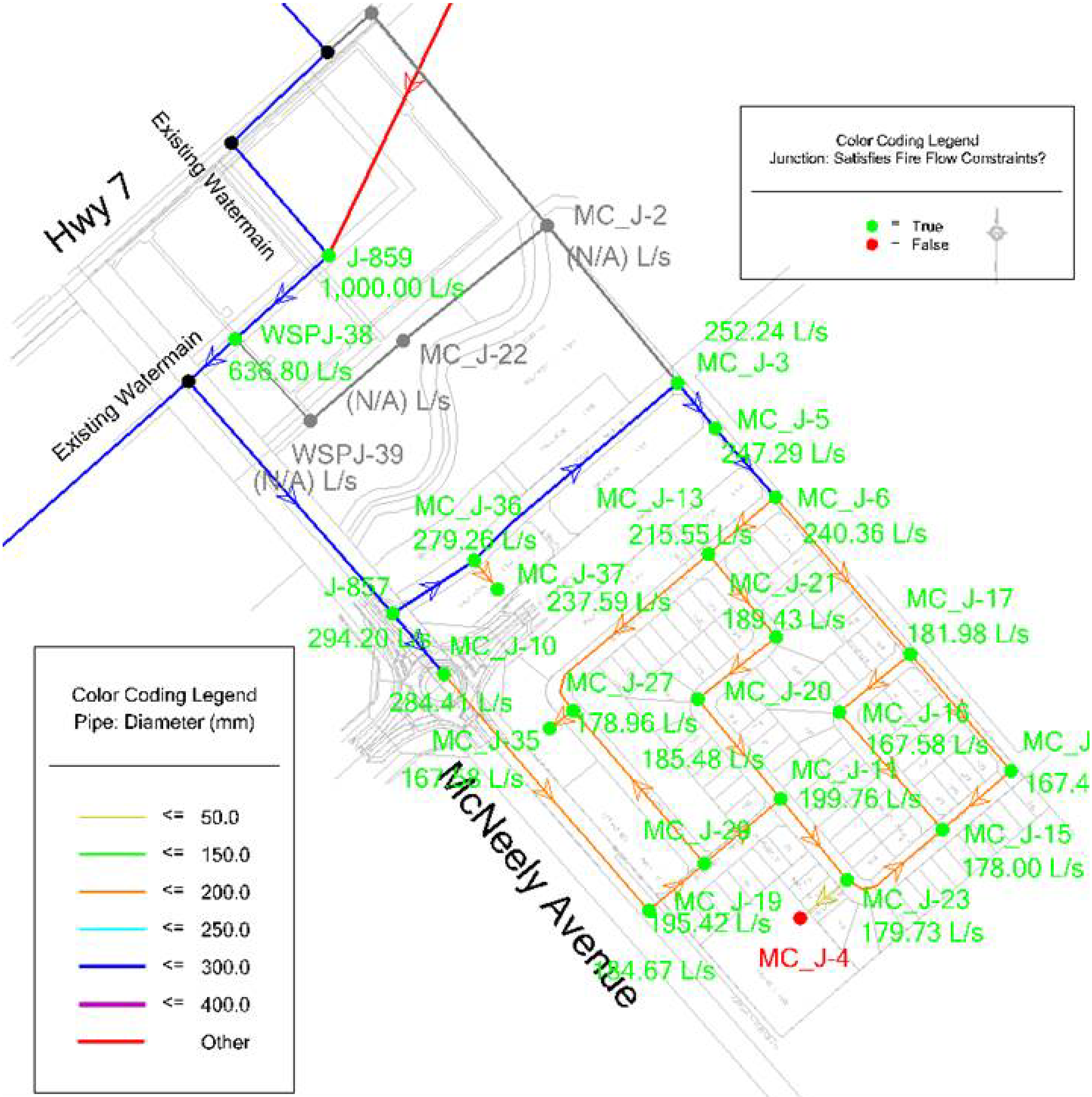
Hello Laura,

Based on the results of the modeling for Miller's Crossing below, the change in location of the watermain connection to either Option #1 or Option #2 will still meet the required fire flow and therefore, does not change the conclusions of our analysis.

Option #1:


Hwy 7
Existing Watermain
Existing Watermain
McNeely Avenue
J-859
1,000.00 L/s
MC_J-2
275.96 L/s
WSPJ-38
726.76 L/s
MC_J-22
(N/A) L/s
WSPJ-39
(N/A) L/s
231.10 L/s
MC_J-3
MC_J-5
225.46 L/s
MC_J-6
218.70 L/s
MC_J-13
202.69 L/s
MC_J-36
219.61 L/s
MC_J-37
195.49 L/s
J-857
216.52 L/s
MC_J-10
214.20 L/s
MC_J-21
179.12 L/s
MC_J-17
174.95 L/s
MC_J-27
169.11 L/s
MC_J-20
175.46 L/s
MC_J-16
161.73 L/s
MC_J-35
159.41 L/s
MC_J-11
186.37 L/s
MC_J-15
170.73 L/s
MC_J-29
MC_J-19
178.79 L/s
MC_J-23
170.86 L/s
MC_J-4
166.64 L/s
Color Coding Legend
Junction: Satisfies Fire Flow Constraints?
= True
= False
Color Coding Legend
Pipe: Diameter (mm)
<= 50.0
<= 150.0
<= 200.0
<= 250.0
<= 300.0
<= 400.0
Other

Option #2:



Best Regards,
Gian

Gian Carlo Manigbas, E.I.T.

WSP Canada Inc.
600 Cochrane Drive, Suite 500

Markham, Ontario L3R 5K3 Canada
Tel. 905-475-7270 #18286
Cel. 647-868-0224

www.wspgroup.com

We were GENIVAR. We are now WSP.

Please consider the environment before printing...

From: Laura Maxwell [mailto:lmaxwell@dsel.ca]
Sent: Saturday, April 09, 2016 3:46 PM
To: Manigbas, Gian Carlo <GianCarlo.Manigbas@wspgroup.com>
Subject: Miller's Crossing Watermain Analysis

HI Gian Carlo,

The watermain connection point for Miller's Crossing will be changing from the original location shown in your report (latest report attached for your records).

There are two options for the new location – see attached sketch.

Can you please confirm by e-mail or by memo that the change in location to either Option #1 or Option #2 does not change the conclusions of your analysis?

Many thanks,

Laura Maxwell, B.Sc.(Civil Eng), M.Pl.
Project Manager

DSEL
david schaeffer engineering ltd.

120 Iber Road, Unit 103
Stittsville, ON K2S 1E9

phone: (613) 836-0856 ext. 527
cell: (613) 293-8750
email: lmaxwell@DSEL.ca

WATER DEMAND DESIGN SHEET

Novatech Project #:	119221
Project Name:	McNeely Landing
Date Prepared:	6/10/2021
Date Revised:	5/13/2022
Input By:	Ben Sweet
Reviewed By:	Sam Bahia
Drawing Reference:	Figure 1.1 & 6.1

Legend:
- PROJECT SPECIFIC INFO
- USER DESIGN INPUT
- CALCULATED AVERAGE DAY CELL OUTPUT
- CALCULATED BASIC DAY CELL OUTPUT
- CALCULATED MAX DAY CELL OUTPUT
- CALCULATED PEAK HOUR CELL OUTPUT
- CALCULATED MAX DAY + RFF CELLOUTPUT

NOVATECH
Engineers, Planners & Landscape Architects

LOCATION		TOTAL WATER DEMAND																					
		RESIDENTIAL DEMAND & AVERAGE DEMAND						INDUSTRIAL / COMMERCIAL / INSTITUTIONAL (ICI) INPUT & AVERAGE DEMAND						BASIC DAY DEMAND DAILY VOLUME (m³)							DESIGN FIRE DEMAND		
								INDUST. AREA							MAXIMUM DAY DEMAND			PEAK HOUR DEMAND			REQUIRED FIRE FLOW (RFF)			
NODE	SMALL SYSTEM *	SINGLES	SEMIS/ TOWNS	APTS (2BR)	APTS (1BR)	POP. EQUIV.	RES. AVERAGE DAY FLOW DEMAND (L/s)	LIGHT (ha.)	HEAVEY (ha.)	COMM. AREA (ha.)	INST. AREA (ha.)	OTHER AREA (m²)	ICI AVERAGE DAY FLOW DEMAND (L/s)		RES. PEAKING FACTOR	ICI PEAKING FACTOR	MAX DAY FLOW DEMAND (L/s)	RES. PEAKING FACTOR	ICI PEAKING FACTOR	PEAK HOUR FLOW DEMAND (L/s)	RFF 1 FUS (L/min)	RFF 2 FUS (L/min)	RFF 3 OBC / NFPA (L/min)	MAX DAY + GOVERNING RFF (L/s)
J1			11	28		88.5	0.359						0.000	17.7	2.50	1.80	0.896	5.50	3.24	1.972	10,000			167.56
J2			21			56.7	0.230						0.000	11.3	2.50	1.80	0.574	5.50	3.24	1.263	10,000			167.24
J3			22			59.4	0.241						0.000	11.9	2.50	1.80	0.602	5.50	3.24	1.323	10,000			167.27
J4		5	5			30.5	0.124						0.000	6.1	2.50	1.80	0.309	5.50	3.24	0.680	10,000			166.98
J5			18			48.6	0.197						0.000	9.7	2.50	1.80	0.492	5.50	3.24	1.083	10,000			167.16
J6			27			72.9	0.295						0.000	14.6	2.50	1.80	0.738	5.50	3.24	1.624	10,000			167.40
J7		3	16			53.4	0.216						0.000	10.7	2.50	1.80	0.541	5.50	3.24	1.190	10,000			167.21
J8			21			56.7	0.230						0.000	11.3	2.50	1.80	0.574	5.50	3.24	1.263	10,000			167.24
J9						0.0	0.000						0.000	0.0	2.50	1.80	0.000	5.50	3.24	0.000	10,000			166.67
J10			11	28		88.5	0.359						0.000	17.7	2.50	1.80	0.896	5.50	3.24	1.972	10,000			167.56
J11		3				10.2	0.041						0.000	2.0	2.50	1.80	0.103	5.50	3.24	0.227	10,000			166.77
J12			19			51.3	0.208						0.000	10.3	2.50	1.80	0.520	5.50	3.24	1.143	10,000			167.19
J13						0.0	0.000						0.000	0.0	2.50	1.80	0.000	5.50	3.24	0.000	10,000			166.67
J15		14				47.6	0.193						0.000	9.5	2.50	1.80	0.482	5.50	3.24	1.061	10,000			167.15
J16		12				40.8	0.165						0.000	8.2	2.50	1.80	0.413	5.50	3.24	0.909	10,000			167.08
J17		17				57.8	0.234						0.000	11.6	2.50	1.80	0.585	5.50	3.24	1.288	10,000			167.25
J18		17				57.8	0.234						0.000	11.6	2.50	1.80	0.585	5.50	3.24	1.288	10,000			167.25
J19		26				88.4	0.358						0.000	17.7	2.50	1.80	0.895	5.50	3.24	1.970	10,000			167.56
J20		24				81.6	0.331						0.000	16.3	2.50	1.80	0.826	5.50	3.24	1.818	10,000			167.49
J21		24				81.6	0.331						0.000	16.3	2.50	1.80	0.826	5.50	3.24	1.818	10,000			167.49
J23		20				68.0	0.275						0.000	13.6	2.50	1.80	0.689	5.50	3.24	1.515	10,000			167.36
J24		17				57.8	0.234						0.000	11.6	2.50	1.80	0.585	5.50	3.24	1.288	10,000			167.25
J25		22				74.8	0.303						0.000	15.0	2.50	1.80	0.758	5.50	3.24	1.667	10,000			167.42
J26						0.0	0.000						0.000	0.0	2.50	1.80	0.000	5.50	3.24	0.000	10,000			166.67
J27						0.0	0.000				1.620		0.525	27.5	2.50	1.80	0.945	5.50	3.24	1.701	10,000			167.61
																								0.00
SUB-TOTAL	NA	204	171	56		1272.9	5.156	0.000	0.000	0.000	1.620	0.000	0.525	282.1	2.50	1.80	13.836	5.50	3.24	30.061	10,000	0	0	180.50

DEMAND PARAMETERS

	Residential				
Unit Type Population Equiv.	Singles	Semis/ Towns	Apts (2-BR)	Apts (2-BR)	Apts (AVG)
	3.4	2.7	2.1	1.4	1.8
Dailly Demand	L/per person/day				
Average Demand	350				
Basic Demand	200				

Res. Peaking Factors		Max Day (x Avg Day)	Peak Hour (x Avg Day)
Population > 500		2.50	5.50
Population < 500*	Pop.	Small Systems *	
Ref: MECP DWS Guideline Table 3-3	0	9.50	14.30
	30	9.50	14.30
	150	4.90	7.40
* Note: Use Drop Down List at ea Node.	300	3.60	5.40
YES	450	3.00	4.50
NO	500	2.90	4.30

Institutional / Commercial / Industrial				
Industrial		Commercial	Institutional	Other Use**
Light	Heavy			
L/gross ha/day				L/m²/day
35,000	55,000	28,000	28,000	5
10,000	17,000	17,000	17,000	3

ICI Peaking Factors	Max Day (x Avg Day)	Peak Hour (x Avg Day)
	1.80	3.24

**Note: Custom Designer defined input/parameter

Vulnerable Service Area (VSA) Review ***
< 50 m³/day
> 50 m³/day

***Note: Designer to review Node / Total VSA.

Quick Fire Flow Reference Guide **

FUS (L/min)	Comments	OBC (L/min)	Comments
> 2,000	Min FUS	< 9,000	Unsprinklered Non- Combustible
	Low Density - Singles/Towns		
10,000	Complies w/ TB2014-01 Cap. (10m rear spacing, 6 units max, <600 m²)		
13,000	Non-complying w/TB2014-01. Calculate.		
15,000	Medium Density Back-to-back Towns.		
	High Density		
20,000	Wood Frame 4-Storey		
5,000	Fire-Resisitve Podium/Multi-Storey		
30,000	High Contiguous / Hazard Areas		
< 45,000	Max FUS		

****Note: Designer to confirm RFF @ each node using FUS / OBC. Use Novatech FUSv2-0 and OBCv2-0 or NFPA.

Novatech Project #: 119221
Project Name: McNeely Landing
Date: 6/10/2021
Date Revised: 05/13/2022

Engineers, Planners & Landscape Architects

MAX PRESSURES DURING AVDY CONDITIONS

JUNCTION ID	ELEVATION (m)	STATIC DEMAND (L/s)	STATIC HEAD (m)	STATIC PRESSURE (m)	STATIC PRESSURE (psi)
J01	132.70	0.36	180.81	48.11	68
J02	133.10	0.23	180.81	47.71	68
J03	133.30	0.24	180.81	47.51	68
J04	133.50	0.12	180.81	47.31	67
J05	134.50	0.20	180.81	46.31	66
J06	136.40	0.30	180.81	44.41	63
J07	136.60	0.22	180.81	44.21	63
J08	137.20	0.23	180.81	43.61	62
J09	136.00	0.00	180.81	44.81	64
J10	133.00	0.36	180.81	47.81	68
J11	132.30	0.04	180.81	48.51	69
J12	137.50	0.21	180.81	43.31	62
J13	132.90	0.00	180.81	47.91	68
J15	133.50	0.19	180.81	47.31	67
J16	136.40	0.17	180.81	44.41	63
J17	133.30	0.23	180.81	47.51	68
J18	133.50	0.23	180.81	47.31	67
J19	136.00	0.36	180.81	44.81	64
J20	137.00	0.33	180.81	43.81	62
J21	140.00	0.33	180.81	40.81	58
J23	137.00	0.28	180.81	43.81	62
J24	137.30	0.23	180.81	43.51	62
J25	140.00	0.30	180.81	40.81	58
J26	131.00	0.00	180.81	49.81	71
J27	134.50	0.53	180.81	46.31	66

Novatech Project #: 119221
Project Name: McNeely Landing
Date: 6/10/2021
Date Revised: 05/13/2022

NOVATECH
Engineers, Planners & Landscape Architects

MIN PRESSURES DURING PKHR CONDITIONS

JUNCTION ID	ELEVATION (m)	STATIC DEMAND (L/s)	STATIC HEAD (m)	STATIC PRESSURE (m)	STATIC PRESSURE (psi)
J01	132.70	1.97	177.91	45.21	64
J02	133.10	1.26	177.89	44.79	64
J03	133.30	1.32	177.87	44.57	63
J04	133.50	0.68	177.86	44.36	63
J05	134.50	1.08	177.86	43.36	62
J06	136.40	1.62	177.85	41.45	59
J07	136.60	1.19	177.85	41.25	59
J08	137.20	1.26	177.85	40.65	58
J09	136.00	0.00	177.85	41.85	60
J10	133.00	1.97	177.89	44.89	64
J11	132.30	0.23	177.86	45.56	65
J12	137.50	1.14	177.85	40.35	57
J13	132.90	0.00	177.90	45.00	64
J15	133.50	1.06	177.91	44.41	63
J16	136.40	0.91	177.88	41.48	59
J17	133.30	1.29	177.87	44.57	63
J18	133.50	1.29	177.87	44.37	63
J19	136.00	1.97	177.85	41.85	60
J20	137.00	1.82	177.85	40.85	58
J21	140.00	1.82	177.85	37.84	54
J23	137.00	1.52	177.84	40.84	58
J24	137.30	1.29	177.84	40.54	58
J25	140.00	1.67	177.84	37.84	54
J26	131.00	0.00	177.97	46.97	67
J27	134.50	1.70	177.85	43.35	62

Novatech Project #: 119221
Project Name: McNeely Landing
Date: 6/10/2021
Date Revised: 05/13/2022

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AVAILABLE FLOW AT 20psi DURING MXDY+FF CONDITIONS

JUNCTION ID	ELEVATION (m)	STATIC DEMAND (L/s)	STATIC HEAD (m)	STATIC PRESSURE (m)	STATIC PRESSURE (psi)	FIRE FLOW DEMAND (L/s)	FIRE FLOW DEMAND (L/min)	AVAILABLE FLOW (L/min)
J01	132.70	0.90	179.89	47.19	67	167	10,000	8,730
J02	133.10	0.57	179.88	46.78	67	167	10,000	8,682
J03	133.30	0.60	179.88	46.58	66	167	10,000	8,646
J04	133.50	0.31	179.88	46.38	66	167	10,000	8,604
J05	134.50	0.49	179.87	45.37	65	167	10,000	8,514
J06	136.40	0.74	179.87	43.47	62	167	10,000	8,310
J07	136.60	0.54	179.87	43.27	62	167	10,000	8,142
J08	137.20	0.57	179.87	42.67	61	167	10,000	7,866
J09	136.00	0.00	179.87	43.87	62	-	-	-
J10	133.00	0.90	179.88	46.88	67	167	10,000	8,538
J11	132.30	0.10	179.88	47.58	68	167	10,000	7,254
J12	137.50	0.52	179.87	42.37	60	167	10,000	7,482
J13	132.90	0.00	179.88	46.98	67	167	10,000	8,592
J15	133.50	0.48	179.89	46.39	66	167	10,000	8,622
J16	136.40	0.41	179.88	43.48	62	167	10,000	8,334
J17	133.30	0.59	179.88	46.58	66	167	10,000	8,562
J18	133.50	0.59	179.88	46.38	66	167	10,000	8,520
J19	136.00	0.90	179.87	43.87	62	167	10,000	8,394
J20	137.00	0.83	179.87	42.87	61	167	10,000	8,190
J21	140.00	0.83	179.87	39.87	57	167	10,000	6,738
J23	137.00	0.69	179.87	42.87	61	167	10,000	6,846
J24	137.30	0.59	179.87	42.57	61	167	10,000	6,786
J25	140.00	0.76	179.87	39.87	57	167	10,000	6,660
J26	131.00	0.00	179.90	48.90	70	-	-	-
J27	134.50	0.95	179.87	45.37	65	-	-	-

1. Background

On behalf of the City of Ottawa, the National Research Council of Canada (NRC) evaluated the City's hydrant spacing guidelines in relation to Required Fire Flow (RFF) as calculated using the Fire Underwriters Survey (FUS) methodology. This work lead to the development of a procedure to be used to establish the appropriate sizing of, and hydrant spacing on, dead-end watermains. This procedure may also be used as an optional watermain network design method to optimize watermain sizing based on RFF and standard hydrant spacing.

The procedure is partially based on the NFPA 1: Fire Code (NFPA1) and the City of Ottawa existing hydrant classification practice (refer to **Attachment A** at the end of this appendix for relevant excerpts of the Fire Code).

2. Rationale for Guideline

Given a Required Fire Flow (RFF) for a certain asset/structure/building, proper planning must ensure that there is a sufficient number of hydrants at sufficient proximities to actually provide the RFF. Both the capacity of the hydrants and their proximity to the asset/structure/building must be considered. Pressure losses (due to friction) in firehoses are proportional to the firehose length. Therefore, the actual fire flow delivered by the nozzle at the end of a very long firehose will be less compared to a short firehose connected to the same hydrant. Table 1 provides conservative values for hydrant fire flow capacity adjusted for firehose length.

3. Hydrant Capacity Requirement

For the purposes of this guidelines, the aggregate fire flow capacity of all contributing fire hydrants within 150 m of a building/asset/structure[1], measured in accordance with Table 1, shall be not less than the RFF.

4. Standard Practice

For the vast majority of developments, hydrant spacing as indicated in Section 4.5, Table 4.9, Ottawa Design Guidelines – Water Distribution, are sufficient to meet the RFF. This has been verified by evaluating approved development plans representing a

[1] Although NFPA 1 considers hydrant contribution at distances of up to 1000ft (305 m), Ottawa Fire Services (OFS) would need two pumpers to deliver flow from such a distance (one pumper midway – acting as a booster). Moreover, OFS cautioned that some redundancy is advisable to account for accessibility limitations in emergency situations, wind effects, etc. Therefore 150 m was considered as the maximum contributing distance

range of land uses and configurations. However, in some instances involving dead-end watermains, standard spacing requirements may not be sufficient to meet RFF.

Standard design practice involves systematic checking of design fire flows at every node in hydraulic models of proposed water distribution systems. Normally the entire design fire flow is applied to each node in succession. Nodes are typically at water main junctions rather than actual hydrant locations. This significantly simplifies the design process and the current software packages that are normally used for this purpose have been developed based on this practice. The "point load assumption" produces a conservative design.

Table 1. Maximum flow to be considered from a given hydrant

Hydrant Class	Distance to asset/structure/building (m)[a]	Contribution to required fire flow (L/min)[b]
AA	≤ 75	5,700
	> 75 and ≤ 150	3,800
A	≤ 75	3,800
	> 75 and ≤ 150	2,850
B	≤ 75	1,900
	> 75 and ≤ 150	1,500
C	≤ 75	800
	> 75 and ≤ 150	800

[a] Distance of contributing hydrant from the structure, measured in accordance with NFPA 1 (Appendix A).

[b] Maximum flow contribution to be considered for a given asset/structure/building, at a residual pressure of 20 psi, measured at the location of the main, at ground level.

4. Intended Application of Guideline

The intent of this procedure is to:

- Determine the appropriate sizing of dead end watermains and associated hydrant requirements.
- Provide an optional approach to local watermain network sizing that will assist the designer in determining the minimum pipe sizing needed to meet RFF.

The procedure permits the designer to: (a) reconcile available hydrant flow with computed RFFs, and (b) allow the distribution of RFFs along multiple hydrants, rather than consider RFF to be a point flow. The application of this protocol may result in reduced watermain diameters compared to those determined based on a traditional design approach. Caution is required in the application of the procedure to ensure that the transmission function of any watermains identified in a Master Servicing Study is not compromised. Normally, watermains 300mm in diameter and larger that are identified in such studies would not be considered for resizing.

5. Application Procedure

5.1 Rated hydrants

The procedure described here would apply to an existing watermain network with existing hydrants (i.e., re-development or infill in existing neighborhoods):

- Identify critical zones within the (re)development area, e.g., high RFF, dead ends, small diameter watermains, low C factor, and/or high geographic elevation zones.
- For the critical zones use Table 1 to examine if there are sufficient hydrants to deliver the RFF (following procedure described in 5.3).
- If hydrant capacity is insufficient, then consider either:
    - adding hydrants as appropriate;
    - determine if the existing hydrants can be upgraded to higher rating; or
    - upgrade existing watermains.

5.2 Un-rated hydrants

There are currently about 24,800 hydrants in the City of Ottawa, of which about 78% are rated. Of the rated hydrants, 96% are AA (Blue), 3% are A (Green). Many of the un-rated hydrants are located in old parts of the City, often installed on water mains with minimum diameter of 6" (150 mm), and would be likely to have a low rating.

Based on a review of hydrants that have been installed as part of recent urban development, approximately 99% of those which were rated are rated AA, and only 1% are rated A.

5.2.1 Un-rated Existing Hydrants

In cases where fire flow is to be evaluated in areas with an established water distribution network and with existing fire hydrants (i.e., re-development or infill in existing neighborhoods), all un-rated hydrants should be tested and rated in accordance with NFPA standard 291. The procedure described in Section 5.1 can then be followed to complete the design.

5.2.2 Planned hydrants

Planned hydrants cannot be tested for rating because they have not been installed yet. Moreover, the rating of a hydrant is an intrinsic property of the hydrant and can therefore not be directly evaluated by simulation. Based on the statistics cited previously, it can be assumed for design purposes that all planned hydrants are AA. However, there could be a situation where the proposed network might not have sufficient capacity to supply 5,700 L/min to a AA-rated hydrant in a specific area. Hydraulic analysis is required to confirm that the distribution network is capable of providing the hydrants with the fire flows in Table 1.

5.3 Hydrant Placement and Watermain Size Optimization

Ottawa design guidelines for watermain sizing and hydrant placement (Section 4) stipulate that the RFF be added to the average hourly rate of a peak day demand. This fire flow is added to hydraulic nodes in the vicinity of the planned development, while ensuring that the residual pressure is at least 140 kPa (measured at the location of the main, at ground level).[2] The following procedure is used to optimize watermain sizing and hydrant placement based on the RFF.

- Place hydrants throughout the development area according to the current Ottawa design guidelines.
- Size water mains and locate hydrants according to standard design procedures. Assume all hydrants are AA-rated.
- Identify the most critical zones in the development area, e.g. highest required fire flows, dead ends, longest distances between junctions, and/or highest elevation. Within these critical zones identify critical structures, i.e. those with highest RFF or greatest distance from proposed hydrant locations. Identify the closest hydrants to these buildings.
- For each critical structure, distribute the RFF according to Table 1 (i.e., assign a flow of 5,700 L/min to all hydrants with a distance of less or equal to 75 m from the test property and 3,800 L/min to all hydrants with a distance of more than 75 m but less or equal to 150 m from the test property) These hydrants are to be represented as hydrant-nodes in the network model, where the hydrant lateral would connect to the proposed water main.

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[2] At the time when this protocol was proposed, the City of Ottawa had in effect Technical Bulletin ISDTB 2014-02, whereby RFF may be capped at 10,000 L/min for single detached dwellings (with a minimum 10 m separation between the backs of adjacent units and for side-by-side town and row houses that comply with the OBC Div. B, subsection 3.1.10 requirement (compartments of no more than 600 m^2 area).

- For each critical structure, run a single fire flow simulation ensuring that the RFF is provided by hydrants within 150 m distance from the test property, with a minimum residual pressure of 140 kPa.
- If the required residual pressure cannot be achieved, consider either re-sizing of pipes, and/or re-spacing of hydrants.

The above procedure is optional except for dead-end watermains servicing cul-de-sacs because (a) based on standard spacing requirements, there would often be insufficient fire flow provided and (b) the watermain would otherwise could be sized larger than necessary and lead to excessive water age and on-going flushing requirements.

Irrespective of the above, if the RFF is equal to or less than 10,000 L/min, then:

- where the distance between two adjacent hydraulic nodes is greater than the inter-hydrant spacing allowed in the guideline, a hydraulic node should be added halfway between the two nodes, and proceed with fire flow simulations to verify watermain sizing, ensuring that the simulation considers RFF at the new hydraulic node.

Attachment A—Excerpts from NFPA 1 Fire Code (2015 Edition)

18.5 Fire Hydrants.

18.5.1 Fire Hydrant Locations and Distribution. Fire hydrants shall be provided in accordance with Section 18.5 for all new buildings, or buildings relocated into the jurisdiction unless otherwise permitted by 18.5.1.1 or 18.5.1.2.

18.5.1.4* The distances specified in Section 18.5 shall be measured along fire department access roads in accordance with 18.2.3.

18.5.1.5 Where fire department access roads are provided with median dividers incapable of being crossed by fire apparatus, or where fire department access roads have traffic counts of more than 30,000 vehicles per day, hydrants shall be placed on both sides of the fire department access road on an alternating basis, and the distances specified by Section 18.5 shall be measured independently of the hydrants on the opposite side of the fire department access road.

18.5.1.6 Fire hydrants shall be located not more than 12 ft (3.7 m) from the fire department access road.

18.5.2 Detached One- and Two-Family Dwellings. Fire hydrants shall be provided for detached one- and two-family dwellings in accordance with both of the following:

(1) The maximum distance to a fire hydrant from the closest point on the building shall not exceed 600 ft (183 m).

(2) The maximum distance between fire hydrants shall not exceed 800 ft (244 m).

18.5.3 Buildings Other than Detached One- and Two-Family Dwellings. Fire hydrants shall be provided for buildings other than detached one- and two-family dwellings in accordance with both of the following:

(1) The maximum distance to a fire hydrant from the closest point on the building shall not exceed 400 ft (122 m).

(2) The maximum distance between fire hydrants shall not exceed 500 ft (152 m).

18.5.4 Minimum Number of Fire Hydrants for Fire Flow.

18.5.4.1 The minimum number of fire hydrants needed to deliver the required fire flow for new buildings in accordance with Section 18.4 shall be determined in accordance with Section 18.5.4.

18.5.4.2 The aggregate fire flow capacity of all fire hydrants within 1000 ft (305 m) of the building, measured in accordance with 18.5.1.4 and 18.5.1.5, shall be not less than the required fire flow determined in accordance with Section 18.4.

18.5.4.3* The maximum fire flow capacity for which a fire hydrant shall be credited shall be as specified by Table 18.5.4.3. Capacities exceeding the values specified in Table 18.5.4.3 shall be permitted when local fire department operations have the ability to accommodate such values as determined by the fire department.

Table 18.5.4.3 Maximum fire flow hydrant capacity

Distance to buildings[a]		Maximum capacity[b]	
(ft)	(m)	(gpm)	(L/min)
≤ 250	≤ 76	1500	5678
> 250 and ≤ 500	> 76 and ≤ 152	1000	3785
> 500 and ≤ 1000	> 152 and ≤ 305	750	2839

[a] Measured in accordance with 18.5.1.4 and 18.5.1.5.

[b] Minimum 20 psi (139.9 kPa) residual pressure.

18.5.4.4 Fire hydrants required by **18.5.2** and **18.5.3** shall be included in the minimum number of fire hydrants for fire flow required by **18.5.4**.

The City of Ottawa design guidelines on hydrant classification conform to the NFPA Standard #291, which recommends the following:

5.1 Classification of Hydrants. Hydrants should be classified in accordance with their rated capacities [at 20 psi (1.4 bar) residual pressure or other designated value as follows:

(1) Class AA — Rated capacity of 1500 gpm (5700L/min) or greater

(2) Class A — Rated capacity of 1000–1499 gpm (3800– 5699 L/min)

(3) Class B — Rated capacity of 500–999 gpm (1900–3799 L/min)

(4) Class C — Rated capacity of less than 500 gpm (1900 L/min)

Appendix F
Geotechnical Investigation

Geotechnical
Engineering

Environmental
Engineering

Hydrogeology

Geological
Engineering

Materials Testing

Building Science

Archaeological Services

patersongroup

Geotechnical Investigation

Proposed Residential Development
Highway 7 at Highway 15, Carleton Place
Ottawa, Ontario

Prepared For

Novatech Engineering Consultants Ltd.

May 1, 2020

Report PG5212-1

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Table of Contents

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Novatech Engineering Consultants Ltd. to conduct a geotechnical investigation for the subject site to be located at Highway 7 at Highway 15, Carleton Place, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- ❑ determine the subsurface soil and groundwater conditions based on available subsoil information and test pit investigation.

- ❑ to provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the proposed development as they are understood at the time of writing this report.

2.0 Proposed Project

Based on available design plans, it is understood that the proposed development will consist of medium density residential dwellings with basement or slab-on-grade construction, attached garages, associated driveways, local roadways and landscaped areas. It is also understood that the development will include a park area as well as a school. It is further anticipated that the site will be municipally serviced by future water, sanitary and storm services, with a stormwater management pond planned within the northernmost corner of the subject site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out on January 9, 2020. A total of 15 test pits were excavated to a maximum depth of 3.8 m below existing grade. It should be noted that previous investigations were conducted by this firm within the subject property in 2012 consisting of a total of 11 test pits excavated to a maximum depth of 2.2 m below existing grade. The test holes were distributed in a manner to provide general coverage of the subject site, with a specific focus on the Northern section of the parcel for storm water management planning. The approximate locations of the test holes are shown on Drawing PG5212-1 - Test Hole Location Plan included in Appendix 2.

The test pits were excavated using a rubber tired backhoe. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test hole procedure consisted of excavating to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples from the test pits from the current investigation were recovered from the side walls of the open excavation and all soil samples were initially classified on site. All samples were transported to our laboratory for further examination and classification. The depths at which the grab samples were recovered from the test holes are shown as G on the Soil Profile and Test Data sheets in Appendix 1.

Undrained shear strength testing, using a hand held vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The subsurface conditions observed at the test pits were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1.

Groundwater

Open hole groundwater infiltration levels were observed at the time of excavation at each test pit location. Our observations are presented in the Soil Profile and Test Data sheets in Appendix 1.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The location of the test pits and ground surface elevation at each test hole location was recovered in the field by Paterson personnel. The ground surface elevation at each test hole location was referenced to a geodetic datum. The location and ground surface elevation at each test hole location is presented on Drawing PG5212-1 - Test Hole Location Plan attached to Appendix 1.

3.3 Laboratory Testing

The soil samples recovered from the subject site were visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by agricultural lands with areas of trees and brush over Steward Lands (RSSR), while Laing/Mutuura Lands are being occupied primarily by trees and dense brush. The ground surface across Stewart Lands is relatively flat, with high points in the southeast and southwest corners sloping North. For Laing/Mutuura Lands a high point just south of the centre of the site was noted sloping downward in all directions at varying slopes (see Drawing PG5212-1 - Test Hole Location Plan in Appendix 1). The site is bordered on the Southeast by rural residential properties, and the Northwest by a vacant forested area (to eventually be Riddell Street just beyond the site boundary). There currently exists two fill piles west of the North corner of the site, at the site boundary, where there also exists an area of dense trees (approximately 3200 sq m). Ditches were noted running Northeast along Highway 15 to the South as well as McNeely Ave to the North at the site boundaries. Directly across McNeely Avenue to the Northwest exists a residential development area.

4.2 Subsurface Profile

Overburden

RSSR or Stewart Lands

The subsoil profile encountered at the test hole locations consist primarily of topsoil overlaying hard to very stiff silty clay (TP 4-20, TP 7-20, TP 12). A layer of glacial till was encountered in most pits, underlaying the silty clay layer (TP 2-20, TP 3-20, TP 5-20, TP 15-20). The glacial till consist of silty sand with gravel, cobbles and boulders within a silty clay soil matrix. In areas of shallow bedrock, growth over bedrock or a thin layer of topsoil was encountered (TP 1-20, TP 6-20, TP13, TP14). Test pit TP 19 near the south corner of the site from the 2012 investigation revealed a layer of silty sand underlaying the topsoil layer and extending down to the bedrock. All test pits in this area were terminated due to practical refusal on inferred bedrock surface refusal was encounter from surface to 3.8 m deep. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each test hole location.

Laing/Mutuura Lands

The subsoil profile encountered at the test hole locations consist primarily of till-like topsoil layer (TP 10-20, TP 12-20, TP 14-20, TP 23, TP 24, TP 25) or topsoil overlaying a silty sand with gravel or glacial till (TP 8-20, TP 9-20, TP 11-20, TP 13-20, TP 20, TP 21, TP 22). All test pits in this area were terminated due to practical refusal on inferred bedrock surface, ranging from depths of 0.22 metres to 1.20 metres from surface. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the subject site is underlain by sandstone and dolomite bedrock of the March Formation extending through most of the subject site. The drift thickness varies between 0 to 1 metres for most of the site, with areas of 2 to 5 metres thickness along the Northeast site boundary.

4.3 Groundwater

Groundwater levels (GWL) were measured in the test pits upon completion of the field program. The results are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings				
Test Pit Number	**Ground Surface Elevation (m)**	**Groundwater Depth (m)**	**Groundwater Level (m)**	**Recording Date**
PG5212 - Highway 7 at Highway 15, Carleton Place				
TP 2-20	130.48	2.40	128.08	January 9, 2020
TP 3-20	131.29	1.45	129.84	January 9, 2020
TP 5-20	130.00	2.40	127.60	January 9, 2020
TP 11-20	135.80	0.80	135.00	January 9, 2020
TP 15-20	132.23	1.10	131.13	January 9, 2020
PG2793 - Highway 7 at Highway 15, Carleton Place				
TP 12	130.15	1.80	128.35	October 22, 2012
TP 19	135.41	1.00	134.41	October 22, 2012

The remaining test pits were dry upon completion. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction. Most shallow test holes were observed to be dry upon completion of the sampling program. The deeper test holes were noted to have minor infiltration through the test pit walls. Based on the moisture levels and colouring of the recovered soil samples, and our experience with the local area, the long-term groundwater table is expected at depths between 4 to 5 m below ground surface. The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed residential dwellings will be founded over conventional shallow footings placed on an undisturbed, very stiff silty clay, compact silty sand, compact glacial till, engineered fill and/or surface-sounded bedrock bearing surface.

Due to the presence of a silty clay deposit, a permissible grade raise restriction is required for the subject site.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Preparation

Stripping Depth

Topsoil, and any deleterious fill, such as those containing organic materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeter. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 metre below final grade.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting may be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed.

A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the structures) should not exceed 25 mm per second during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical side walls. A minimum 1 m horizontal ledge, should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system. However, should the entire area be required to accommodate the parking garage, drilled piles into the weathered portion of the bedrock can be used to support the upper levels of the excavation and can be placed at the property boundary.

Vibration Considerations

Construction operations are also the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipments could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles or sheet piling will require the use of this equipment. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). It should be noted that these guidelines are for today's construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey be completed to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If excavated stiff brown silty clay, free of organics and deleterious materials, is to be used to build up the subgrade level for areas to be paved, the silty clay, under dry conditions, should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

In-filling the existing ditches should be completed in a stepped fashion within the lateral support of the proposed buildings. The fill should consist of clean imported granular fill, such as OPSS Granular A or Granular B Type II material. The steps should have a minimum horizontal length of 1.5 m and minimum vertical height of 0.5 m and should be compacted using suitable compaction equipment to a minimum 98% of the material's SPMDD. All backfilling and compaction efforts should be reviewed and approved by Paterson personnel at the time of construction.

5.3 Foundation Design

Shallow Foundation

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, very stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit state (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit state (ULS) of **225 kPa**.

Footings placed on an undisturbed, compact glacial till bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa**.

Footings placed on an undisturbed, compact silty sand bearing surface can be designed using a bearing resistance value at SLS of **100 kPa** and a factored bearing resistance value at ULS of **175 kPa**.

Footings placed over an approved engineered fill bearing surface over an undisturbed, very stiff silty clay or compact silty sand bearing surface can be designed using a bearing resistance value at SLS of **150 kPa** and a factored bearing resistance value at ULS of **225 kPa.**

Footings placed over an approved engineered fill bearing surface over a clean, surface sounded bedrock bearing surface can be designed using a factored bearing resistance value at ULS of **1,000 kPa** using a geotechnical factor of 0.5**.**

Footings designed using the above noted bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

Footings bearing on a clean, surface sounded bedrock and designed using the above noted bearing resistance values will be subjected to negligible post-construction total and differential settlements. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance values at ULS.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near or surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Permissible Grade Raise

A **permissible grade raise restriction of 2.0 m** is recommended for areas where building foundations are founded over a silty clay deposit. Areas affected by a permissible grade raise restriction due to the presence of a silty clay deposit are indicated in Drawing PG5212-2 - Permissible Grade Raise Areas in Appendix 2. Footings bearing on a compact glacial till, silty sand and/or bedrock bearing surface will not subjected to permissible grade raise restrictions.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a stiff silty clay above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1H:6V passing through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock should be provided with a lateral support zone of 1.5H:1V.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for foundations considered for the subject site. A higher seismic site class such as Class A or B may be applicable for foundations located within the eastern portion of the subject site where shallow bedrock was encountered. However, the higher site class would have to be confirmed by site specific shear wave velocity testing. The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest version of the Ontario Building Code (OBC) 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab / Slab on Grade Construction

With the removal of all topsoil and deleterious fill from within the footprint of the proposed buildings, the native soil surface or approved fill will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab.

For structures with slab-on-grade construction, the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

For structures with basement slabs, it is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone.

5.6 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of driveways, local residential streets and roadways with bus traffic. It should be noted that for residential driveways and car only parking areas, an Ontario Traffic Category A is applicable. For local roadways and roadways with bus traffic, an Ontario Traffic Category B and Category D should be used for design purposes, respectively.

Table 2 - Recommended Pavement Structure - Driveways/Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either approved fill, in situ soil or OPSS Granular B Type I or Type II material placed over in situ soil or approved fill	

Table 3 - Recommended Pavement Structure - Local Residential Roadways

Thickness (mm)	Material Description
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either approved fill, in situ soil or OPSS Granular B Type I or Type II material placed over in situ soil or approved fill	

Table 4 - Recommended Pavement Structure - Roadways with Bus Traffic	
Thickness mm	**Material Description**
40	**Wear Course -** Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** - Superpave 19.0 Asphaltic Concrete
50	**Lower Binder Course** - Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
550	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil or OPSS Granular B Type I or Type II material placed over in situ soil or approved fill	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for driveways and local roadways and PG 64-34 asphalt cement should be used for roadways with bus traffic. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Where silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction as per City of Ottawa standards. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or an approved equivalent. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. However, the bedding thickness should be increased to 300 mm for areas over a bedrock subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe).

The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 99% of the material’s SPMDD.

Based on the soil profile encountered, the subgrade for the services will be placed in both bedrock and overburden soils. It is recommended that the subgrade medium be inspected in the field to determine how steeply the bedrock surface, where encountered, drops off. A transition should be provided where the bedrock slopes more than 3H:1V. At these locations, the bedrock should be excavated and replaced with addition bedding materials to provide a 3H:1V (or flatter) transition from the bedrock subgrade towards the soil subgrade. This treatment reduced the propensity for bending stress to occur in the service pipes.

Generally, it should be possible to re-use the moist, not wet, silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches. The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

A temporary Ministry of the Environment and Climate Change (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. These results are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The results of the chloride content, pH and resistivity indicate the presence of a non-aggressive to slightly aggressive environment for exposed ferrous metals at this site.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation services program including the following aspects be performed by the geotechnical consultant.

❏ Grading plan review from a geotechnical perspective, once the final grading plan is available.

❏ Observation of all bearing surfaces prior to the placement of concrete.

❏ Sampling and testing of the concrete and fill materials used.

❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

❏ Observation of all subgrades prior to backfilling.

❏ Field density tests to determine the level of compaction achieved.

❏ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the grading plan once available. Paterson's recommendations should also be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and the test hole logs are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests to be notified immediately in order to permit reassessment of the recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Novatech Engineering Consultants Ltd. or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Joey R. Villeneuve, M.A.Sc.,P.Eng

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

- ❏ Novatech Engineering Consultants Ltd.
- ❏ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

FILE NO. PG5212

HOLE NO. TP 1-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	129.60		
TOPSOIL 0.10									
End of Test Pit TP terminated on bedrock surface at 0.10m depth (TP dry upon completion)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

FILE NO. PG5212

REMARKS

HOLE NO. TP 2-20

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	130.48		
TOPSOIL (to 0.30)		G	1						
Hard, brown **SILTY CLAY** (to 1.70)		G	2			1	129.48	▲ 200	
GLACIAL TILL: Brown silty clay with gravel, cobbles and boulders (to 3.80)		G	3			2	128.48		∇
		G	4			3	127.48		
End of Test Pit									

TP terminated on inferred bedrock surface at 3.80m depth

(Groundwater infiltration at 2.4m depth)

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

FILE NO. PG5212

REMARKS

HOLE NO. TP 3-20

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	131.29		
TOPSOIL		G	1						
0.40									
Hard, brown **SILTY CLAY**		G	2			1	130.29	▲ 200	
1.30									∇
GLACIAL TILL: Grey-brown silty clay with gravel, cobbles and boulders		G	3			2	129.29		
2.80									
End of Test Pit TP terminated on inferred bedrock surface at 2.80m depth (Groundwater infiltration at 1.45m depth)									

Water Content % axis: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

FILE NO. PG5212

HOLE NO. TP 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	130.88		
TOPSOIL 0.20		G	1						
Very stiff, brown **SILTY CLAY,** trace organics 0.50		G	2						
End of Test Pit TP terminated on inferred bedrock surface at 0.50m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

FILE NO. PG5212

REMARKS

HOLE NO. TP 5-20

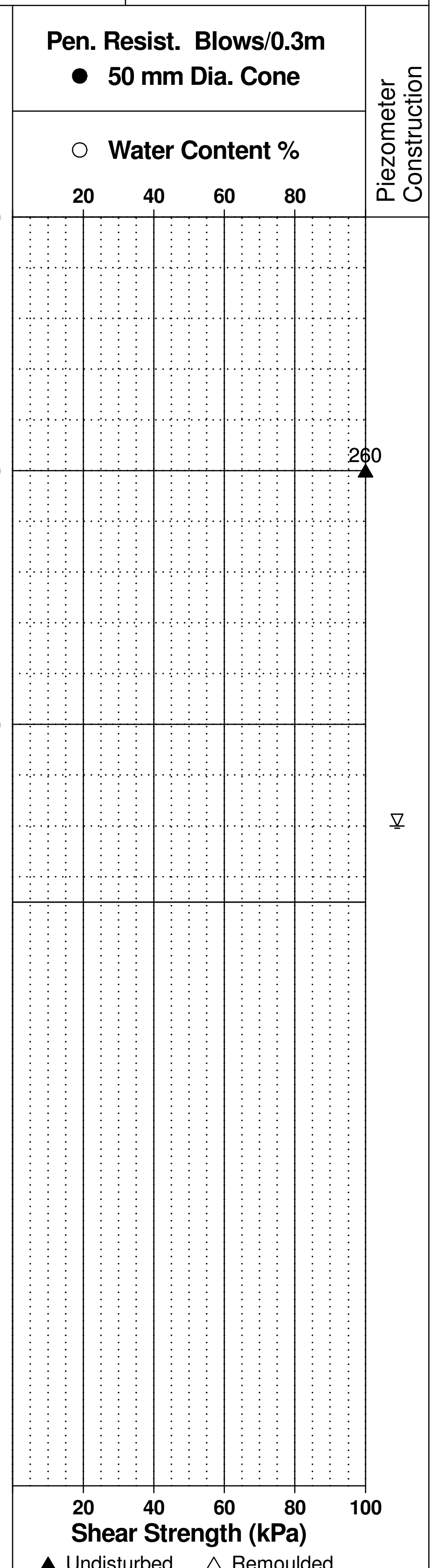
BORINGS BY Hydraulic Shovel

DATE 2020 January 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	130.00
TOPSOIL (to 0.48)		G	1				
Hard, brown **SILTY CLAY** (0.48 to 1.40)		G	2			1	129.00
GLACIAL TILL: Brown silty clay with gravel, cobbles and boulders (1.40 to 2.70)		G	3			2	128.00
End of Test Pit (2.70)							

TP terminated on inferred bedrock surface at 2.70m depth

(Groundwater infiltration at 2.4m depth)



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

FILE NO. PG5212

REMARKS

HOLE NO. TP 6-20

BORINGS BY Hydraulic Shovel **DATE** 2020 January 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	132.34		
TOPSOIL 0.25		G	1						
End of Test Pit TP terminated on inferred bedrock surface at 0.25m depth (TP dry upon completion)									

Shear Strength (kPa): 20, 40, 60, 80, 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

FILE NO. PG5212

HOLE NO. TP 7-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	131.26		
TOPSOIL 0.30		G	1						
Very stiff, brown **SILTY CLAY,** trace sand 0.50		G	2						
End of Test Pit TP terminated on inferred bedrock surface at 0.50m depth (TP dry upon completion)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

FILE NO. PG5212

HOLE NO. TP 8-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	136.66		
TOPSOIL ... 0.30		G	1						
GLACIAL TILL: Brown silty clay with gravel, cobbles and boulders ... 0.85		G	2						
End of Test Pit TP terminated on inferred bedrock surface at 0.85m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic
FILE NO. PG5212
REMARKS
HOLE NO. TP 9-20
BORINGS BY Hydraulic Shovel
DATE 2020 January 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	140.07	20 40 60 80	
Loose to compact, brown **SILTY SAND** with gravel, cobbles and boulders, some clay		G	1						
0.80									
End of Test Pit TP terminated on inferred bedrock surface at 0.80m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

FILE NO. PG5212

HOLE NO. TP10-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	136.14		
TOPSOIL 0.40		G	1						
End of Test Pit TP terminated on inferred bedrock surface at 0.40m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

FILE NO. PG5212

HOLE NO. TP11-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	135.80		
TOPSOIL 0.30		G	1						
Brown **SILTY SAND** with clay, trace gravel 0.95		G	2						∇
End of Test Pit TP terminated on inferred bedrock surface at 0.95m depth (Groundwater infiltration at 0.8m depth)									

Water Content % scale: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

FILE NO. PG5212

HOLE NO. TP12-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	135.22		
TOPSOIL 0.35		G	1						
End of Test Pit TP terminated on inferred bedrock surface at 0.35m depth (TP dry upon completion)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

FILE NO. PG5212

HOLE NO. TP13-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	137.22		
TOPSOIL 0.40									
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders 1.20		G	1			1	136.22		
End of Test Pit TP terminated on inferred bedrock surface at 1.20m depth (TP dry upon completion)									

Shear Strength (kPa): 20, 40, 60, 80, 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

FILE NO. PG5212

REMARKS

HOLE NO. TP14-20

BORINGS BY Hydraulic Shovel

DATE 2020 January 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	138.07		
TOPSOIL		G	1						
0.50									
End of Test Pit TP terminated on inferred bedrock surface at 0.50m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Highway No. 7
Carleton Place, Ontario

DATUM Geodetic

FILE NO. PG5212

REMARKS

HOLE NO. TP15-20

BORINGS BY Hydraulic Shovel

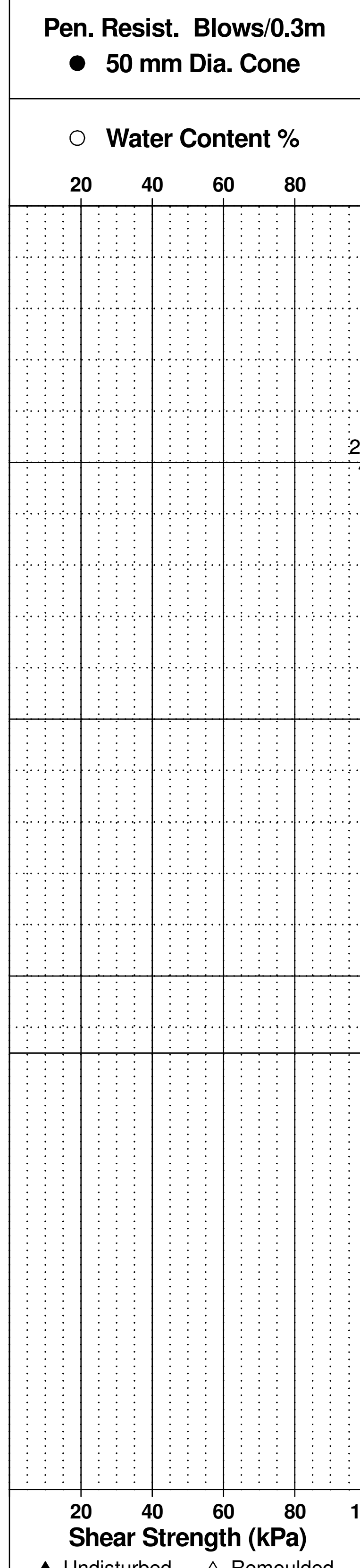
DATE 2020 January 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	132.23
TOPSOIL (to 0.25)		G	1				
Brown **SILTY SAND** (0.25 to 0.40)		G	2				
Hard, brown **SILTY CLAY** (0.40 to 1.70)		G	3			1	131.23
GLACIAL TILL: Grey-brown silty clay with sand, gravel, cobbles and boulders (1.70 to 3.30)		G	4			2	130.23
		G	5			3	129.23
End of Test Pit (3.30)							

End of Test Pit

TP terminated on inferred bedrock surface at 3.30m depth

(Groundwater infiltration at 1.1m depth)



Piezometer Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG2793

REMARKS

HOLE NO. TP12

BORINGS BY Backhoe

DATE October 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	130.15		
Brown **SILTY CLAY** with sand						1	129.15		
						2	128.15		▽
2.24									
End of Test Pit Practical shovel refusal on inferred bedrock at 2.24m depth (Water infiltration at 1.8m depth)									

Water Content %: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG2793

REMARKS

HOLE NO. TP13

BORINGS BY Backhoe

DATE October 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	137.82		
TOPSOIL 0.04									
End of Test Pit Practical shovel refusal on inferred bedrock at 0.04m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG2793

REMARKS

HOLE NO. TP14

BORINGS BY Backhoe

DATE October 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	133.03		
TOPSOIL 0.28		G	1						
End of Test Pit Practical shovel refusal on inferred bedrock at 0.28m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG2793

REMARKS

HOLE NO. TP15

BORINGS BY Backhoe

DATE October 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	130.68		
TOPSOIL 0.12									
Brown **SILTY CLAY,** trace sand 0.59		G	1						
End of Test Pit Practical shovel refusal on inferred bedrock at 0.59m depth (TP dry upon completion)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. **PG2793**

REMARKS

HOLE NO. **TP19**

BORINGS BY Backhoe

DATE October 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	135.41
TOPSOIL 0.15							
Brown to grey **SILTY SAND,** trace clay and gravel 1.60						1	134.41
End of Test Pit Practical shovel refusal on inferred bedrock at 1.60m depth (Water infiltration at 1.0m depth)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY Backhoe

DATE October 22, 2012

FILE NO. PG2793

HOLE NO. TP20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	135.24		
TOPSOIL 0.11									
Brown **SILTY SAND** with clay 0.43		G	1						
End of Test Pit Practical shovel refusal on inferred bedrock at 0.43m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG2793

REMARKS

HOLE NO. TP21

BORINGS BY Backhoe

DATE October 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	135.50		
TOPSOIL 0.15									
Brown **SILTY SAND** with clay 0.49									
End of Test Pit									
Practical shovel refusal on inferred bedrock at 0.49m depth									
(TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG2793

REMARKS

HOLE NO. TP22

BORINGS BY Backhoe

DATE October 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	135.76		
TOPSOIL 0.23									
Brown **SILTY SAND** with clay 0.98		G	1						
End of Test Pit Practical shovel refusal on inferred bedrock at 0.98m depth (TP dry upon completion)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

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FILE NO. PG2793

REMARKS

HOLE NO. TP23

BORINGS BY Backhoe

DATE October 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	135.71		
TOPSOIL 0.38									
End of Test Pit Practical shovel refusal on inferred bedrock at 0.38m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

FILE NO. PG2793

REMARKS

HOLE NO. TP24

BORINGS BY Backhoe

DATE October 22, 2012

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	138.56		
TOPSOIL 0.22		G	1						
End of Test Pit Practical shovel refusal on inferred bedrock at 0.22m depth (TP dry upon completion)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Preliminary Geotechnical Investigation
Proposed Residential Development - Highway 7
Carleton Place, Ontario

DATUM Ground surface elevations provided by Novatech Engineering Consultants Ltd.

REMARKS

BORINGS BY Backhoe

DATE October 22, 2012

FILE NO. PG2793

HOLE NO. TP25

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	139.04		
TOPSOIL 0.27									
End of Test Pit Practical shovel refusal on inferred bedrock at 0.27m depth (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

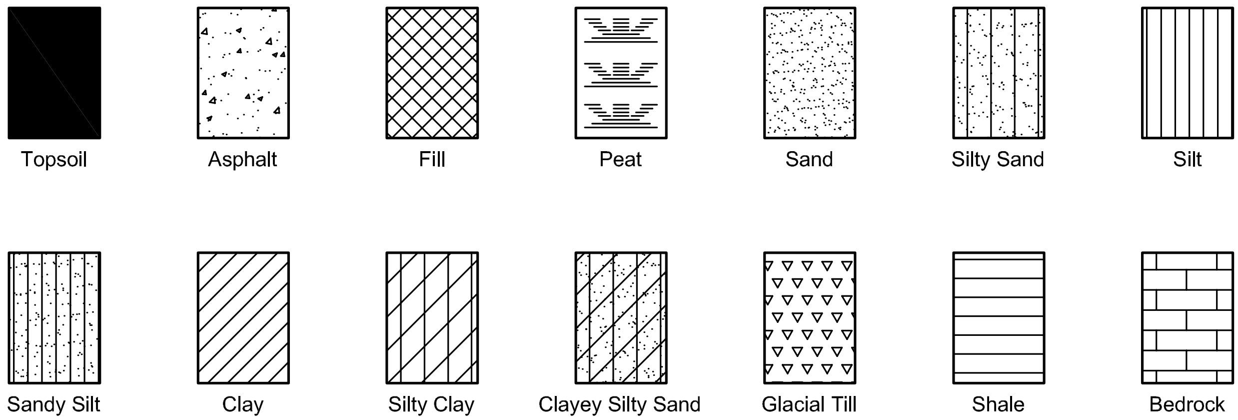
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

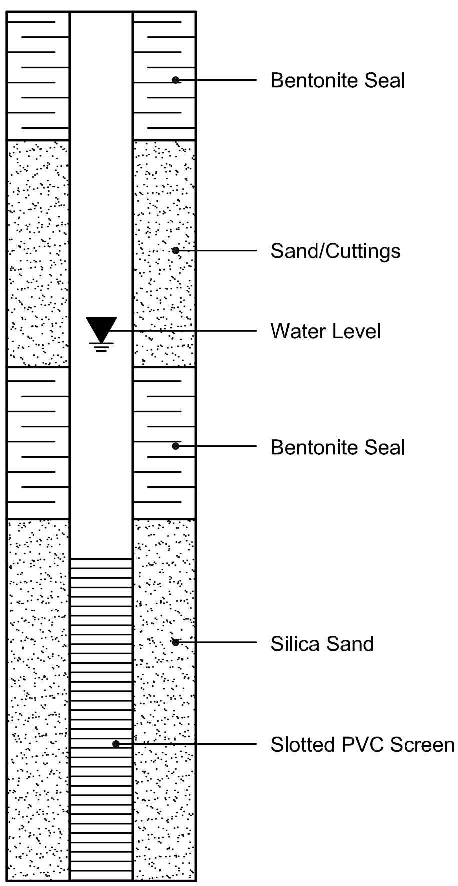
SYMBOLS AND TERMS (continued)

STRATA PLOT

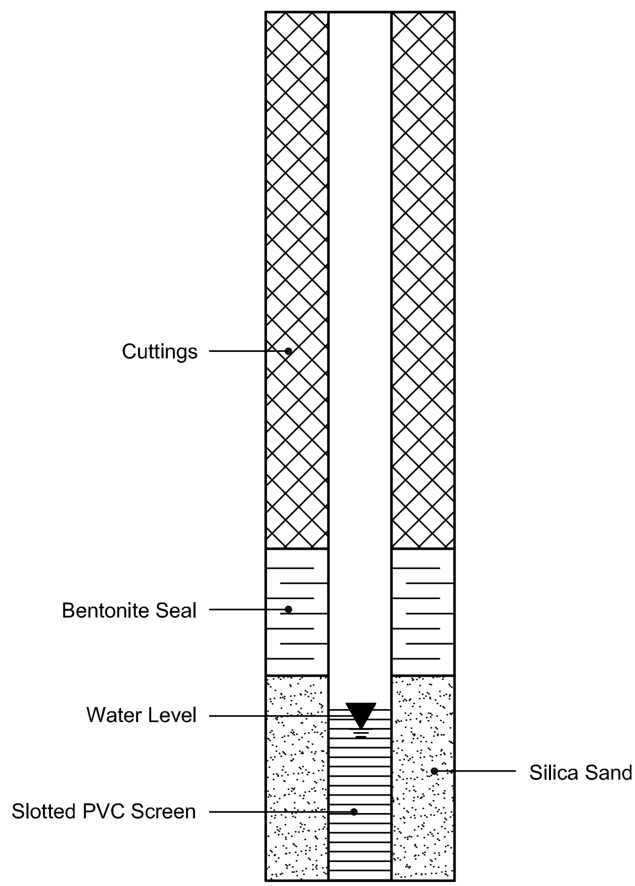


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 25635

Report Date: 15-Jan-2020

Order Date: 13-Jan-2020

Project Description: PG5212

	Client ID:	TPR GS4 - 1.9 To 2.0	-	-	-
	Sample Date:	09-Jan-20 13:00	-	-	-
	Sample ID:	2003096-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	73.4	-	-	-
General Inorganics					
pH	0.05 pH Units	7.80	-	-	-
Resistivity	0.10 Ohm.m	73.2	-	-	-
Anions					
Chloride	5 ug/g dry	11	-	-	-
Sulphate	5 ug/g dry	16	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5212-1- TEST HOLE LOCATION PLAN

DRAWING PG5212-2 - PERMISSIBLE GRADE RAISE AREAS

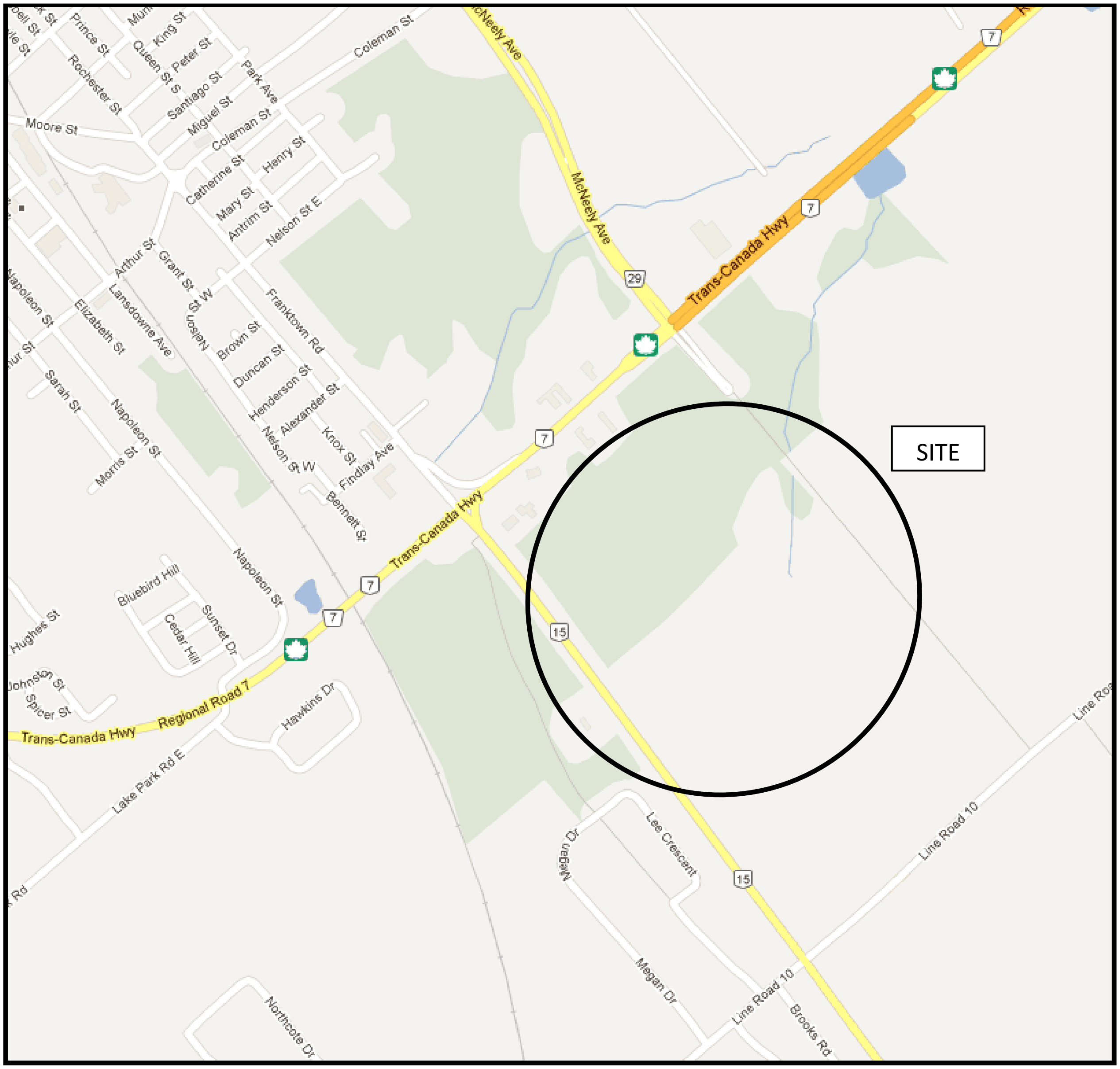
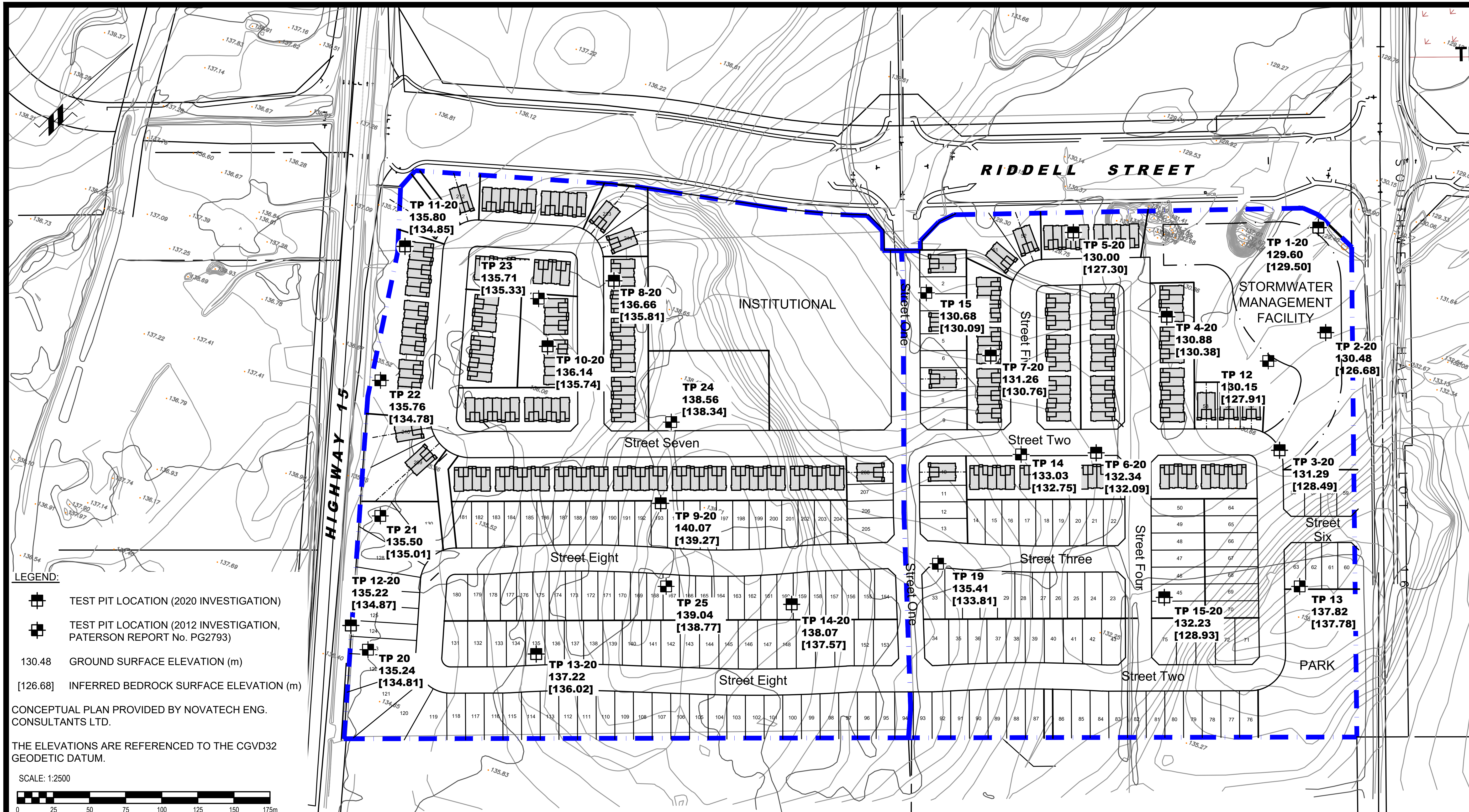


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

- TEST PIT LOCATION (2020 INVESTIGATION)
- TEST PIT LOCATION (2012 INVESTIGATION, PATERSON REPORT No. PG2793)
- 130.48 GROUND SURFACE ELEVATION (m)
- [126.68] INFERRED BEDROCK SURFACE ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY NOVATECH ENG. CONSULTANTS LTD.

THE ELEVATIONS ARE REFERENCED TO THE CGVD32 GEODETIC DATUM.

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO NEW CONCEPTUAL PLAN	05/19/2021	FAS

REGIONAL GROUP c/o NOVATECH ENGINEERING CONSULTANTS LTD.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - HIGHWAY 7

CARLETON PLACE, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2500	Date: 01/2020
Drawn by: NFRV	Report No.: PG5212-1
Checked by: DP	Dwg. No.: PG5212-1
Approved by: FAS	Revision No.: 1

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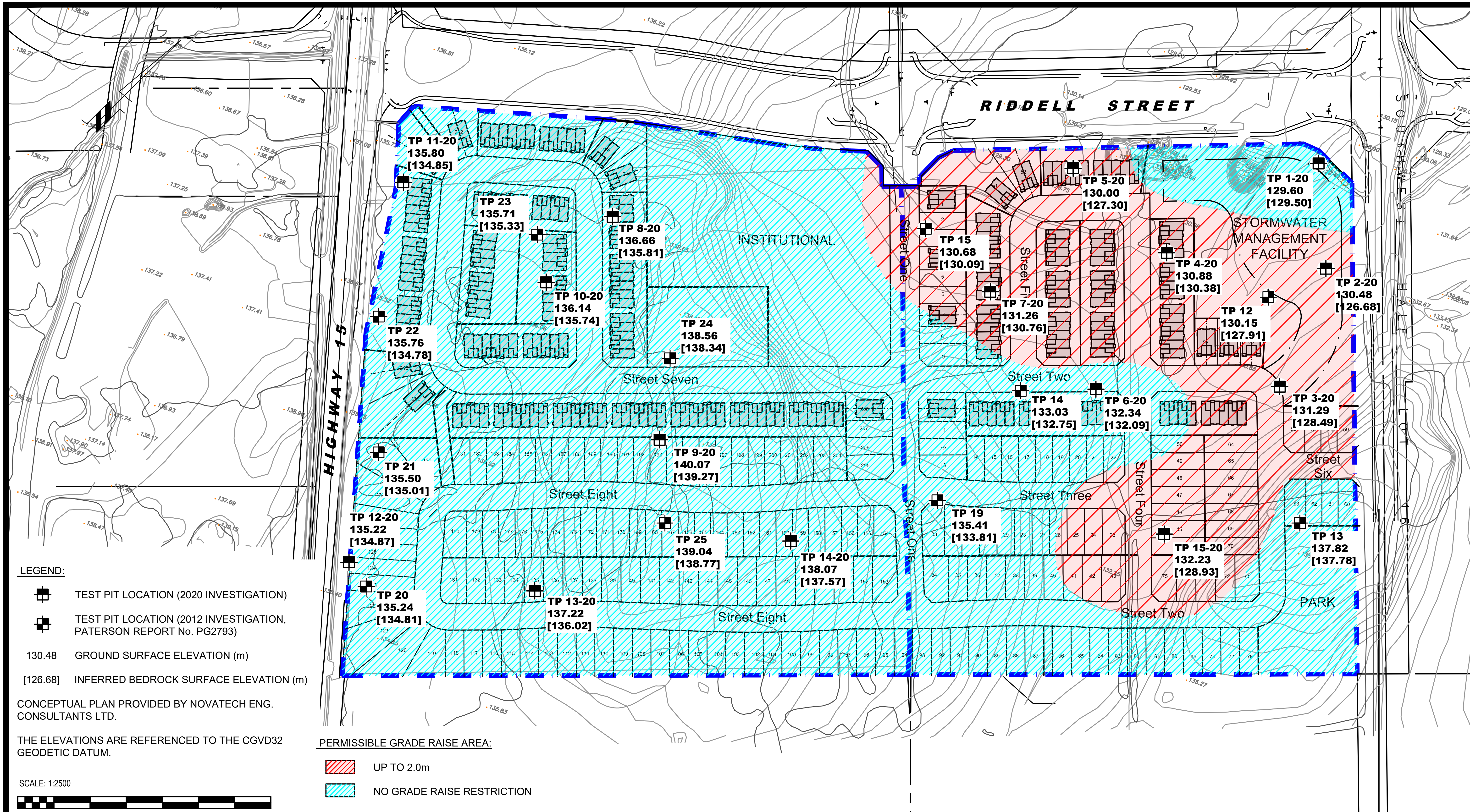

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CONCEPTUAL PLAN PROVIDED BY NOVATECH ENG. CONSULTANTS LTD.

THE ELEVATIONS ARE REFERENCED TO THE CGVD32 GEODETIC DATUM.

SCALE: 1:2500

0 25 50 75 100 125 150 175m

PERMISSIBLE GRADE RAISE AREA:

- UP TO 2.0m
- NO GRADE RAISE RESTRICTION

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Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO NEW CONCEPTUAL PLAN	05/19/2021	FAS

REGIONAL GROUP c/o NOVATECH ENGINEERING CONSULTANTS LTD.

GEOTECHNICAL INVESTIGATION
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CARLETON PLACE, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:2500	Date: 01/2020
Drawn by: NFRV	Report No.: PG5212-1
Checked by: DP	Dwg. No.: PG5212-2
Approved by: FAS	Revision No.: 1

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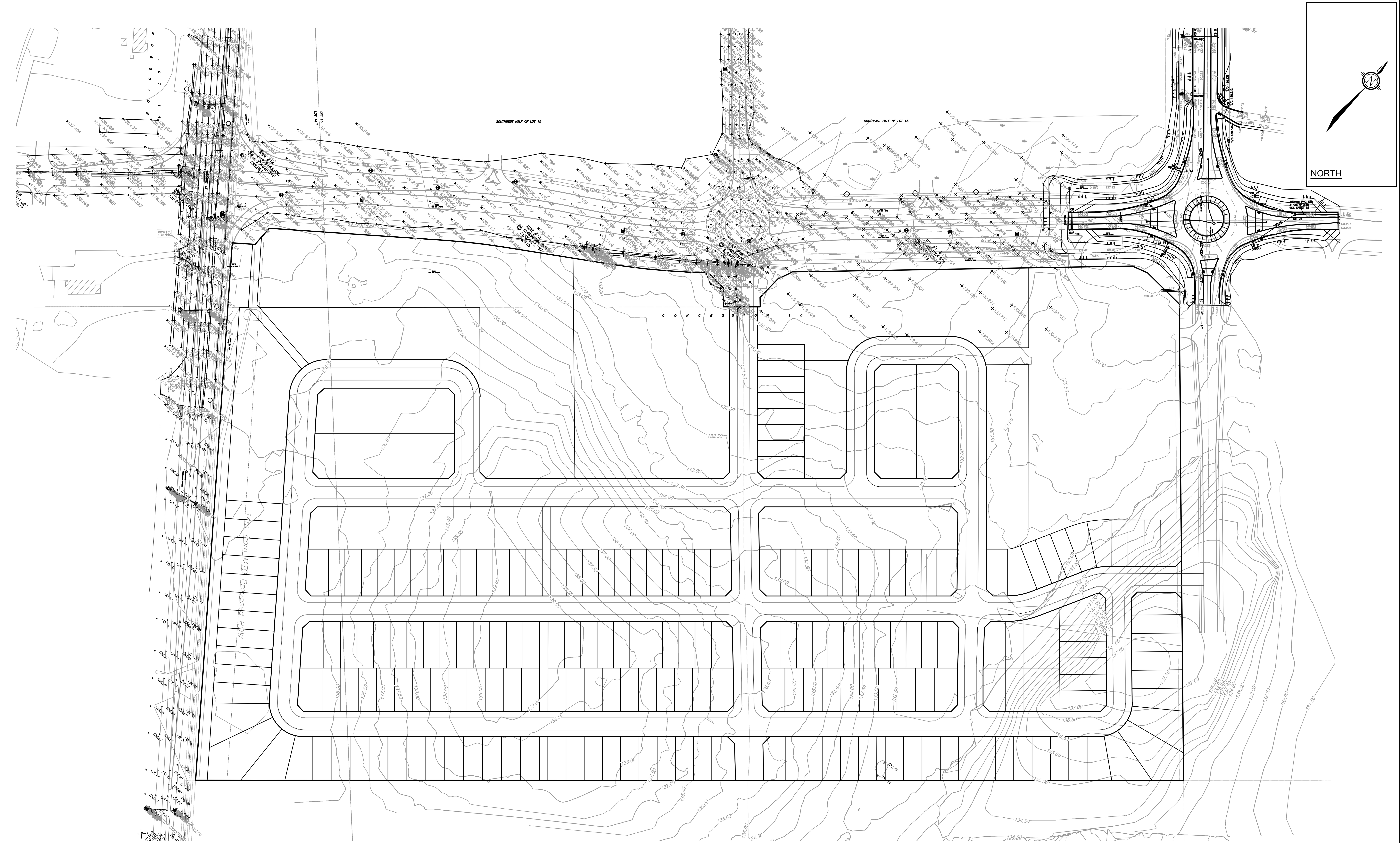
List of Enclosures

Topographic Survey



NOTE

TOPOGRAPHIC SURVEY CONSISTS OF COMPILED SURVEYS FROM CALLON DIETZ (2021), NOVATECH (2021), AND RH PRECISION UNMANNED INC. (2019)

COORDINATE SYSTEM

MODIFIED TRANSVERSE MERCATOR (M.T.M.) GRID, MTM ZONE 9, NAD83 (CSRS)(2010)

NOTE:
THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION OF ALL SUCH UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

No.	REVISION	DATE	BY
1.	ISSUED FOR INFORMATION	MAY 16/22	BHB

SCALE 1:1250

DESIGN XXX
CHECKED XXX
DRAWN XXX
CHECKED XXX
APPROVED XXX

FOR REVIEW ONLY

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Ottawa, Ontario, Canada K2M 1P6
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Website www.novatech-eng.com

LOCATION
TOWN OF CARLETON PLACE
MCNEELY LANDING

DRAWING NAME
TOPOGRAPHIC SURVEY

PROJECT No. 119221-00
REV REV # 1
DRAWING No. 119221-TOPO

PLANA1.DWG - 841mmx594mm