Kanata West Block 29 – Servicing and Stormwater Management Report

Stantec Project No. 160401608

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Prepared for:

Richcraft Group of Companies Inc.

Prepared by:

Stantec Consulting Ltd.
400-1331 Clyde Avenue
Ottawa ON K2C 3G4

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Prepared by ______________________________

Daniel Chochlinski, EIT

Reviewed by ______________________________

Ana Paerez, P.Eng.

LICENSED PROFESSIONAL ENGINEER
A.M. PAEREZ FERNANDEZ
100105130
2021-03-25
PROVINCE OF ONTARIO

Approved by ______________________________

Sheridan Gillis

Table of Contents

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

Richcraft Group of Companies Inc. (Richcraft) has commissioned Stantec Consulting Ltd. (Stantec) to prepare the following Servicing and Stormwater Management Report in support of the Site Plan Application for Block 29 of the Kanata West Subdivision.

The subject site is within the First Registration Phase of the Kanata West Subdivision in the city of Ottawa, bound by Maize Street to the south, Poole Creek to the west and north, and Roger Griffiths Avenue to the east (refer to **Figure 1** below). The First Registration Phase of the Kanata West Subdivision has been approved and is currently under construction. It is expected that Roger Griffiths Avenue and Maize Street will be completed before the servicing works for Block 29 begin.

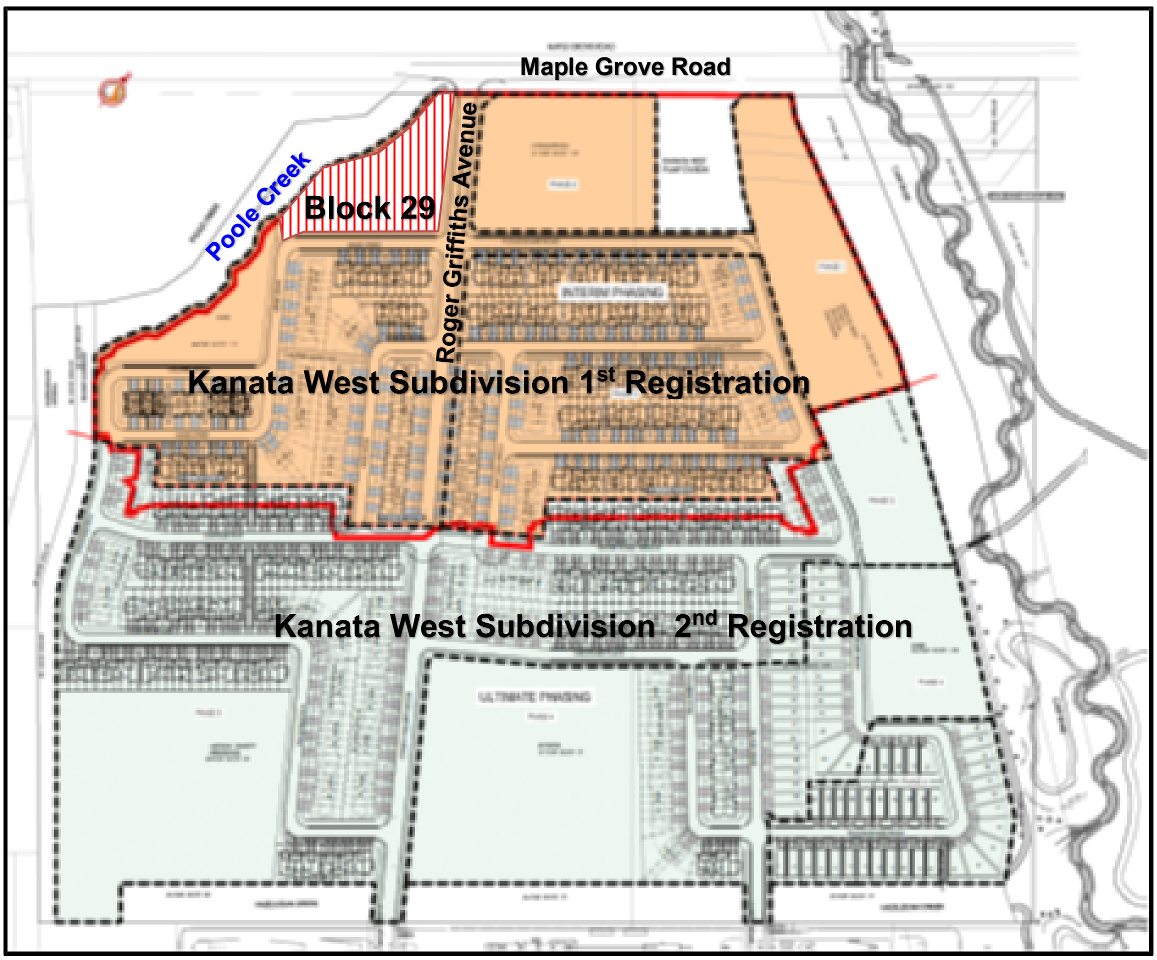


Figure 1: Key Map of Kanata West Block 29

The subject property is currently zoned R4Z (Residential Fourth Density) and occupies 0.74 ha of land. The site is currently undeveloped. The proposed development consists of forty-eight (48) 2-bedroom terrace units as shown in the draft plan included in **Appendix E**.

Preliminary servicing and stormwater management analysis for the Block 29 site was completed as part of the *Kanata West Development First Registration Phase Servicing and Stormwater Management* (SWM) Report completed by Stantec Consulting Ltd. in February 2020 and referenced throughout this report.

1.1 OBJECTIVE

This site servicing and stormwater management (SWM) report has been prepared to present an internal servicing scheme that is free of conflicts, uses existing/approved infrastructure, and meets all design criteria as identified in background documents and City of Ottawa design guidelines.

2.0 REFERENCE DOCUMENTS

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

- *City of Ottawa Sewer Design Guidelines*, 2nd Edition, City of Ottawa, October 2012.
- *City of Ottawa Design Guidelines – Water Distribution*, 1st Edition, Infrastructure Services Department, City of Ottawa, July 2010.
- *Technical Bulletin ISDTB-2014-02 Revision to Ottawa Design Guidelines – Water*, City of Ottawa, May 2014.
- *Technical Bulletin PIEDTB-2016-01 Revisions to Ottawa Design Guidelines – Sewer,* City of Ottawa, September 2016.
- *Technical Bulletin ISTB-2018-01 Revision to Ottawa Design Guidelines – Sewer,* City of Ottawa, March 2018.
- *Technical Bulletin ISTB-2018-02 Revision to Ottawa Design Guidelines – Water Distribution,* City of Ottawa, March 2018.
- *Richcraft Kanata West Development First Registration Phase, 1620 Maple Grove Road (D07-16-04-0017) – Servicing and Stormwater Management Report*, Stantec Consulting Ltd., February 12, 2020.
- *Interim Kanata West Pond 5 (with Ultimate Carp River) Design Brief – 1620 Maple Grove Road – D07-16-04-0017*, Stantec Consulting Ltd., July 12, 2019.
- *Geotechnical Investigation – Proposed Residential Development – Kanata West Block 29*, Paterson Group Inc., July 14, 2020.
- *Phase I – Environmental Site Assessment Update – Block 29 of Kanata West Development*, Paterson Group Inc., July 6, 2020.

3.0 POTABLE WATER SERVICING

3.1 BACKGROUND

The proposed development is located within Zone 3W of the City of Ottawa's water distribution system. This zone is fed by the Glen Cairn Pump Station. The site will be fed by a 300 mm diameter watermain on Maize Street, to be constructed as part of the 1^{st} construction phase of the Kanata West Development.

As part of the First Registration Phase of the Kanata West Development, which provides all municipal services for Block 29, three H2OMAP Water hydraulic analyses were completed to assess different build-out scenarios: i) 1^{st} Construction Phase, ii) First Registration Phase (i.e., Interim Condition), and iii) Ultimate Condition (First and Second Registration Phases). To be conservative, boundary conditions from the worst case of these three build-out scenarios have been used in the hydraulic analysis for Block 29.

The Block 29 population was previously estimated as 76 persons in the *Kanata West Development First Registration Phase Servicing and Stormwater Management Report* (Stantec, February 2020).

3.2 PROPOSED WATERMAIN SIZING AND LAYOUT

3.2.1 Connections to Existing Infrastructure

The proposed watermain alignment and sizing for the development is demonstrated on **Drawing SSP-1**. A 204 mm diameter watermain is proposed to follow the alignment of the private roads within the subject property with a connection to the existing 300 mm diameter watermain on Maize Street at the entrance to the Block 29 site.

Figure 2 shows the layout of the proposed watermain network and the connection to the existing watermain on Maize Street.

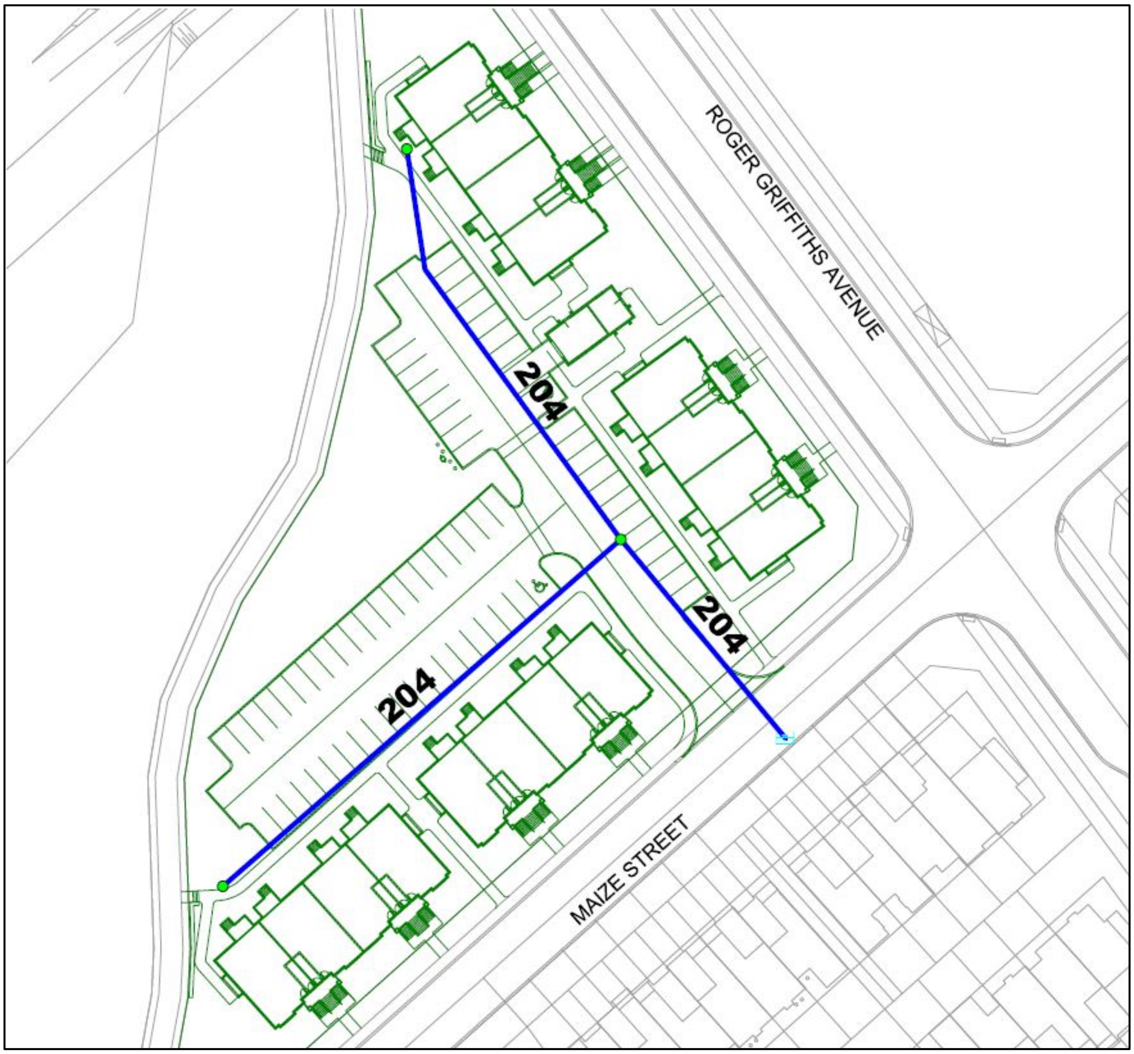


Figure 2: Proposed Watermain Layout and Pipe Diameters (mm)

3.2.2 Ground Elevations

Proposed ground elevations throughout the Block 29 site range from approximately 97.83 m to 98.23 m at nodes in the watermain network.

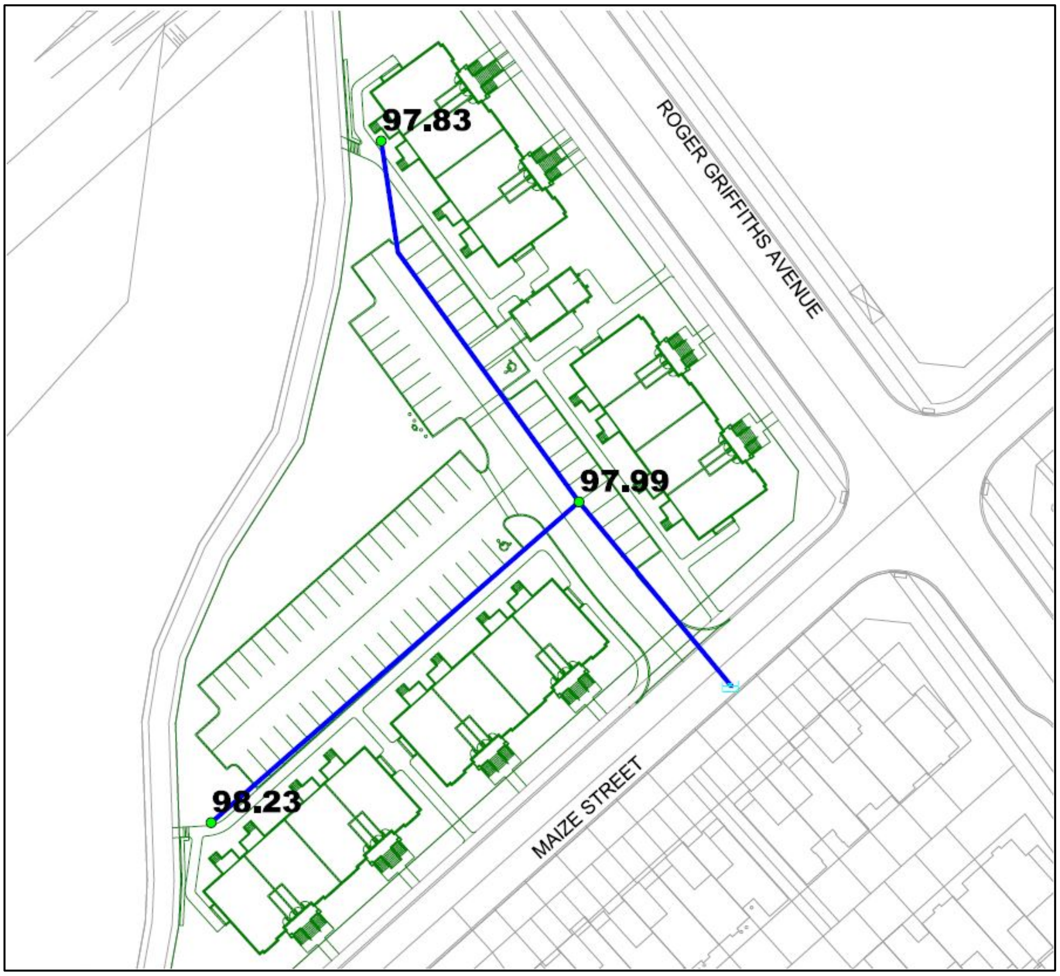


Figure 3: Ground Elevations (m) at Nodes

3.2.3 Domestic Water Demands

Kanata West Block 29 contains a total of forty-eight (48) 2-bedroom terrace units, with an estimated total population of 101 persons. Refer to **Appendix A.1** for detailed domestic water demand calculations.

Water demands for the development were estimated using the City of Ottawa's Water Distribution Design Guidelines. For residential developments, the average day (AVDY) per capita water demand is 350 L/cap/d. For maximum day (MXDY) demand, AVDY was multiplied by a factor of 2.5 and for peak hour (PKHR) demand, MXDY was multiplied by a factor of 2.2. The calculated residential water consumption is represented in **Table 1**.

Table 1: Residential Water Demands for Block 29

Unit Type	Units	Persons/Unit	Population	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
2-Bedroom Terrace Units	48	2.1	101	0.41	1.02	2.25

3.3 LEVEL OF SERVICE

3.3.1 Allowable Pressures

The City of Ottawa Water Distribution Design Guidelines state that the desired range of system pressures under normal demand conditions (i.e., basic day, maximum day, and peak hour) should be in the range of 350 to 552 kPa (50 to 80 psi) and no less than 275 kPa (40 psi) at the ground elevation on the streets (i.e., at hydrant level). The maximum pressure at any point in the distribution system in occupied areas outside of the public right-of-way is 552 kPa (80 psi). As per the Ontario Building Code (OBC) & Guide for Plumbing, if pressures greater than 552 kPa (80 psi) are anticipated, pressure relief measures are required. The maximum pressure at any point in the distribution system in unoccupied areas shall not exceed 689 kPa (100 psi). Under emergency fire flow conditions, the minimum pressure objective in the distribution system is 138 kPa (20 psi).

3.3.2 Fire Flow Demands

Fire flow calculations were completed using the Fire Underwriters Survey (FUS) methodology. Refer to **Appendix A.2** for detailed FUS calculations. The results of the fire flow calculations are summarized in **Table 2**.

Table 2: Fire Flow Calculations Using FUS Methodology

Unit Type	Description	Required Fire Flow (L/min)	Required Fire Flow (L/s)
2-Bedroom Terrace Units	3-storey building with twelve 2-bedroom terrace units (worst-case exposures: Block 3).	15,000	250

The highest fire flow requirement of 15,000 L/min (250 L/s) is used for the purpose of the fire flow analysis.

3.4 HYDRAULIC ANALYSIS

A hydraulic model for the proposed Block 29 watermain layout was completed using H2OMAP Water with the boundary condition from the hydraulic model prepared for the Richcraft Kanata West Development First Registration Phase Servicing and SWM Report (Stantec, February 2020). Three hydraulic models were prepared for the Kanata West Development as follows: i) 1st Construction Phase, ii) First Registration Phase (i.e., "Interim Condition"), and iii) Ultimate Condition (First and Second Registration Phases).

The boundary conditions from the Kanata West Development hydraulic models at the node nearest to Block 29 (intersection of Roger Griffiths Avenue and Maize Street) are summarized in **Table 3**. The 267 L/s fire flow demand condition from the Kanata West Development hydraulic model serves as a conservative boundary condition for the 250 L/s (15,000 L/min) fire flow demand calculated for the Block 29 site.

Table 3: Boundary Conditions for Block 29 from KW Development Hydraulic Model

	1st Construction Phase	1st Registration Phase (Interim Condition)	Ultimate Condition (1st and 2nd Registration Phases complete)
Maximum HGL (AVDY), Head (m)	161.74	**161.66**	161.38
PKHR, Head (m)	157.73	**157.59**	157.66
MXDY+FF (267 L/s), Head (m)	149.42	**149.27**	153.95

1. Boundary conditions taken from Node 10 in the Kanata West Development H2OMAP Water hydraulic model (Stantec, 2020). Ground elevation at Node 10 = 97.88 m in the model.

The MXDY plus fire flow head is lowest for the First Registration Phase (Interim Condition). Therefore, these values have been used as conservative boundary conditions for Block 29.

The anticipated pressures in this development were assessed to meet minimum servicing requirements (average day and peak hour demands). A fire flow analysis was also performed under maximum day conditions. Detailed results are shown in **Appendix A.3**.

3.4.1 Model Development

New watermains were added to the hydraulic model to simulate the proposed distribution system. Hazen-Williams coefficients ("C-Factors") were applied to the new watermain in accordance with the City of Ottawa's Water Distribution Design Guidelines (**Table 4**).

Table 4: C-Factors Applied Based on Watermain Diameter

Nominal Pipe Diameter (mm)	C-Factor
150	100
200 to 250	110
300 to 600	120
Over 600	130

3.4.1.1 Average Day & Peak Hour

The hydraulic model results show that the maximum pressures (AVDY condition) are anticipated to be approximately 622-626 kPa (90.2-90.7 psi) within the Block 29 site. Minimum pressures during PKHR conditions are anticipated to be approximately 582-586 kPa (84.4-85.0 psi) for Block 29. These pressures are well above the minimum allowable pressure of 276 kPa (40 psi). Because the pressures exceed 80 psi, pressure reducing valves (PRVs) are required for all proposed units.

Figure 4 and **Figure 5** below identify the minimum and maximum pressure results for the simulation, respectively.

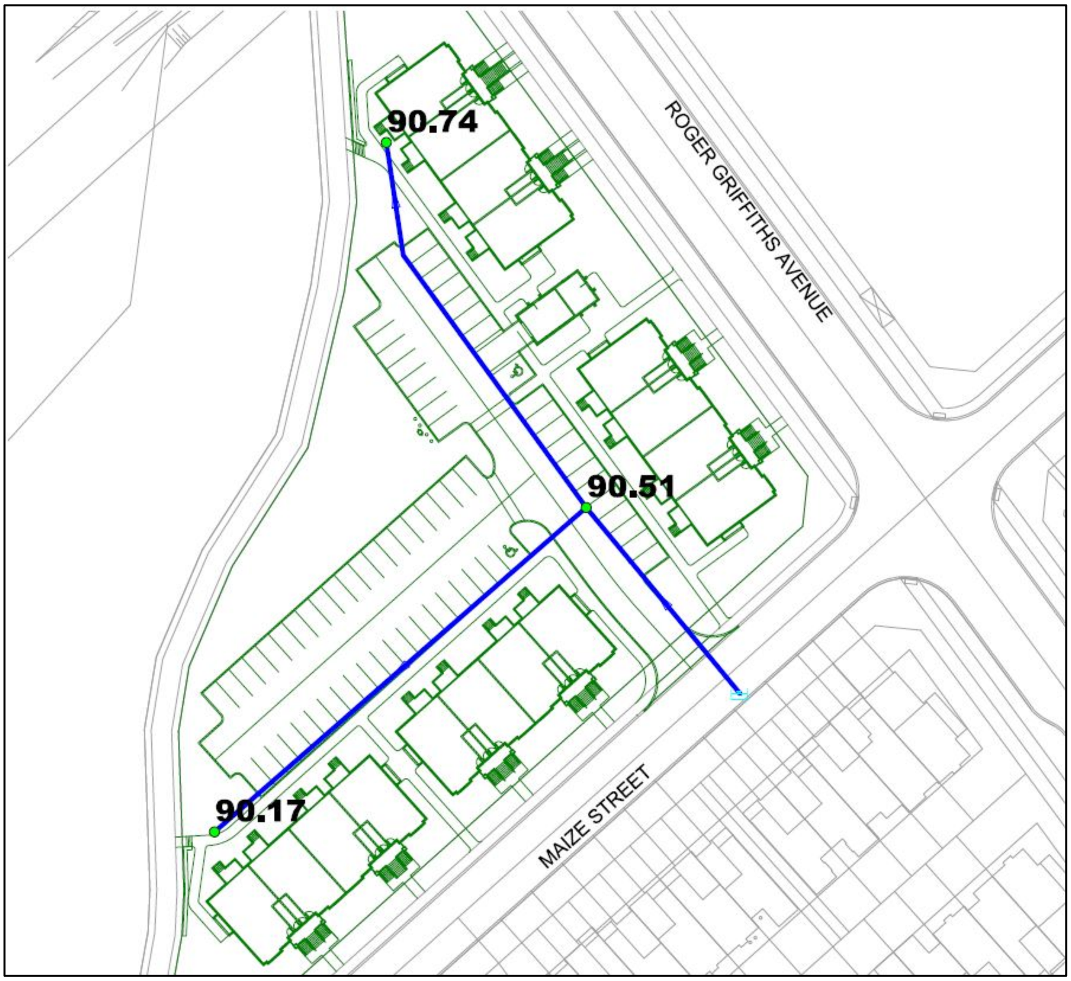


Figure 4: Maximum Pressures (psi) in Block 29 During AVDY Conditions

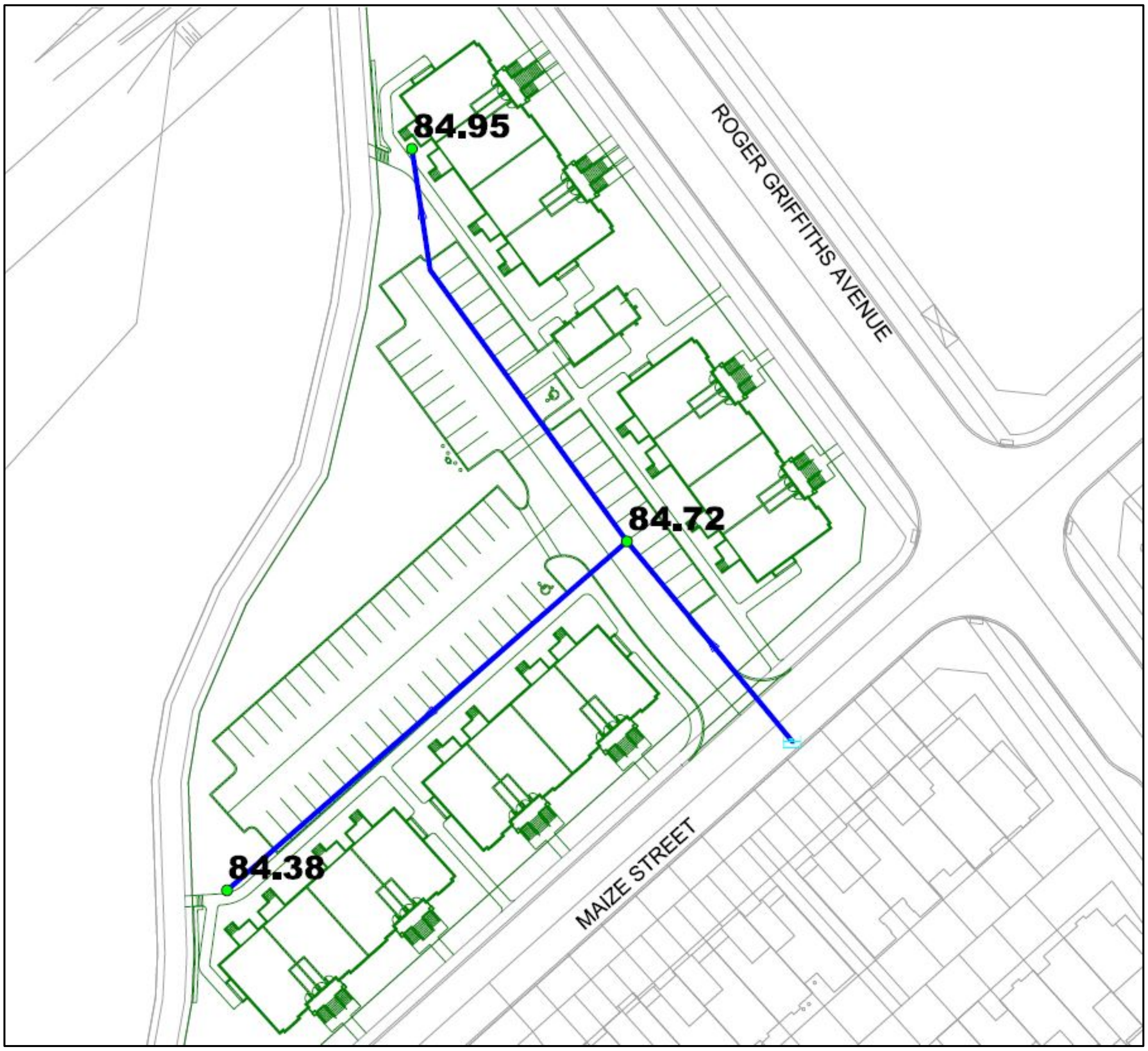


Figure 5: Minimum Pressures (psi) in Block 29 During PKHR Conditions

3.4.1.2 Maximum Day Plus Fire flow

An analysis was carried out using the hydraulic model to determine if the proposed development, under maximum day demands, can achieve a fire flow of 15,000 L/min (250 L/s) while maintaining a residual pressure of 138 kPa (20 psi). This was accomplished using a steady-state maximum day demand scenario along with the automated fire flow simulation feature of H2OMAP Water. The available flows are shown in **Figure 6**.

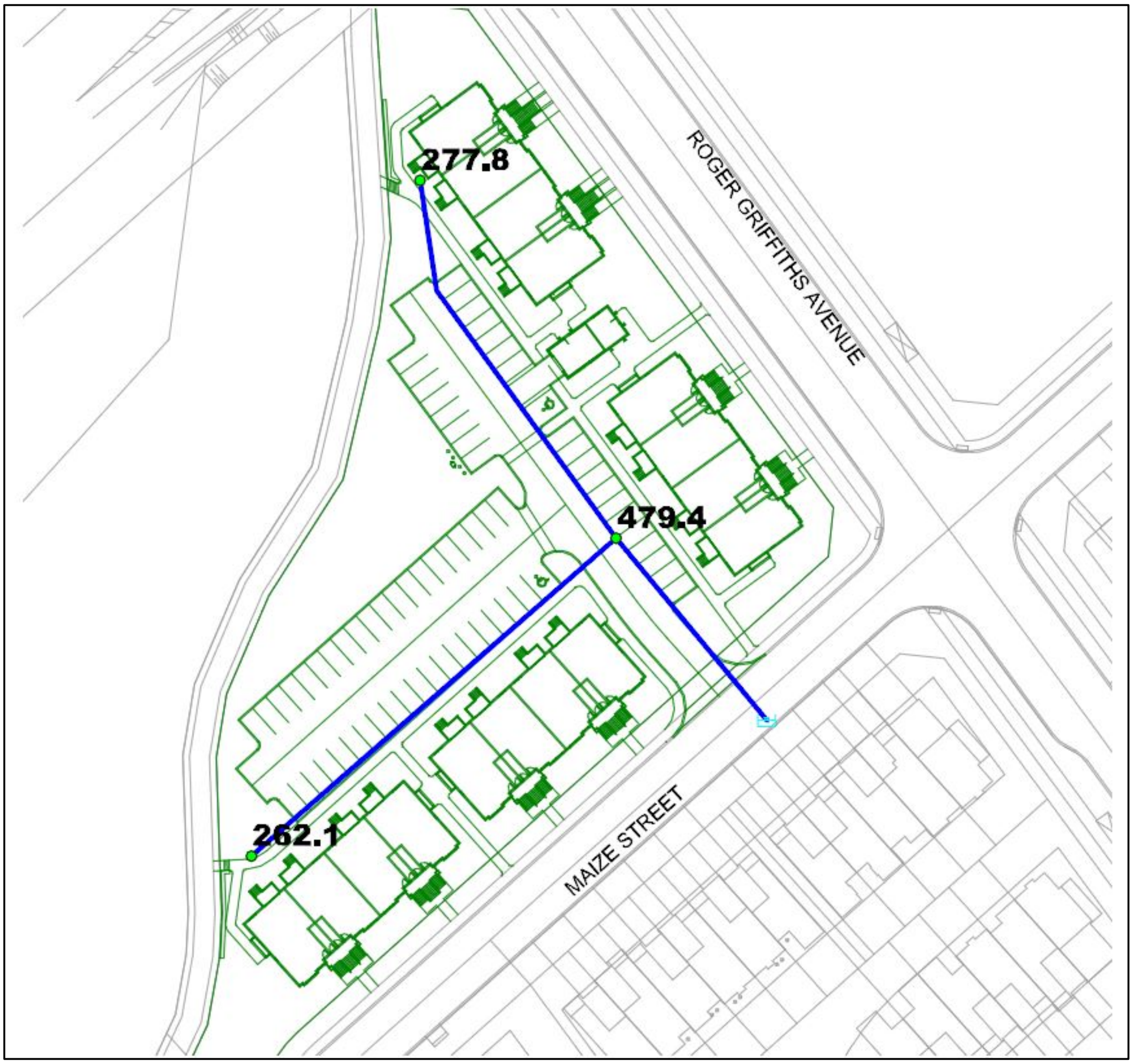


Figure 6: Available Fire Flows (L/s) in Block 29 During MXDY Conditions

Using the proposed pipe layout and sizing, a fire flow of 15,000 L/min (250 L/s) can be achieved while maintaining at least 20 psi residual pressure at all locations upon development of Block 29.

4.0 WASTEWATER SERVICING

4.1 BACKGROUND

As indicated in the *Kanata West Development First Registration Phase Servicing and Stormwater Management Report* (Stantec, February 2020), wastewater from the Kanata West Development is conveyed to the existing 1200 mm diameter sanitary sewer on Maple Grove Road via a free flow gravity trunk sewer. Wastewater from the Kanata West Development is ultimately conveyed to the Kanata West Pump Station.

The *Kanata West Development First Registration Phase Servicing and Stormwater Management Report* (Stantec, February 2020) identifies Block 29 as sanitary drainage area 'R4B.' Furthermore, this report specifies that the sanitary outlet for Block 29 is to be made downstream of SAN MH 4 on Maize Street. The population of Block 29 was previously estimated as 76 persons.

4.2 DESIGN CRITERIA

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

- Minimum full flow velocity – 0.6 m/s
- Maximum full flow velocity – 3.0 m/s
- Manning's roughness coefficient for all smooth-walled pipes – 0.013
- Single family home persons per unit – 3.4
- Townhouse persons per unit – 2.7
- 2-bedroom apartments persons per unit – 2.1
- Extraneous flow allowance – 0.33 L/s/ha
- Residential average flows – 280 L/cap/day
- Commercial/mixed-use flows – 28,000 L/ha/day
- Maintenance hole spacing – 120 m for pipes under 450 mm diameter, 150 m for pipes 450 mm diameter and larger
- Minimum cover – 2.5 m
- Harmon correction factor – 0.8

In addition, a residential peak factor based on Harmon's Equation was used to determine the peak design flows, per the City of Ottawa Sewer Design Guidelines.

Refer to **Appendix B** for the sanitary sewer design sheet for the proposed Kanata West Block 29 site.

4.3 SANITARY SERVICING DESIGN

200 mm diameter sanitary sewers are proposed throughout the Block 29 site. A 200 mm diameter sanitary sewer on Maize Street will be installed as part of the Kanata West Subdivision First Registration servicing works and will serve as the sanitary outlet for the site. Sanitary flows will then be directed

northwards along Roger Griffiths Avenue, then eastwards along Maple Grove Road towards the Kanata West Pump Station. The proposed sanitary sewers within the Block 29 site will not convey any upstream sanitary flows. The proposed sanitary sewer layout for the subject site is shown in **Drawings SSP-1** and **SA-1** in **Appendix F**. The sanitary sewer design sheet is included in **Appendix B.1**.

The proposed peak flows from Block 29 are summarized in **Table 5** below.

Table 5: Sanitary Peak Flow at Proposed SAN MH 1

MH ID	Total area (ha)	Population	Peak Flow (L/s)	Sewer Diameter (mm)
SAN MH 1, Block 29 contribution	0.74	101	1.4	200

The Servicing and SWM Report for the First Registration of the Kanata West Development assumed a population of 76 for the Block 29 site, which has since been revised to 101 based on the proposed site plan. To ensure that the sanitary sewer on Maize Street is sufficiently sized to accept additional peak flows from the Block 29 development given the population increase, the sanitary sewer design sheet from the First Registration of the Kanata West Subdivision was used to assess the capacity of the approved sanitary sewers. The results are summarized in **Table 6** below. Background information, including the Kanata West Development sanitary sewer design sheet, is provided in **Appendix B.2**.

Table 6: Comparison of Expected Residential Sanitary Peak Flows for Block 29

MH ID	Formerly Expected Population for Block 29 from First Registration KW Subdivision Report	Revised Expected Population for Block 29	Formerly Expected Sanitary Peak Flow (L/s)	Revised Expected Sanitary Peak Flow (L/s)
KW SAN 3	76	101	1.1	1.4

The above table shows a 0.3 L/s increase in the expected sanitary peak flows due to the higher population density of the Block 29 development and as can be seen in the Kanata West Subdivision sanitary sewer design sheet attached in **Appendix B.2**, the residual capacity of the approved downstream sanitary sewers ranges between 12.0 L/s and 252.2 L/s, which is well above the proposed sanitary peak flow increase.

5.0 STORMWATER MANAGEMENT AND STORM SERVICING

The proposed development encompasses approximately 0.74 ha of land within the Block 29 property. The entire development is residential containing three-storey terrace flat units. As shown on **Drawing SD-1**, post-development minor system peak flows from the development will be discharged to the approved 675 mm diameter storm sewer on Maize Street. Overland flows during major storm events will be directed to Maize Street and then along the Ploughshare Road right-of-way to the Kanata West Pond 5, located east of the site. Stormwater quality control (80% TSS removal) is provided by the existing interim Kanata West Pond 5, as described in the *Interim Kanata West Pond 5 (with Ultimate Carp River) Design Brief* (Stantec, July 2019). Refer to **Appendix C.6** for the storm drainage plan and storm sewer design sheet for the First Registration of the Kanata West Subdivision (Stantec, 2020).

In the existing condition, a portion of the site drains to the northwest into Poole Creek and the remainder of the site drains southwards towards Maize Street. An interim drainage ditch runs through the southern portion of Block 29 draining from west to east.

5.1 BACKGROUND

Stantec Consulting Ltd. completed the detailed design of the First Registration Richcraft Kanata West (KW) Subdivision in February 2020. The design of the storm sewers and KW Pond 5 in the KW site accounted for the future development in Block 29 (previously referred to as Block 115 or MD1).

Major and minor system flows are to be conveyed to the interim Kanata West Pond 5 for quality and quantity control per the *Kanata West Development First Registration Servicing and Stormwater Management Report* (Stantec, February 2020).

Additional SWM criteria from this report are listed in the proceeding sections.

5.2 STORMWATER MANAGEMENT DESIGN

5.2.1 Design Criteria and Constraints

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

General

- Use of the dual drainage principle (City of Ottawa).
- Wherever feasible and practical, site-level measures should be used to reduce and control the volume and rate of runoff (City of Ottawa).
- Using the 3-Hour Chicago design event, assess the impact of the 2-year storm, 100-year storm, and 100-year+20% climate change event, as outlined in the City of Ottawa Sewer Design Guidelines, on the major and minor drainage system (City of Ottawa).

Storm Sewer & Inlet Controls

- Proposed site to discharge to the existing 675 mm diameter storm sewer on Maize Street, downstream of STM MH 123 (*Kanata West Development First Registration Servicing and Stormwater Management Report*, Stantec).
- Minor system discharge rate from Block 29 not to exceed **111** L/s; major system overflows from the site not to exceed **480 L/s** in the 100-year event (*Kanata West Development First Registration Servicing and Stormwater Management Report*, Stantec).
- Size storm sewers to convey the 2-year storm event under free-flow conditions using 2012 City of Ottawa I-D-F parameters. (City of Ottawa).
- 100-year storm hydraulic grade line (HGL) to be a minimum of 0.30 m below building foundation footing (City of Ottawa).
- Climate change event HGL to be below building foundation footing (City of Ottawa).

Surface Storage & Overland Flow

- Overland flow from Block 29 to be directed to the Maize Street right-of-way (Stantec)
- No surface ponding is permitted within the site during the 2-year storm event (City of Ottawa).
- Maximum depth of flow under either static or dynamic conditions shall be less than 0.35m for design storm events (i.e., up to 100-year storm) (City of Ottawa).
- Minimum clearance depth of 0.15m to be provided from spill elevations within the proposed rights-of-way to building envelopes in proximity of overland flow routes or ponding areas.
- Water must not encroach upon proposed building envelopes and must remain below all proposed building openings during the climate change event (City of Ottawa).
- Provide adequate emergency overflow conveyance off-site (City of Ottawa).
- No rear-yard ponding volumes to be accounted for in SWM model preparation (City of Ottawa).
- The product of depth times velocity on streets not to be greater than 0.6 during the 100-year storm event (City of Ottawa).

The site is to be designed using the “dual drainage” principle, whereby the minor (pipe) system in local roads is designed to convey the peak rate of runoff from the 2-year design storm and runoff from larger events is conveyed by both minor (pipe) and major (overland) channels, such as roadways and walkways, safely off site without impacting proposed or existing downstream properties.

In keeping with the 2-year inlet restriction criterion, inlet control devices (ICDs) or orifice plates are specified for all street catchbasins to limit the inflow to the minor system. Restricted inlet rates to the sewer are necessary to prevent the hydraulic grade line from surcharging storm sewers into basements during major storms. **Drawing SD-1** outlines the proposed storm sewer alignment and drainage divides.

5.3 POST-DEVELOPMENT MODELLING

Hydrologic and hydraulic modeling of the storm system was completed using PCSWMM modeling software which uses the EPA-SWMM 5.1.015 computational engine for analysis. The included models can also be opened and reviewed using the free EPA-SWMM GUI. PCSWMM model layout, input parameters, and example input file are provided in **Appendix C**. Modelling files have been provided as part of the digital submission. The following sections summarize the input parameters used in the post-development model.

5.3.1 Allowable Release Rate

The minor and major system allowable release rates from the Block 29 site are based on the *Kanata West Development First Registration Servicing and Stormwater Management Report* (Stantec, February 2020). The minor and major system target release rates are summarized in **Table 7** below.

Table 7: Block 29 Minor and Major System Target Release Rates

Minor System Target Release Rate (L/s)	Major System Target Release Rate (L/s)
111.0	480.0

1. Block 29 was shown to discharge its minor system to STM MH 123 in the PCSWMM Model for the *Richcraft Kanata West Development First Registration Phase Servicing and Stormwater Management Report* (Stantec, February 2020). Block 29 is now proposed to outlet to proposed STM MH 101, which will be placed immediately downstream of STM MH 123.

5.3.2 Modelling Rationale

A comprehensive hydrologic modeling exercise was completed with PCSWMM, accounting for the estimated major and minor systems to evaluate the storm sewer infrastructure. The use of PCSWMM for modeling of the site hydrology and hydraulics allowed for an analysis of the systems response during various storm events. Surface storage estimates were based on the final grading plan design (see **Drawing GP-1**). The following assumptions were applied to the detailed model:

- Hydrologic parameters as per Ottawa Sewer Design Guidelines, including Manning's 'n', and depression storage values.
- Subcatchment infiltration parameters per Horton Infiltration method per Ottawa Sewer Design Guidelines.
- 3-hour Chicago Storm distribution for the 2-year, 100-year, and 100-year+20% events. All roof runoff is to be directed to parking areas where it will be controlled.
- To 'stress test' the system, a 'climate change' scenario was created by adding 20% of the individual intensity values of the 100-year Chicago storm event at their specified time step.
- Percent imperviousness (imp.) calculated based on actual soft and hard surfaces on each subcatchment, converted to equivalent Runoff Coefficient (C) using the relationship C = (imp. x 0.7) + 0.2.
- Subcatchment areas are defined from high-point to high-point where sags occur. Subcatchment width determined by multiplying street segment length x 2 (length of overland flow path measured from high point to high point) for street (double-sided) catchments, multiplying by 1.5 for single-loaded roads, multiplying by 1.0 for single-sided catchments, or by multiplying the subcatchment area by 225m where a street segment flow path has not otherwise been defined.
- Number of catchbasins based on proposed servicing plans (**Drawing SSP-1**)
- Catchbasin inflow restricted with inlet-control devices (ICDs) as necessary to maintain inflow target rate, maximize use of surface storage, and ensure no standing water during the 2-year event (5-year event level of service for collector roads).

- Surface storage within the site was modelled with tabular storage curves. The storage volumes at each ponding area in the site were determined using the conical volume equation based on the ponding areas and depths determined in the grading plan. As PCSWMM uses the average area equation for determining volumes within storage curves, an equivalent area was calculated to match the volume calculated using the conical volume equation at the maximum static ponding depth identified per the grading plan.
- For Block 29, weirs representing the roadway width were used to model the major system flows between adjacent low points. Active storage volumes were applied at each low point node corresponding to catchbasin surface ponding volumes as noted on **Drawing SD-1**.

5.3.2.1 SWM Dual Drainage Methodology

The proposed development is modelled in one modelling program as a dual conduit system (see **Figure 7**), with: 1) circular conduits representing the sewers and junction nodes representing manholes; 2) weirs representing the spill grade elevations between low points at the top of static ponding, and storage nodes representing catchbasins. The dual drainage systems are connected via orifices from storage node (i.e., CB) to junction (i.e., MH), and represent inlet control devices (ICDs). Subcatchments are linked to the storage node on the surface so that generated hydrographs are directed there first.

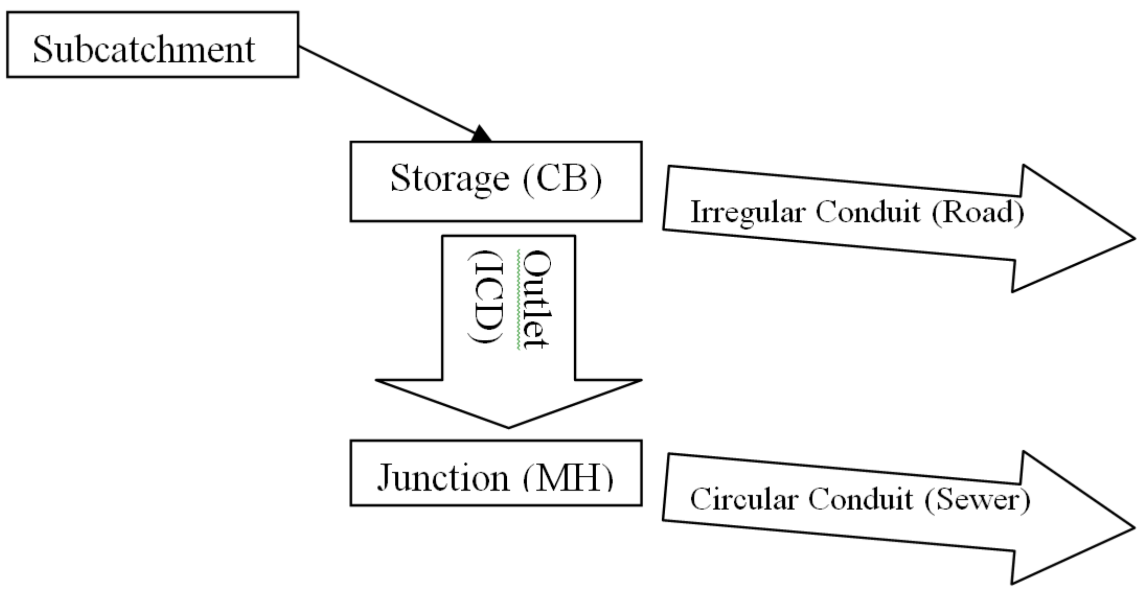


Figure 7: Schematic Representing Model Object Roles

Storage nodes are used in the model to represent catchbasins (CBs). The invert of the storage node represents the invert of the CB and the rim of the storage node represents the top of the CB plus the allowable flow depth on the segment. CB inverts have been set based on actual inverts noted on **Drawing SSP-1**, and a 0.40m buffer has been applied to rim elevations to model surface water depths above the CB.

The proposed Block 29 site conveys its minor system peak flows to the approved 675 mm diameter storm sewer on Maize Street via a connection at a proposed maintenance hole (STM MH 101). The site's major system peak flows are also directed to Maize Street. Due to grading restrictions, a small portion of Block 29 site will drain uncontrolled to Roger Griffiths Avenue, Maize Street, and Poole Creek.

5.3.3 Boundary Conditions

The downstream storm sewer, labelled as Outfall 101, was modelled as an outfall with a fixed boundary condition obtained from the PCSWMM model for the First Registration Kanata West Development (Stantec, 2020) as the maximum HGL at STM MH 123.The fixed boundary conditions for each storm event are summarized in the table below.

Table 8: Fixed Boundary Condition at Block 29 Outlet

Node from KW Development Model (Stantec 2020)	2-year Maximum HGL (m^3/s)	100-year Maximum HGL (m)	100-year+20% Maximum HGL (m)
STM MH 123	95.50	95.64	95.74

5.3.4 Modelling Parameters

Table 9 presents the general subcatchment parameters used.

Table 9: General Subcatchment Parameters

Subcatchment Parameter	Value
Infiltration Method	Horton
Max. Infiltration Rate (mm/hr)	76.2
Min. Infiltration Rate (mm/hr)	13.2
Decay Constant (1/hr)	4.14
N Imperv	0.013
N Perv	0.25
Dstore Imperv (mm)	1.57
Dstore Perv (mm)	4.67

Table 10 presents the individual subcatchments' parameters.

Table 10: Subcatchment Parameters

Area ID	Outlet	Area (ha)	Width (m)	Slope (%)	% Impervious	Runoff Coefficient	Subarea Routing	Percent Routed
L103A	L103A-S	0.077	50	2.5	88.6	0.82	OUTLET	100
L104A	L104A-S	0.156	79	3.0	88.6	0.82	OUTLET	100
L106A	L106A-S	0.244	114	2.0	80.0	0.76	OUTLET	100
UNC-1	OF-1	0.109	165	25.0	10.0	0.27	PERVIOUS	100
UNC-2	OF-2	0.152	238	5.0	28.6	0.40	PERVIOUS	100

Table 11 summarizes the storage node parameters used in the model.

Table 11: Storage Node Parameters

Storage Node	Invert Elevation (m)	Rim Elevation (m)	Total Depth (m)	Curve Name	Static Storage Available (m^3)
L103A-S	96.40	98.30	1.90	L103A-V	5.8
L104A-S	96.44	98.22	1.78	L104A-V	30.9
L106A-S	96.54	98.32	1.78	L106A-V	20.1

5.3.4.1 Hydraulic Parameters

As per the Ottawa Sewer Design Guidelines (OSDG 2012), Manning's roughness values of 0.013 were used for sewer modeling and overland flow corridors representing roadways.

Table 12 presents the parameters for the outlet and orifice link objects in the model, which represent inlet control devices (ICDs). All orifices were assigned a discharge coefficient of 0.572 to correspond to manufacturer supplied discharge curves for IPEX Tempest HF/MHF models. Should an approved equivalent model be required, the peak outlet rate of the selected model will be required to match that of the modeled ICD at the maximum head noted in the model results portion of this report.

Table 12: Orifice Parameters

Orifice Name	Inlet	Outlet	Inlet Elevation (m)	Type	Diameter (m)
L103A-IC	L103A-S	103	96.40	CIRCULAR	0.083
L104A-IC1	L104A-S	104	96.44	CIRCULAR	0.083
L104A-IC2	L104A-S	104	96.44	CIRCULAR	0.083
L106A-IC	L106A-S	106	96.54	CIRCULAR	0.152

Exit losses at maintenance holes were set for all pipe segments based on the flow angle through the structure. Exit losses were assigned as per City guidelines (Appendix 6b, Sewer Design Guidelines), as shown in **Table 13**.

Table 13: Exit Loss Coefficients for Bends at Maintenance Holes

Degrees	Coefficient
11	0.060
22	0.140
30	0.210
45	0.390
60	0.640
90	1.320
180	0.020

5.4 MODELLING RESULTS AND DISCUSSION

The following section summarizes the key hydrologic and hydraulic model results. For detailed model results or inputs please refer to the example input file in **Appendix C** and to the model files included in the digital submission.

Table 14 summarizes the minor system peak discharge rate from the proposed Block 29 for the modelled storm events.

Table 14: Storm Event Peak Discharge Rates (Minor System)

Model	Outlet Node	2-yr Peak Flow Rate (L/s)	100-yr Peak Flow Rate (L/s)
Block 29 PCSWMM Model (current)	STM MH 101	92.8	107.3
KW Development Allowable Minor System Release Rate (Stantec, 2020)	STM MH 123[1]	111.0	

1. Block 29 was shown to discharge its minor system to STM MH 123 in the PCSWMM Model for the *Richcraft Kanata West Development First Registration Phase Servicing and Stormwater Management Report* (Stantec, February 2020). Block 29 is now proposed to outlet to proposed STM MH 101, which will be placed immediately downstream of STM MH 123.

The minor system peak flow rate from the proposed Block 29 site is lower than the allowable during all storm events up the 100-year storm event.

Table 15 summarizes the major system peak outflows from the proposed site to Maize Street. The major system peak flows were determined by adding the major system peak outflow from area L103A and uncontrolled runoff from area UNC-2 which sheet flows to Roger Griffith Avenue and Maize Street.

Table 15: Storm Event Peak Discharge Rates (Major System)

Model	2-yr Peak Flow Rate (L/s)	100-yr Peak Flow Rate (L/s)
Block 29 PCSWMM Model (current)	26.5	113.6
KW Development Allowable Major System Release Rate (Stantec, 2020)	480	

Table 16 summarizes the HGL results within the proposed development for the 100-year, 3-hour Chicago storm event and the ‘climate change’ scenario required by the City of Ottawa Sewer Design Guidelines (2012), where intensities are increased by 20% from the 100-year event.

Table 16: Hydraulic Grade Line (HGL) Results

STM MH or STUB	Lowest USF (m)	100yr 3hr Chicago		100yr + 20% 3hr Chicago	
		HGL (m)	USF-HGL Clearance (m)	HGL (m)	USF-HGL Clearance (m)
102	96.19	95.71	0.48	95.82	0.37
103	96.19	95.78	0.41	95.88	0.31

STM MH or STUB	Lowest USF (m)	100yr 3hr Chicago		100yr + 20% 3hr Chicago	
		HGL (m)	USF-HGL Clearance (m)	HGL (m)	USF-HGL Clearance (m)
104	96.21	95.82	0.39	95.93	0.28
106	96.48	95.99	0.49	96.09	0.39
At STM Service for Block 2 (b/w MH 106 and 103)[1]	96.20	95.83	0.37	95.93	0.27

1. This HGL was interpolated using linear interpolation between the maximum HGLs at STM MH 106 and 103.

As is demonstrated in the table above, the worst-case scenario results in HGL elevations that remain at least 0.37 m below the proposed underside of footings in the 100-year event, and HGL elevations remain below the proposed underside of footing elevations during the 'climate change' scenario.

Table 17 presents the proposed ICDs with their corresponding heads and flows in the 2-year and 100-year storm events.

Table 17: 2-year and 100-year Heads and Flow Rates at ICDs

Orifice Name	CB Name	Diameter (mm)	Invert (m)	2-year Head (m)	2-year Flow (L/s)	100-year Head (m)	100-year Flow (L/s)
L103A-IC[1]	CB103A-1	83	96.40	1.22	14.9	1.67	17.5
L104A-IC1	CB104A-1	83	96.44	1.36	15.7	1.60	17.1
L104A-IC2	CB104A-2	83	96.44	1.36	15.7	1.60	17.1
L106A-IC	CB106A-1	152	96.54	1.12	47.0	1.54	55.7

1. CB L103A-1 and L103A-2 to be interconnected and controlled by a single ICD at CB L103A-1.

Table 18 presents the maximum total surface water depths (static ponding depth + dynamic flow) above the top-of-grate of catchbasins for the 100-year design storm and climate change storm. Based on the model results, the total ponding depth (static + dynamic) does not exceed the 0.35 m maximum in the 100-year event. Total ponding depths during the climate change scenario are below adjacent building openings and are not expected to impact proposed buildings within the development. There is no ponding in the 2-year event (refer to **Appendix C.3**).

Table 18: Maximum Surface Water Depths

Storage Node ID	Invert Elevation (m)	Rim Elevation (m)	100yr 3hr Chicago		100yr 3hr Chicago + 20%		Adjacent Lowest Building Opening (m)
			Max. Surface HGL (m)	Total Surface Water Depth (m)	Max. Surface HGL (m)	Total Surface Water Depth (m)	
L103A-S	96.40	98.30	98.07	0.17	98.09	0.19	98.35
L104A-S	96.44	98.22	98.04	0.22	98.09	0.27	98.32
L106A-S	96.54	98.32	98.08	0.16	98.10	0.18	98.35

1. Rim elevation = (catchbasin top of grate elevation) + 0.4 m.

6.0 GEOTECHNICAL CONSIDERATIONS AND GRADING

6.1 GEOTECHNICAL INVESTIGATION

A geotechnical investigation report for Kanata West Block 29 was completed by Paterson Group on July 14, 2020. Field testing consisting of the advancement of three (3) boreholes throughout the subject site was completed on June 24, 2020. Data from an existing borehole immediately northwest of the subject site was also used. The geotechnical investigation report is included in **Appendix D.1**.

The site is undeveloped and mostly covered in grass. The grade across the site is generally level at an elevation of approximately 97 m. The subsurface profile within Block 29 generally consists of 0.2 to 0.5m of brown silty sand fill with some clay and crushed stone, underlain by a silty clay deposit. This silty clay deposit is generally very stiff to stiff brown silty clay crust within the upper 3-4 m below original ground surface. This brown silty clay transitions to a firm, grey silty clay as the depth increases.

Groundwater levels were taken at the three (3) boreholes advanced in 2020. The long-term groundwater table is anticipated to be at a 3 to 4 m depth, subject to seasonal fluctuations.

The site is considered suitable for the proposed development from a geotechnical perspective. Conventional shallow foundations placed on undisturbed stiff to firm silty clay, compacted silty sand to sandy silt, or engineered compacted fill, can be used for the proposed buildings.

A permissible grade raise restriction of 1.5 m above original ground surface is recommended by Paterson due to the silty clay deposit. If higher-than-permissible grade raises are needed, pre-loading, lightweight fill, or other measures should be investigated to reduce the risks of unacceptable long-term total and differential settlements.

6.1.1 Limits of Hazard Lands along Poole Creek

A slope stability analysis was completed as part of the geotechnical investigation to determine the required setback from the top of slope using a factor of safety of 1.5. Toe erosion and 6 m erosion access allowances were also considered in the determination of the limit of hazard lands, which is demonstrated in the geotechnical investigation report.

The existing vegetation on the face of the slope along Poole Creek should not be removed as it provides stability to the slope and reduces erosion.

The limit of hazard lands, 5 m toe erosion allowance, 3 m erosion access allowance, Poole Creek top of valley, and MVCA floodplains are shown on all plans in **Appendix F**.

6.1.2 Proposed Pavement Structure

Table 19 and **Table 20** summarize the recommended pavement structures for the development.

Table 19: Recommended Pavement Structure for Access Lanes

Thickness (mm)	Material Description
40	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course – HL-8 or Superpave 19.0 Asphaltic Concrete
150	Base – OPSS Granular A Crushed Stone Compacted to Min. 100% SPMDD
450	Subbase – OPSS Granular B Type II Compacted to Min. 100% SPMDD
-	Subgrade – Either fill, in situ soil, or OPSS Granular B Type II material placed over in situ soil or fill. Geoxtextile (such as Terratrack 200 or equivalent) or thicker subbase may be required if soft spots develop in the subgrade.

Table 20: Recommended Pavement Structure for 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 Compacted to Min. 100% SPMDD
300	Subbase – OPSS Granular B Type II Compacted to Min. 100% SPMDD
-	Subgrade – Either fill, in situ soil, or OPSS Granular B Type II material placed over in situ soil or fill

6.1.3 Sewer/Watermain Installation

The subsurface soils are considered to be Type 2 and 3 according to the Occupational Health and Safety Act and Regulations for Construction Projects. For excavations up to 3 m deep, 1H:1V slopes or shallower are recommended. A shallow slope should be used if the excavation is below the groundwater table. A trench box is required for all steep or vertical side slopes where workers are present.

At least 150mm of OPSS Granular A crushed stone compacted to 95% SPMDD is recommended as bedding for watermains and sewers, up to the springline of the pipes. The base thickness should be increased to 300 mm in the presence of the firm to stiff grey silty clay. OPSS Granular A crushed stone is to be used as cover material at least 300mm above the obvert of the pipes and compacted to a minimum of 95% SPMDD.

If the excavation and filling operations are carried out in dry weather, the moist brown silty clay is expected to be suitable as backfill material (above the cover material). Wet silty clay materials will be difficult to reuse without an extensive drying period. The trench backfill material within the frost zone (about 1.8 m below finished grade) should match the existing soils at the trench walls. Clay seals are recommended at no more than 60 m intervals in the service trenches and at strategic locations to reduce long-term lowering of the groundwater level in the site.

Open sumps and pumps are anticipated to be sufficient in providing groundwater control for relatively shallow excavations due to the impervious nature of the silty clay present throughout the site. A temporary Permit to Take Water (PTTW) from the Ontario Ministry of the Environment, Conservation and

Parks (MECP) may be required if more than 400,000 L/day of ground and/or surface water need to be pumped during the construction phase (to be determined by the geotechnical consultant). The review/issuance of the permit may take upwards of 4 months. For typical ground/surface water pumping volumes (50,000 L/day to 400,000 L/day), registration on the Environmental Activity and Sector Registry (EASR) will be required. Two to four weeks should be allotted for the completion of this registration and the preparation of a Water Taking and Discharge Plan by a Qualified Person as required under O.Reg. 63/16.

The founding stratum should be protected from freezing temperatures if winter construction is anticipated. The trench excavations should also be completed in a manner that will avoid the introduction of frozen materials into the trenches.

6.2 GRADING PLAN

Proposed grading for Block 29 is shown on **Drawing GP-1**. Proposed grading for the Block 29 site directs most overland flows from the proposed development to Maize Street, as per the intent from the *Kanata West Development First Registration Servicing and Stormwater Management Report* (Stantec, February 2020) and City standards. A small northwestern portion of the site, containing mostly landscaped area, drains uncontrolled to Poole Creek due to grading restrictions with the existing topography. The proposed grading implements sags in the parking areas for surface stormwater detention.

The proposed grading has been developed to match the existing road grades along Maize Street to the south and Roger Griffiths Avenue to the east. The 1.5 m grade raise restriction outlined in the geotechnical investigation report has been generally respected throughout the site, with minor exceedances of 0.3-0.5 m in certain locations. Paterson Group has confirmed that this grade raise exceedance is allowable without any additional measures. Please refer to correspondence included in **Appendix D.2.**

All grading, in-filling and backfilling works are to be completed as per the geotechnical recommendations made in Paterson's geotechnical investigation report (summarized above in **Section 6.1**).

7.0 UTILITIES

Utility infrastructure for Bell, Rogers, Hydro Ottawa, and Enbridge exists within underground plant servicing urbanized rights-of-way adjacent to the subject site. Coordination regarding the exact size, location, and routing of utilities will begin following design circulation.

8.0 APPROVALS

The City of Ottawa will review and approve most development applications as they relate to the provision of water supply, wastewater collection and disposal, and stormwater conveyance and treatment.

An Environmental Compliance Approval (ECA) is not expected to be required from the Ontario Ministry of the Environment, Conservation and Parks (MECP) for the proposed servicing works within the proposed private block so long as part lot control is not pursued for this development (i.e., as long as the property will be held under single ownership). The Mississippi Valley Conservation Authority (MVCA) will be circulated on this submission.

An MECP Permit to Take Water (PTTW) or registration on the Environmental Activity and Sector Registry (EASR) may be required for the site. The geotechnical consultant shall confirm at the time of application whether a PTTW or EASR registration is required.

No other approval requirements from other regulatory agencies are anticipated.

9.0 EROSION CONTROL

In order to protect downstream water quality and prevent sediment build up in catch basins and storm sewers, erosion and sediment control measures must be implemented during construction. The following recommendations will be included in the contract documents and communicated to the Contractor.

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

The Contractor will also be required to complete inspections and guarantee the proper performance of their erosion and sediment control measures at least after every rainfall. The inspections are to include:

- Verification that water is not flowing under silt barriers.
- Cleaning and changing the sediment traps placed on catch basins.

As described in the geotechnical investigation report for the site (see **Appendix D.1**), the vegetation along the existing slope to Poole Creek should be retained as it serves to stabilize the slope and protect it from erosion.

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

10.0 CONCLUSIONS AND RECOMMENDATIONS

10.1 POTABLE WATER SERVICING

The proposed watermain network is capable of achieving the level of service required by the City. Based on the hydraulic analysis, the following conclusions were made:

- The proposed water distribution system in the Block 29 site is recommended to consist of a 200 mm diameter watermain connecting to the existing 300 mm diameter watermain on Maize Street at a single connection point.
- The Block 29 proposed watermain network operates above the maximum pressure objective of 552 kPa (80 psi) in both the average day (AVDY) and peak hour (PKHR) conditions. Therefore, pressure reducing valves will be required on all water services for the site.
- During maximum day domestic demands with a fire flow demand of 15,000 L/min (250 L/s), the Block 29 proposed watermain network is capable of providing sufficient fire flow while maintaining a residual pressure of 138 kPa (20 psi) in all areas within the development.

10.2 WASTEWATER SERVICING

Wastewater from the proposed Block 29 development will be conveyed to the existing sanitary sewer on Maize Street constructed as part of the Kanata West Development First Registration servicing works. The wastewater will ultimately reach the Kanata West Pump Station off Maple Grove Road.

200 mm diameter sanitary sewers are proposed throughout Block 29. The capacity of the existing sanitary sewers on Maize Street and Roger Griffiths Avenue were verified with the estimated peak wastewater flows from the Block 29 site and their relative increase from the estimates made in the *Kanata West Development First Registration Servicing and Stormwater Management Report* (Stantec, 2020). The analysis confirmed that there is sufficient capacity within the downstream sanitary sewer system to service the Block 29 site.

Peak wastewater flows from Block 29 are expected to be 1.4 L/s.

10.3 STORMWATER MANAGEMENT AND SERVICING

The proposed stormwater management plan is in compliance with the requirements outlined in the background documents, the City of Ottawa Sewer Design Guidelines and the Ontario Ministry of the Environment, Conservation and Parks (MECP) Stormwater Management Planning and Design Manual.

Inlet control devices were defined for each subcatchment to restrict inflow rates to the storm sewers to that of the 2-year runoff for the Block 29 site, as per City and background report design criteria. Major system peak flows from the entire site will be directed to Maize Street, except for a small uncontrolled area in the west which will drain directly to Poole Creek and another small uncontrolled area in the east which will drain to the Roger Griffiths Avenue right-of-way. Minor system peak flows will be directed to the existing 675 mm diameter storm sewer on Maize Street. Quantity and quality control (80% TSS removal) of stormwater runoff will be provided at the downstream Kanata West Pond 5.

10.4 GRADING

Proposed grading for the Block 29 site directs most overland flows from the proposed development to Maize Street, as per the intent from the *Kanata West Development First Registration Servicing and Stormwater Management Report* (Stantec, February 2020) and City standards. A small northwestern portion of the site, containing mostly landscaped area, drains uncontrolled to Poole Creek due to grading restrictions with the existing topography. Another small eastern portion of the site drains uncontrolled to the Roger Griffiths Avenue right-of-way. The proposed grading implements sags in the parking areas for surface stormwater detention.

The existing grades along Maize Street to the south of the site and along Roger Griffiths Avenue to the east of the site are to be maintained. All grading, in-filling and backfilling works are to be completed as per the geotechnical recommendations made in Paterson Group's geotechnical investigation report for the site (summarized in **Section 6.1**).

10.5 APPROVALS/PERMITS

An MECP Environmental Compliance Approval (ECA) may be required for the installation of the proposed storm and sanitary sewers within the private Block 29 site should part lot control be pursued to sever the property into separate parcels at a later date. A Permit to Take Water or registration on the EASR may be required for dewatering works during sewer/watermain installation, pending confirmation by the geotechnical consultant. The Mississippi Valley Conservation Authority (MVCA) will need to be consulted in order to obtain municipal approval for site development. No other approval requirements from other regulatory agencies are anticipated.

10.6 UTILITIES

Utility infrastructure for Bell, Rogers, Hydro Ottawa, and Enbridge exists within underground plant servicing urbanized rights-of-way adjacent to the subject site. Coordination regarding the exact size, location, and routing of utilities will begin following design circulation.

APPENDICES

Appendix A - POTABLE WATER SERVICING

A.1 DOMESTIC WATER DEMAND CALCULATIONS

Kanata West Block 29 - Domestic Water Demand Estimates

Based on Site Plan prepared by M. David Blakely Architect Inc. dated March 17, 2021
Last updated on March 19, 2021

Population densities as per City of Ottawa Guidelines:

2-Bedroom Apt.	2.1	ppu

Building ID	Number of Units	Population	Daily Demand Rate (L/cap/day)	Avg. Day Demand		Max. Day Demand [1]		Peak Hour Demand [1]	
				(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
2-Bedroom Terrace Units	48	101	350	24.5	0.41	61.3	1.02	134.8	2.25
Total Site :	**48**	**101**		**24.5**	**0.41**	**61.3**	**1.02**	**134.8**	**2.25**

1 Water demand criteria used to estimate peak demand rates for residential areas are as follows:
maximum daily demand rate = 2.5 x average day demand rate
peak hour demand rate = 2.2 x maximum day demand rate

2 Terrace units assumed to be 2-bedroom units.

A.2 FUS CALCULATION SHEETS

FUS Fire Flow Calculation Sheet

Stantec Project #: 160401608
Project Name: Kanata West Block 29
Date: 2021-03-24
Fire Flow Calculation #: 1
Description: 12-unit terrace flats (Block 1).

Notes: 3-storey building with 412 m2 footprint and 12 2-bedroom units (4 adjoining units each stacked 3 high).

Step	Task	Notes						Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Wood Frame						1.5	-
2	Determine Ground Floor Area of One Unit	-						104	-
	Determine Number of Adjoining Units	Includes adjacent wood frame structures separated by 3m or less						4	-
3	Determine Height in Storeys	Does not include floors >50% below grade or open attic space						3	-
4	Determine Required Fire Flow	($F = 220 \times C \times A^{1/2}$). Round to nearest 1000 L/min						-	12000
5	Determine Occupancy Charge	Limited Combustible						-15%	10200
6	Determine Sprinkler Reduction	None						0%	0
		Non-Standard Water Supply or N/A						0%	
		Not Fully Supervised or N/A						0%	
		% Coverage of Sprinkler System						0%	
7	Determine Increase for Exposures (Max. 75%)	Direction	Exposure Distance (m)	Exposed Length (m)	Exposed Height (Stories)	Length-Height Factor (m x stories)	Construction of Adjacent Wall	-	-
		North	> 45	32.4	3	91-120	Wood Frame or Non-Combustible	0%	2856
		East	3.1 to 10	12.8	3	31-60	Wood Frame or Non-Combustible	18%	
		South	20.1 to 30	32.4	3	91-120	Wood Frame or Non-Combustible	10%	
		West	> 45	12.8	3	31-60	Wood Frame or Non-Combustible	0%	
8	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min							13000
		Total Required Fire Flow in L/s							216.7
		Required Duration of Fire Flow (hrs)							2.50
		Required Volume of Fire Flow (m^3)							1950

FUS Fire Flow Calculation Sheet

Stantec

Stantec Project #: 160401608
Project Name: Kanata West Block 29
Date: 2021-03-24
Fire Flow Calculation #: 2
Description: 12-unit terrace flats (Block 2).

Notes: 3-storey building with 412 m2 footprint and 12 2-bedroom units (4 adjoining units each stacked 3 high).

Step	Task	Notes						Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Wood Frame						1.5	-
2	Determine Ground Floor Area of One Unit	-						104	-
	Determine Number of Adjoining Units	Includes adjacent wood frame structures separated by 3m or less						4	-
3	Determine Height in Storeys	Does not include floors >50% below grade or open attic space						3	-
4	Determine Required Fire Flow	(F = 220 x C x $A^{1/2}$). Round to nearest 1000 L/min						-	12000
5	Determine Occupancy Charge	Limited Combustible						-15%	10200
6	Determine Sprinkler Reduction	None						0%	0
		Non-Standard Water Supply or N/A						0%	
		Not Fully Supervised or N/A						0%	
		% Coverage of Sprinkler System						0%	
7	Determine Increase for Exposures (Max. 75%)	Direction	Exposure Distance (m)	Exposed Length (m)	Exposed Height (Stories)	Length-Height Factor (m x stories)	Construction of Adjacent Wall	-	-
		North	> 45	32.4	3	91-120	Wood Frame or Non-Combustible	0%	4182
		East	10.1 to 20	12.8	3	31-60	Wood Frame or Non-Combustible	13%	
		South	20.1 to 30	32.4	3	91-120	Wood Frame or Non-Combustible	10%	
		West	3.1 to 10	12.8	3	31-60	Wood Frame or Non-Combustible	18%	
8	Determine Final Required Fire Flow	**Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min**							**14000**
		Total Required Fire Flow in L/s							**233.3**
		Required Duration of Fire Flow (hrs)							**3.00**
		Required Volume of Fire Flow (m^3)							**2520**

FUS Fire Flow Calculation Sheet

Stantec Project #: 160401608
Project Name: Kanata West Block 29
Date: 2021-03-24
Fire Flow Calculation #: 3
Description: 12-unit terrace flats (Block 3).

Notes: 3-storey building with 412 m2 footprint and 12 2-bedroom units (4 adjoining units each stacked 3 high).

Step	Task	Notes						Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Wood Frame						1.5	-
2	Determine Ground Floor Area of One Unit	-						104	-
	Determine Number of Adjoining Units	Includes adjacent wood frame structures separated by 3m or less						4	-
3	Determine Height in Storeys	Does not include floors >50% below grade or open attic space						3	-
4	Determine Required Fire Flow	($F = 220 \times C \times A^{1/2}$). Round to nearest 1000 L/min						-	12000
5	Determine Occupancy Charge	Limited Combustible						-15%	10200
6	Determine Sprinkler Reduction	None						0%	0
		Non-Standard Water Supply or N/A						0%	
		Not Fully Supervised or N/A						0%	
		% Coverage of Sprinkler System						0%	
7	Determine Increase for Exposures (Max. 75%)	Direction	Exposure Distance (m)	Exposed Length (m)	Exposed Height (Stories)	Length-Height Factor (m x stories)	Construction of Adjacent Wall	-	-
		North	3.1 to 10	12.8	3	31-60	Wood Frame or Non-Combustible	18%	4692
		East	30.1 to 45	32.4	3	91-120	Wood Frame or Non-Combustible	5%	
		South	20.1 to 30	12.8	3	31-60	Wood Frame or Non-Combustible	8%	
		West	10.1 to 20	32.4	3	91-120	Wood Frame or Non-Combustible	15%	
8	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min							15000
		Total Required Fire Flow in L/s							250.0
		Required Duration of Fire Flow (hrs)							3.00
		Required Volume of Fire Flow (m^3)							2700

Stantec

FUS Fire Flow Calculation Sheet

Stantec Project #: 160401608
Project Name: Kanata West Block 29
Date: 2021-03-24
Fire Flow Calculation #: 4
Description: 12-unit terrace flats (Block 4).

Notes: 3-storey building with 412 m2 footprint and 12 2-bedroom units (4 adjoining units each stacked 3 high).

<table>
<tr><th>Step</th><th>Task</th><th colspan="6">Notes</th><th>Value Used</th><th>Req'd Fire Flow (L/min)</th></tr>
<tr><td>1</td><td>Determine Type of Construction</td><td colspan="6">Wood Frame</td><td>1.5</td><td>-</td></tr>
<tr><td rowspan="2">2</td><td>Determine Ground Floor Area of One Unit</td><td colspan="6">-</td><td>104</td><td>-</td></tr>
<tr><td>Determine Number of Adjoining Units</td><td colspan="6">Includes adjacent wood frame structures separated by 3m or less</td><td>4</td><td>-</td></tr>
<tr><td>3</td><td>Determine Height in Storeys</td><td colspan="6">Does not include floors >50% below grade or open attic space</td><td>3</td><td>-</td></tr>
<tr><td>4</td><td>Determine Required Fire Flow</td><td colspan="6">($F = 220 \times C \times A^{1/2}$). Round to nearest 1000 L/min</td><td>-</td><td>12000</td></tr>
<tr><td>5</td><td>Determine Occupancy Charge</td><td colspan="6">Limited Combustible</td><td>-15%</td><td>10200</td></tr>
<tr><td rowspan="4">6</td><td rowspan="4">Determine Sprinkler Reduction</td><td colspan="6">None</td><td>0%</td><td rowspan="4">0</td></tr>
<tr><td colspan="6">Non-Standard Water Supply or N/A</td><td>0%</td></tr>
<tr><td colspan="6">Not Fully Supervised or N/A</td><td>0%</td></tr>
<tr><td colspan="6">% Coverage of Sprinkler System</td><td>0%</td></tr>
<tr><td rowspan="5">7</td><td rowspan="5">Determine Increase for Exposures (Max. 75%)</td><td>Direction</td><td>Exposure Distance (m)</td><td>Exposed Length (m)</td><td>Exposed Height (Stories)</td><td>Length-Height Factor (m x stories)</td><td>Construction of Adjacent Wall</td><td>-</td><td>-</td></tr>
<tr><td>North</td><td>> 45</td><td>12.8</td><td>3</td><td>31-60</td><td>Wood Frame or Non-Combustible</td><td>0%</td><td rowspan="4">2346</td></tr>
<tr><td>East</td><td>30.1 to 45</td><td>32.4</td><td>3</td><td>91-120</td><td>Wood Frame or Non-Combustible</td><td>5%</td></tr>
<tr><td>South</td><td>3.1 to 10</td><td>11.5</td><td>3</td><td>31-60</td><td>Wood Frame or Non-Combustible</td><td>18%</td></tr>
<tr><td>West</td><td>> 45</td><td>32.4</td><td>3</td><td>91-120</td><td>Wood Frame or Non-Combustible</td><td>0%</td></tr>
<tr><td rowspan="4">8</td><td rowspan="4">Determine Final Required Fire Flow</td><td colspan="7">Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min</td><td>13000</td></tr>
<tr><td colspan="7">Total Required Fire Flow in L/s</td><td>216.7</td></tr>
<tr><td colspan="7">Required Duration of Fire Flow (hrs)</td><td>2.50</td></tr>
<tr><td colspan="7">Required Volume of Fire Flow (m^3)</td><td>1950</td></tr>
</table>

A.3 WATERMAIN HYDRAULIC ANALYSIS RESULTS

Kanata West Block 29
H2OMAP Water - Hydraulic Modelling Results
Stantec Project No. 160401608
Model last revised on 2021-03-24

Hydraulic Modelling Results - Average Day (AVDY) Demands

Junction Results

ID	Demand	Elevation	Head	Pressure	
	(L/s)	(m)	(m)	(psi)	(kPa)
1	0.20	97.99	161.66	90.51	624.05
2	0.10	98.23	161.66	90.17	621.70
3	0.10	97.83	161.66	90.74	625.63

Pipe Results

ID	From Node	To Node	Length	Diameter	Roughness	Flow	Velocity
			(m)	(mm)		(L/s)	(m/s)
1	1000	1	35.56	204	110	0.40	0.01
2	1	2	72.56	204	110	0.10	0.00
3	1	3	62.59	204	110	0.10	0.00

Hydraulic Modelling Results - Peak Hour (PKHR) Demands

Junction Results

ID	Demand	Elevation	Head	Pressure	
	(L/s)	(m)	(m)	(psi)	(kPa)
1	1.13	97.99	157.59	84.72	584.13
2	0.56	98.23	157.59	84.38	581.78
3	0.56	97.83	157.59	84.95	585.71

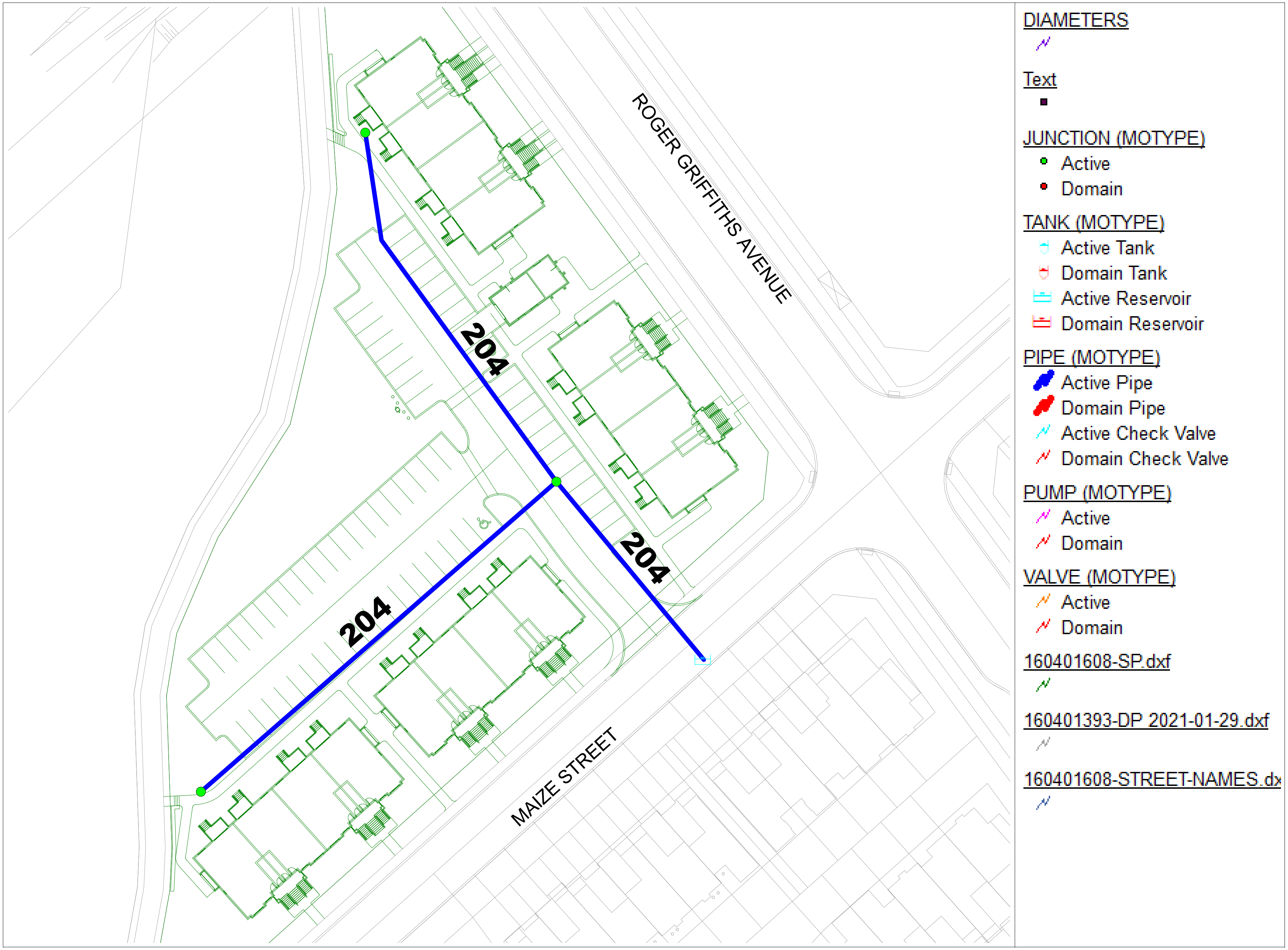
Pipe Results

ID	From Node	To Node	Length	Diameter	Roughness	Flow	Velocity
			(m)	(mm)		(L/s)	(m/s)
1	1000	1	35.56	204	110	2.25	0.07
2	1	2	72.56	204	110	0.56	0.02
3	1	3	62.59	204	110	0.56	0.02

Hydraulic Modelling Results - Maximum Day + Fire Flow (250 L/s) Analysis

ID	Static Demand	Static Pressure		Static Head	Fireflow Demand	Residual Pressure		Available Flow at Hydrant	Available Flow Pressure	
	(L/s)	(psi)	(kPa)	(m)	(L/s)	(psi)	(kPa)	(L/s)	(psi)	(KPa)
1	0.51	72.90	502.63	149.27	250	56.97	392.80	479.41	20.00	137.90
2	0.26	72.56	500.29	149.27	250	24.31	167.61	262.10	20.00	137.90
3	0.26	73.13	504.22	149.27	250	29.32	202.16	277.76	20.00	137.90

Kanata West Block 29 - Watermain Diameters (mm)

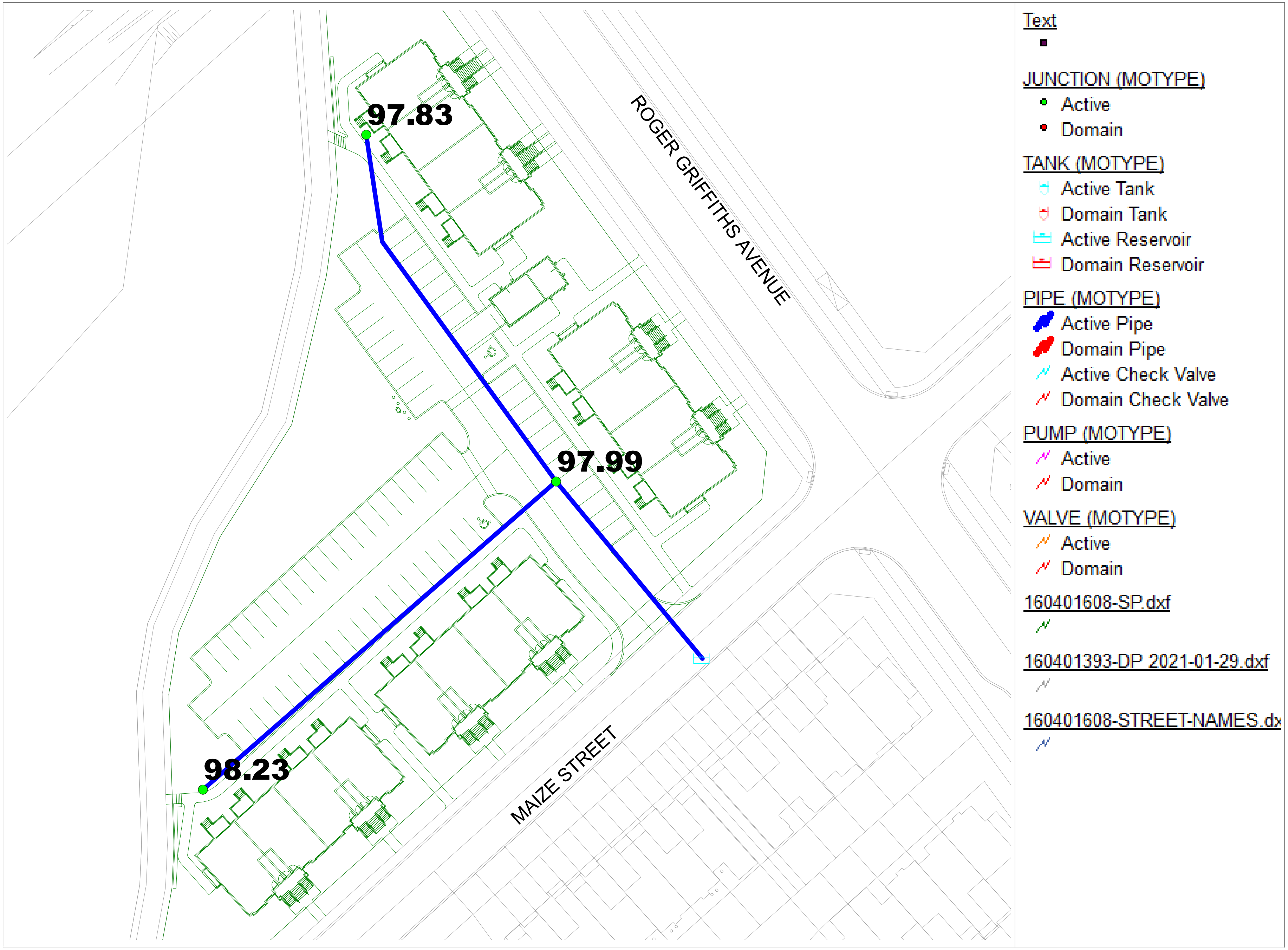


Prepared By: Stantec Consulting Ltd.

Stantec Project No. 160401608

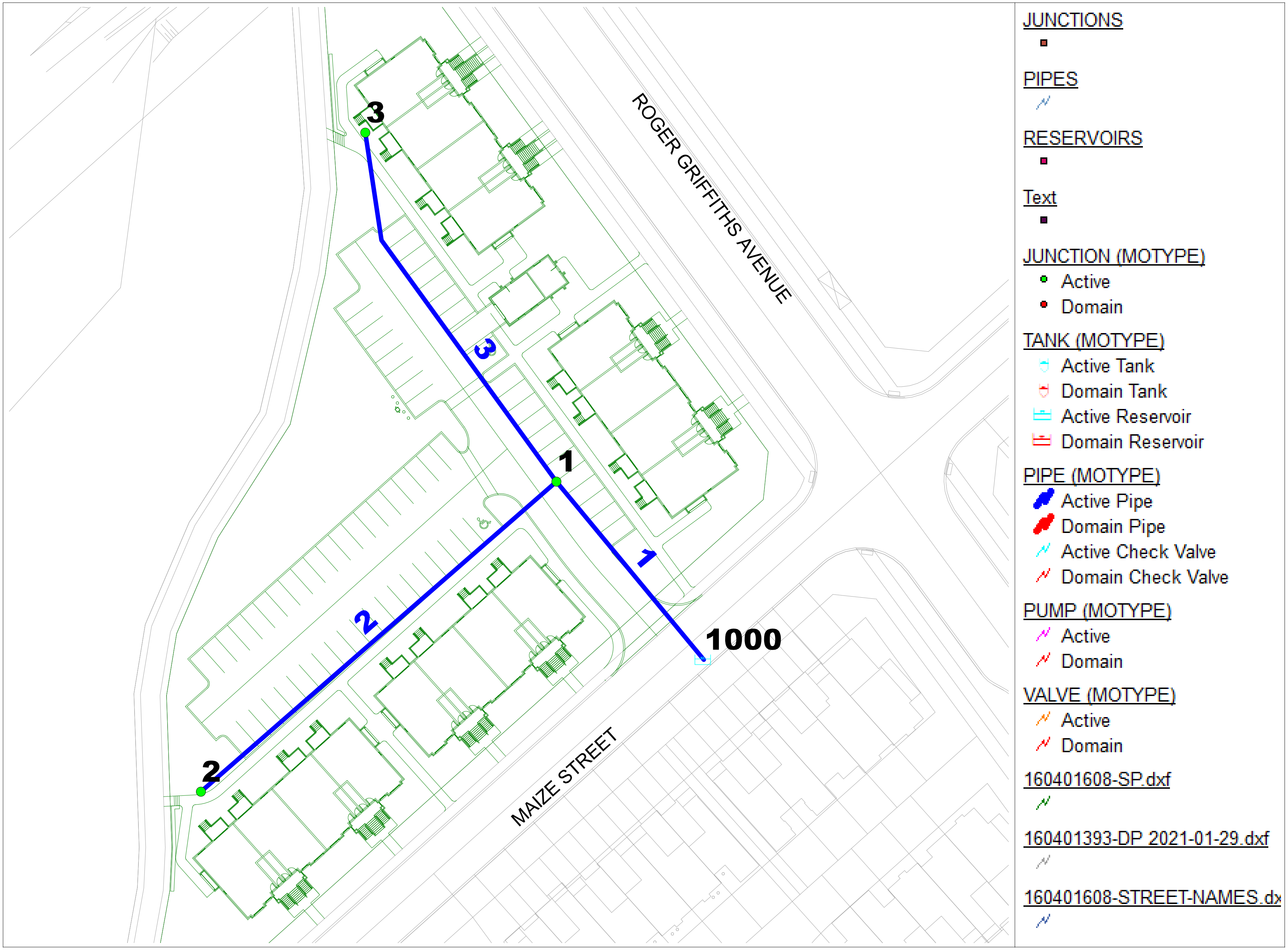
Date: Thursday, February 25, 2021

Kanata West Block 29 - Ground Elevations (m) at Nodes



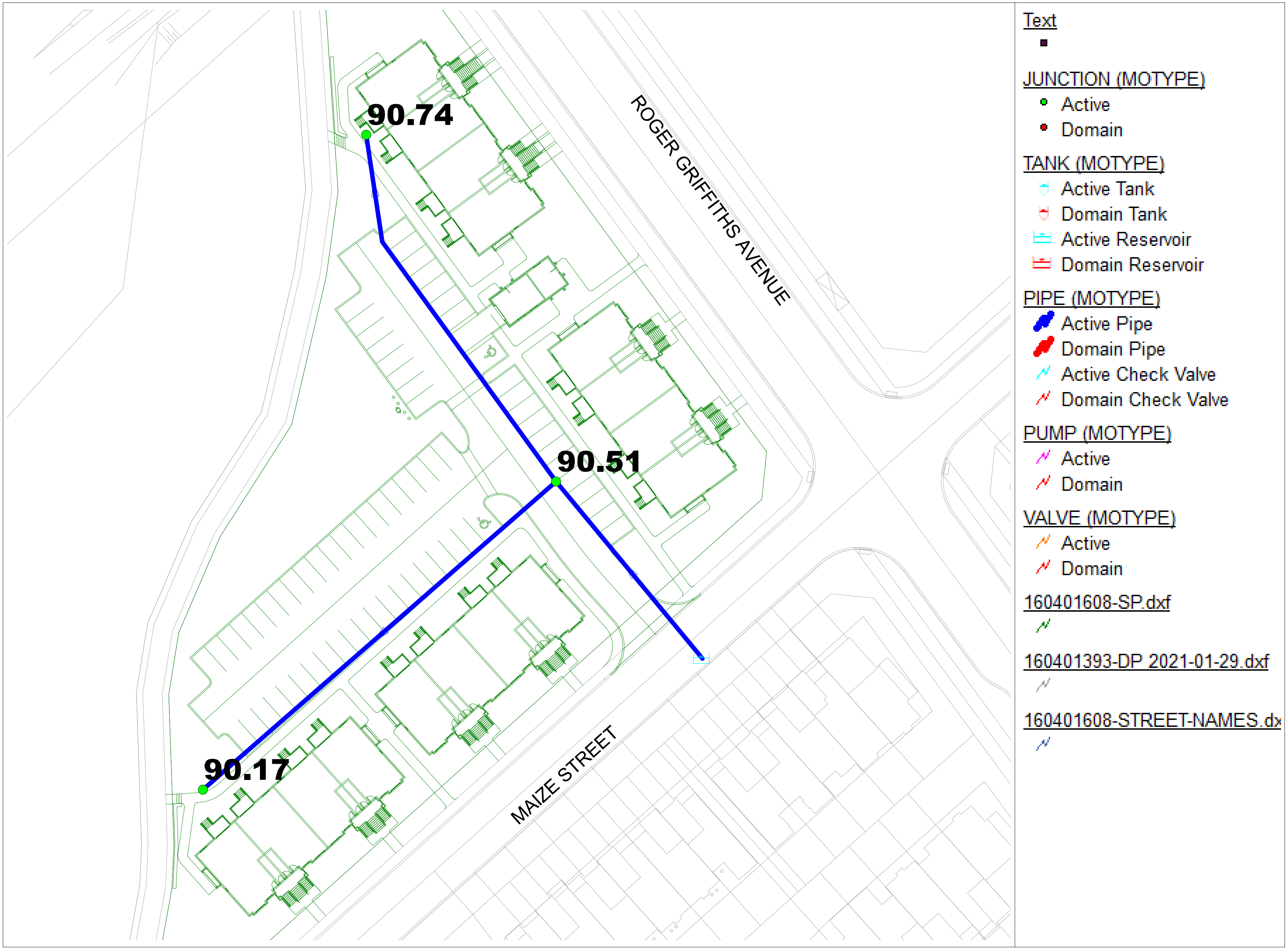
Prepared By: Stantec Consulting Ltd. Stantec Project No. 160401608 Date: Thursday, February 25, 2021

Kanata West Block 29 - Pipe IDs (Blue) and Junction/Reservoir IDs (Black)



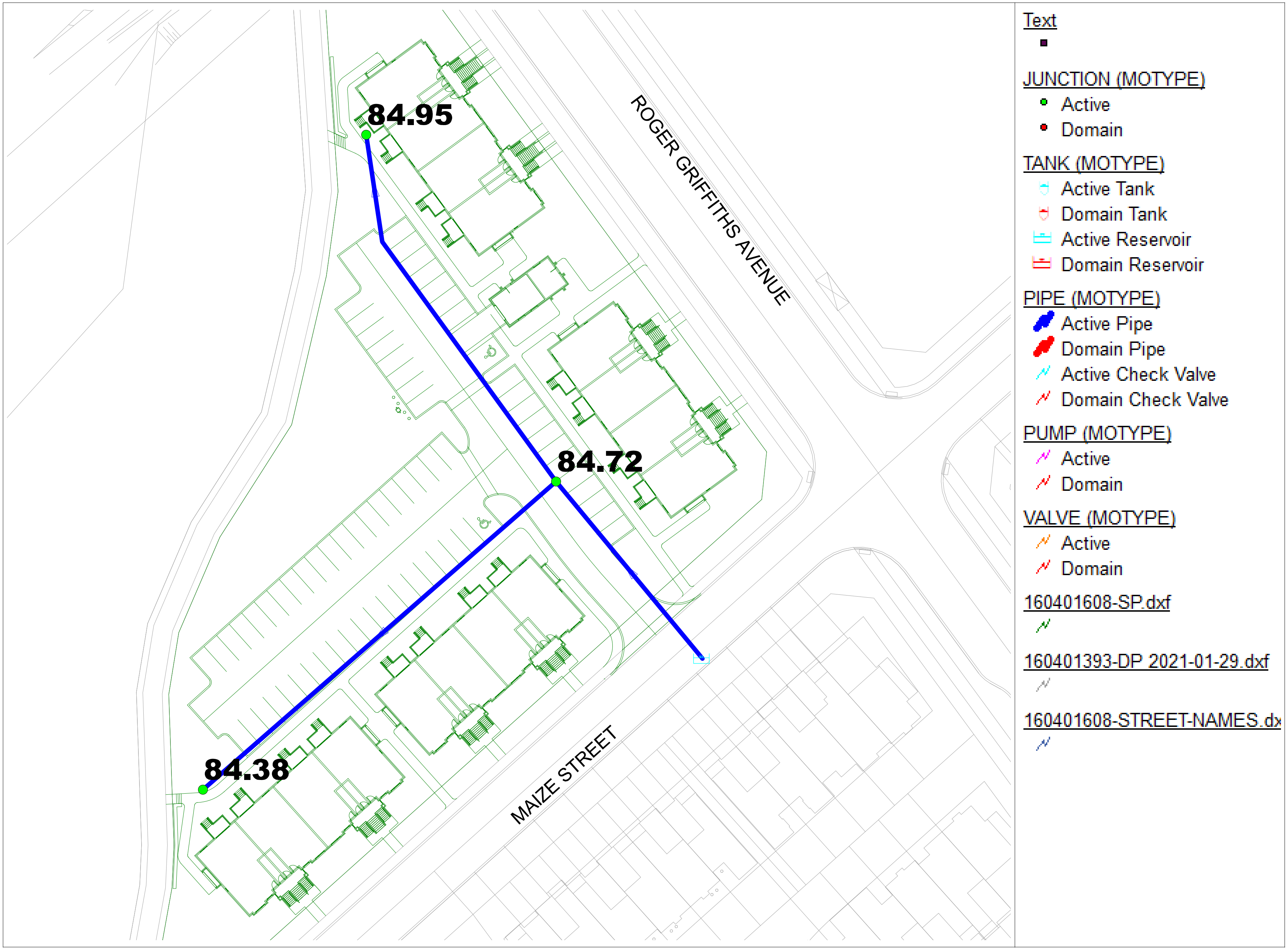
Prepared By: Stantec Consulting Ltd. Stantec Project No. 160401608 Date: Thursday, February 25, 2021

Kanata West Block 29 - Pressures (psi) in AVDY Scenario



Prepared By: Stantec Consulting Ltd.　　Stantec Project No. 160401608　　Date: Thursday, February 25, 2021

Kanata West Block 29 - Pressures (psi) in PKHR Scenario

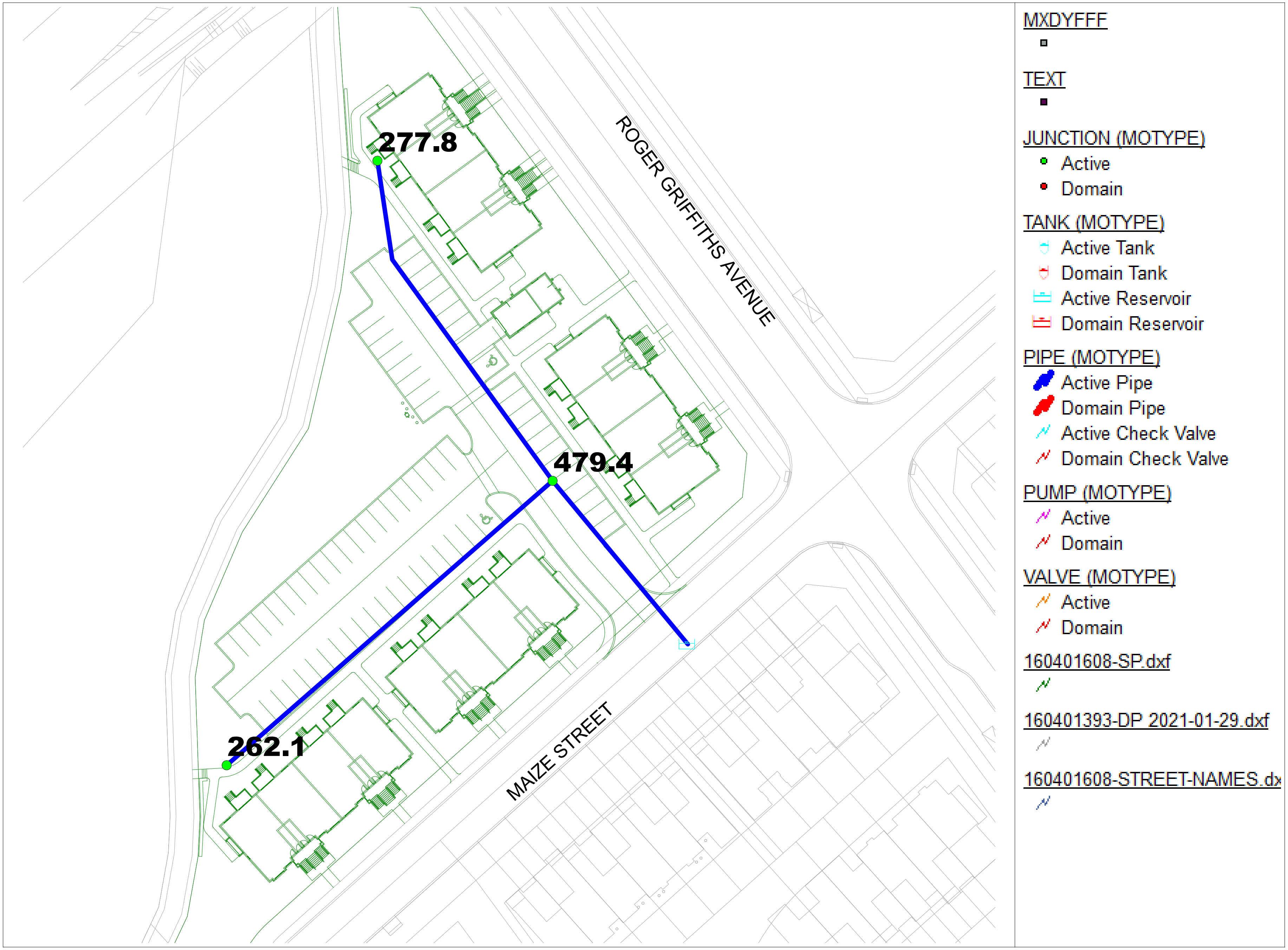


Prepared By: Stantec Consulting Ltd.

Stantec Project No. 160401608

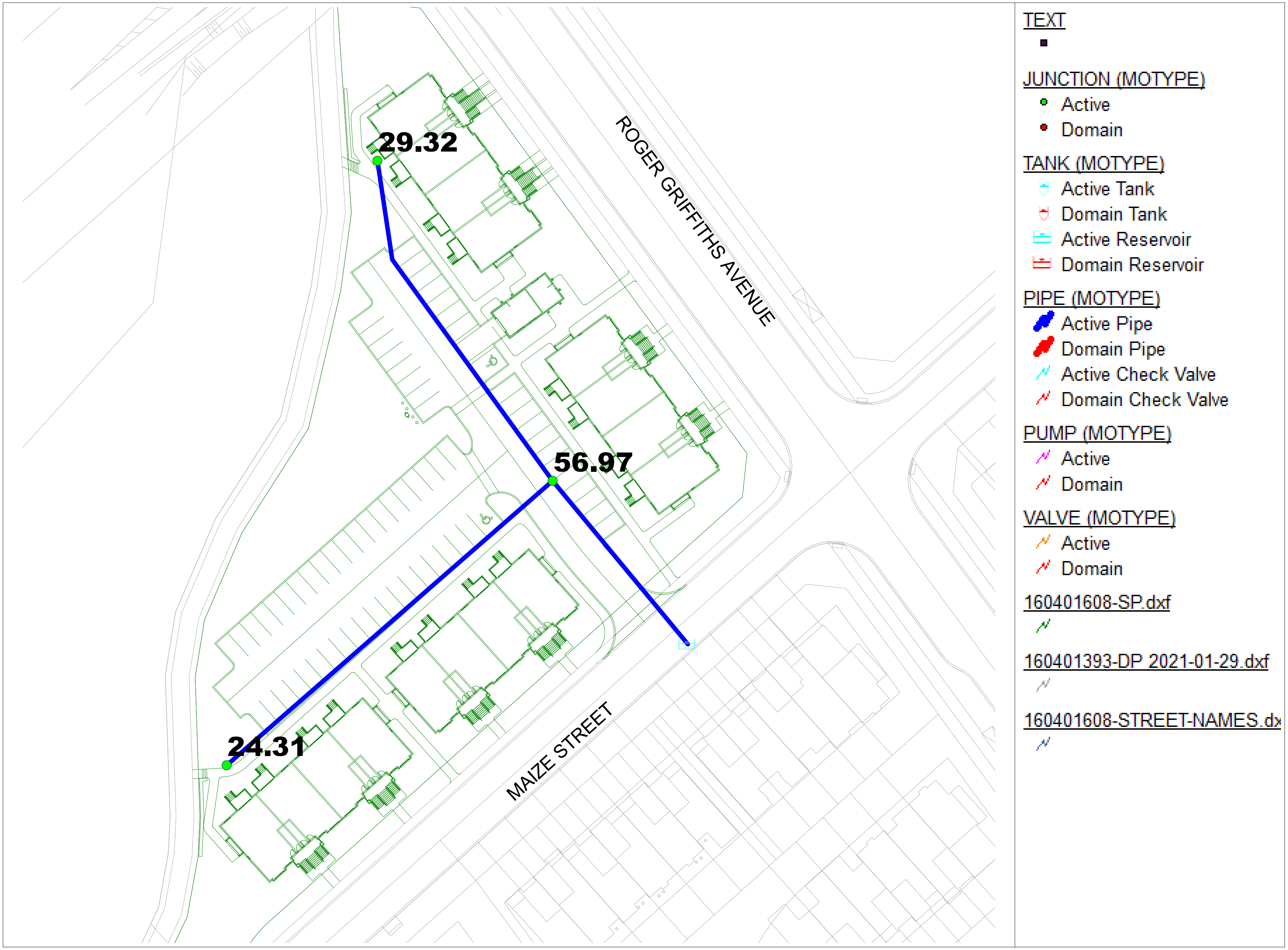
Date: Thursday, February 25, 2021

Kanata West Block 29 - Available Fire Flows (L/s) in MXDY+FF (FF=250 L/s) Scenario



Prepared By: Stantec Consulting Ltd. | Stantec Project No. 160401608 | Date: Wednesday, March 24, 2021

Kanata West Block 29 - Residual Pressures (psi) in MXDY+FF (FF=250 L/s) Scenario



Prepared By: Stantec Consulting Ltd. Stantec Project No. 160401608 Date: Wednesday, March 24, 2021

A.4 POTABLE WATER EXCERPTS FROM KANATA WEST DEVELOPMENT FIRST REGISTRATION PHASE REPORT (STANTEC, FEBRUARY 2020)

2.0 POTABLE WATER

Detailed potable water servicing analyses have been completed and are included in **Appendix A.** Three separate analyses were created to evaluate the proposed potable water distribution network during the first construction phase, the proposed first registration phase(interim condition), and ultimate development conditions of the Richcraft Kanata West Development.

2.1 BACKGROUND

The proposed development is located between Maple Grove Road to the North, Hazeldean Road to the South, Terry Fox Drive to the East, and Huntmar Road to the West. The proposed development is within Zone 3W of the city of Ottawa water distribution system. This zone is fed by the Glen Cairn Pump Station. The ultimate development consists of a mix of single-family units and townhouse units, a school, two parks, a commercial block, and two medium density residential private blocks.

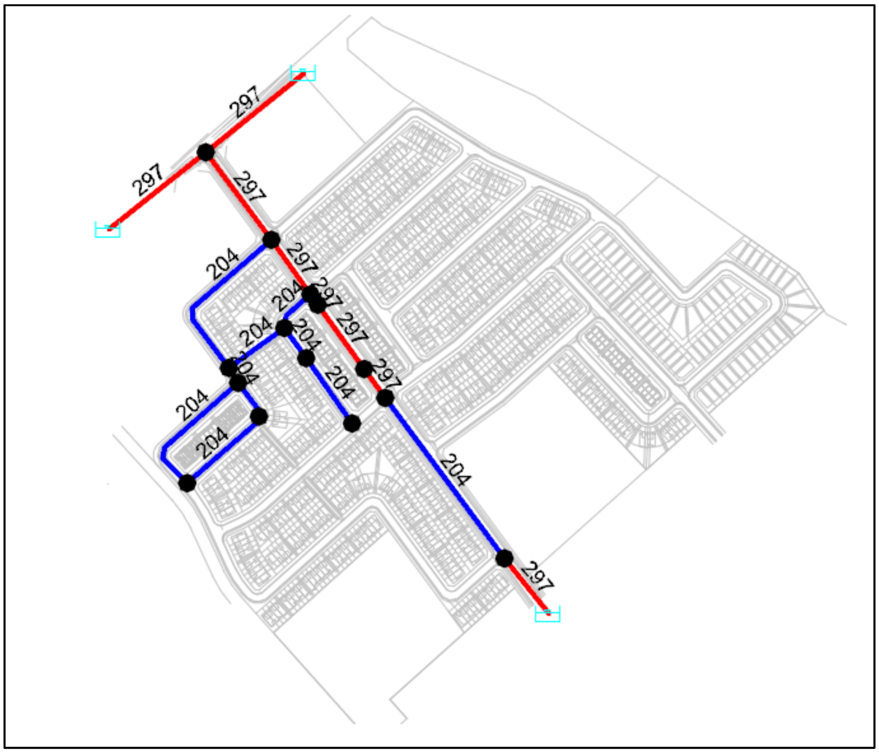
A 300 mm diameter watermain on Maple Grove Road will be extended from the Kanata West Pump Station across the subdivision frontage and connected to the existing 300 mm diameter watermain located at the intersection of Maple Grove Road and the future north-south arterial to service the proposed development from the north as shown on **Drawing OSSP-2**. In the ultimate development condition, the proposed water distribution network will be looped through the internal roadway network to connect to the 914 mm diameter feeder main on Hazeldean Road through two connections to the watermain network within the Trinity commercial development to the south (see **Figure 2.3** and Drawing WM-1 from the KWMSS included in **Appendix D.1**).

The first construction phase and proposed first registration phase (interim condition) will have only one connection through the Trinity development as shown on **Drawing OSSP-1, Figure 2.1** and **Figure 2.2**. Due to construction constraints during ultimate development conditions, it is not feasible to install the ultimate development condition watermain along the future alignment of Roger Griffiths Avenue to connect to the Trinity development watermain network. As a result, a temporary 200 mm diameter watermain will be installed adjacent to the future alignment of Roger Griffiths Avenue from Cartage Way to the future Holstein Road intersection. The temporary watermain will be connected to a proposed 300 mm diameter watermain that will connect to the Trinity development watermain network to service the site under all development conditions. Prior to ultimate build out, a 300 mm diameter watermain will be installed along the future Roger Griffiths Avenue and the temporary 200 mm diameter watermain will be decommissioned.

Although the watermains within the Trinity Development are currently private, these private roadways and the associated infrastructure are planned to be transferred over to the City of Ottawa prior to the watermain connections for the Richcraft Kanata West Development.

For the purpose of modelling the proposed watermain network, the specific connection points provided in the boundary conditions by the City were used in the analysis as shown in **Figure 2.1** to **Figure 2.3**.

Figure 2.1: First Construction Phase Proposed Potable Water Distribution Network



It should be noted that during the proposed first registration phase of the development there will be two temporary dead ends; one on Sheaf Row and a second one on Corn Husk Lane. However, as can be seen on **Drawing OSSP-1,** only thirty one (31) units will be connected to the temporary dead end on Sheaf Row and only sixteen (16) units will be connected to the temporary dead end on Corn Husk Lane. Water age calculations for these temporary dead ends are provided in **Section 2.6.**

Figure 2.2: Interim Development (First Registration) Proposed Potable Water Distribution Network

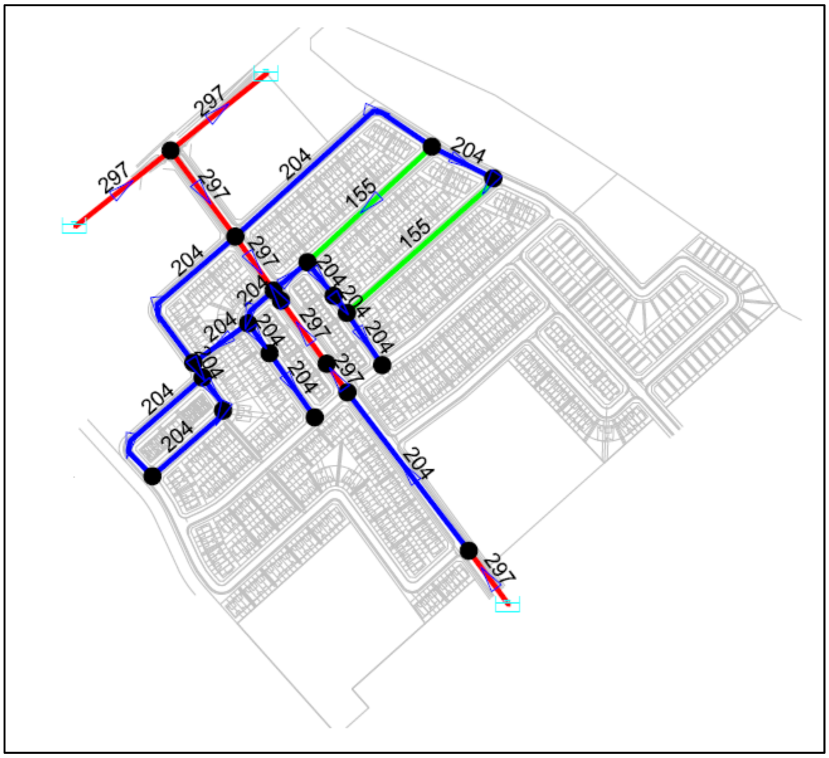
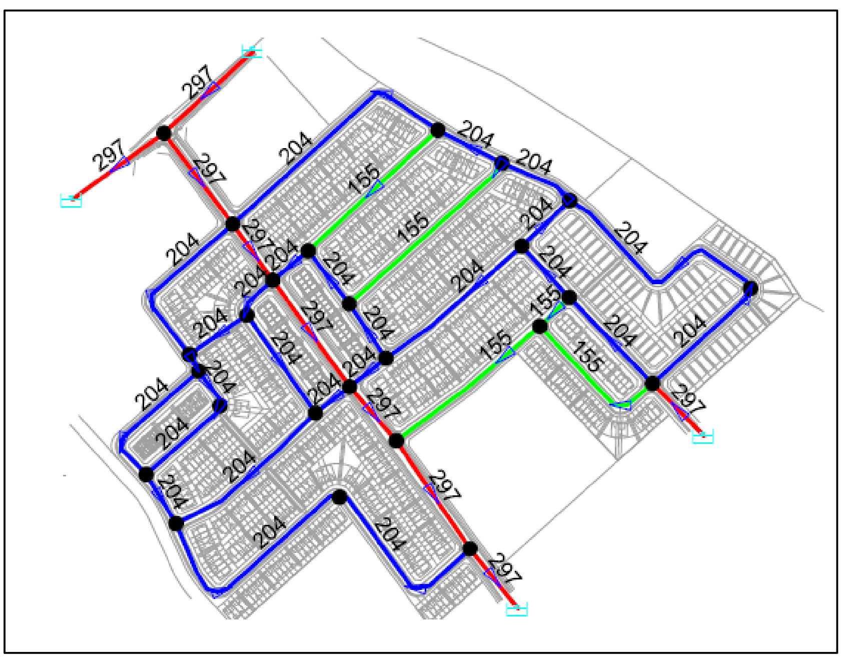


Figure 2.3: Ultimate Development Proposed Potable Water Distribution Network



with a minimum two-hour fire-resistance rating that comply with OBC Div. B, Subsection 3.1.10, are constructed to separate townhouse blocks to the lesser of seven dwelling units and 600 m^2 of building area, and that a minimum 10 m separation exists between rear yards (see Technical Bulletin ISDTB-2014-02 in **Appendix D.1**). As a result, fire flow requirements for most of the proposed development have been capped at 10,000 L/min, and FUS calculations have been provided for all other townhomes and back to back units that do not meet these conditions as described in the following sections.

In addition, the KWMSS allowed a greater maximum pressure of 100 psi within the watermain network for peak hour conditions while the current guidelines state that as per the Ontario Building Code, the static pressure at any fixture shall not exceed 552 kPa (80 psi).

2.3 WATER DEMANDS

Water demands for the first construction phase, interim (proposed first registration phase), and ultimate development scenarios were estimated using the City of Ottawa Water Distribution Design Guidelines. A daily rate of 28,000 L/ha/d was used for the proposed school and the commercial block. The population for the medium density residential blocks was estimated based on a density of 38 units/ha. See **Appendix A** for detailed domestic water demand estimates.

In the ultimate development condition, the average day demand (AVDY) was determined to be 10.8 L/s. The maximum daily demand (MXDY) was determined to be 25.6 L/s and was calculated as 1.5 times the AVDY for school and commercial blocks and 2.5 times the AVDY for all other residential areas. The peak hour demand (PKHR) totaled 55.5 L/s and was calculated as 1.8 times the MXDY for school and commercial blocks, and 2.2 times the MXDY for all other residential areas.

Similarly, the average day demand (AVDY) for the first construction phase and the proposed interim phase development was determined to be 1.7 L/s and 4.2 L/s, while the maximum daily demand (MXDY) was determined to be 4.3 L/s and 9.9 L/s, and the peak hour demand (PKHR) totaled 9.4 L/s and 21.5 L/s. The calculated residential, Institutional and commercial water consumption for the ultimate development scenario is shown in **Table 2.3** and **Table 2.4**.

Table 2.3: Ultimate Development Residential Water Demands

Unit Type	Units	Area (ha)	Person/Unit	Population	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
Singles	72	-	3.4	245	0.99	2.48	5.45
Townhomes	503	-	2.7	1358	5.50	13.75	30.26
Back-to-back	130	-	2.7	351	1.42	3.55	7.82
MD1	-	0.74	-	76	0.31	0.76	1.68
MD2	-	2.80	-	287	1.16	2.91	6.40

distribution system in unoccupied areas shall not exceed 689 kPa (100 psi). Under emergency fire flow conditions, the minimum pressure objective in the distribution system is 138 kPa (20 psi).

Model Development

New watermains were added to the hydraulic model to simulate the proposed distribution system under the different development conditions. Hazen-Williams coefficients ("C-Factors") were applied to the new watermain in accordance with the City of Ottawa's Water Distribution Design Guidelines (**Table 2.6**).

Table 2.6: Proposed Watermain C-Factors

Pipe Diameter (mm)	C-Factor
150	100
200 to 250	110
300 to 600	120
> 600	130

2.5 HYDRAULIC MODEL RESULTS

Three hydraulic models were created to simulate the first construction phase, the proposed first registration phase (interim condition), and the ultimate development condition based on boundary conditions provided by the City of Ottawa. The hydraulic analyses were completed with H2OMAP Water Software and assessed the internal network and connections to the surrounding infrastructure. The models were tested under peak hour, average day, and maximum day plus fire flow conditions.

First Construction Phase Scenario

The results from the first construction phase scenario show that the maximum pressure modeled was approximately 93.4 psi (643 kPa) and the minimum pressure during the peak hour scenario was approximately 81.0 psi (558 kPa) as shown in **Figure 2.4** and **Figure 2.5** respectively. These pressures are above the serviceable limit of 50 to 80 psi (345 to 552 kPa) and therefore all units will require pressure reducing valves.

Figure 2.4: AVDY Pressure Results for the First Construction Phase Condition Scenario (psi)

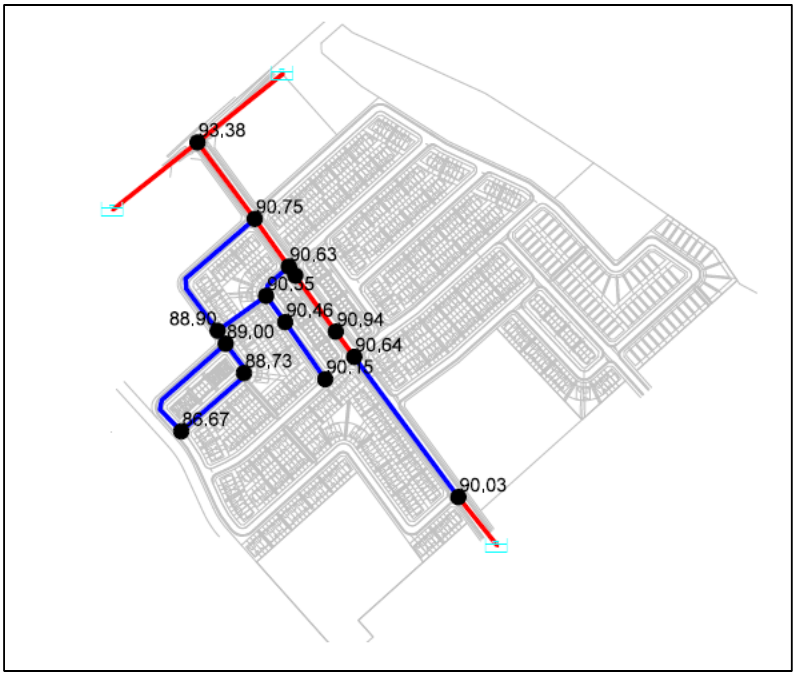
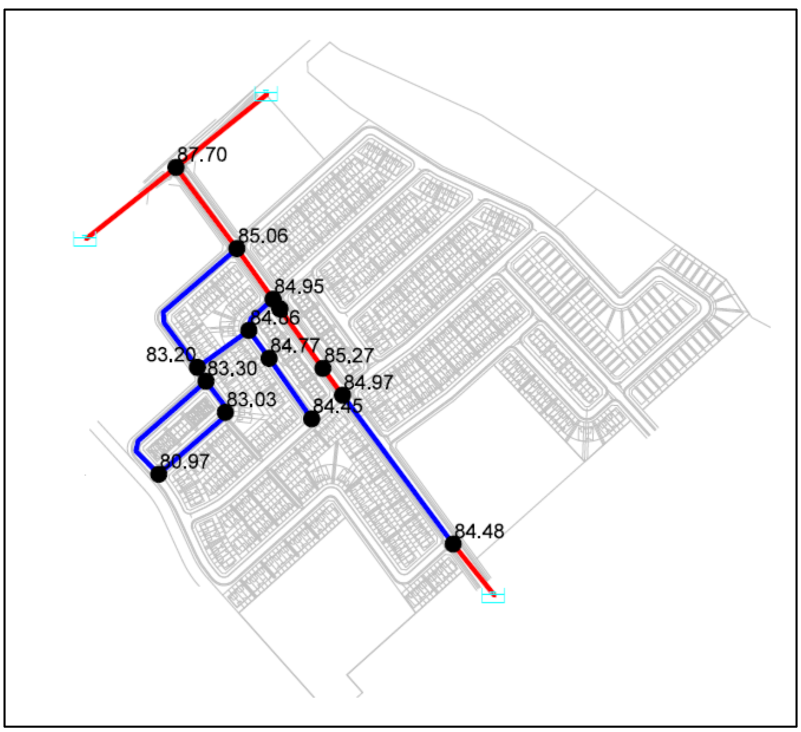


Figure 2.5: PKHR Pressure Results for the First Construction Phase Condition Scenario (psi)



A hydraulic model was used to assess the fire flow conditions of the first construction phase of the proposed development. The model was carried out to determine the anticipated amount of flow that could be provided under maximum day demands and a fire flow requirement of 267 L/s for Block 46 on Sheaf Row (nodes 29 and 30) as shown in **Appendix A.1**. As mentioned above all other fire flow requirements are just under 200 L/s.

Results of the modeling analysis indicate that flows in excess of 12,633 L/min (211 L/s) can be delivered for the units that require 200 L/s or less, and 16,214 L/min (270 L/s can be delivered to units that require 267 L/s while still maintaining a residual pressure of 140 kPa (20 psi). The residual pressures for the different fire flow analyses are shown in **Figure 2.6** and **Figure 2.7**. Results of the hydraulic modeling are included for reference in **Appendix A1**.

Figure 2.6: MXDY + 200 L/s Fire Flow Results for the First Construction Phase Condition Scenario (Residual Pressure (psi))

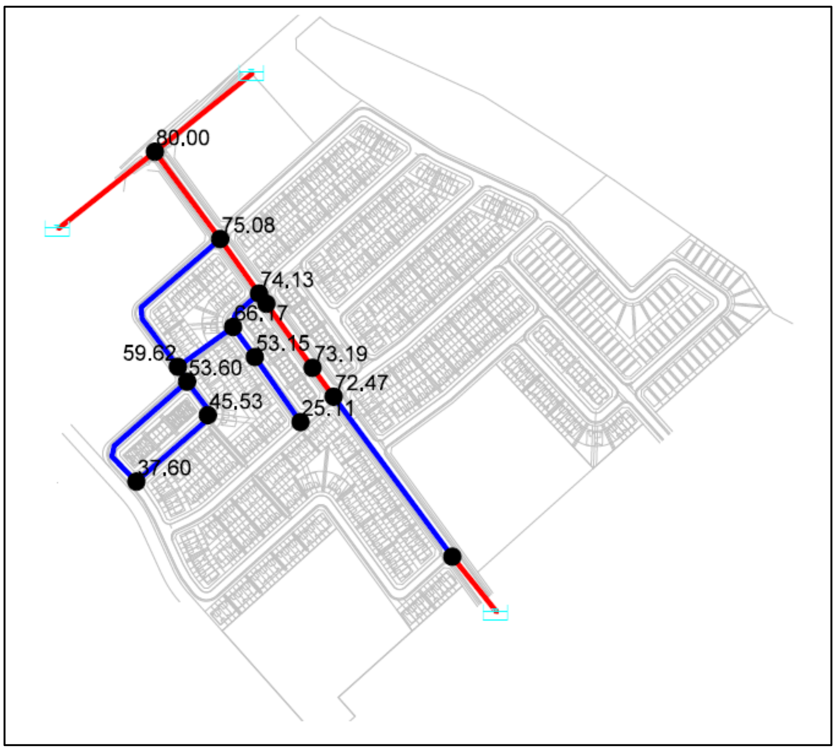
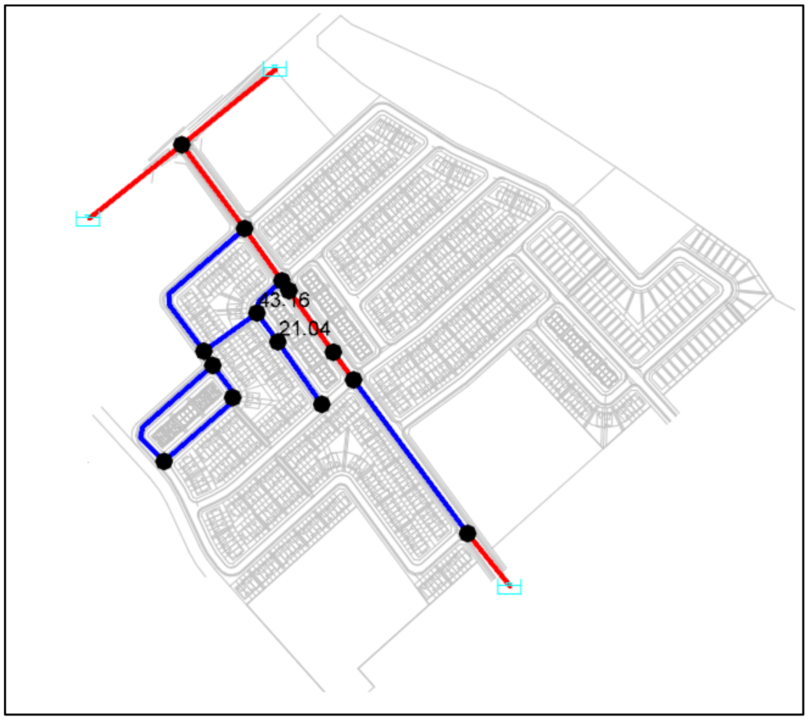


Figure 2.7: MXDY + 267 L/s Fire Flow Results for the First Construction Phase Condition Scenario (Residual Pressure (psi))



Proposed First Phase Registration - Interim Scenario

The maximum pressure modeled during the average day scenario was approximately 93.4 psi (644 kPa), and the minimum pressure during the peak hour scenario was approximately 80.8 psi (557 kPa)as shown in **Figure 2.8** and **Figure 2.9** respectively. These pressures are above the serviceable limit of 50 to 80 psi (345 to 552 kPa) and therefore, all the proposed units will require pressure reducing valves. Results of the hydraulic modeling are included for reference in **Appendix A.2**.

Figure 2.8: AVDY Pressure Results for Interim Condition Scenario (psi)

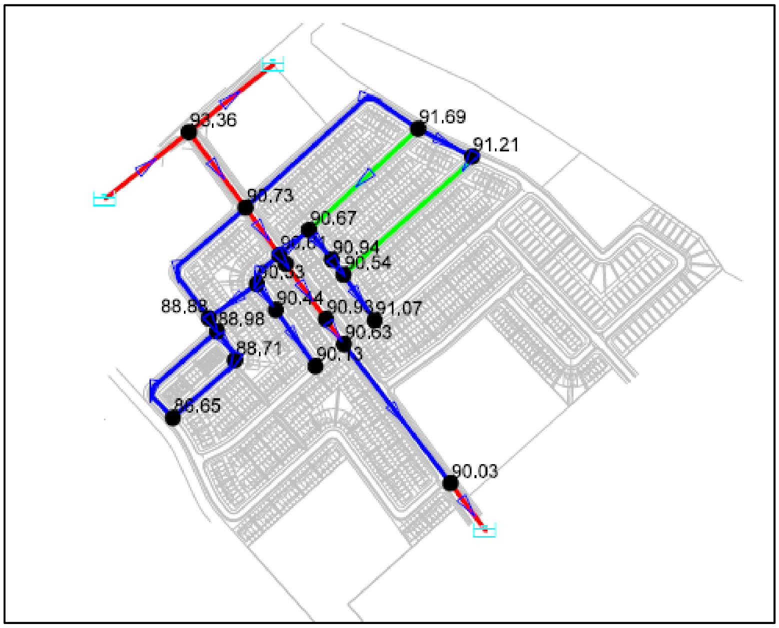
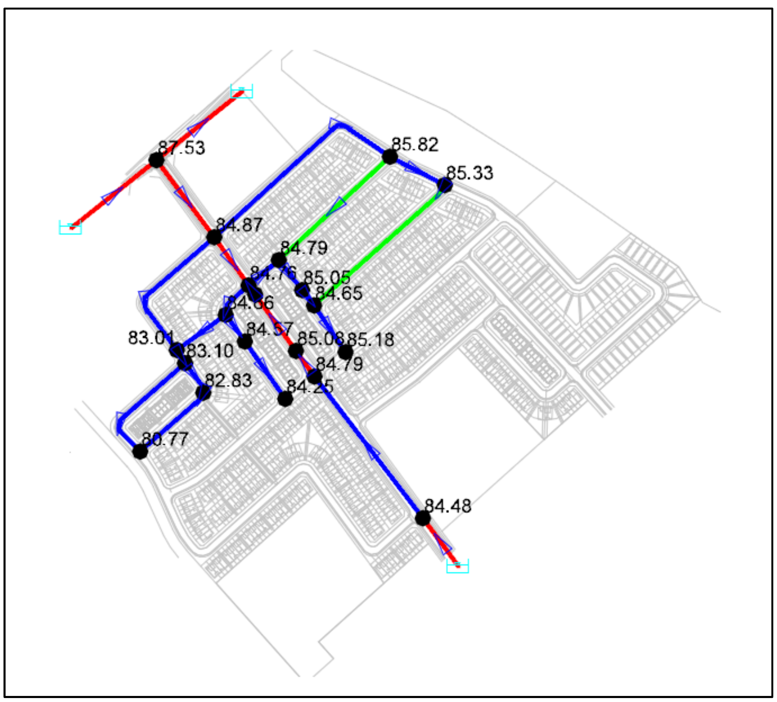
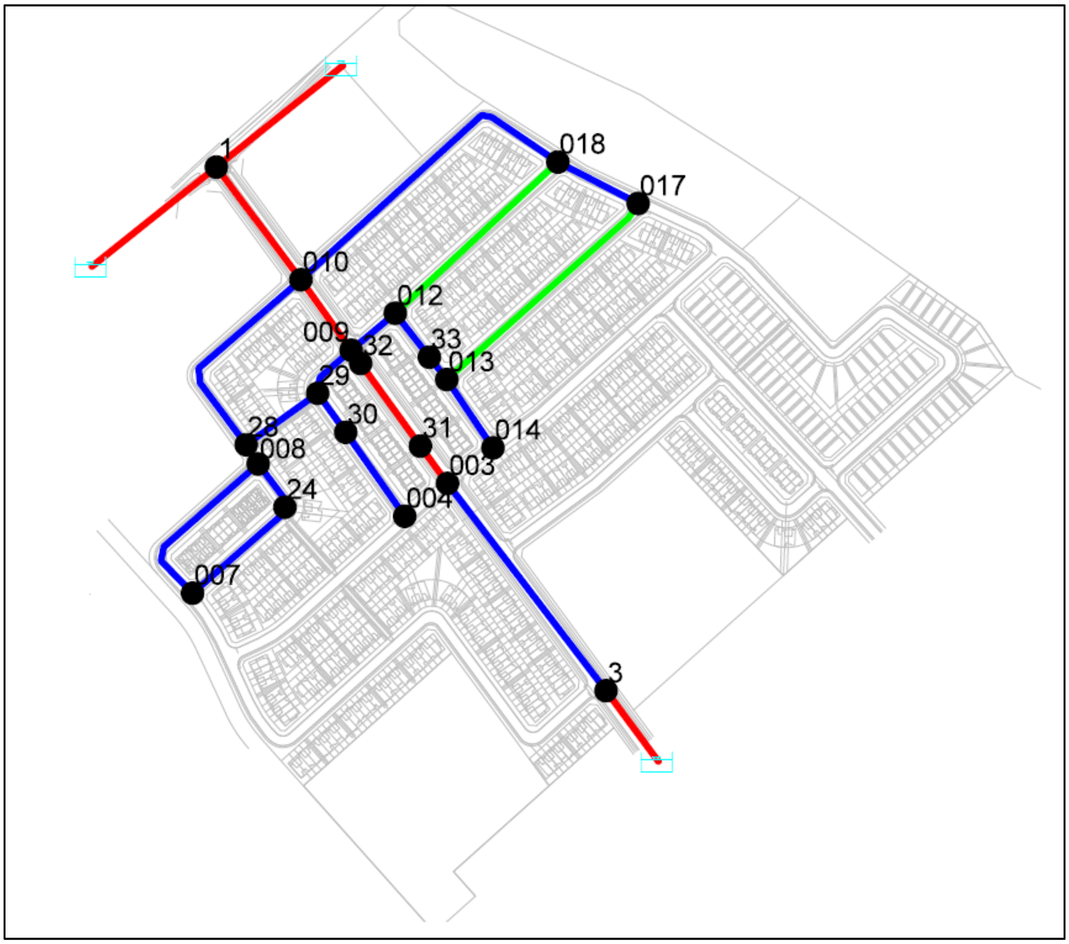


Figure 2.9: PKHR Pressure Results for Interim Condition Scenario (psi)



Due to phasing and construction restrictions in the proposed interim condition, two temporary dead ends are located on Sheaf Row and Corn Husk Lane. Both temporary dead-ends service less than 49 units. Alternative looping options were considered along winter Wheat Terrace; however, due to future deep sanitary and storm servicing along Winter Wheat Terrace in the ultimate condition scenario, the addition of a watermain to connect the interim dead ends would result in throw away pipe. **Figure 2.10** shows the node IDs used in the interim condition model.

Figure 2.10: Interim Condition Scenario Node ID



A fire flow analysis was carried out in the hydraulic model to determine the anticipated amount of flow that could be provided across the proposed watermain network under maximum day demands and a fire flow requirement of 200 L/s and 267 L/s as per the worst-case conditions for the back-to-back units and townhome blocks.

As mentioned above, additional FUS calculation were provided for Block 1 and Block 2 as the rear yard separation distance to the adjacent side yard was measured to be less than 10 m.

Results of the interim condition modeling analysis indicate that flows in excess of 12,595 L/min (210 L/s) can be delivered for the units that require 200 L/s or less, and 16,180 L/min (270 L/s) can be delivered to the proposed back to back units on Sheaf Row and Cornhusk Lane, which require 267 L/s while still maintaining a residual pressure greater than 140 kPa (20 psi). The resulting residual pressures for the proposed first registration phase of the development are shown in **Figure 2.11** and **Figure 2.12**. Results of the hydraulic modeling are included for reference in **Appendix A.2.**

Figure 2.11: MXDY + 200 L/s Fire Flow Results for Interim Condition Scenario (Residual Pressure (psi))

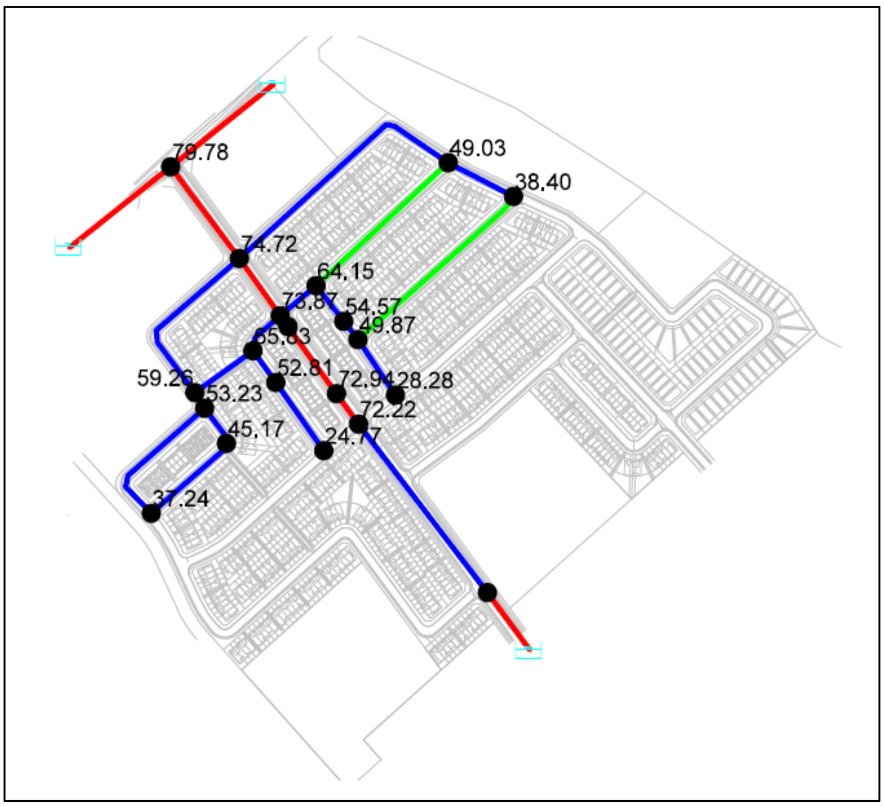
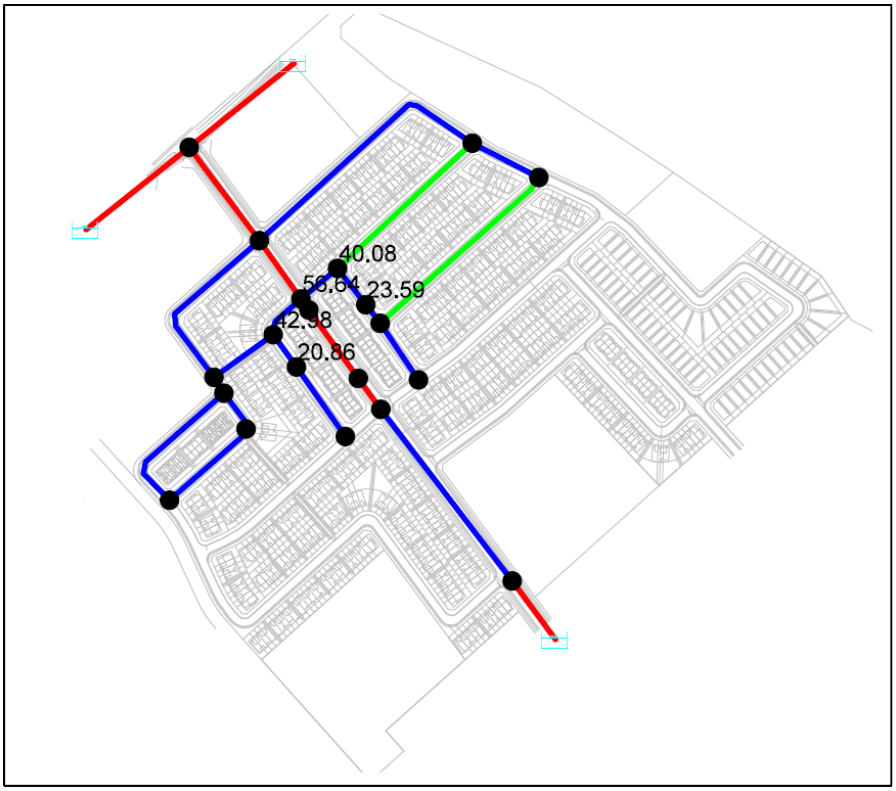


Figure 2.12: MXDY + 267 L/s Fire Flow Results Interim Condition Scenario (Residual Pressure (psi))



Ultimate Condition Scenario

In the ultimate development condition, the maximum pressure modeled was approximately 91.3 psi (630 kPa) and the minimum pressure during the peak hour scenario was approximately 80.9 psi (557 kPa) as shown in **Figure 2.13** and **Figure 2.14** respectively. These pressures are above the serviceable limit of 50 to 80 psi (345 to 552 kPa) and therefore all units will require pressure reducing valves.

Figure 2.13: AVDY Pressure Results for Ultimate Condition Scenario (psi)

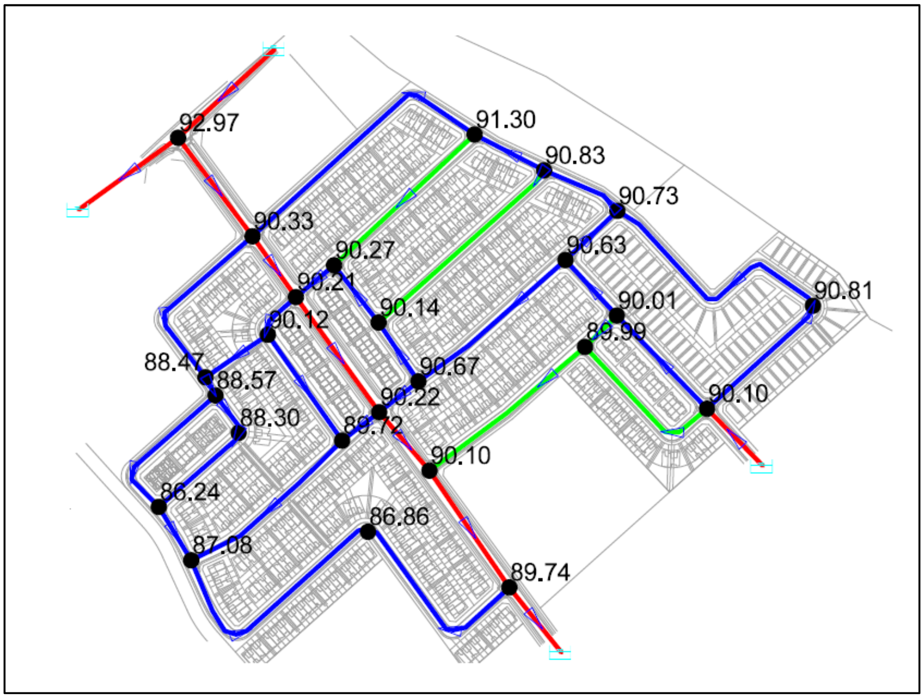
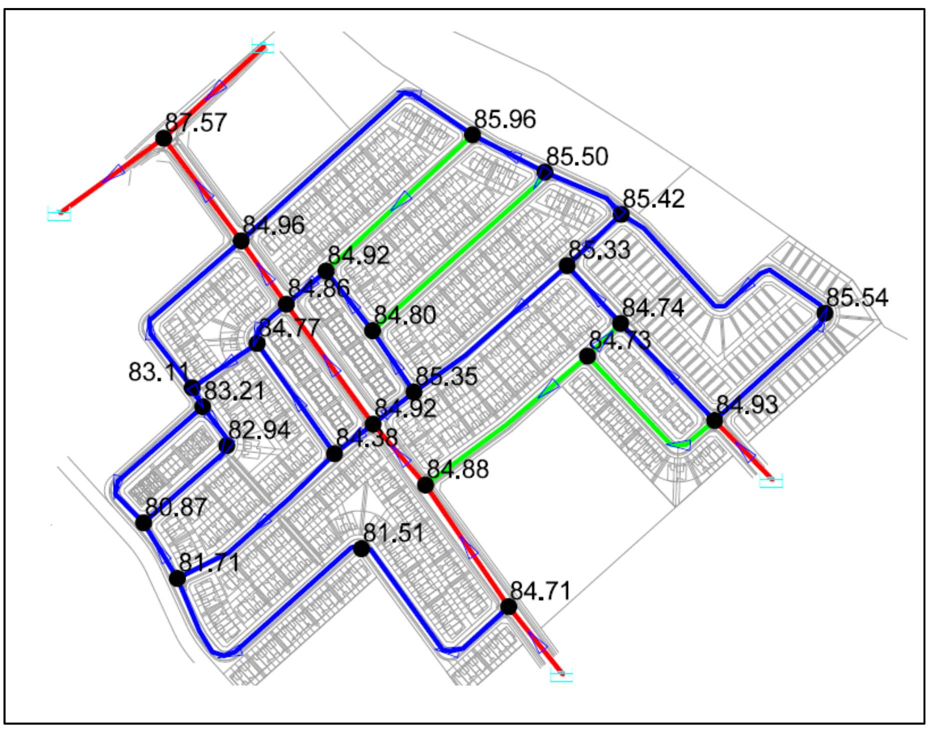


Figure 2.14: PKHR Pressure Results for Ultimate Condition Scenario (psi)



Similar to the previous two scenarios, a fire flow analysis was carried out in the hydraulic model to determine the anticipated amount of flow that could be provided for the proposed development under maximum day demands and a fire flow requirements of 200 L/s, and 267 L/s for the back to back units.

Results of the ultimate conditions modeling analysis indicate that flows in excess of 20,220 L/min (337 L/s) can be delivered for the units that require 200 L/s or less, and 32,700 L/min (545 L/s) can be delivered to units that require 267 L/s while still maintaining a residual pressure of 140 kPa (20 psi). The residual pressures for the different fire flow analyses are shown in **Figure 2.15, Figure 2.16**. Results of the hydraulic modeling are included for reference in **Appendix A3**.

Figure 2.15: MXDY + 200 L/s Fire Flow Results for Ultimate Condition Scenario (Residual Pressure (psi))

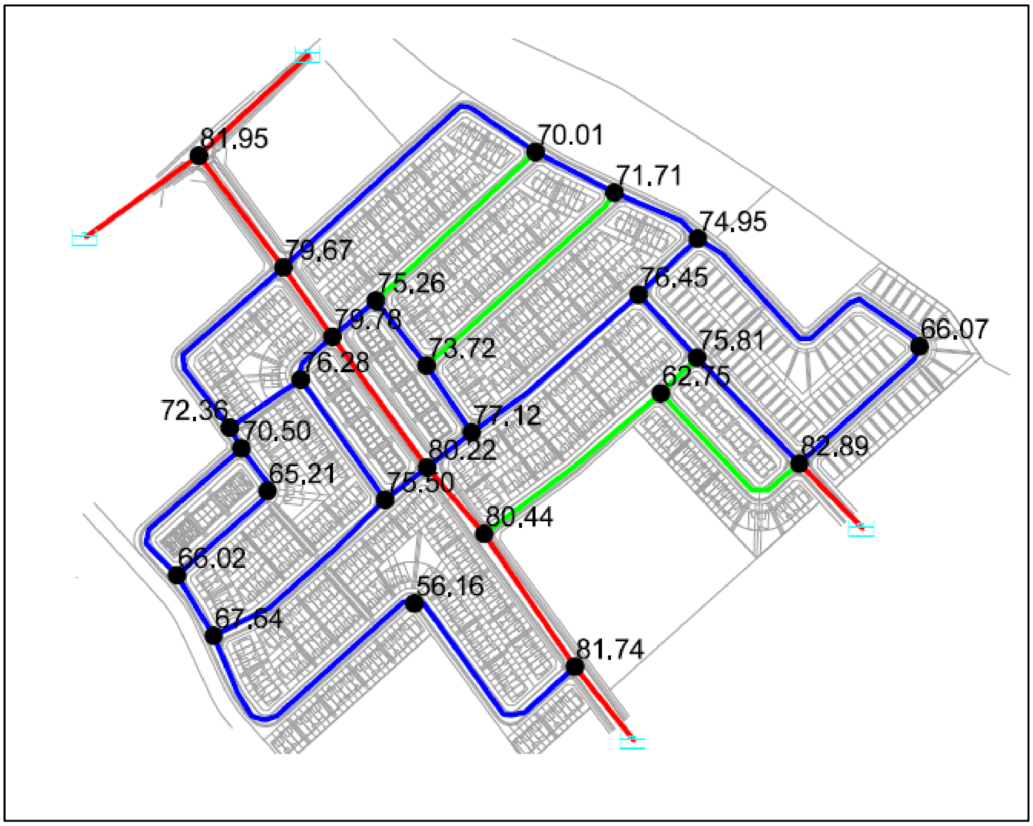
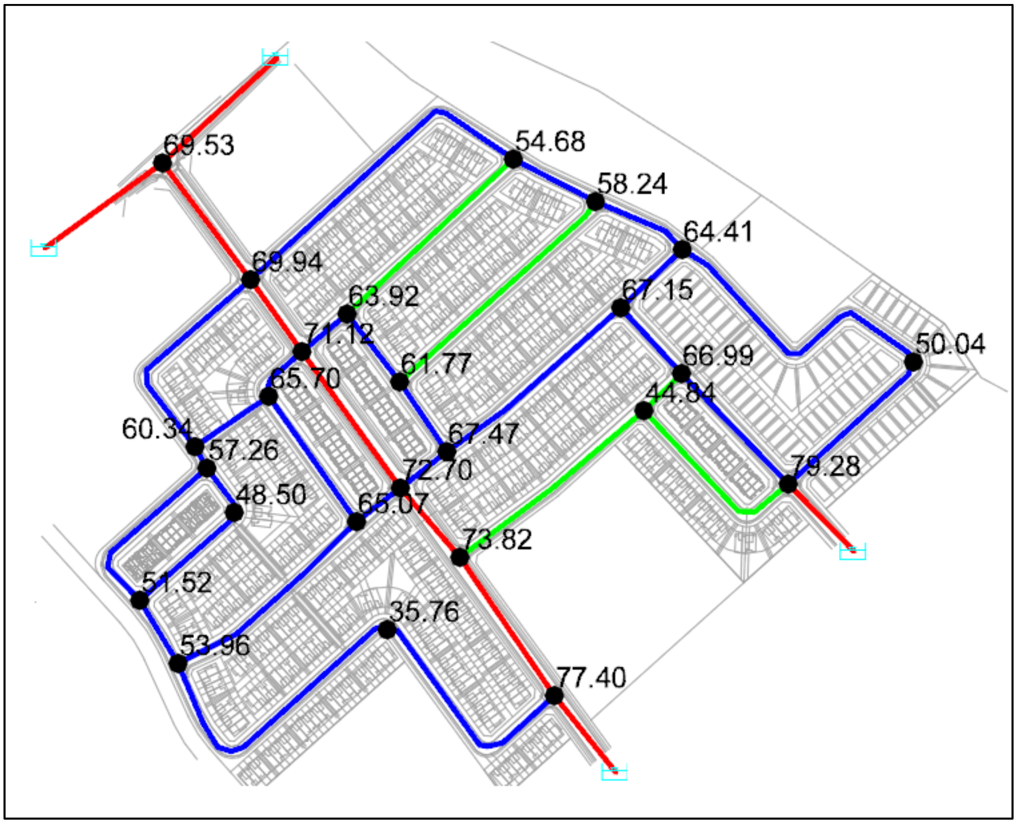


Figure 2.16:MXDY + 267 L/s Fire Flow Results for Ultimate Condition Scenario (Residual Pressure (psi))



2.6 WATER AGE CALCULATIONS

Estimated daily consumption for the first construction phase and interim conditions is based on an average usage of 350 L/c/d and 2.7 people per townhome and back-to-back unit.

2.6.1 First Construction Phase

Total Site

The water volume calculation is based on the length and size of all watermain piping within the first construction phase. The average daily consumption is anticipated to be more than the total pipe volume of the first construction phase.

Volume of 204 mm diameter pipe = 33.14 m^3

Volume of 297 mm diameter pipe = 35.38 m^3

Total Pipe Volume = 68.52 m^3

Appendix B - WASTEWATER SERVICING CALCULATIONS

B.1 SANITARY SEWER DESIGN SHEET

SANITARY SEWER DESIGN SHEET
(City of Ottawa)

Stantec

SUBDIVISION:
Kanata West Block 29

DATE: 2021-03-19
REVISION: 1
DESIGNED BY: WAJ
CHECKED BY: DJC

FILE NUMBER: 160401608

DESIGN PARAMETERS

MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	280 l/p/day	MINIMUM VELOCITY	0.60 m/s
MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	28,000 l/ha/day	MAXIMUM VELOCITY	3.00 m/s
PEAKING FACTOR (INDUSTRIAL):	2.4	INDUSTRIAL (HEAVY)	55,000 l/ha/day	MANNINGS n	0.013
PEAKING FACTOR (ICI >20%):	1.5	INDUSTRIAL (LIGHT)	35,000 l/ha/day	BEDDING CLASS	B
PERSONS / SINGLE	3.4	INSTITUTIONAL	28,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / 2BR TERRACE FLAT	2.1	INFILTRATION	0.33 l/s/Ha	HARMON CORRECTION FACTOR	0.8
PERSONS / APARTMENT	1.8				

LOCATION			RESIDENTIAL AREA AND POPULATION									COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+I+I	INFILTRATION			TOTAL	PIPE								
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	UNITS SINGLE	UNITS 2BR	UNITS APT	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)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
R5A	5	4	0.09	0	6	0	13	0.09	13	3.72	0.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.09	0.09	0.0	0.2	17.6	200	PVC	SDR 35	0.65	27.0	**0.67%**	0.85	0.21
R4A	4	3	0.19	0	12	0	25	0.28	38	3.67	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.19	0.28	0.1	0.5	43.0	200	PVC	SDR 35	0.50	23.6	**2.29%**	0.74	0.26
R6A	6	3	0.36	0	24	0	50	0.36	50	3.65	0.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.36	0.36	0.1	0.7	70.2	200	PVC	SDR 35	0.50	23.6	**3.02%**	0.74	0.28
R3A	3	2	0.06	0	6	0	13	0.70	101	3.59	1.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.06	0.70	0.2	1.4	13.9	200	PVC	SDR 35	0.50	23.6	**5.94%**	0.74	0.34
R2A	2	1	0.04	0	0	0	0	0.74	101	3.59	1.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.04	0.74	0.2	1.4	22.5	200	PVC	SDR 35	0.50	23.6	**6.00%**	0.74	0.34
																												200							

B.2 SANITARY EXCERPTS FROM KANATA WEST DEVELOPMENT FIRST REGISTRATION PHASE REPORT (STANTEC, FEBRUARY 2020)

the ultimate condition were 281 m³ and 810 m³ respectively, while the wet weather peak flow of 1,250 L/s was used as the pumping rate.

4.5.4 Sanitary HGL Analysis Results

The following tables summarize the HGL results from the catastrophic failure (annual parameters) scenarios and the normal operating conditions (rare parameters) scenario across the proposed first registration phase of the Richcraft KW Development. Table summarizing the HGL results across the entire Richcarft KW site have been included in **Appendix C.2**.

Table 4.4: Catastrophic Pump Station Failure Sanitary HGL – Annual Parameters

MH ID	Road Grade (m)	USF (m)	Catastrophic HGL and USF – HGL Clearance							
			Scenario 1		Scenario 2		Scenario 3		Scenario 4	
			HGL (m)	Clearance (m)	HGL (m)	Clearance (m)	HGL (m)	Clearance (m)	HGL (m)	Clearance (m)
1A	96.20	N/A	95.35	-	95.29	-	95.32	-	95.21	-
1B	96.60	N/A	95.31	-	95.27	-	95.28	-	95.17	-
1C	95.30	N/A	95.24	-	95.22	-	95.23	-	95.10	-
2	97.45	N/A	95.36	-	95.30	-	95.33	-	95.22	-
3	97.89	95.79	95.39	0.40	95.33	0.46	95.36	0.43	95.25	0.54
4	98.19	96.40	95.40	1.00	95.34	1.06	95.36	1.04	95.26	1.14
5	98.63	96.64	95.43	1.21	95.43	1.21	95.43	1.21	95.43	1.21
6	98.67	96.99	95.52	1.47	95.52	1.47	95.52	1.47	95.52	1.47
7	99.11	96.99	96.01	0.98	96.01	0.98	96.01	0.98	96.01	0.98
8	100.21	97.69	97.15	0.54	97.15	0.54	97.15	0.54	97.15	0.54
9	99.23	97.44	96.21	1.23	96.21	1.23	96.21	1.23	96.21	1.23
10	99.43	97.67	96.31	1.36	96.31	1.36	96.31	1.36	96.31	1.36
11	100.75	97.67	96.90	0.77	96.90	0.77	96.90	0.77	96.90	0.77
12	100.60	98.39	97.03	1.36	97.03	1.36	97.03	1.36	97.03	1.36
3A	97.75	95.71	95.39	0.32	95.33	0.38	95.36	0.35	95.25	0.46
3B	97.66	95.71	95.39	0.32	95.33	0.38	95.36	0.35	95.25	0.46
3C	97.53	95.71	95.39	0.32	95.33	0.38	95.36	0.35	95.25	0.46
14	97.61	N/A	95.40	-	95.34	-	95.37	-	95.26	-
16	97.35	95.71	95.41	0.30	95.35	0.36	95.38	0.33	95.27	0.44
17	97.29	N/A	95.41	-	95.35	-	95.38	-	95.27	-
17A	97.32	95.63	95.41	0.22	95.35	0.28	95.38	0.25	95.27	0.36
17B	97.37	95.63	95.41	0.22	95.35	0.28	95.38	0.25	95.27	0.36

MH ID	Road Grade (m)	USF (m)	Catastrophic HGL and USF – HGL Clearance							
			Scenario 1		Scenario 2		Scenario 3		Scenario 4	
			HGL (m)	Clearance (m)	HGL (m)	Clearance (m)	HGL (m)	Clearance (m)	HGL (m)	Clearance (m)
18	97.17	95.63	95.42	0.21	95.36	0.27	95.38	0.25	95.28	0.35
18A	97.65	95.69	95.42	0.27	95.36	0.33	95.38	0.31	95.28	0.41
19	97.43	95.63	95.42	0.21	95.36	0.27	95.39	0.24	95.28	0.35
20	97.50	95.63	95.43	0.20	95.37	0.26	95.40	0.23	95.29	0.34
22	97.96	96.15	95.41	0.74	95.35	0.80	95.38	0.77	95.27	0.88
22A	97.83	96.10	95.41	0.69	95.35	0.75	95.38	0.72	95.27	0.83
22B	97.84	96.10	95.41	0.69	95.35	0.75	95.38	0.72	95.27	0.83
23	98.10	96.54	95.41	1.13	95.35	1.19	95.38	1.16	95.27	1.27
24	98.05	96.30	95.41	0.89	95.35	0.95	95.38	0.92	95.27	1.03
25	98.00	96.30	95.41	0.89	95.35	0.95	95.38	0.92	95.27	1.03
26	98.10	96.30	95.41	0.89	95.35	0.95	95.38	0.92	95.27	1.03
27	97.92	95.75	95.41	0.34	95.35	0.40	95.38	0.37	95.27	0.48
30	98.01	95.90	95.41	0.49	95.35	0.55	95.38	0.52	95.27	0.63
31	97.52	95.70	95.43	0.27	95.37	0.33	95.40	0.30	95.29	0.41
32	97.68	95.90	95.43	0.47	95.37	0.53	95.40	0.50	95.29	0.61
33	97.62	96.00	95.41	0.59	95.35	0.65	95.38	0.62	95.27	0.73
35	97.96	96.04	95.43	0.61	95.37	0.67	95.40	0.64	95.29	0.75
35A	97.94	N/A	95.42	-	95.36	-	95.39	-	95.28	-

As can be seen in the above table, the worst-case annual HGL (Scenario 1) remains below the proposed USF elevations across the proposed development.

Table 4.5: Normal Operating Conditions Sanitary HGL

Sanitary Manhole	Rim Elevation (m)	USF (m)	HGL (m)	USF - HGL Clearance
1A	96.20	N/A	88.79	-
1B	96.60	N/A	88.61	-
1C	95.30	N/A	87.66	-
2	97.45	N/A	89.01	-
3	97.89	95.79	91.11	4.68
4	98.19	96.40	94.10	2.30

Stantec

SUBDIVISION:

Richcraft Kanata West Subdivision - First Registration Phase

DATE: 8/27/2019
REVISION: 3
DESIGNED BY: WAJ
CHECKED BY: AMP

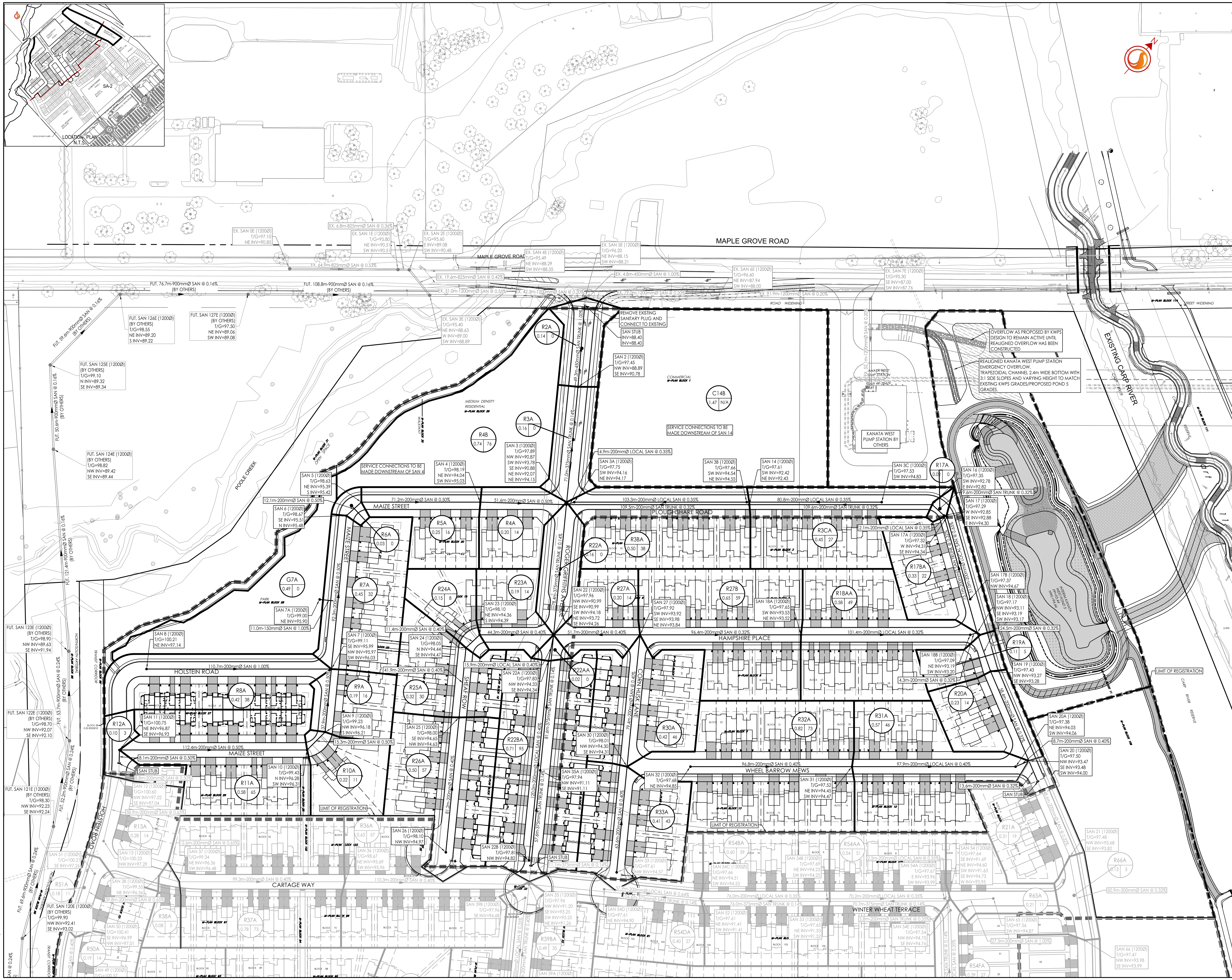
SANITARY SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160401393

DESIGN PARAMETERS

MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	280 L/p/day	MINIMUM VELOCITY	0.60 m/s
MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	28,000 L/ha/day	MAXIMUM VELOCITY	3.00 m/s
PEAKING FACTOR (INDUSTRIAL):	2.4	INDUSTRIAL (HEAVY)	55,000 L/ha/day	MANNINGS n	0.013
PEAKING FACTOR (ICI >20%):	1.5	INDUSTRIAL (LIGHT)	35,000 L/ha/day	BEDDING CLASS	B
PERSONS / SINGLE	3.4	INSTITUTIONAL	28,000 L/ha/day	MINIMUM COVER	2.50 m
PERSONS / TOWNHOME	2.7	INFILTRATION	0.33 L/s/ha	HARMON CORRECTION FACTOR	0.8
PERSONS / APARTMENT	1.8	Medium Denisty Residential Blcoks	38 units/ha		

LOCATION			RESIDENTIAL AREA AND POPULATION									COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+I+I	INFILTRATION			TOTAL	PIPE								
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	UNITS SINGLE	UNITS TOWN	UNITS APT	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)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (L/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (L/s)	FLOW (L/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
PROPOSED FIRST REGISTRATION PHASE																																			
R8A	8	7	0.42	0	14	0	38	0.42	38	3.67	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.42	0.42	0.1	0.6	110.7	200	PVC	SDR 35	1.00	33.4	1.8%	1.05	0.34
FUT R13A	*13*	*12*	*0.28*	*0*	*5*	*0*	*14*	*0.28*	*14*	*3.72*	*0.2*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.00*	*0.0*	*0.28*	*0.28*	*0.1*	*0.3*	*38.9*	*200*	*PVC*	*SDR 35*	*0.65*	*27.0*	*0.9%*	*0.85*	*0.23*
R12A	12	11	0.10	0	1	0	3	0.38	16	3.71	0.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.10	0.38	0.1	0.3	16.8	200	PVC	SDR 35	0.50	23.6	1.4%	0.74	0.21
R11A	11	10	0.58	0	24	0	65	0.96	81	3.61	0.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.58	0.96	0.3	1.3	112.4	200	PVC	SDR 35	0.50	23.6	5.4%	0.74	0.33
R10A	10	9	0.22	0	4	0	11	1.18	92	3.60	1.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.22	1.18	0.4	1.5	15.3	200	PVC	SDR 35	0.50	23.6	6.2%	0.74	0.34
R9A	9	7	0.19	0	6	0	16	1.37	108	3.59	1.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.19	1.37	0.5	1.7	37.9	200	PVC	SDR 35	0.50	23.6	7.2%	0.74	0.36
G7A, R7A	7	6	0.45	0	12	0	32	2.24	178	3.53	2.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.49	0.49	0.0	0.93	2.73	0.9	2.9	92.3	200	PVC	SDR 35	0.50	23.6	12.4%	0.74	0.42
R6A	6	5	0.03	0	0	0	0	2.27	178	3.53	2.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.49	0.0	0.03	2.76	0.9	3.0	12.1	200	PVC	SDR 35	0.50	23.6	12.5%	0.74	0.42
R5A	5	4	0.25	0	6	0	16	2.53	194	3.52	2.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.49	0.0	0.25	3.01	1.0	3.2	71.2	200	PVC	SDR 35	0.50	23.6	13.6%	0.74	0.43
R4A, R4B	4	3	0.94	0	5	0	90	3.47	284	3.47	3.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.49	0.0	0.94	3.95	1.3	4.5	51.6	200	PVC	SDR 35	0.50	23.7	19.0%	0.74	0.48
R33A	33	30	0.41	0	16	0	43	0.41	43	3.66	0.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.41	0.41	0.1	0.6	63.6	200	PVC	SDR 35	0.40	21.1	3.1%	0.67	0.25
R30A	30	27	0.42	0	17	0	46	0.83	89	3.61	1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.42	0.83	0.3	1.3	79.4	200	PVC	SDR 35	0.40	21.1	6.2%	0.67	0.31
R27A	27	22	0.20	0	5	0	14	1.02	103	3.59	1.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.20	1.02	0.3	1.5	51.7	200	PVC	SDR 35	0.40	21.1	7.2%	0.67	0.32
	35	35A	0.00	0	0	0	0	13.99	1196	3.20	12.4	0.00	13.19	0.00	0.00	0.00	0.00	0.00	3.99	0.00	1.28	8.4	0.00	32.44	10.7	31.5	68.7	375	PVC	SDR 35	0.14	60.1	52.4%	0.57	0.49
	35A	22	0.00	0	0	0	0	13.99	1196	3.20	12.4	0.00	13.19	0.00	0.00	0.00	0.00	0.00	3.99	0.00	1.28	8.4	0.00	32.44	10.7	31.5	81.6	375	PVC	SDR 35	0.14	60.7	51.8%	0.58	0.50
R26A	26	25	0.50	0	21	0	57	0.50	57	3.64	0.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.50	0.50	0.2	0.8	81.2	200	PVC	SDR 35	0.40	21.1	3.9%	0.67	0.27
R25A	25	24	0.32	0	11	0	30	0.82	86	3.61	1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.32	0.82	0.3	1.3	41.9	200	PVC	SDR 35	0.40	21.2	6.0%	0.67	0.30
R24A	24	23	0.15	0	3	0	8	0.97	95	3.60	1.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.15	0.97	0.3	1.4	11.4	200	PVC	SDR 35	0.40	21.1	6.7%	0.67	0.31
R23A	23	22	0.19	0	5	0	14	1.16	108	3.59	1.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.19	1.16	0.4	1.6	44.3	200	PVC	SDR 35	0.40	21.1	7.7%	0.67	0.33
R22BA	22B	22A	0.71	0	35	0	95	0.71	95	3.60	1.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.71	0.71	0.2	1.3	120.0	200	PVC	SDR 35	0.40	21.1	6.3%	0.67	0.31
R22AA	22A	22	0.02	0	0	0	0	0.73	95	3.60	1.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.02	0.73	0.2	1.3	15.9	200	PVC	SDR 35	0.40	21.1	6.4%	0.67	0.31
R22A	22	3	0.16	0	0	0	0	17.06	1501	3.14	15.3	0.00	13.19	0.00	0.00	0.00	0.00	0.00	3.99	0.00	1.28	8.4	0.16	35.52	11.7	35.4	78.4	375	PVC	SDR 35	0.14	61.3	57.7%	0.58	0.52
R32A	32	31	0.82	0	27	0	73	0.82	73	3.62	0.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.82	0.82	0.3	1.1	96.8	200	PVC	SDR 35	0.40	21.1	5.3%	0.67	0.29
R31A	31	20A	0.57	0	17	0	46	1.39	119	3.58	1.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.57	1.39	0.5	1.8	97.9	200	PVC	SDR 35	0.40	21.1	8.7%	0.67	0.34
	20A	20	0.00	0	0	0	0	1.39	119	3.58	1.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	1.39	0.5	1.8	8.7	200	PVC	SDR 35	0.40	21.1	8.7%	0.67	0.34
R20A	20	19	0.23	0	5	0	14	4.67	328	3.45	3.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.97	0.0	0.23	5.64	1.9	5.5	58.4	200	PVC	SDR 35	0.32	18.9	29.2%	0.60	0.43
R19A	19	18	0.11	0	2	0	5	4.78	333	3.45	3.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.97	0.0	0.11	5.75	1.9	5.6	24.5	200	PVC	SDR 35	0.32	18.9	29.7%	0.60	0.43
R27B	27	18A	0.65	0	22	0	59	0.65	59	3.64	0.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.65	0.65	0.2	0.9	96.4	200	PVC	SDR 35	0.32	18.9	4.8%	0.60	0.25
R18AA	18A	18B	0.58	0	18	0	49	1.24	108	3.59	1.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.58	1.24	0.4	1.7	101.4	200	PVC	SDR 35	0.32	18.9	8.8%	0.60	0.30
	18B	18	0.00	0	0	0	0	1.24	108	3.59	1.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	1.24	0.4	1.7	4.3	200	PVC	SDR 35	0.32	18.9	8.8%	0.60	0.30
	18	17	0.00	0	0	0	0	6.02	441	3.40	4.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.97	0.0	0.00	6.99	2.3	7.2	73.1	200	PVC	SDR 35	0.32	18.9	37.9%	0.60	0.47
R17BA	17B	17A	0.33	0	8	0	22	0.33	22	3.70	0.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.33	0.33	0.1	0.4	51.0	200	PVC	SDR 35	0.65	27.0	1.4%	0.85	0.24
	17A	17	0.00	0	0	0	0	0.33	22	3.70	0.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	0.33	0.1	0.4	2.1	200	PVC	SDR 35	0.35	19.8	1.9%	0.62	0.20
R17A	17	16	0.02	0	0	0	0	6.36	463	3.39	5.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.97	0.0	0.02	7.33	2.4	7.5	9.6	200	PVC	SDR 35	0.32	18.9	39.7%	0.60	0.47
	16	14	0.00	0	0	0	0	6.36	463	3.39	5.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.97	0.0	0.00	7.33	2.4	7.5	109.6	200	PVC	SDR 35	0.32	18.9	39.7%	0.60	0.47
C14B	14	3	0.00	0	0	0	0	6.36	463	3.39	5.1	1.47	1.47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.97	0.5	1.47	8.80	2.9	8.5	109.5	200	PVC	SDR 35	0.32	18.9	44.8%	0.60	0.49
R3CA	3C	3B	0.45	0	10	0	27	0.45	27	3.69	0.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.45	0.45	0.1	0.5	80.8	200	PVC	SDR 35	0.35	19.8	2.4%	0.62	0.21
R3BA	3B	3A	0.50	0	14	0	38	0.95	65	3.63	0.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.50	0.95	0.3	1.1	103.3	200	PVC	SDR 35	0.35	19.8	5.4%	0.62	0.27
	3A	3	0.00	0	0	0	0	0.95	65	3.63	0.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	0.95	0.3	1.1	4.9	200	PVC	SDR 35	0.35	19.8	5.4%	0.62	0.27
R3A	3	2	0.16	0	0	0	0	28.00	2313	3.03	22.7	0.00	14.66	0.00	0.00	0.00	0.00	0.00	3.99	0.00	2.74	9.1	0.16	49.39	16.3	48.1	71.0	375	PVC	SDR 35	0.14	60.1	80.0%	0.57	0.56
R2A	2	1A	0.14	0	0	0	0	28.14	2313	3.03	22.7	0.00	14.66	0.00	0.00	0.00	0.00	0.00	3.99	0.00	2.74	9.1	0.14	49.52	16.3	48.1	49.3	450	PVC	SDR 35	1.00	300.3	16.0%	1.83	1.11
	1A	1	0.00	0	0	0	0	28.14	2313	3.03	22.7	0.00	14.66	0.00	0.00	0.00	0.00	0.00	3.99	0.00	2.74	9.1	0.00	49.52	16.3	48.1	4.8	450	PVC	SDR 35	1.00	300.1	16.0%	1.83	1.11
																												450							



ORIGINAL SHEET - ARCH D

Stantec

Stantec Consulting Ltd.
400 - 1331 Clyde Avenue
Ottawa ON
Tel. 613.722.4420
www.stantec.com

Legend

- 1 / 0.38 / 14 — SANITARY DRAINAGE AREA ID#; POPULATION; SANITARY DRAINAGE AREA ha.
- SANITARY DRAINAGE AREA
- PROPOSED SANITARY SEWER
- EXISTING SANITARY SEWER

1. CONTROLLED SETTLEMENT JOINTS TO BE USED ON EACH BUILDING SERVICE CONNECTION AS PER CITY STANDARD S11

9	ISSUED FOR ECA APPROVAL	WAJ	SGG	20.02.12
8	REVISED AS PER CITY COMMENTS	WAJ	SGG	20.01.31
7	ISSUED FOR TENDER	WAJ	KJH	20.01.09
6	REVISED AS PER CITY COMMENTS	WAJ	SGG	19.11.18
5	REVISED CROSSING DESIGN	WAJ	SGG	19.09.23
4	REVISED AS PER CITY COMMENTS	WAJ	SGG	19.08.30
3	REVISED AS PER CITY COMMENTS	WAJ	SGG	19.05.10
2	REVISED AS PER CITY COMMENTS	WAJ	SGG	18.12.10
1	ISSUED FOR FIRST SUBMISSION	WAJ	SGG	18.03.02
Revision		By	Appd.	YY.MM.DD

File Name: 1060401393-DB	WAJ	SGG	WAJ	17.12.20
	Dwn.	Chkd.	Dsgn.	YY.MM.DD

Permit-Seal

Client/Project

RICHCRAFT HOMES
1st REGISTRATION

RICHCRAFT KANATA WEST
1620 MAPLE GROVE ROAD
OTTAWA, ON

Title

SANITARY DRAINAGE AREA PLAN

Project No. 160401393

Scale 1:1000 (0 10 30 50m)

Drawing No. SA-1

Sheet 42 of 44

Revision 9

CITY FILE No. D07-16-04-0017 CITY PLAN No. 17529

Appendix C - STORMWATER MANAGEMENT

C.1 STORM SEWER DESIGN SHEET

Stantec	Kanata West Block 29		STORM SEWER DESIGN SHEET (City of Ottawa)
	DATE:	2021-03-19	
	REVISION:	1	
	DESIGNED BY:	WAJ	FILE NUMBER: 160401608
	CHECKED BY:	DJC	

DESIGN PARAMETERS

$I = a / (t+b)^c$ (As per City of Ottawa Guidelines, 2012)

	1:2 yr	1:5 yr	1:10 yr	1:100 yr					
a =	732.951	998.071	1174.184	1735.688	MANNING'S n =	0.013		BEDDING CLASS =	B
b =	6.199	6.053	6.014	6.014	MINIMUM COVER:	2.00	m		
c =	0.810	0.814	0.816	0.820	TIME OF ENTRY	10	min		

LOCATION / DRAINAGE AREA

AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (2-YEAR) (ha)	AREA (5-YEAR) (ha)	AREA (10-YEAR) (ha)	AREA (100-YEAR) (ha)	AREA (ROOF) (ha)	C (2-YEAR) (-)	C (5-YEAR) (-)	C (10-YEAR) (-)	C (100-YEAR) (-)
L104A	104	103	0.16	0.00	0.00	0.00	0.00	0.82	0.00	0.00	0.00
L106A	106	103	0.24	0.00	0.00	0.00	0.00	0.76	0.00	0.00	0.00
L103A	103	102	0.08	0.00	0.00	0.00	0.00	0.82	0.00	0.00	0.00
	102	101	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

DRAINAGE AREA (continued)

AREA ID NUMBER	A x C (2-YEAR) (ha)	ACCUM AxC (2YR) (ha)	A x C (5-YEAR) (ha)	ACCUM. AxC (5YR) (ha)	A x C (10-YEAR) (ha)	ACCUM. AxC (10YR) (ha)	A x C (100-YEAR) (ha)	ACCUM. AxC (100YR) (ha)	T of C (min)	$I_{2\text{-YEAR}}$ (mm/h)	$I_{5\text{-YEAR}}$ (mm/h)	$I_{10\text{-YEAR}}$ (mm/h)	$I_{100\text{-YEAR}}$ (mm/h)	$Q_{CONTROL}$ (L/s)	ACCUM. $Q_{CONTROL}$ (L/s)	Q_{ACT} (CIA/360) (L/s)
L104A	0.131	0.131	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	28.0
									10.83							
L106A	0.185	0.185	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	39.5
									10.97							
L103A	0.065	0.382	0.000	0.000	0.000	0.000	0.000	0.000	10.97	73.28	99.35	116.43	170.18	0.0	0.0	77.7
	0.000	0.382	0.000	0.000	0.000	0.000	0.000	0.000	11.19	72.52	98.30	115.20	168.36	0.0	0.0	76.9
									11.54							

PIPE SELECTION

AREA ID NUMBER	LENGTH (m)	PIPE WIDTH OR DIAMETEI (mm)	PIPE HEIGHT (mm)	PIPE SHAPE (-)	MATERIAL (-)	CLASS (-)	SLOPE %	Q_{CAP} (FULL) (L/s)	% FULL (-)	VEL. (FULL) (m/s)	VEL. (ACT) (m/s)	TIME OF FLOW (min)
L104A	38.7	300	300	CIRCULAR	PVC	SDR 35	0.50	68.0	**41.22%**	0.97	0.78	0.83
L106A	50.1	300	300	CIRCULAR	PVC	SDR 35	0.50	68.0	**58.15%**	0.97	0.86	0.97
L103A	13.9	375	375	CIRCULAR	PVC	SDR 35	0.50	116.6	**66.69%**	1.11	1.03	0.22
	21.7	375	375	CIRCULAR	PVC	SDR 35	0.50	116.6	**65.99%**	1.11	1.03	0.35
		375	375									

C.2 RUNOFF COEFFICIENT/IMPERVIOUSNESS CALCULATIONS

Kanata West Block 29
Stantec Project No. 160401608
Runoff Coefficient Calculations
Last updated on 2021-03-22

Runoff Coefficient for Hard (Impervious) Areas, C=0.90
Runoff Coefficient for Soft (Pervious) Areas, C=0.20

Subcatchment ID	Total Area (m^2)	Hard Area (m^2)	Soft Area (m^2)	C-Value	Imperviousness (%)
UNC-1	1087	114	973	0.27	10.00
UNC-2	1523	425	1098	0.40	28.57
L103A	768	681	87	0.82	88.57
L104A	1557	1377	180	0.82	88.57
L106A	2439	1952	487	0.76	80.00

C.3 PONDING DEPTHS AT CATCHBASINS

Kanata West Block 29 - Ponding Depths in 2-year, 100-year and 100-year + 20% Chicago Events
Stantec Project. No. 160401608
Last updated on 2021-03-19

Storage Node ID	Invert Elevation (m)	Rim Elevation (m)	2-year 3-hour Chicago		100-year 3-hour Chicago		100-year 3-hour Chicago + 20%		Adjacent Lowest Building Opening (m)
			Max. Surface HGL (m)	Total Surface Water Depth (m)	Max. Surface HGL (m)	Total Surface Water Depth (m)	Max. Surface HGL (m)	Total Surface Water Depth (m)	
L103A-S	96.40	98.30	97.62	0.00	98.07	0.17	98.09	0.19	98.35
L104A-S	96.44	98.22	97.80	0.00	98.04	0.22	98.09	0.27	98.32
L106A-S	96.54	98.32	97.66	0.00	98.08	0.16	98.10	0.18	98.35

C.4 SAMPLE PCSWMM INPUT FILE

```
[TITLE]
160401608 - Kanata West Block 29 - PCSWMM Input File for 100-year 3-hour
Chicago Storm

[OPTIONS]
;;Option              Value
FLOW_UNITS            LPS
INFILTRATION          HORTON
FLOW_ROUTING          DYNWAVE
LINK_OFFSETS          ELEVATION
MIN_SLOPE             0
ALLOW_PONDING         NO
SKIP_STEADY_STATE     NO

START_DATE            02/22/2021
START_TIME            00:00:00
REPORT_START_DATE     02/22/2021
REPORT_START_TIME     00:00:00
END_DATE              02/22/2021
END_TIME              08:00:00
SWEEP_START           01/01
SWEEP_END             12/31
DRY_DAYS              0
REPORT_STEP           00:01:00
WET_STEP              00:05:00
DRY_STEP              00:05:00
ROUTING_STEP          10
RULE_STEP             00:00:00

INERTIAL_DAMPING      PARTIAL
NORMAL_FLOW_LIMITED   BOTH
FORCE_MAIN_EQUATION   H-W
VARIABLE_STEP         0.75
LENGTHENING_STEP      0
MIN_SURFAREA          0
```

```
MAX_TRIALS           8
HEAD_TOLERANCE       0.0015
SYS_FLOW_TOL         5
LAT_FLOW_TOL         5
MINIMUM_STEP         0.5
THREADS              4

[FILES]
;;Interfacing Files
USE HOTSTART "C:\Users\dchochlinski\Desktop\Local Docs\1604 
Projects\160401608 KW Block 29\PCSWMM\100C.HSF"

[EVAPORATION]
;;Data Source    Parameters
;;-------------- ----------------
CONSTANT         0.0
DRY_ONLY         NO

[RAINGAGES]
;;Name           Format    Interval SCF      Source    
;;-------------- --------- -------- -------- ----------
RG1              INTENSITY 0:10     1.0      TIMESERIES 
Chicago_100yr_3hr_10m_Ottawa

[SUBCATCHMENTS]
;;Name           Rain Gage        Outlet           Area     %Imperv  Width   
%Slope   CurbLen  SnowPack        
;;-------------- ---------------- ---------------- -------- -------- --------
-------- -------- ----------------
;0.82
L103A            RG1              L103A-S          0.076827 88.57    50      
2.5      0                        
;0.82
L104A            RG1              L104A-S          0.155656 88.57    79      
3        0                        
;0.76
```

```
L106A            RG1              L106A-S          0.243851 80       114
2        0

;0.27

UNC-1            RG1              OF-1             0.108724 10       165
25       0

;0.40

UNC-2            RG1              OF-2             0.152323 28.57    238
5        0


[SUBAREAS]

;;Subcatchment   N-Imperv   N-Perv     S-Imperv   S-Perv     PctZero
RouteTo    PctRouted

;;-------------- ---------- ---------- ---------- ---------- ---------- -----
----- ----------

L103A            0.013      0.25       1.57       4.67       0
OUTLET

L104A            0.013      0.25       1.57       4.67       0
OUTLET

L106A            0.013      0.25       1.57       4.67       0
OUTLET

UNC-1            0.013      0.25       1.57       4.67       0
PERVIOUS   100

UNC-2            0.013      0.25       1.57       4.67       0
PERVIOUS   100


[INFILTRATION]

;;Subcatchment   Param1     Param2     Param3     Param4     Param5

;;-------------- ---------- ---------- ---------- ---------- ----------

L103A            76.2       13.2       4.14       7          0

L104A            76.2       13.2       4.14       7          0

L106A            76.2       13.2       4.14       7          0

UNC-1            76.2       13.2       4.14       7          0

UNC-2            76.2       13.2       4.14       7          0


[OUTFALLS]

;;Name           Elevation  Type       Stage Data       Gated    Route To

;;-------------- ---------- ---------- ---------------- -------- ------------
----

;Minor system outlet
```

```
101              95.1       FIXED      95.64            NO
;Maize Overland Flow
MAIZE            97.9       FREE                        NO
OF-1             0          FREE                        NO
OF-2             0          FREE                        NO

[STORAGE]
;;Name           Elev.    MaxDepth   InitDepth  Shape      Curve Name/Params
N/A      Fevap    Psi      Ksat      IMD
;;-------------- -------- ---------- ---------- ---------- ------------------
---------- -------- --------          -------- --------
102              94.909   3.097      0          FUNCTIONAL 0          0
1.13     0        0
103              94.99    3.116      0          FUNCTIONAL 0          0
1.13     0        0
104              95.261   2.913      0          FUNCTIONAL 0          0
1.13     0        0
106              95.324   2.863      0          FUNCTIONAL 0          0
1.13     0        0
;T/G at CB=97.90
L103A-S          96.4     1.9        0          TABULAR    L103A-V
0        0
;T/G at CB=97.82
L104A-S          96.44    1.78       0          TABULAR    L104A-V
0        0
;T/G at CB=97.92
L106A-S          96.54    1.78       0          TABULAR    L106A-V
0        0

[CONDUITS]
;;Name           From Node        To Node          Length     Roughness
InOffset   OutOffset  InitFlow   MaxFlow
;;-------------- ---------------- ---------------- ---------- ---------- ----
------ ---------- ---------- ----------
102-101          102              101              19.825     0.013
95.209     95.1       0          0
103-102          103              102              15.81      0.013
95.29      95.22      0          0
104-103          104              103              39.454     0.013
95.561     95.368     0          0
```

```
106-103          106              103              51.158     0.013
95.624     95.368     0          0


[ORIFICES]
;;Name           From Node        To Node          Type         Offset
Qcoeff     Gated    CloseTime
;;-------------- ---------------- ---------------- ------------ ---------- --
-------- -------- ----------
L103A-IC         L103A-S          103              SIDE         96.4
0.572      NO       0
;Single ICD
L104A-IC1        L104A-S          104              SIDE         96.44
0.572      NO       0
;Single ICD
L104A-IC2        L104A-S          104              SIDE         96.44
0.572      NO       0
;Single ICD
L106A-IC         L106A-S          106              SIDE         96.54
0.572      NO       0


[WEIRS]
;;Name           From Node        To Node          Type         CrestHt
Qcoeff     Gated    EndCon   EndCoeff   Surcharge  RoadWidth  RoadSurf
Coeff. Curve
;;-------------- ---------------- ---------------- ------------ ---------- --
-------- -------- -------- ---------- ---------- ---------- ---------- ------
----------
;Overland Flow
W103A            L103A-S          MAIZE            TRANSVERSE   98.05
1.74       NO       0        0          YES
;Overland Flow
W104A            L104A-S          L103A-S          TRANSVERSE   98.07
1.74       NO       0        0          YES
;Overland Flow
W106A            L106A-S          L103A-S          TRANSVERSE   98.06
1.74       NO       0        0          YES


[XSECTIONS]
;;Link           Shape        Geom1            Geom2      Geom3      Geom4
Barrels    Culvert
```

```
;;-------------- ------------ ---------------- ---------- ---------- --------
-- ---------- ----------
102-101          CIRCULAR     0.375            0          0          0
1
103-102          CIRCULAR     0.375            0          0          0
1
104-103          CIRCULAR     0.3              0          0          0
1
106-103          CIRCULAR     0.3              0          0          0
1
L103A-IC         CIRCULAR     0.083            0          0          0
L104A-IC1        CIRCULAR     0.083            0          0          0
L104A-IC2        CIRCULAR     0.083            0          0          0
L106A-IC         CIRCULAR     0.152            0          0          0
W103A            RECT_OPEN    0.15             6.7        0          0
W104A            RECT_OPEN    0.15             14.8       0          0
W106A            RECT_OPEN    0.15             8          0          0

[TRANSECTS]
;;Transect Data in HEC-2 format
;
NC 0.013    0.025    0.013
X1 Access            6        0.0      9.5      0.0      0.0      0.0
0.0      0.0
GR 0.28     0        0.25     1.5      0.1      1.5      0        9.5
0.15     9.5
GR 0.21     12.5

[LOSSES]
;;Link           Kentry     Kexit      Kavg       Flap Gate  Seepage
;;-------------- ---------- ---------- ---------- ---------- ----------
103-102          0          0.053      0          NO         0
104-103          0          0.022      0          NO         0
106-103          0          1.344      0          NO         0

[CURVES]
;;Name           Type       X-Value    Y-Value
```

```
;;-------------- ---------- ---------- ----------
L103A-V          Storage    0          0
L103A-V                     1.51       0
L103A-V                     1.66       77.3
L103A-V                     1.661      77.3
L103A-V                     1.91       77.3

L104A-V          Storage    0          0
L104A-V                     1.38       0
L104A-V                     1.63       247.2
L104A-V                     1.631      247.2
L104A-V                     1.78       247.2

L106A-V          Storage    0          0
L106A-V                     1.38       0
L106A-V                     1.52       287.1
L106A-V                     1.521      287.1
L106A-V                     1.78       287.1


[COORDINATES]
;;Node           X-Coord            Y-Coord
;;-------------- ------------------ ------------------
101              350894.8           5017092
MAIZE            350885.965         5017105.383
OF-1             350828.486         5017121.445
OF-2             350876.943         5017080.391
102              350881.6           5017106
103              350872.3           5017119
104              350848.9           5017151
106              350834.2           5017085
L103A-S          350884.17          5017114.403
L104A-S          350863.549         5017143.636
L106A-S          350849.499         5017104.206
```

C.5 SAMPLE PCSWMM OUTPUT FILE

[TITLE]

160401608 – Kanata West Block 29 – PCSWMM Report File for 100-year 3-hour Chicago Storm

```
*************
Element Count
*************
Number of rain gages ...... 1
Number of subcatchments ... 5
Number of nodes ........... 11
Number of links ........... 11
Number of pollutants ...... 0
Number of land uses ....... 0

****************
Raingage Summary
****************
                                                               Data        Recording
Name                 Data Source                               Type        Interval
------------------------------------------------------------------------------------
RG1                  Chicago_100yr_3hr_10m_Ottawa              INTENSITY    10 min.

********************
Subcatchment Summary
********************
Name                          Area     Width   %Imperv    %Slope Rain Gage
Outlet
----------------------------------------------------------------------------
--------------------------------
L103A                         0.08     50.00     88.57    2.5000 RG1
L103A-S
L104A                         0.16     79.00     88.57    3.0000 RG1
L104A-S
L106A                         0.24    114.00     80.00    2.0000 RG1
L106A-S
UNC-1                         0.11    165.00     10.00   25.0000 RG1
OF-1
```

```
  UNC-2                         0.15     238.00      28.57     5.0000 RG1
OF-2


  ************
  Node Summary
  ************
                                              Invert      Max.    Ponded
External
  Name                 Type                    Elev.     Depth      Area
Inflow
  ----------------------------------------------------------------------------
----
  101                  OUTFALL                 95.10      0.38       0.0
  MAIZE                OUTFALL                 97.90      0.00       0.0
  OF-1                 OUTFALL                  0.00      0.00       0.0
  OF-2                 OUTFALL                  0.00      0.00       0.0
  102                  STORAGE                 94.91      3.10       0.0
  103                  STORAGE                 94.99      3.12       0.0
  104                  STORAGE                 95.26      2.91       0.0
  106                  STORAGE                 95.32      2.86       0.0
  L103A-S              STORAGE                 96.40      1.90       0.0
  L104A-S              STORAGE                 96.44      1.78       0.0
  L106A-S              STORAGE                 96.54      1.78       0.0


  ************
  Link Summary
  ************
  Name             From Node        To Node          Type            Length
%Slope Roughness
  ----------------------------------------------------------------------------
------------------
  102-101          102              101              CONDUIT           19.8
0.5498    0.0130
  103-102          103              102              CONDUIT           15.8
0.4428    0.0130
  104-103          104              103              CONDUIT           39.5
0.4892    0.0130
  106-103          106              103              CONDUIT           51.2
0.5004    0.0130
```

```
  L103A-IC          L103A-S           103               ORIFICE
  L104A-IC1         L104A-S           104               ORIFICE
  L104A-IC2         L104A-S           104               ORIFICE
  L106A-IC          L106A-S           106               ORIFICE
  W103A             L103A-S           MAIZE             WEIR
  W104A             L104A-S           L103A-S           WEIR
  W106A             L106A-S           L103A-S           WEIR

  *********************
  Cross Section Summary
  *********************
                                        Full     Full     Hyd.     Max.   No.
of     Full
  Conduit           Shape              Depth     Area     Rad.    Width
Barrels     Flow
  ---------------------------------------------------------------------------
------------
  102-101           CIRCULAR            0.38     0.11     0.09     0.38
1   130.01
  103-102           CIRCULAR            0.38     0.11     0.09     0.38
1   116.67
  104-103           CIRCULAR            0.30     0.07     0.07     0.30
1    67.64
  106-103           CIRCULAR            0.30     0.07     0.07     0.30
1    68.41

  ****************
  Transect Summary
  ****************

  Transect Access
  Area:
            0.0006     0.0023     0.0052     0.0093     0.0145
            0.0209     0.0284     0.0371     0.0470     0.0580
            0.0702     0.0835     0.0980     0.1137     0.1305
            0.1485     0.1676     0.1879     0.2086     0.2294
            0.2501     0.2708     0.2915     0.3122     0.3329
```

```
              0.3537       0.3744       0.3956       0.4176       0.4403
              0.4637       0.4878       0.5127       0.5383       0.5646
              0.5916       0.6194       0.6478       0.6763       0.7047
              0.7332       0.7617       0.7902       0.8187       0.8472
              0.8763       0.9062       0.9367       0.9680       1.0000
Hrad:
              0.0168       0.0336       0.0504       0.0672       0.0840
              0.1009       0.1177       0.1345       0.1513       0.1681
              0.1849       0.2017       0.2185       0.2353       0.2521
              0.2690       0.2858       0.3049       0.3381       0.3711
              0.4041       0.4370       0.4697       0.5024       0.5350
              0.5676       0.6000       0.6316       0.6616       0.6902
              0.7174       0.7433       0.7680       0.7916       0.8141
              0.8357       0.8563       0.8768       0.8981       0.9198
              0.9420       0.9644       0.9872       1.0102       1.0218
              1.0141       1.0082       1.0040       1.0013       1.0000
Width:
              0.0358       0.0717       0.1075       0.1434       0.1792
              0.2150       0.2509       0.2867       0.3226       0.3584
              0.3942       0.4301       0.4659       0.5018       0.5376
              0.5734       0.6093       0.6400       0.6400       0.6400
              0.6400       0.6400       0.6400       0.6400       0.6400
              0.6400       0.6448       0.6672       0.6896       0.7120
              0.7344       0.7568       0.7792       0.8016       0.8240
              0.8464       0.8688       0.8800       0.8800       0.8800
              0.8800       0.8800       0.8800       0.8800       0.8880
              0.9104       0.9328       0.9552       0.9776       1.0000


*********************************************************
NOTE: The summary statistics displayed in this report are
based on results found at every computational time step,
not just on results from each reporting time step.
*********************************************************
```

```
****************
Analysis Options
****************
Flow Units ............... LPS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ NO
  Water Quality .......... NO
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Surcharge Method ......... EXTRAN
Starting Date ............ 02/22/2021 00:00:00
Ending Date .............. 02/22/2021 08:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:01:00
Wet Time Step ............ 00:05:00
Dry Time Step ............ 00:05:00
Routing Time Step ........ 10.00 sec
Variable Time Step ....... YES
Maximum Trials ........... 8
Number of Threads ........ 1
Head Tolerance ........... 0.001500 m


**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Initial LID Storage ......         0.001         0.970
Total Precipitation ......         0.053        71.663
```

```
Evaporation Loss .........         0.000          0.000
Infiltration Loss ........         0.010         14.114
Surface Runoff ...........         0.043         58.050
Final Storage ............         0.001          0.970
Continuity Error (%) .....        -0.690


**************************        Volume         Volume
Flow Routing Continuity        hectare-m       10^6 ltr
**************************     ---------      ---------
Dry Weather Inflow .......         0.000          0.000
Wet Weather Inflow .......         0.043          0.428
Groundwater Inflow .......         0.000          0.000
RDII Inflow ..............         0.000          0.000
External Inflow ..........         0.000          0.001
External Outflow .........         0.043          0.430
Flooding Loss ............         0.000          0.000
Evaporation Loss .........         0.000          0.000
Exfiltration Loss ........         0.000          0.000
Initial Stored Volume ....         0.001          0.010
Final Stored Volume ......         0.001          0.010
Continuity Error (%) .....        -0.165


***************************
Time-Step Critical Elements
***************************
Link 103-102 (54.40%)


********************************
Highest Flow Instability Indexes
********************************
All links are stable.
```

```
*************************
Routing Time Step Summary
*************************
Minimum Time Step           :     0.50 sec
Average Time Step           :     6.49 sec
Maximum Time Step           :    10.00 sec
Percent in Steady State     :    -0.00
Average Iterations per Step :     2.02
Percent Not Converging      :     0.14
Time Step Frequencies       :
   10.000 -  5.493 sec      :    45.56 %
    5.493 -  3.017 sec      :    46.19 %
    3.017 -  1.657 sec      :     1.35 %
    1.657 -  0.910 sec      :     6.15 %
    0.910 -  0.500 sec      :     0.74 %


***************************
Subcatchment Runoff Summary
***************************
```

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Imperv Runoff mm	Perv Runoff mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff LPS	Runoff Coeff
L103A	71.66	0.00	0.00	3.95	63.58	4.37	67.95	0.05	37.77	0.948
L104A	71.66	0.00	0.00	3.95	63.59	4.38	67.98	0.11	76.53	0.949

```
  L106A                      71.66       0.00       0.00       6.96       57.47
7.69       65.15        0.16   118.85   0.909

  UNC-1                      71.66       0.00       0.00      32.37        7.17
40.06       40.06        0.04    50.21   0.559

  UNC-2                      71.66       0.00       0.00      28.05       20.48
44.38       44.38        0.07    71.12   0.619



  ******************

  Node Depth Summary

  ******************


  ----------------------------------------------------------------------------
------

                              Average  Maximum  Maximum  Time of Max
Reported

                                Depth    Depth      HGL   Occurrence    Max
Depth

  Node                 Type    Meters   Meters   Meters  days hr:min
Meters

  ----------------------------------------------------------------------------
------

  101                  OUTFALL     0.54     0.54    95.64     0  00:00
0.54

  MAIZE                OUTFALL     0.00     0.00    97.90     0  00:00
0.00

  OF-1                 OUTFALL     0.00     0.00     0.00     0  00:00
0.00

  OF-2                 OUTFALL     0.00     0.00     0.00     0  00:00
0.00

  102                  STORAGE     0.74     0.81    95.71     0  01:03
0.81

  103                  STORAGE     0.66     0.79    95.78     0  01:03
0.79

  104                  STORAGE     0.39     0.56    95.82     0  01:03
0.56

  106                  STORAGE     0.35     0.66    95.99     0  01:03
0.66

  L103A-S              STORAGE     0.11     1.67    98.07     0  01:01
1.67

  L104A-S              STORAGE     0.13     1.60    98.04     0  01:04
1.60
```

L106A-S	STORAGE	0.12	1.54	98.08	0	01:01	1.54

****

Node Inflow Summary

****

Node	Type	Maximum Lateral Inflow LPS	Maximum Total Inflow LPS	Time of Max Occurrence days	Time of Max Occurrence hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
101	OUTFALL	0.00	107.34	0	01:04	0	0.307	0.000
MAIZE	OUTFALL	0.00	42.47	0	01:01	0	0.0131	0.000
OF-1	OUTFALL	50.21	50.21	0	01:00	0.0436	0.0436	0.000
OF-2	OUTFALL	71.12	71.12	0	01:00	0.0677	0.0677	0.000
102	STORAGE	0.00	107.34	0	01:03	0	0.308	0.000
103	STORAGE	0.00	107.34	0	01:03	0	0.307	-0.020
104	STORAGE	0.00	34.22	0	01:04	0	0.108	0.005
106	STORAGE	0.00	55.69	0	01:01	0	0.15	-0.028
L103A-S	STORAGE	37.77	65.91	0	01:01	0.0523	0.0619	-0.013
L104A-S	STORAGE	76.53	76.53	0	01:00	0.106	0.107	-0.315
L106A-S	STORAGE	118.85	118.85	0	01:00	0.159	0.159	-0.199

```
  **********************
  Node Surcharge Summary
  **********************

  No nodes were surcharged.


  *********************
  Node Flooding Summary
  *********************

  No nodes were flooded.


  **********************
  Storage Volume Summary
  **********************
```

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow LPS
102	0.001	24	0	0	0.001	26	0 01:03	107.34
103	0.001	21	0	0	0.001	25	0 01:03	107.34
104	0.000	13	0	0	0.001	19	0 01:03	37.75
106	0.000	12	0	0	0.001	23	0 01:03	55.65

```
  L103A-S                  0.000      1     0     0         0.007      28
0  01:01      65.73
  L104A-S                  0.001      1     0     0         0.024      35
0  01:04      34.22
  L106A-S                  0.001      1     0     0         0.027      28
0  01:01      91.95


  ***********************
  Outfall Loading Summary
  ***********************

  -----------------------------------------------------------
                        Flow       Avg       Max       Total
                        Freq      Flow      Flow      Volume
  Outfall Node          Pcnt       LPS       LPS    10^6 ltr
  -----------------------------------------------------------
  101                  96.22     13.52    107.34       0.307
  MAIZE                 1.24     23.85     42.47       0.013
  OF-1                 21.33      5.25     50.21       0.044
  OF-2                 26.70      7.46     71.12       0.068
  -----------------------------------------------------------
  System               36.37     50.08    259.16       0.431


  ********************
  Link Flow Summary
  ********************

  ---------------------------------------------------------------------------
--
                                 Maximum  Time of Max   Maximum    Max/
Max/
                                  |Flow|   Occurrence   |Veloc|    Full
Full
  Link                 Type          LPS  days hr:min     m/sec    Flow
Depth
```

```
  ---------------------------------------------------------------------------
--
  102-101              CONDUIT    107.34     0  01:04      0.97    0.83
1.00
  103-102              CONDUIT    107.34     0  01:03      0.97    0.92
1.00
  104-103              CONDUIT     37.75     0  01:20      0.64    0.56
0.93
  106-103              CONDUIT     55.65     0  01:02      0.79    0.81
1.00
  L103A-IC             ORIFICE     17.52     0  01:01
1.00
  L104A-IC1            ORIFICE     17.11     0  01:04
1.00
  L104A-IC2            ORIFICE     17.11     0  01:04
1.00
  L106A-IC             ORIFICE     55.69     0  01:01
1.00
  W103A                WEIR        42.47     0  01:01
0.16
  W104A                WEIR         5.74     0  01:01
0.02
  W106A                WEIR        36.26     0  01:01
0.15


  ***************************
  Flow Classification Summary
  ***************************


  ---------------------------------------------------------------------------
----------
                      Adjusted    ---------- Fraction of Time in Flow Class -
---------
                       /Actual         Up    Down  Sub   Sup   Up    Down
Norm  Inlet
  Conduit               Length    Dry  Dry   Dry   Crit  Crit  Crit  Crit
Ltd   Ctrl
  ---------------------------------------------------------------------------
----------
  102-101                 1.00   0.00  0.00  0.00  1.00  0.00  0.00  0.00
0.00  0.00
```

```
  103-102                 1.00   0.00  0.00  0.00  1.00  0.00  0.00  0.00
0.00  0.00
  104-103                 1.00   0.00  0.00  0.00  1.00  0.00  0.00  0.00
0.01  0.00
  106-103                 1.00   0.00  0.00  0.00  1.00  0.00  0.00  0.00
0.34  0.00


  *************************
  Conduit Surcharge Summary
  *************************


  ---------------------------------------------------------------------------
-
                                                          Hours        Hours
                         --------- Hours Full --------   Above Full
Capacity
  Conduit                Both Ends  Upstream  Dnstream   Normal Flow
Limited
  ---------------------------------------------------------------------------
-
  102-101                     8.00       8.00      8.00        0.01         0.01
  103-102                     0.59       0.59      8.00        0.01         0.01
  104-103                     0.01       0.01      0.57        0.01         0.01
  106-103                     0.42       0.42      0.57        0.01         0.01


  Analysis begun on:  Fri Mar 19 10:56:14 2021
  Analysis ended on:  Fri Mar 19 10:56:14 2021
  Total elapsed time: < 1 sec
```

C.6 SWM EXCERPTS FROM KANATA WEST DEVELOPMENT FIRST REGISTRATION PHASE REPORT (STANTEC, FEBRUARY 2020)

3.0 STORM DRAINAGE

The following sections describe the stormwater management (SWM) design for the Richcraft Kanata West Development in the context of the background documents and governing criteria.

3.1 PROPOSED CONDITIONS

The initial 15.9 ha phase of the overall 36.5 ha development comprises a mixture of townhomes and back to back units, a park, a commercial block, and a medium density residential block. The commercial block and medium density residential private block will be developed as separate site plan applications.

The Kanata West Pond 5 is to be constructed in two stages, interim and ultimate conditions. In the interim development condition, it is proposed to construct an interim KW SWM Pond 5 to provide quality and quantity control for runoff from the proposed first registration phase of the Richcraft Kanata West Development, as well as runoff from the Kanata West Pumping Station (KWPS) for a total interim drainage area to the interim KW SWM Pond 5 of 15.5 ha at 61% imperviousness as shown on **Figure 1.3**. The interim pond will eventually become part of the ultimate KW SWM Pond 5 in the future. The KW Pond 5 will be designed to provide 'Normal' level of quality control of stormwater runoff which corresponds to 70% TSS removal prior to discharging into the Carp River. The *Interim Kanata West Pond 5 (with Ultimate Carp River) Design Brief* (Stantec, April 2019) has been submitted under separate cover and should be read in conjunction with this report.

Site storm sewers for the proposed interim phase will be directed to the interim Kanata West Pond 5 through the west forebay, while a future trunk sewer along Cartage Way and Winter Wheat Terrace will direct runoff from the future phase of the Richcraft Kanata West Development to the east forebay of the ultimate Kanata West Pond 5 which will also service the existing Mattamy and Trinity developments, as well as an existing section of Hazeldean Road once their respective interim SWM facilities are decommissioned. In addition, the ultimate condition KW Pond 5 and future site trunk sewers will service the future transitway/arterial road, and a future commercial block as shown on **Figure 1.4**.

The ultimate development condition with the ultimate KW Pond 5 configuration and the inclusion of runoff from external areas results in the worst-case scenario for the proposed first registration phase of the Richcraft Kanata West Development and as such, the results of the SWM analyses presented in this report correspond to the ultimate development conditions.

The overall approach for storm servicing and stormwater management for the Richcraft Kanata West Development was initially outlined in the Kanata West Master Servicing Study (KWMSS) prepared by Stantec in 2006. In accordance with this document, inlet control devices have been used at road low points to restrict inflow rates to the storm sewers and to provide attenuating surface storage. The major overland system comprising of swales, roadway sags, streets, etc. has been designed to handle peak flows beyond the storm sewer capacity up to the 100-year storm and to be directed to the KW Pond 5.

3.2 STORMWATER BACKGROUND AND DESIGN CRITERIA

3.2.1 Kanata West Master Servicing Study SWM Criteria

The KWMSS established the SWM criteria for the Kanata West community, the SWM pond locations and their respective outlet locations. The following summarizes the design requirements outlined in the KWMSS for the future developments as shown in the report excerpts included in **Appendix D.2**.

- Width parameter: Twice the street length to be used for arterial roadways, and 225 m/ha to be used for residential and mixed-use developments where no street layout is available.
- Major system storage: major system storage of 40 m^3/ha and 50 m^3/ha was assumed for arterial roads and residential/non-residential lands respectively.
- Inlet control devices to be used to restrict runoff to 85 L/s/ha on average for residential and non-residential lands.
- The capture rates for arterial roadways to be equal to the10-year storm design.
- Sizing of local storm sewers to use inlet times of 15 minutes for typical split-lot drainage in residential developments.
- All sewer hydraulic analysis is to use a static boundary condition equal to the MVC 1983 flood limit of 94.60 m.
- Pond 5 is proposed upstream of the confluence of Poole Creek and the Carp River along the south-west bank of the Carp River. Pond 5 will service residential and retail developments between Poole Creek, the Carp River and Hazeldean Road. Normal and 100-year water levels in the pond are 93.44 m and 94.94 m, respectively.
- The Carp River Watershed/Subwatershed Study (Robinson Consultants, November 2004) proposed target infiltration rates of 104 mm/year and 73 mm/year for areas of moderate and low recharge respectively within the KW Community. Post development infiltration rates are to be increased by 25% above the pre-development rate. This rate of infiltration was established to compensate for those areas (i.e. Roadway corridors) that can not provide infiltration. The Richcraft Kanata West subdivision is identified as having an estimated infiltration rate between 50-70 mm/yr.

3.2.2 Proposed Deviations from the Kanata West Master Servicing Study

The proposed SWM design for the Richcraft Kanata West Development contains deviations from the KWMSS. Firstly, an interim pond has been introduced to support the development of the first registration phase of the Richcraft Development which will be serviced through the proposed west forebay trunk sewer, prior to constructing a second forebay to the east and corresponding system segments in the ultimate development condition. The use of PCSWMM for modeling of the site hydrology and hydraulics allowed for an analysis of the systems' response during various storm events. The following assumptions were applied to the detailed model:

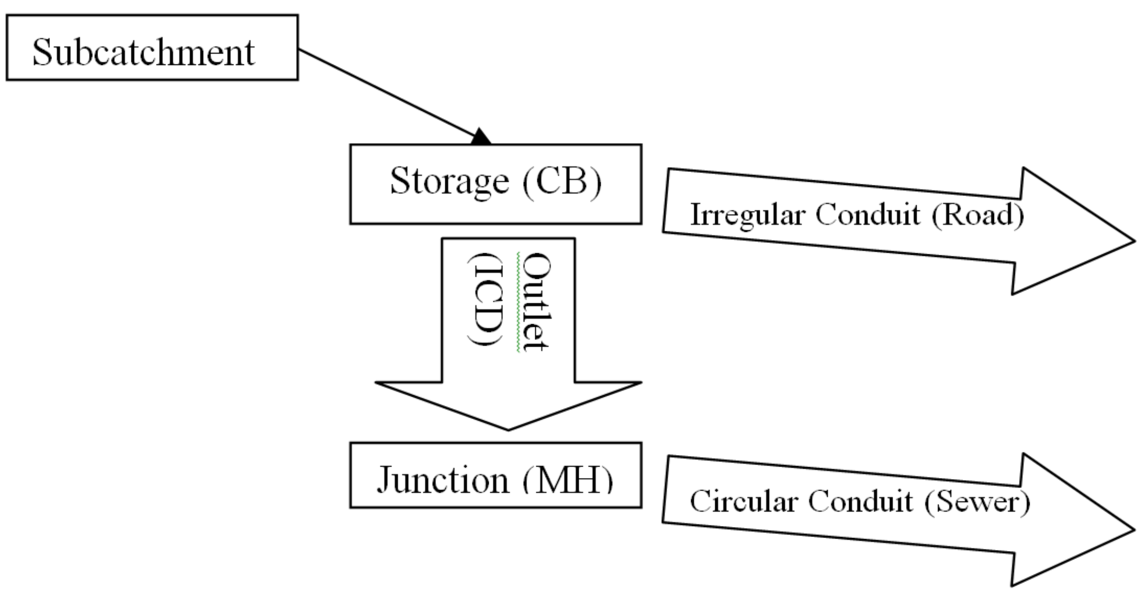
- Hydrologic parameters as per Ottawa Sewer Design Guidelines, including Horton infiltration, Manning's 'n', and depression storage values.
- 3-hour Chicago Storm distribution for the 2-year, 5-year, and 10-year to determine inlet capture rates for the different catchments.
- Minor and major system response assessed for the 100-year using the 3-hour Chicago Storm Distribution and the 12-hour SCS Type II distribution with a fixed water level in the Carp River of 94.60 m.
- To 'stress test' the system a 'climate change' scenario was created by adding 20% of the individual intensity values of the 100-year storms at their specified time step.
- Assess the minor and major systems response during the July 1, 1979 historical with a fixed water level in the Carp River of 94.60 m.
- Percent imperviousness calculated based on actual soft and hard surfaces for representative catchments and converted to equivalent runoff coefficient using the relationship C = (Imp. x 0.7) + 0.2.
- Runoff coefficients for future medium density residential and school blocks of 0.70, and 0.85 for commercial blocks.
- Subcatchment areas are defined from high-point to high-point where sags occur, and detailed grading is available.
- Width parameter was taken as twice the length of the street/swale segment for two-sided catchments and as the length of the street/swale segment for one-sided catchments.
- Where detailed grading was not available, subcatchment areas were defined by the limits of the future development blocks and the width of the subcatchment was defined as 225 m/ha as per the City of Ottawa Sewer Design Guidelines.
- Catchbasin inflow restricted with inlet-control devices (ICDs) as per City guidelines.
- Surface ponding in sag storage calculated based on grading plans (**Drawings PD-1** - **PD-6**).
- Different segment cross-section types defined, accounting for varying right-of-way widths with 3% cross slope, swales, and spillways.
- Future school block (area L218A) to restrict minor system peak flows up to the 100-year storm to 443 L/s and to restrict 100-year overflows to Shropshire Place to 565 L/s.

- Future medium density residential blocks, areas L123B and L225B, to limit minor system peak flows up to the 100-year storm to 111 L/s and to 389 L/s and to restrict 100-year overflows to Maize Street and Holstein Road to 480 L/s and 799 L/s respectively.
- Future commercial block, area L121B, to restrict minor system peak flows up to the 100-year storm to 299 L/s and to provide on-site storage for 100-year overflows.
- Inlet control devices (ICDs) to have a minimum orifice diameter of 83 mm.

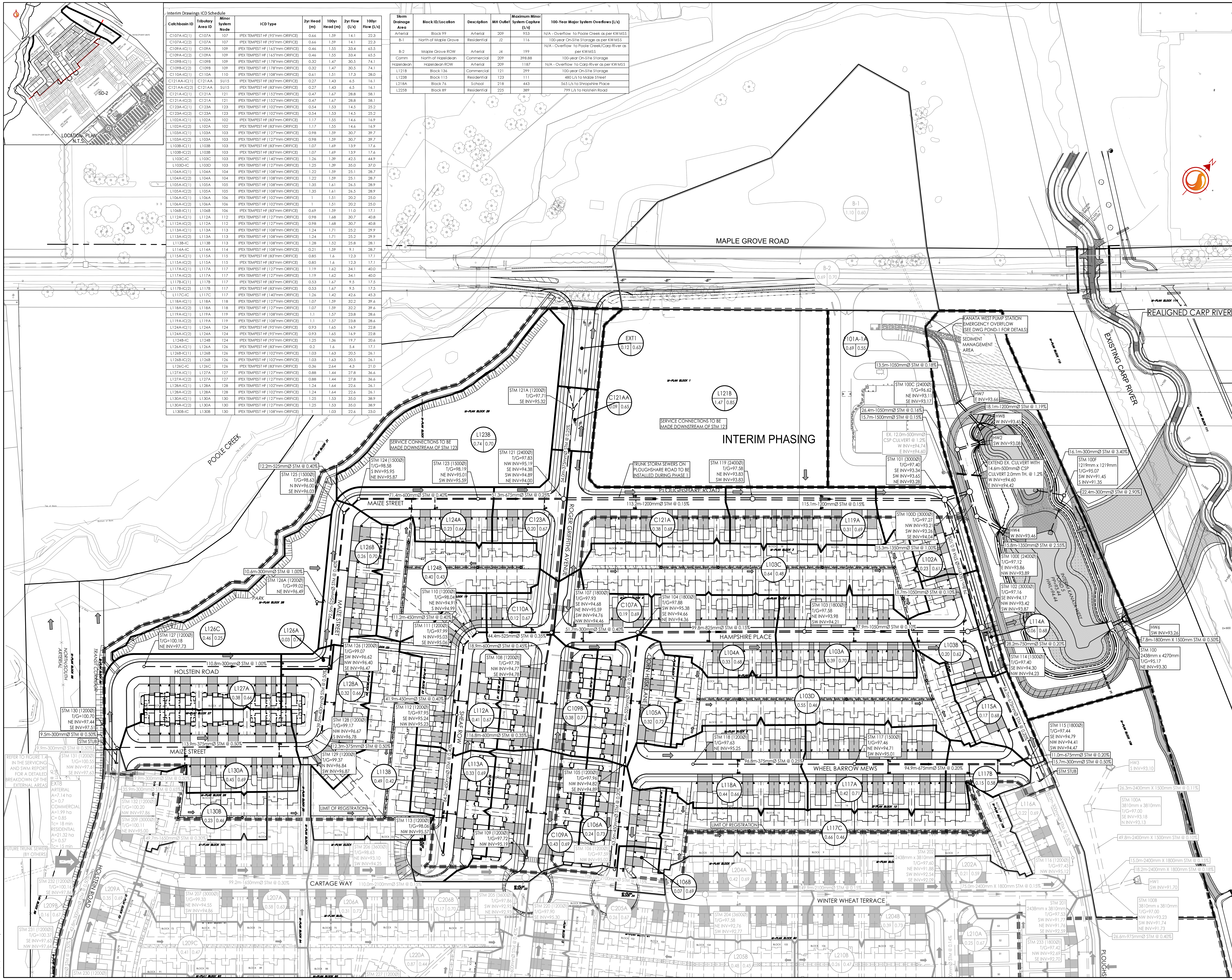
3.4.1 SWMM Dual Drainage Methodology

The proposed development is modeled in one modeling program as a dual conduit system (see **Figure 3.1**), with: 1) circular conduits representing the sewers & storage nodes representing manholes; 2) irregular conduits using street-shaped cross-sections to represent the saw-toothed overland road network from high-point to low-point and storage nodes representing catchbasins and high points. The dual drainage systems are connected via orifice link objects from storage node (i.e. CB) to junction (i.e. MH), and represent inlet control devices (ICDs). Subcatchments are linked to the storage node on the surface so that generated hydrographs are directed there firstly.

Figure 3.1: Schematic Representing Model Object Roles



Storage nodes are used in the model to represent catchbasins as well as major system junctions. For storage nodes representing catchbasins (CBs), the invert of the storage node represents the invert of the CB and the rim of the storage node represents the maximum allowable flow depth elevation above the storage node (equal to the top of the CB plus an additional 0.35 m or



Interim Drawings ICD Schedule

Catchbasin ID	Tributary Area ID	Minor System Node	ICD Type	2yr Head (m)	100yr Head (m)	2yr Flow (L/s)	100yr Flow (L/s)
C107A-IC(1)	C107A	107	IPEX TEMPEST HF (95"mm ORIFICE)	0.66	1.59	14.1	22.3
C107A-IC(2)	C107A	107	IPEX TEMPEST HF (95"mm ORIFICE)	0.66	1.59	14.1	22.3
C109A-IC(1)	C109A	109	IPEX TEMPEST HF (165"mm ORIFICE)	0.46	1.55	33.4	65.5
C109A-IC(2)	C109A	109	IPEX TEMPEST HF (165"mm ORIFICE)	0.46	1.55	33.4	65.5
C109B-IC(1)	C109B	109	IPEX TEMPEST HF (178"mm ORIFICE)	0.32	1.47	30.5	74.1
C109B-IC(2)	C109B	109	IPEX TEMPEST HF (178"mm ORIFICE)	0.32	1.47	30.5	74.1
C110A-IC(1)	C110A	110	IPEX TEMPEST HF (108"mm ORIFICE)	0.61	1.51	17.3	28.0
C121AA-IC(1)	C121AA	SU15	IPEX TEMPEST HF (83"mm ORIFICE)	0.27	1.43	6.5	16.1
C121AA-IC(2)	C121AA	SU15	IPEX TEMPEST HF (83"mm ORIFICE)	0.27	1.43	6.5	16.1
C121A-IC(1)	C121A	121	IPEX TEMPEST HF (152"mm ORIFICE)	0.47	1.67	28.8	58.1
C121A-IC(2)	C121A	121	IPEX TEMPEST HF (152"mm ORIFICE)	0.47	1.67	28.8	58.1
C123A-IC(1)	C123A	123	IPEX TEMPEST HF (102"mm ORIFICE)	0.54	1.53	14.5	25.2
C123A-IC(2)	C123A	123	IPEX TEMPEST HF (102"mm ORIFICE)	0.54	1.53	14.5	25.2
L102A-IC(1)	L102A	102	IPEX TEMPEST HF (83"mm ORIFICE)	1.17	1.55	14.6	16.9
L102A-IC(2)	L102A	102	IPEX TEMPEST HF (83"mm ORIFICE)	1.17	1.55	14.6	16.9
L103A-IC(1)	L103A	103	IPEX TEMPEST HF (127"mm ORIFICE)	0.98	1.59	30.7	39.7
L103A-IC(2)	L103A	103	IPEX TEMPEST HF (127"mm ORIFICE)	0.98	1.59	30.7	39.7
L103B-IC(1)	L103B	103	IPEX TEMPEST HF (83"mm ORIFICE)	1.07	1.69	13.9	17.6
L103B-IC(2)	L103B	103	IPEX TEMPEST HF (83"mm ORIFICE)	1.07	1.69	13.9	17.6
L103C-IC	L103C	103	IPEX TEMPEST HF (140"mm ORIFICE)	1.26	1.39	42.5	44.9
L103D-IC	L103D	103	IPEX TEMPEST HF (127"mm ORIFICE)	1.25	1.39	35.0	37.0
L104A-IC(1)	L104A	104	IPEX TEMPEST HF (108"mm ORIFICE)	1.22	1.59	25.1	28.7
L104A-IC(2)	L104A	104	IPEX TEMPEST HF (108"mm ORIFICE)	1.22	1.59	25.1	28.7
L105A-IC(1)	L105A	105	IPEX TEMPEST HF (108"mm ORIFICE)	1.35	1.61	26.5	28.9
L105A-IC(2)	L105A	105	IPEX TEMPEST HF (108"mm ORIFICE)	1.35	1.61	26.5	28.9
L106A-IC(1)	L106A	106	IPEX TEMPEST HF (102"mm ORIFICE)	1	1.51	20.2	25.0
L106A-IC(2)	L106A	106	IPEX TEMPEST HF (102"mm ORIFICE)	1	1.51	20.2	25.0
L106B-IC(1)	L106B	106	IPEX TEMPEST HF (83"mm ORIFICE)	0.69	1.59	11.0	17.1
L112A-IC(1)	L112A	112	IPEX TEMPEST HF (127"mm ORIFICE)	0.98	1.68	30.7	40.8
L112A-IC(2)	L112A	112	IPEX TEMPEST HF (127"mm ORIFICE)	0.98	1.68	30.7	40.8
L113A-IC(1)	L113A	113	IPEX TEMPEST HF (108"mm ORIFICE)	1.24	1.71	25.2	29.9
L113A-IC(2)	L113A	113	IPEX TEMPEST HF (108"mm ORIFICE)	1.24	1.71	25.2	29.9
L113B-IC	L113B	113	IPEX TEMPEST HF (108"mm ORIFICE)	1.28	1.52	25.8	28.1
L114A-IC	L114A	114	IPEX TEMPEST HF (108"mm ORIFICE)	0.21	1.59	9.1	28.7
L115A-IC(1)	L115A	115	IPEX TEMPEST HF (83"mm ORIFICE)	0.85	1.6	12.3	17.1
L115A-IC(2)	L115A	115	IPEX TEMPEST HF (83"mm ORIFICE)	0.85	1.6	12.3	17.1
L117A-IC(1)	L117A	117	IPEX TEMPEST HF (127"mm ORIFICE)	1.19	1.62	34.1	40.0
L117A-IC(2)	L117A	117	IPEX TEMPEST HF (127"mm ORIFICE)	1.19	1.62	34.1	40.0
L117B-IC(1)	L117B	117	IPEX TEMPEST HF (83"mm ORIFICE)	0.53	1.67	9.5	17.5
L117B-IC(2)	L117B	117	IPEX TEMPEST HF (83"mm ORIFICE)	0.53	1.67	9.5	17.5
L117C-IC	L117C	117	IPEX TEMPEST HF (140"mm ORIFICE)	1.26	1.42	42.6	45.3
L118A-IC(1)	L118A	118	IPEX TEMPEST HF (127"mm ORIFICE)	1.07	1.59	32.2	39.6
L118A-IC(2)	L118A	118	IPEX TEMPEST HF (127"mm ORIFICE)	1.07	1.59	32.2	39.6
L119A-IC(1)	L119A	119	IPEX TEMPEST HF (108"mm ORIFICE)	1.1	1.57	23.8	28.6
L119A-IC(2)	L119A	119	IPEX TEMPEST HF (108"mm ORIFICE)	1.1	1.57	23.8	28.6
L124A-IC(1)	L124A	124	IPEX TEMPEST HF (95"mm ORIFICE)	0.93	1.65	16.9	22.8
L124A-IC(2)	L124A	124	IPEX TEMPEST HF (95"mm ORIFICE)	0.93	1.65	16.9	22.8
L124B-IC	L124B	124	IPEX TEMPEST HF (95"mm ORIFICE)	1.25	1.36	19.7	20.6
L126A-IC(1)	L126A	126	IPEX TEMPEST HF (83"mm ORIFICE)	0.2	1.6	5.4	17.1
L126B-IC(1)	L126B	126	IPEX TEMPEST HF (102"mm ORIFICE)	1.03	1.63	20.5	26.1
L126B-IC(2)	L126B	126	IPEX TEMPEST HF (102"mm ORIFICE)	1.03	1.63	20.5	26.1
L126C-IC	L126C	126	IPEX TEMPEST HF (83"mm ORIFICE)	0.36	2.64	4.3	21.0
L127A-IC(1)	L127A	127	IPEX TEMPEST HF (127"mm ORIFICE)	0.88	1.44	27.8	36.6
L127A-IC(2)	L127A	127	IPEX TEMPEST HF (127"mm ORIFICE)	0.88	1.44	27.8	36.6
L128A-IC(1)	L128A	128	IPEX TEMPEST HF (102"mm ORIFICE)	1.24	1.64	22.6	26.1
L128A-IC(2)	L128A	128	IPEX TEMPEST HF (102"mm ORIFICE)	1.24	1.64	22.6	26.1
L130A-IC(1)	L130A	130	IPEX TEMPEST HF (127"mm ORIFICE)	1.25	1.53	35.0	38.9
L130A-IC(2)	L130A	130	IPEX TEMPEST HF (127"mm ORIFICE)	1.25	1.53	35.0	38.9
L130B-IC	L130B	130	IPEX TEMPEST HF (108"mm ORIFICE)	1	1.03	22.6	23.0

Storm Drainage Area	Block ID/Location	Description	MH Outlet	Maximum Minor System Capture (L/s)	100-Year Major System Overflows (L/s)
Arterial	Block 99	Arterial	209	953	N/A - Overflow to Poole Creek as per KWMSS
B-1	North of Maple Grove	Residential	J2	116	100-year On-Site Storage as per KWMSS
B-2	Maple Grove ROW	Arterial	J4	199	N/A - Overflow to Poole Creek/Carp River as per KWMSS
Comm	North of Hazeldean	Commercial	209	398.88	100-year On-Site Storage
Hazeldean	Hazeldean ROW	Arterial	209	1187	N/A - Overflow to Carp River as per KWMSS
L121B	Block 136	Commercial	121	299	100-year On-Site Storage
L123B	Block 115	Residential	123	111	480 L/s to Maize Street
L218A	Block 76	School	218	443	565 L/s to Shropshire Place
L225B	Block 89	Residential	225	389	799 L/s to Holstein Road

Stantec

Stantec Consulting Ltd.
400 - 1331 Clyde Avenue
Ottawa ON
Tel. 613.722.4420
www.stantec.com

Legend

- PROPOSED STORM SEWER
- PROPOSED DRAINAGE BOUNDARY
- PROPOSED CATCHBASIN
- ST101A / 0.38 / 0.70: STORM DRAINAGE AREA ID#; C-COEFFICIENT VALUE; DRAINAGE AREA (HA)

DRAINAGE AREA PREFIX
- L — LOCAL ROAD, 2-YEAR DESIGN EVENT
- C — COLLECTOR ROAD, 5-YEAR DESIGN EVENT
- F — FULL CAPTURE AREA, 100-YEAR DESIGN EVENT
- EXT — EXTERNAL DRAINAGE AREA
- DIRECTION OF OVERLAND FLOW
- EXTERNAL DRAINAGE
- CIRCULAR ORIFICE (SEE DWG SD-1)
- SERVICE LATERAL LOCATION
- PROPOSED BIOSWALE (SEE NOTE 5)

NOTES:
1. PROPOSED IPEX TEMPEST HF ICDS ARE TO HAVE CIRCULAR ORIFICES AS SPECIFIED AND ARE REQUIRED TO BE VERTICAL SLIDING TYPE WITH FLOATABLE TRAP.

		By	Appd.	YY.MM.DD
9	ISSUED FOR ECA APPROVAL	WAJ	SGG	20.02.12
8	REVISED AS PER CITY COMMENTS	WAJ	SGG	20.01.31
7	ISSUED FOR TENDER	WAJ	KJH	20.01.09
6	REVISED AS PER CITY COMMENTS	WAJ	SGG	19.11.18
5	REVISED CROSSING DESIGN	WAJ	SGG	19.09.23
4	REVISED AS PER CITY COMMENTS	WAJ	SGG	19.08.30
3	REVISED AS PER CITY COMMENTS	WAJ	SGG	19.05.10
2	REVISED AS PER CITY COMMENTS	WAJ	SGG	18.12.10
1	ISSUED FOR FIRST SUBMISSION	WAJ	SGG	18.03.02

Revision

File Name: 1060401393-DB	Dwn.	Chkd.	Dsgn.	YY.MM.DD
	WAJ	SGG	WAJ	17.12.20

Permit-Seal

Client/Project

RICHCRAFT HOMES
1st REGISTRATION

RICHCRAFT KANATA WEST
1620 MAPLE GROVE ROAD
OTTAWA, ON

Title

ULTIMATE CONDITION STORM DRAINAGE AREA PLAN

Project No. 160401393

Scale 1:1000 (0 10 30 50m)

Drawing No. SD-1

Sheet 40 of 44

Revision 9

CITY FILE No. D07-16-04-0017 CITY PLAN No. 17529

ORIGINAL SHEET - ARCH D

C.7 EXCERPTS FROM INTERIM KANATA WEST POND 5 (WITH ULTIMATE CARP RIVER) DESIGN BRIEF (STANTEC, JULY 2019)

1.0 INTRODUCTION AND BACKGROUND

1.1 OVERVIEW

This revised Interim Kanata West Pond 5 (with ultimate Carp River) design brief has been prepared to address City comments to the previous submission dated April 1, 2019. A letter summarizing the City comments and Stantec's responses has been included in **Appendix I**. Specifically, the pond calculations have been revised to show that sufficient storage can be provided in the proposed interim pond to provide 'Enhanced' level of quality treatment and as such the Environmental Compliance Approval (ECA) application has been revised to go through the Transfer of Review program. In addition, the ultimate condition overflow weir has been revised to include a concrete cut-off wall, and the block number references in this report and on the drawings have been revised to reflect the latest 4M-Draft Plan in support of registration of Phase 1. The integrity of the design remains the same as that previously submitted.

Stantec Consulting Ltd. has been retained to complete the design of the interim Kanata West stormwater management Pond 5 (KW SWM Pond 5). The interim facility will be designed as an off-line SWM facility to provide end-of-pipe treatment for the initial phases of the Richcraft Kanata West development located between the Carp River and the future transitway and bounded at the north by Poole Creek and at the south by future Cartage Way/Winter Wheat Terrace as shown in **Figure 1** and **Drawing PH-1**. The 14.8 ha initial development area consists of medium density residential areas, a private commercial block (Block 1), a park block (Block 28), a private medium density residential block (Block 29), and a stormwater management (SWM) block (Block 11). The location of the interim KW SWM Pond 5 is consistent with the information presented in the June 16, 2006 Kanata West Master Servicing Study (KWMSS) by Stantec Consulting Ltd. and IBI Group. Refer to **Drawing OSDI-1** for details.

The Kanata West Pond 5 is to be constructed in two stages, interim and ultimate conditions (see **Drawing PH-1** and **Drawing POND-5**). In the interim development condition, it is proposed to construct an interim KW SWM Pond 5 (Block 11) to provide quality and quantity control for runoff from the initial phases of the Richcraft Kanata West Development, as well as runoff from the Kanata West Pumping Station (KWPS) for a total interim drainage area to the interim KW SWM Pond 5 of 15.5 ha at 61% imperviousness. The interim pond will eventually become part of the ultimate KW SWM Pond 5 (Block 144) in the future. **Figure 1** below indicates the general tributary area and the location of the Interim KW SWM Pond 5 while **Figure 2** shows the overall drainage areas to the ultimate KW SWM Pond 5.

2.0 PROPOSED DEVELOPMENT STORMWATER MANAGEMENT PLAN

The following sections summarize the stormwater management (SWM) plan for the initial phases of the Richcraft KW Development tributary to the proposed interim KW SWM Pond 5.

2.1 PROPOSED DEVELOPMENT CONDITIONS

The interim SWM facility will receive runoff from 15.5 ha of land from the initial phases of the Richcraft KW Development, the interim SWM pond footprint area, and the Kanata West pump station. The interim facility will provide quantity and quality control (80% TSS removal) of runoff before discharging to the Carp River.

Detailed design of the interim SWM pond has been done using a detailed hydrologic/hydraulic model of the proposed development phases to determine the inflow rates to the pond. Subdivision and sewer design modelling used the "dual drainage" principle, whereby the minor (pipe) system is designed to convey the peak rate of runoff based on a 2-year return period storm capture rate for local streets and a 5-year return period storm capture rate for collector roads as per the City design criteria. Runoff from larger events will be conveyed via engineered channels (roadways, pathways and swales) to a proposed spillway that will discharge into the Interim SWM Pond 5. Outlet links and orifices have been specified in the model to limit the inlet capture rate to the minor system and thus control the hydraulic grade line during major storms.

Drawing OSDI-1 shows the overall major and minor flow paths as well as the proposed interim SWM Pond layout.

2.2 FACILITY DESIGN CRITERIA

2.2.1 Water Quality Control

The Interim KW SWM Pond 5 achieves 'enhanced' level of treatment of urban runoff according to Ministry of the Environment, Conservation and Parks(MECP) criteria – representing a 80% removal of total suspended solids (TSS). However, 'Normal' level of treatment which corresponds to 70% removal of total suspended solids (TSS) was established as the treatment criteria based on the Carp River Watershed/Subwatershed Study (Robinson Consultants, December 2004).

The facility, as outlined in **Section 3.0**, has been designed with sufficient permanent pool and extended detention storage to provide 'Enhanced' level of quality control. The end-of-pipe facility has been designed according to the recommendations of the Ministry of the Environment Stormwater Management Planning and Design Manual, as provided in **Section 3.4**, and therefore no quality control infrastructure is required within the proposed development tributary to the pond.

2.2.2 Water Quantity Control

The proposed interim SWM Pond will service the initial phases of the Richcraft development and discharge into the fully-restored Carp River. The Richcraft lands to the south, future commercial block between Hazeldean Road and the existing Trinity development, as well as the future transitway and arterial road to the south will remain undeveloped in the interim condition (see **Figure 2**). Additionally, the interim Fairwinds and Trinity Ponds will remain in place under the proposed condition.

As specified in the PCSWMM Model Documentation and as outlined in correspondence obtained from the City (see correspondence in **Appendix J**), all applications within the Kanata West Development are required to demonstrate that the 2 to 100-year discharges from the proposed development are consistent with the target hydrographs from the Carp River ultimate condition model. If any departures from the assumptions supporting the ultimate condition model are proposed, and/or the detailed design response does not match the target hydrographs, the proponent will be required to update the model with a simplified/lumped version of the site's detailed dual drainage model and to provide the following information:

- a comparison of the site's detailed design output hydrographs with the future condition model target hydrographs;
- an updated simplified/lumped version of the site's detailed dual drainage/SWM pond modeling that provides the same response as the proposed outlets;
- 100-year peak flow and water level summaries at key locations along the Carp River.

The proposed interim conditions are considered a departure from the assumptions supporting the future condition model, hence the City required to create a Carp River full restoration model that represents the proposed interim development condition as described above.

Similarly, in the ultimate condition, the development on 2731 Hazeldean Road (identified as the Welling's Development) which was originally planned to be serviced through the ultimate KW Pond 5, will discharge directly into Poole Creek (see **Figure 2**). This is also considered a departure from the assumptions supporting the City's Carp River Ultimate Condition model and as such, the City recommended the model be revised as part of this submission to reflect the revised outlet for the 2731 Hazeldean Road site.

2.2.2.1 Carp River Full Restoration/Interim Development Model

The new Carp River full restoration interim development model was created as follows. A detailed breakdown of the Carp River model changes made is provided in **Appendix C.**

Appendix D - GEOTECHNICAL INFORMATION

D.1 GEOTECHNICAL INVESTIGATION REPORT (PATERSON GROUP, JULY 14, 2020)

Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

patersongroup

Geotechnical Investigation

Proposed Residential Development
Kanata West - Block 29 - Ottawa

Prepared For

Richcraft Group of Companies

July 14, 2020

Report: PG5398-1

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

Table of Contents

Page

Appendices

Appendix 1

Soil Profile and Test Data Sheets
Symbols and Terms
Unidimensional Consolidation Test Sheets
Atterberg Limits Results
Grain Size Distribution Results
Analytical Testing Results

Appendix 2

Figure 1 - Key Plan
Figure 2 - Section B - Static Conditions
Figure 3 - Section B - Seismic Loading
Figure 4 - Section C - Static Conditions
Figure 5 - Section C - Seismic Loading
Drawing PG5398-1 - Test Hole Location Plan
Drawing PG5398-2 - Limit of Hazard Lands Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Richcraft Group of Companies to conduct a geotechnical investigation for Block 29 of the proposed Kanata West development to be located along Maple Grove Road, in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objectives of the geotechnical investigation were to:

❑ determine the subsoil and groundwater conditions at this site by means of test holes.

❑ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its 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 subject development as they are understood at the time of writing this report.

2.0 Proposed Development

It is understood that the proposed development will consist of 4 townhouse blocks, each with one basement level. The proposed townhouse blocks will be surrounded by asphalt paved access lanes and parking areas with landscaped margins. It is also understood that the site will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

The field program for the current geotechnical investigation was carried out on June 24, 2020 and consisted of advancing 3 boreholes (BH 1 to BH 3) to a maximum depth of 6.7 m below existing ground surface. One borehole from a previous investigation (BH 7) was also located within the boundaries of the subject site. The test hole locations were determined in the field by Paterson personnel taking into consideration site features and underground services. The test hole locations are presented on Drawing PG5398-1 - Test Hole Location Plan included in Appendix 2.

The test holes were advanced using a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples from the current and previous investigations were recovered using a 50 mm diameter split-spoon sampler or 73 mm diameter thin walled Shelby tubes in combination with a piston sampler. Auger cutting samples were recovered from surficial soils. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. The Shelby tubes were sealed at both ends. All samples were transported to our laboratory. The depths at which the auger, split-spoon and Shelby tube samples were recovered from the test holes are shown as AU, SS and TW, respectively, on the applicable Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils.

Overburden thickness was evaluated during the course of the current and previous investigations by dynamic cone penetration testing (DCPT) at BH 2 and BH 7. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment. Due to the low resistance exerted by the silty clay in some boreholes, the cone was pushed using the hydraulic head of the drill rig until resistance to penetration was encountered. The hammer was then used to further advance the cone to practical refusal.

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

Sample Storage

All samples from the current investigation 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. All samples from the previous investigations have been discarded.

3.2 Field Survey

The test holes from the current investigation were located and surveyed in the field by Paterson personnel. The locations and ground surface elevations for the current investigation were determined using a hand held GPS unit and are referenced to a geodetic datum. Test hole BH 7, from the previous investigation, was located and surveyed by Annis, Vollebekk and O'Sullivan, and is understood to be referenced to a geodetic datum.

The locations of the test holes and the ground surface elevation at each test hole location are presented on Drawing PG5398-1 - Test Hole Location Plan included in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the test holes were examined in our laboratory to review the results of the field logging. From the 3 current test holes, 13 split spoon samples were submitted for moisture content testing. Among these samples, 3 samples were submitted for Atterberg Limits testing, and 1 sample was submitted for grain size distribution testing.

One (1) soil sample from the previous borehole BH 7 was also submitted for unidimensional consolidation testing. This is discussed further in Section 5.3.

The results of the Atterberg Limits testing, grain size distribution testing, and unidimensional consolidation testing are presented in Appendix 1 and are further discussed in Sections 4 and 5.

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 sample. 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, Block 29, is currently vacant and grass covered across the majority of the site. The site, which has an approximate triangular shape, is bordered by Maple Grove Road to the north, Poole Creek to the northwest, and vacant undeveloped lands to the south and west. The existing ground surface across the site is generally level at approximate geodetic elevation 97 m.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the borehole locations consists of an approximate 0.2 to 0.5 m thickness of fill underlying the existing ground surface. The fill was generally observed to consist of a brown silty sand with some clay and crushed stone. An approximate 140 mm thickness of topsoil was also encountered underlying the fill at BH 1.

On the northwest end of the site, within BH 1 and BH 7, a layer of loose to compact, brown silty sand to sandy silt was encountered underlying the fill and/or topsoil, extending to an approximate depth of 1.5 m below the existing ground surface.

A silty clay deposit was encountered underlying the fill, topsoil, and/or silty sand to sandy silt. The silty clay deposit had a very stiff to stiff, brown silty clay crust in the upper 3 to 4 m, becoming a stiff to firm, grey silty clay with depth. Boreholes BH 1 through BH 3 were terminated in the silty clay deposit at approximate depths of 5.9 to 6.7 m below the existing ground surface.

A glacial till deposit was encountered in BH 7 underlying the silty clay at an approximate depth of 7 m. The glacial till was generally observed to consist of a compact, grey silty sand with gravel. Borehole BH 7 was terminated in the glacial till deposit at an approximate depth of 8.2 m below the existing ground surface.

Practical refusal to the DCPT was encountered at depths ranging from 9.1 m in BH 7, at the northwest end of the site, to 16 m at BH 2, located at the southeast end of the site.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Laboratory Testing

Atterberg limits testing, as well as associated moisture content testing, was completed on the recovered silty clay samples at 3 selected locations throughout the subject site.

The results of the Atterberg limits tests are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1. The tested silty clay samples classify as inorganic clays of low plasticity (CL) in accordance with the Unified Soil Classification System.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 1	2.0	33	20	13	CL
BH 2	2.6	42	19	23	CL
BH 3	2.6	39	20	19	CL

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
CH: Inorganic Clay of High Plasticity

The results of the shrinkage limit test indicate a shrinkage limit of 20% and a shrinkage ratio of 1.77.

Grain size distribution (sieve and hydrometer analysis) was also completed on one selected soil sample. The result of the grain size analysis is summarized in Table 2 and presented on the Grain Size Distribution Results sheet in Appendix 1.

Table 2 - Summary of Grain Size Distribution Analysis

Test Hole	Sample	Gravel (%)	Sand (%)	Silt & Clay (%)
BH 2	SS 4	0.0	13.9	86.1

Bedrock

Based on available geological mapping, the bedrock in this area consists of interbedded limestone and shale of the Verulam Formation with an overburden drift thickness of approximately 10 to 15 m depth.

4.3 Groundwater

Based on groundwater level measurements, field observations during excavation, knowledge of the groundwater within the local area of the subject site, and the recovered soil samples' moisture levels, consistency and colouring, the long-term groundwater table can be expected between a 3 to 4 m depth. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed buildings be founded on conventional shallow foundations bearing on the undisturbed, stiff to firm silty clay, compact silty sand to sandy silt, or on engineered fill which is placed and compacted directly over the undisturbed stiff to firm silty clay or compact silty sand to sandy silt.

Due to the presence of a silty clay deposit, the subject site will be subjected to a permissible grade raise restriction.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, asphalt, and deleterious fill, such as material containing a high content of organic materials, should be stripped from under the proposed building footprints and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed buildings should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. 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 building and paved areas should be compacted to at least 98% of the material's 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. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Using continuously applied loads, footings for the proposed buildings can be designed with the following bearing resistance values presented in Table 3.

Table 3 - Bearing Resistance Values

Undisturbed Bearing Surface	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
Compact Silty Sand/Sandy Silt	100	150
Stiff Silty Clay	120	180
Firm Silty Clay	80	120
Engineered Fill	100	150

Note: Strip footings, up to 2 m wide, and pad footings, up to 3 m wide, placed over an undisturbed, silty clay bearing surface can be designed using the abovenoted bearing resistance values.

If the silty sand subgrade is observed to be in a loose state of compactness, the material should be proof rolled using suitable vibratory equipment making several passes under dry conditions and above freezing temperatures and approved by Paterson at the time of construction.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

The bearing resistance value at SLS given for footings will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

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 silty clay, silty sand to sandy silt, or engineered fill bearing surface 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.

Settlement/Grade Raise

During the previous investigations, 1 consolidation test was completed within the boundaries of the subject site. The results of the consolidation test from the previous investigation are presented in Table 4 and in Appendix 1.

The value for p'_c is the preconsolidation pressure and p'_o is the effective overburden pressure of the test sample. The difference between these values is the available preconsolidation. The increase in stress on the soil due to the cumulative effects of the fill surcharge, the footing pressures, the slab loadings and the lowering of the groundwater should not exceed the available preconsolidation if unacceptable settlements are to be avoided.

The values for C_{cr} and C_c are the recompression and compression indices, respectively. These soil parameters are a measure of the compressibility due to stress increases below and above the preconsolidation pressures. The higher values for the C_c, as compared to the C_{cr}, illustrate the increased settlement potential above, as compared to below, the preconsolidation pressure.

Table 4 - Summary of Consolidation Test Results

Borehole	Sample	Elevation (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c
BH7	TW 6	91.56	107	79	0.025	0.742

The values of p'_c, p'_o, C_{cr} and C_c are determined using standard engineering testing procedures and are estimates only. Natural variations within the soil deposit will affect the results. The p'_o parameter is directly influenced by the groundwater level. Groundwater levels were measured during the site investigation. Groundwater levels vary seasonally which has an impact on the available preconsolidation. Lowering the groundwater level increases the p'_o and therefore reduces the available preconsolidation. Unacceptable settlements could be induced by a significant lowering of the groundwater level. The p'_o values for the consolidation tests during the investigation are based on the long term groundwater level being at 0.5 m below the existing groundwater table. The groundwater level is based on the colour and undrained shear strength profile of the silty clay.

The total and differential settlements will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

The potential post construction total and differential settlements are dependent on the position of the long term groundwater level when buildings are situated over deposits of compressible silty clay. Efforts can be made to reduce the impacts of the proposed development on the long term groundwater level by placing clay dykes in the service trenches, reducing the sizes of paved areas, leaving green spaces to allow for groundwater recharge or limiting planting of trees to areas away from the buildings. However, it is not economically possible to control the groundwater level.

Buildings on silty clay deposits increases the likelihood of movements and therefore of cracking. The use of steel reinforcement in foundations placed at key structural locations will tend to reduce foundation cracking compared to unreinforced foundations.

Based on the test hole information and consolidation testing results, a permissible grade raise restriction of **1.5 m** is recommended for grading within the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class D**. Soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

5.5 Basement Slab Construction

With the removal of all topsoil and fill, containing significant amounts of deleterious or organic materials, the existing fill or native soil subgrade approved by the geotechnical consultant at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for basement slab construction. Where the subgrade consists of existing fill, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as Granular B Type II.

It is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprints of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its MPMDD.

A sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided under the basement slabs. The spacing of the sub-slab drainage pipes can be determined at the time of construction to confirm groundwater infiltration levels, if any. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45\text{-}a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\, K_o\, \gamma\, H^2$, where K_o = 0.5 for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

Where required at the subject site, the recommended pavement structures for car only parking areas and access lanes are shown in Tables 5 and 6.

Table 5 - Recommended Pavement Structure - Car Only Parking Areas

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

Table 6 - Recommended Pavement Structure - Access Lanes	
Thickness (mm)	**Material Description**
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

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 I or II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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.

Due to the impervious nature of the subgrade materials consideration should be given to installing subdrains during the pavement construction. These drains should be installed at each catch basin, be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone which is placed at the footing level around the exterior perimeter of each structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Sub-slab Drainage

Sub-slab drainage is recommended to control water infiltration. For preliminary design purposes, we recommend that 150 mm diameter perforated pipes be placed at approximate 6 m centres underlying the basement slabs. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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 drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. 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 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 heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill 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. However, it is expected that sufficient room will be available for the greater part of the excavation 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 excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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.

The pipe bedding for sewer and water pipes should consist of a minimum of 150 mm of OPSS Granular A material. Where the bedding is located within the firm to stiff grey silty clay, the thickness of the bedding material should be increased to a minimum of 300 mm. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 95% of its SPMDD. The bedding material should extent at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 95% of its SPMDD.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Wet silty clay materials will be difficult to re-use, as the high water contents make compacting impractical without an extensive drying period.

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 a 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 a maximum 225 mm thick loose lifts 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

Due to the relatively impervious nature of the silty clay and existing groundwater level, it is anticipated that groundwater infiltration into the excavations should be low to medium and controllable using open sumps. A perched groundwater condition may be encountered within the silty sand to sandy silt deposit which may produce significant temporary groundwater infiltration levels. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations.

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application 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.

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.

Long-term Groundwater Control

Our recommendations for the long-term groundwater control for proposed construction are presented in Subsection 6.1. Any groundwater encountered along the proposed structure's perimeter or sub-slab drainage system will be directed to the proposed structure's sump pit. It is expected that groundwater flow will be low (i.e.- less than 10,000 L/day) with peak periods noted after rain events.

6.6 Winter Construction

The subsoil conditions at this site mostly 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. Precautions should be taken if winter construction is considered for this project.

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, 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.

The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

The results on analytical testing show that the sulphate content is less than 0.1%. The results are indicative that Type 10 Portland Cement would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity in indicative of a moderate to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Setbacks

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. Grain size distribution testing was also completed on a selected soil sample from BH 2. The above-noted soil samples were recovered from elevations below the anticipated design underside of footing elevation and 3.5 m depth below anticipated finished grade. The results of our testing are presented in Subsection 4.2 and in Appendix 1.

Based on the results of our review, a low to medium sensitivity clay soil is present within the proposed development.

Low/Medium Sensitivity Clay Soils

Based on our Atterberg Limits test results, the modified plasticity limit does not exceed 40% at the subject site. The following tree planting setbacks are recommended for the low to medium sensitivity area. Large trees (mature height over 14 m) can be planted within these areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to **4.5 m** for small (mature tree height up to 7.5 m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the conditions noted below are met:

❏ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

❏ A small tree must be provided with a minimum of 25 m^3 of available soil volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

❏ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

❏ The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

- ❑ Grading surround the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

Swimming Pools

The in-situ soils are considered to be acceptable for in-ground swimming pools. Above ground swimming pools must be placed at least 5 m away from the residence foundation and neighbouring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Aboveground Hot Tubs

Additional grading around the hot tub should not exceed permissible grade raises. Otherwise, hot tub construction is considered routine, and could be constructed in accordance with the manufacturer’s specifications.

Installation of Decks or Additions

Additional grading around proposed deck or addition should not exceed permissible grade raises. Otherwise, standard construction practices are considered acceptable.

6.9 Limit of Hazard Lands

Poole Creek

A section of Poole Creek is located within the west portion of the site. The slope condition was reviewed by Paterson field personnel as part of the geotechnical investigation. One (1) slope cross-section (Section B) was studied as the worst case scenario, where Poole Creek has meandered in close proximity (less than 1 m) from the toe of the upper slope or valley corridor. In addition, a second slope cross-section (Section C) was also analyzed at Block 29. The cross section locations are presented on Drawing PG5398-2 - Limit of Hazard Lands in Appendix 2. The subject section of Poole Creek is approximately 2 to 3 m wide, approximately 0.3 to 0.6 m depth, and meanders across the valley floor.

Poole Creek is observed within a 15 to 25 m wide flood plain. A 3 to 4 m high stable slope confines the flood plain. The upper slope is observed to be well vegetated and stable with little to no signs of active erosion. Signs of erosion were noted along the subject section of Poole Creek where the watercourse has meandered in close proximity to the toe of the corridor wall. The majority of the subject slope was shaped between a 2.2H:1V to 3.5H:1V slope.

A slope stability analysis was carried out to determine the required stable slope allowance setback from the top of slope based on a factor of safety of 1.5. A toe erosion and 6 m erosion access allowances were also considered in the determination of Limit of Hazard Lands and are discussed on the following pages. The proposed Limit of Hazard Lands, including the stable slope allowance, where required, toe erosion allowance, 6 m erosion access allowance, and top of slope are shown on Drawing PG5398-2 - Limit of Hazard Lands in Appendix 2.

Slope Stability Assessment

The analysis of slope stability was carried out using SLIDE, a computer program that permits a two-dimensional slope stability analysis using several methods, including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favouring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain than the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16 g was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The cross-sections were analyzed taking into account a groundwater level at ground surface, which represents a worse-case scenario that can be reasonably expected to occur in cohesive soils. The stability analysis assumes full saturation of the soil with groundwater flow parallel to the slope face. Subsoil conditions at the cross-sections were inferred based on the findings at borehole locations along the top of slope and general knowledge of the area's geology.

Stable Slope Allowance

The results of the stability analysis for static conditions at Sections B and C are presented in Figures 2 and 4 in Appendix 2. Section B requires a stable slope allowance due to the slope stability factor of safety being less than 1.5. It should be noted that the cross-section was analyzed as the worst case scenario for the subject slope. The remainder of the slope reviewed along the subject section of Poole Creek was noted to be shaped to at least a 3H:1V profile.

Based on the soil conditions observed and slope profile along the subject section of Poole Creek, the remainder of the slope has a slope stability factor of safety of greater than 1.5 and does not require a stable slope allowance.

The results of the analyses including seismic loading are shown in Figures 3 and 5 for the slope sections. The results indicate that the factor of safety for the sections are greater than 1.1 for the sections.

The existing vegetation on the slope face should not be removed as it contributes to the stability of the slope and reduces erosion. If the existing vegetation needs to be removed, it is recommended that a 100 to 150 mm of topsoil mixed with a hardy seed or an erosional control blanket be placed across the exposed slope face.

Toe Erosion and Erosion Access Allowance

The toe erosion allowance for the valley corridor wall slope was based on the cohesive nature of the soils, the observed current erosional activities and the width and location of the current watercourse. Signs of erosion were noted along the subject section of Poole Creek where the watercourse has meandered in close proximity to the toe of the corridor wall.

It is considered that in areas where the water course has meandered in close proximity (less than 15 m) to the toe of the upper slope, a toe erosion allowance of 5 m and an erosion access allowance of 6 m are required from the top of slope. Where the watercourse is greater than 15 m from the toe of the slope, the toe erosion allowance should be taken from the watercourse edge. The Limit of Hazard Lands, which includes these allowances, is indicated on Drawing PG5398-2 - Limit of Hazard Lands in Appendix 2.

Minimum Setback Requirements of the Official Plan

Minimum setbacks have been established by Council for the Official Plan for rivers, lakes, streams and other surface water features. It should be noted that where a council-approved watershed, sub-watershed or environmental management plan does not exist, the minimum setback will be the greater of the following:

- ❏ Development limits as established by the regulatory flood line
- ❏ Development limits as established by the geotechnical Limit of the Hazard Lands
- ❏ 30 m from normal high water mark of rivers, lakes and streams as determined in consultation with the conservation authority, or

- 15 m from existing top of bank, where there is a defined bank.

However, it should also be noted that where the geotechnical Limit of Hazard Lands line and regulatory flood line are within 15 m of top of slope, the development limits can be established as the geotechnical limit of hazard lands line provided the Conservation Authority approves.

7.0 Recommendations

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

- ❑ A review of the final grading plan should be completed from a geotechnical perspective.
- ❑ 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 material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation of this nature is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

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 Richcraft Group of Companies or their agents is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group

LICENSED PROFESSIONAL ENGINEER
July 14, 2020
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Yolanda Tang, M.Sc.Eng

Scott S. Dennis, P.Eng.

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

UNIDIMENSIONAL CONSOLIDATION TEST SHEETS

ATTERBERG LIMITS RESULTS

GRAIN SIZE DISTRIBUTION RESULTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Block 29 - Maple Grove Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5398

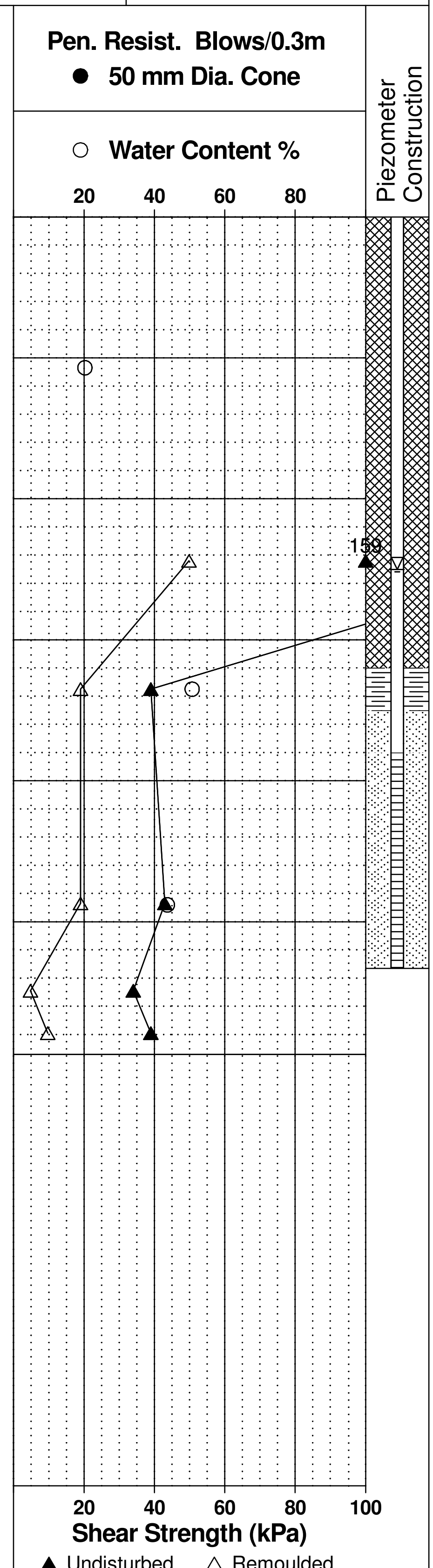
REMARKS

HOLE NO. BH 1

BORINGS BY Track-Mount Power Auger

DATE June 24, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	96.70
FILL: Brown silty sand, some crushed stone (to 0.46)		AU	1				
TOPSOIL (to 0.60)							
Compact, brown **SANDY SILT,** trace clay to **CLAYEY SILT** (to 1.52)		SS	2	58	16	1	95.70
Very stiff to stiff, brown **SILTY CLAY,** trace sand		SS	3	83	2	2	94.70
- firm and grey by 3.0m depth						3	93.70
		SS	4	100	P	4	92.70
(to 5.94)		SS	5	100	P	5	91.70
End of Borehole							
(GWL @ 2.5m depth based on field observations)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Block 29 - Maple Grove Road
Ottawa, Ontario

DATUM Geodetic

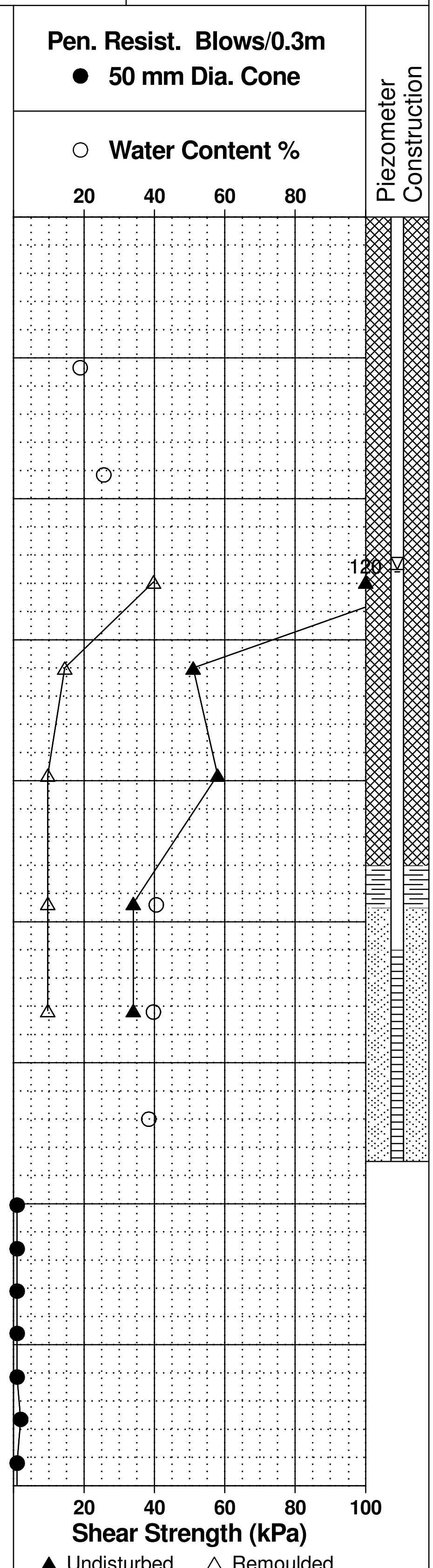
REMARKS

BORINGS BY Track-Mount Power Auger **DATE** June 24, 2020

FILE NO. PG5398

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	97.08
FILL: Brown silty sand, some clay, crushed stone and boulders — 0.36	AU	1				
Brown **CLAYEY SILT,** trace sand	SS	2	58	19	1	96.08
	SS	3	67	7	2	95.08
— 2.20						
Very stiff to stiff, brown **SILTY CLAY,** trace sand - stiff to firm and grey by 3.0m depth	SS	4	100	P	3	94.08
					4	93.08
	SS	5	100	P	5	92.08
	SS	6	100	P	6	91.08
	SS	7	100	P		
— 6.70						
Dynamic Cone Penetration Test commenced at 6.70m depth.					7	90.08
					8	89.08
					9	88.08



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Block 29 - Maple Grove Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5398

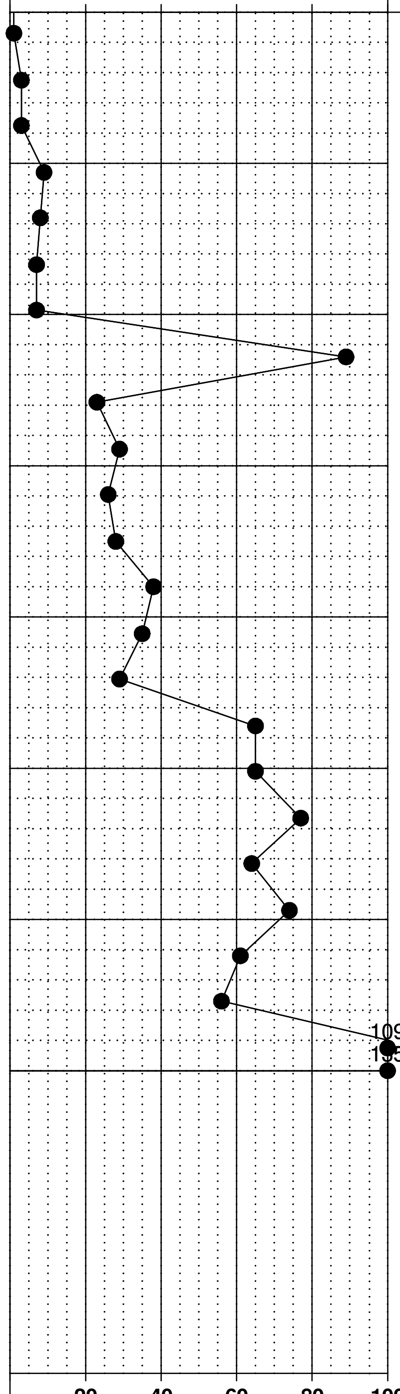
REMARKS

HOLE NO. BH 2

BORINGS BY Track-Mount Power Auger

DATE June 24, 2020

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 %	Piezometer Construction
GROUND SURFACE								20 40 60 80	
						9	88.08		
						10	87.08		
						11	86.08		
						12	85.08		
						13	84.08		
						14	83.08		
						15	82.08		
16.00						16	81.08		
End of Borehole Practical DCPT refusal at 16.00m depth (GWL @ 2.5m depth based on field observations)									



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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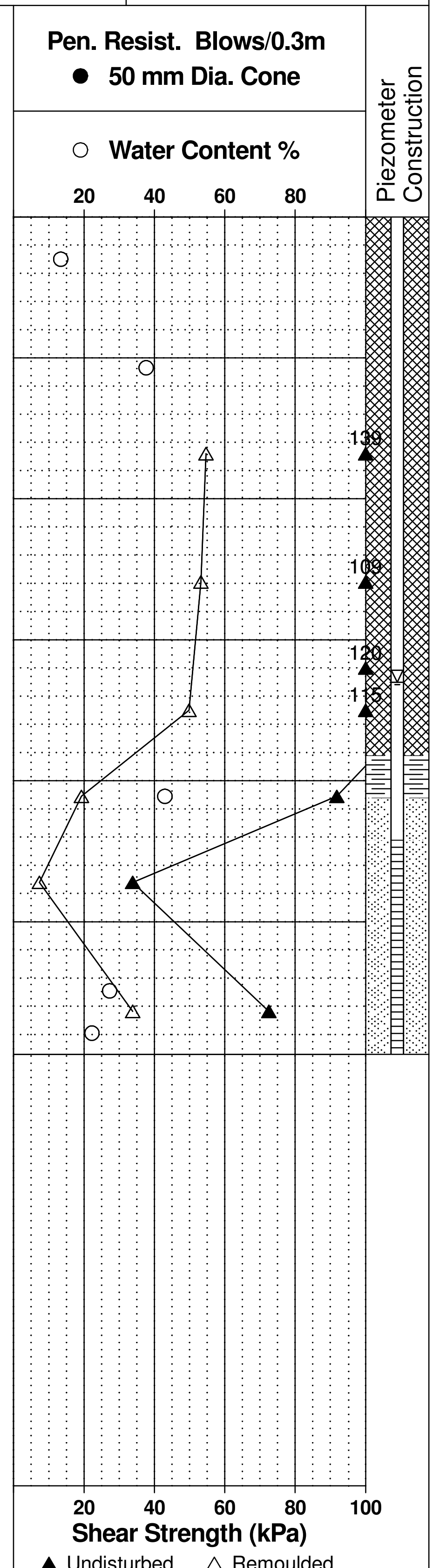
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Block 29 - Maple Grove Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE June 24, 2020

FILE NO. PG5398
HOLE NO. BH 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.24
FILL: Brown silty sand, trace crushed stone (0.20)		SS	1	17	6		
		SS	2	75	6	1	96.24
Very stiff to stiff, brown **SILTY CLAY,** trace sand						2	95.24
		SS	3	100	P		
						3	94.24
- firm to stiff and grey by 3.8m depth						4	93.24
		SS	4	100			
						5	92.24
(5.94)		SS	5	50	3		
End of Borehole							
(GWL @ 3.3m depth based on field observations)							



patersongroup Consulting Engineers

154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Kanata West Subdivision
Ottawa (Kanata), Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 75 Power Auger

DATE Aug 1, 03

FILE NO. G9012

HOLE NO. BH 7

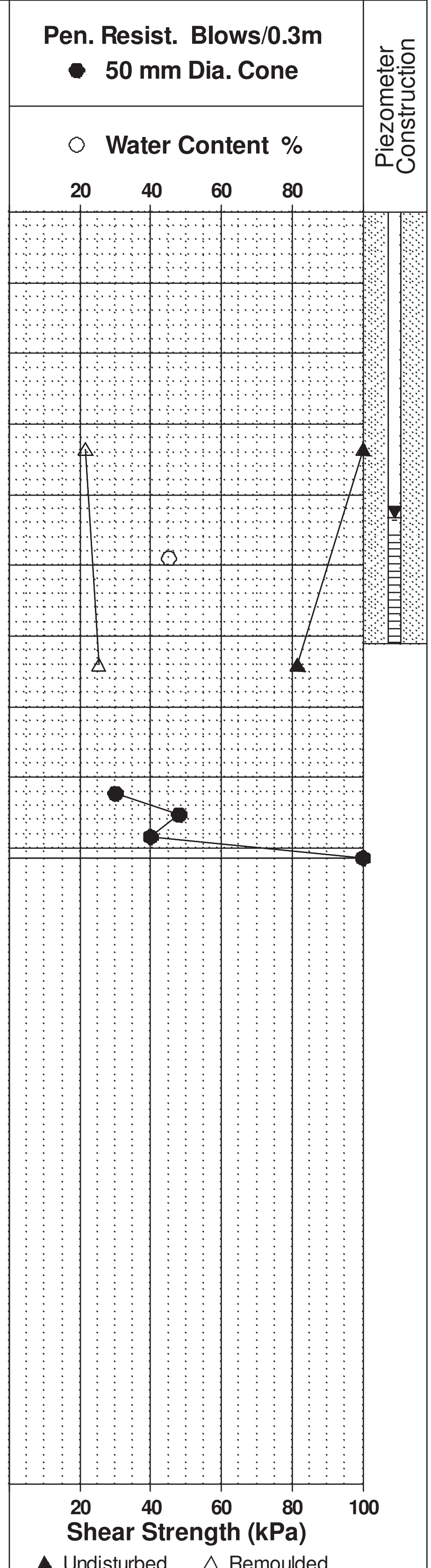
SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.23				
Loose, brown **SILTY** fine **SAND**	1.45	SS	15	29	8
Very stiff to stiff, brown **SILTY CLAY** - grey by 4.0m depth	7.01	SS	16	96	5
		TW	6		
GLACIAL TILL: Compact, grey silty fine sand with gravel	8.23	SS	17	75	18
Dynamic Cone Penetration Test commenced @ 8.23m depth Inferred **GLACIAL TILL**	9.14				

End of Borehole

DCPT refusal @ 9.14m depth

(GWL @ 4.33m-Aug. 18/03)

DEPTH (m)	ELEV. (m)
0	96.46
1	95.46
2	94.46
3	93.46
4	92.46
5	91.46
6	90.46
7	89.46
8	88.46
9	87.46



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 strength of cohesionless soils is the relative density, 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.

Relative Density	'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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture 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 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

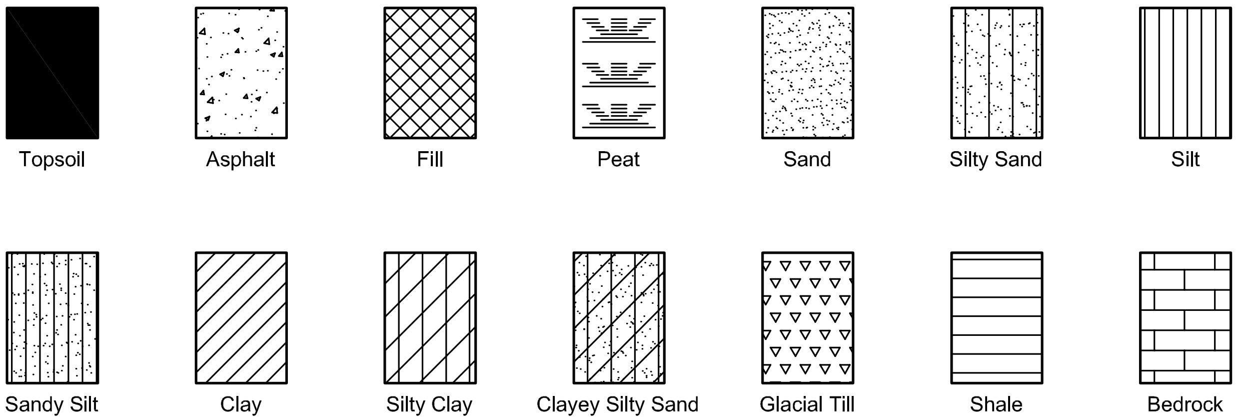
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

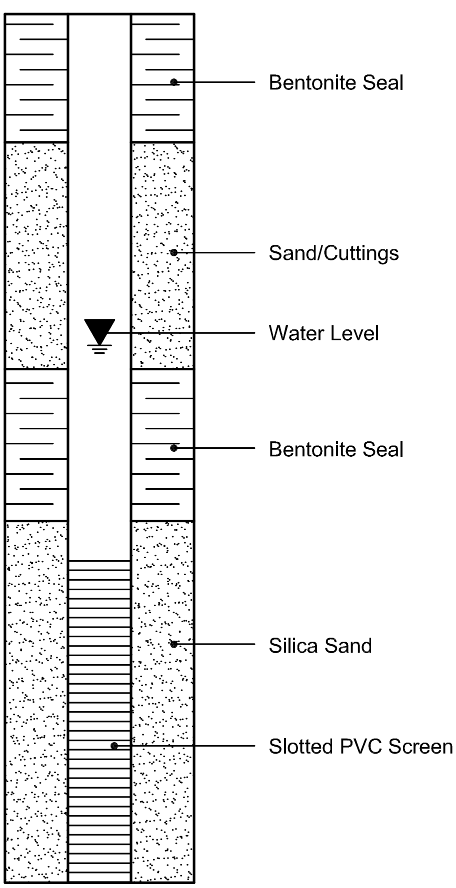
SYMBOLS AND TERMS (continued)

STRATA PLOT

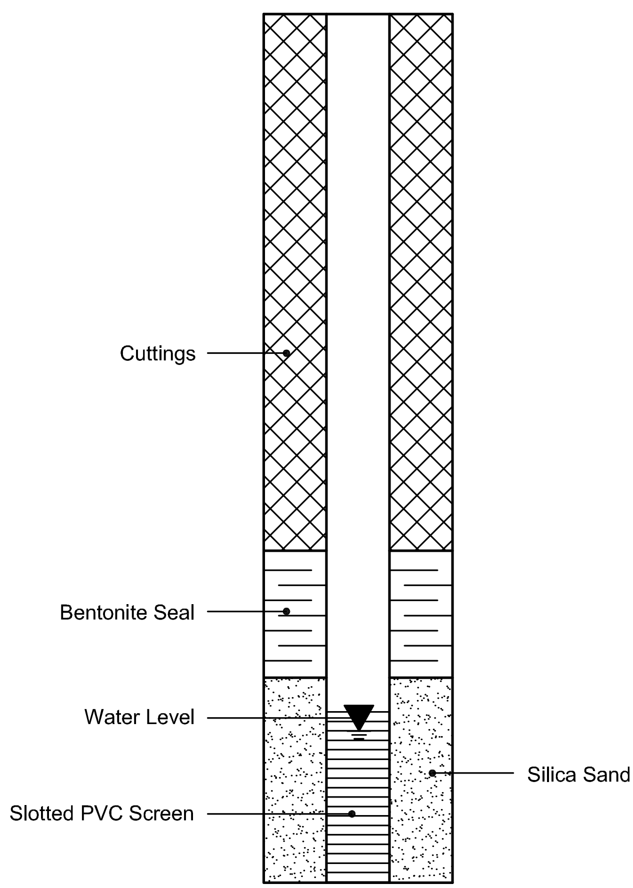


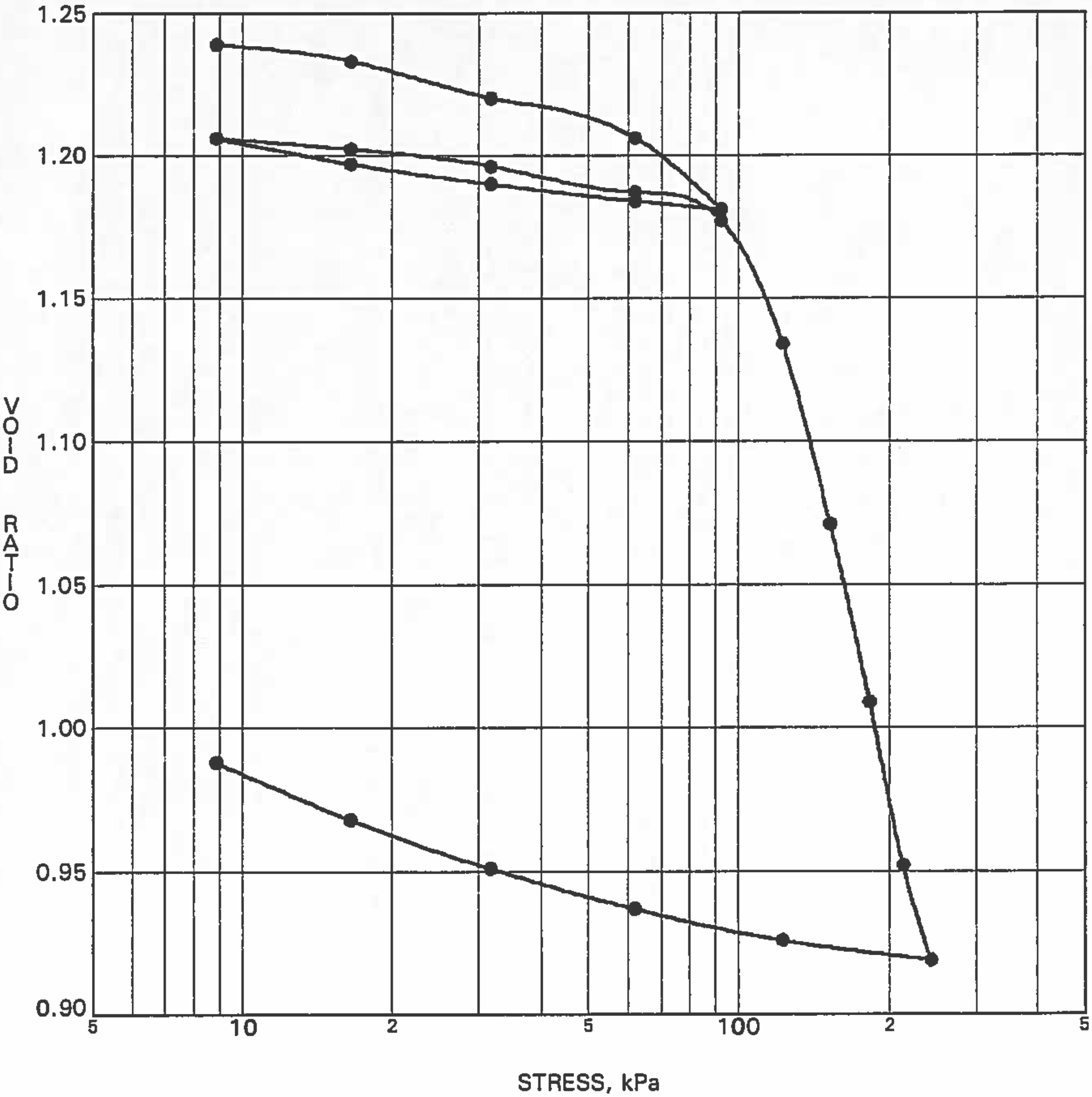
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	BH 7	p'_o	79 kPa	Ccr	0.025
Sample No.	TW 6	p'_c	107 kPa	Cc	0.742
Sample Depth	4.90 m	OC Ratio	1.4	Wo	45.1 %
Sample Elev.	91.56 m	Void Ratio	1.240	Unit Wt.	17.5 kN/m^3

CLIENT Richcraft Homes

FILE NO. G9012

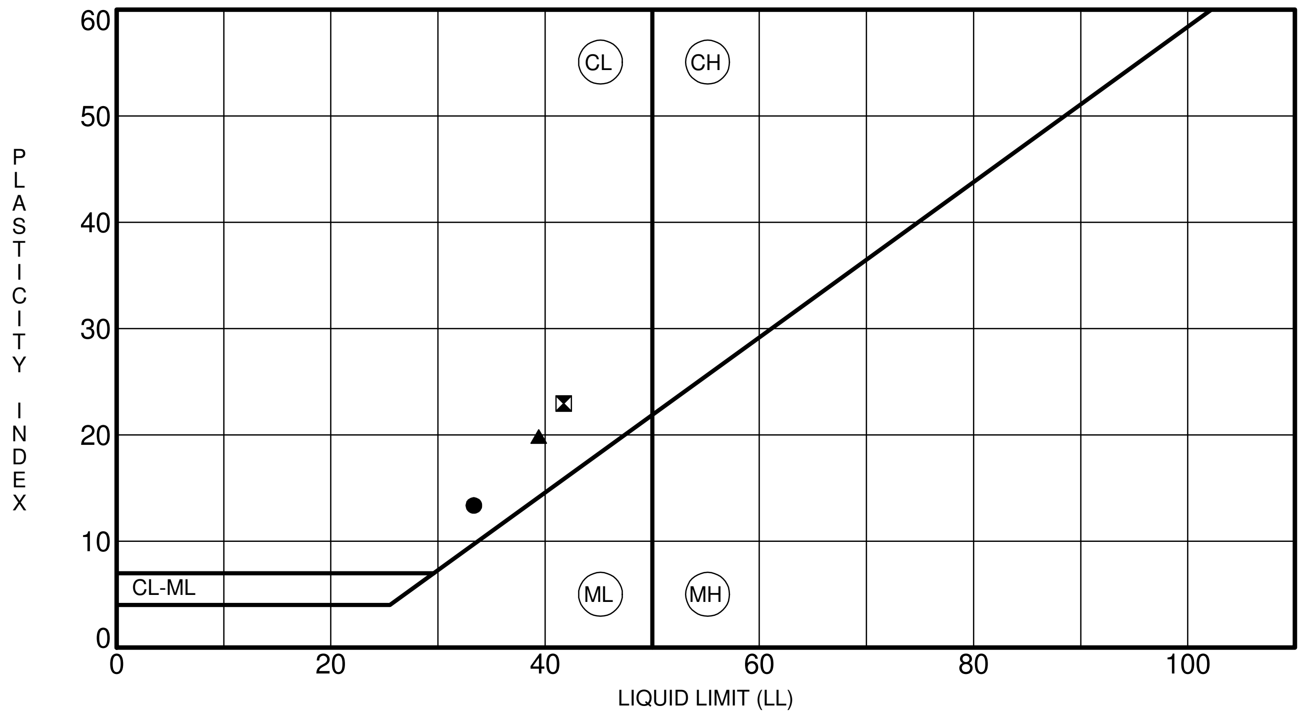
PROJECT Geotechnical Investigation - Proposed Kanata West Subdivision

DATE 19/08/03

CONSOLIDATION TEST

JOHN D. PATERSON & ASSOCIATES LTD.

Unit 1, 28 Concourse Gate, Nepean, Ontario K2E 7T7



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1**	**SS 3**	**33**	**20**	**13**		**CL - Inorganic clays of low plasticity**
⊠	**BH 2**	**SS 4**	**42**	**19**	**23**		**CL - Inorganic clays of low plasticity**
▲	**BH 3**	**SS 3**	**39**	**20**	**20**		**CL - Inorganic clays of low plasticity**

CLIENT **Richcraft Homes**

FILE NO. **PG5398**

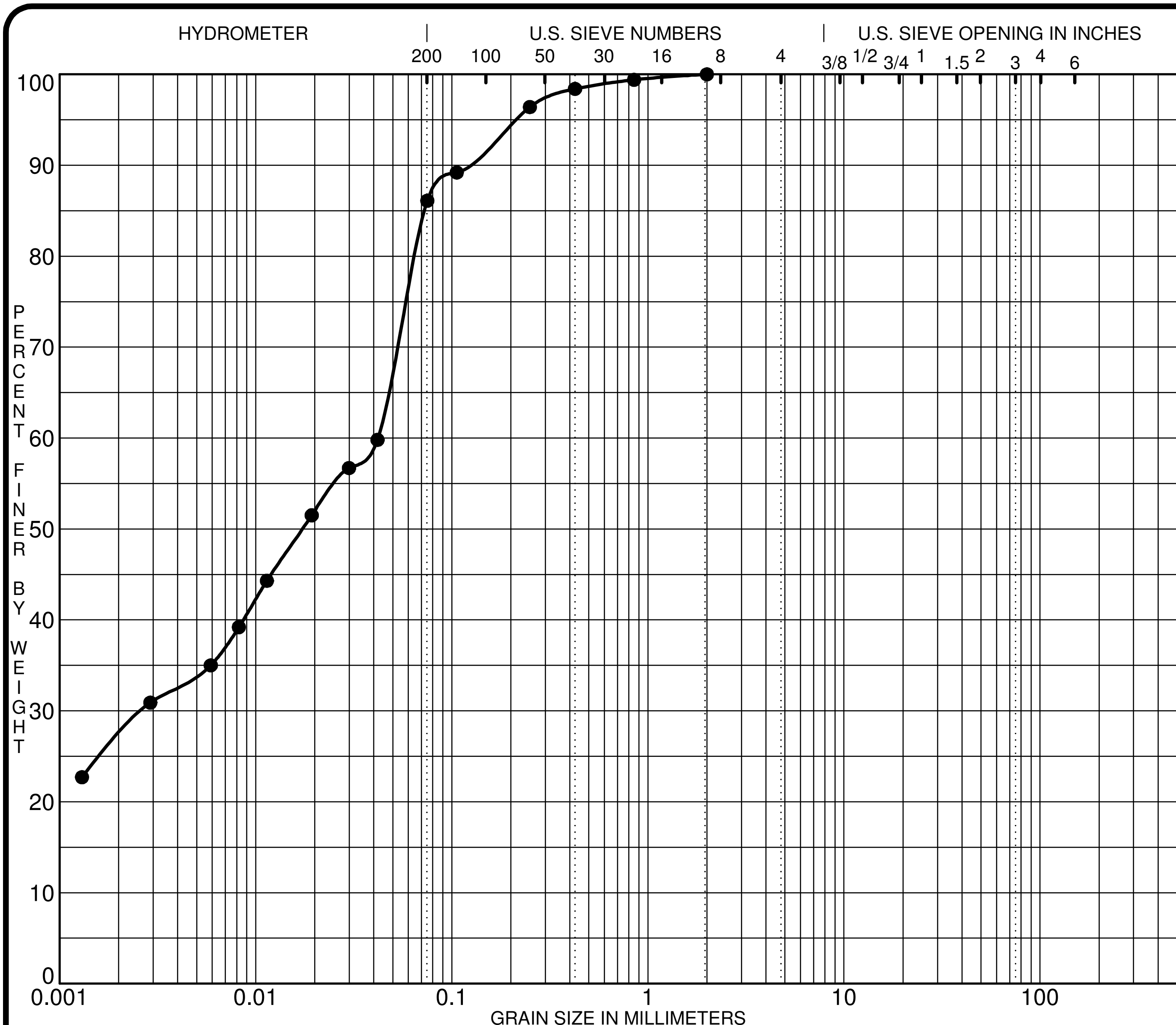
PROJECT **Geotechnical Investigation - Kanata West Block 29 - Maple Grove Road**

DATE **24 Jun 20**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
● BH 2	SS 4	**CL - Inorganic clays of low plasticity**						

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● BH 2	SS 4	**2.00**	**0.04**	**0.003**		**0.0**	**13.9**	**86.1**	

CLIENT **Richcraft Homes**

FILE NO. **PG5398**

PROJECT **Geotechnical Investigation - Kanata West Block 29 - Maple Grove Road**

DATE **24 Jun 20**

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 29945

Report Date: 02-Jul-2020

Order Date: 25-Jun-2020

Project Description: PG5398

	Client ID:	BH3-SS2	-	-	-
	Sample Date:	24-Jun-20 12:00	-	-	-
	Sample ID:	2026396-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	76.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.27	-	-	-
Resistivity	0.10 Ohm.m	135	-	-	-
Anions					
Chloride	5 ug/g dry	11	-	-	-
Sulphate	5 ug/g dry	6	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - SECTION B - STATIC CONDITIONS

FIGURE 3 - SECTION B - SEISMIC LOADING

FIGURE 4 - SECTION C - STATIC CONDITIONS

FIGURE 5 - SECTION C - SEISMIC LOADING

DRAWING PG5398-1 - TEST HOLE LOCATION PLAN

DRAWING PG5398-2 - LIMIT OF HAZARD LANDS

FIGURE 1

KEY PLAN

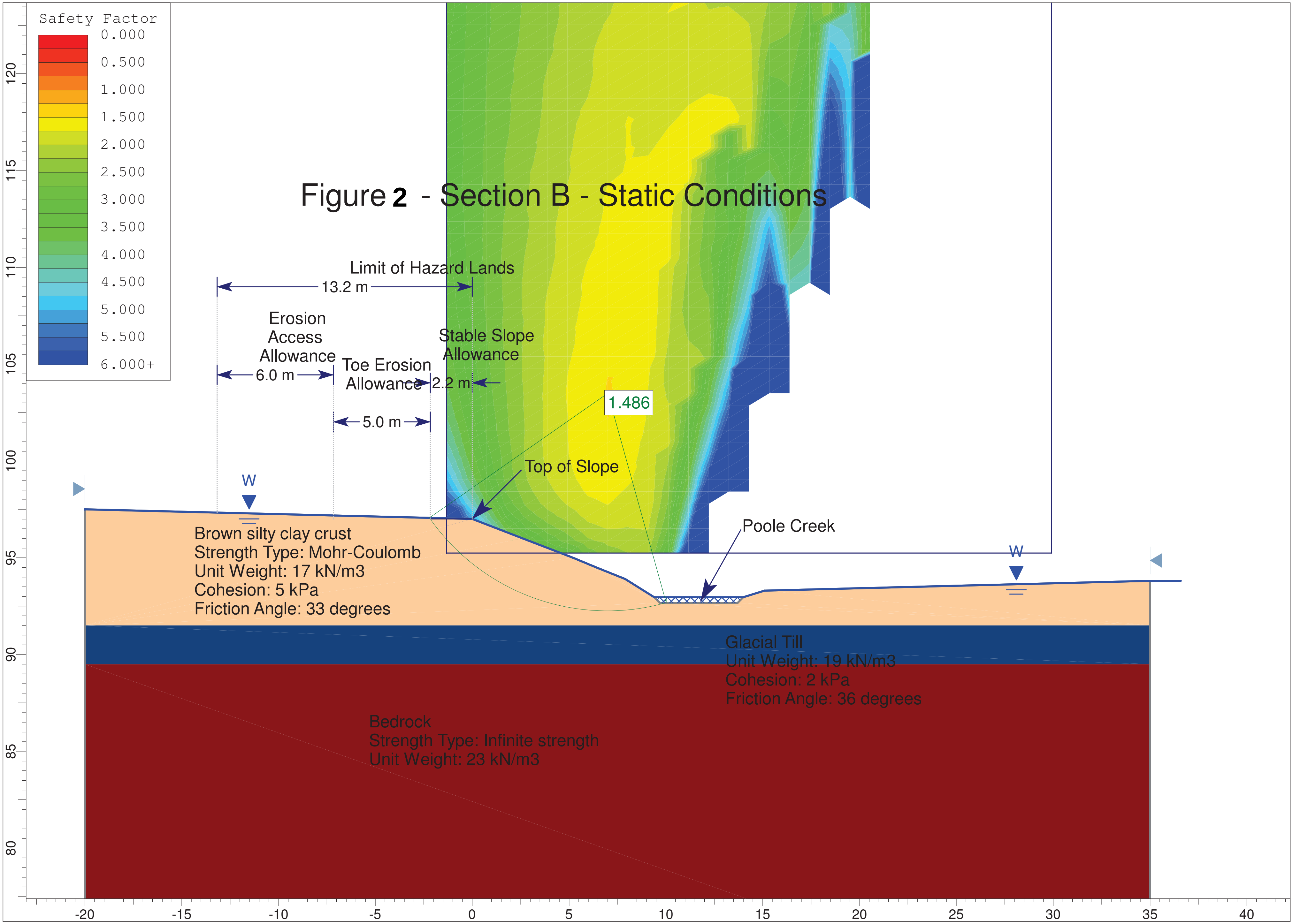
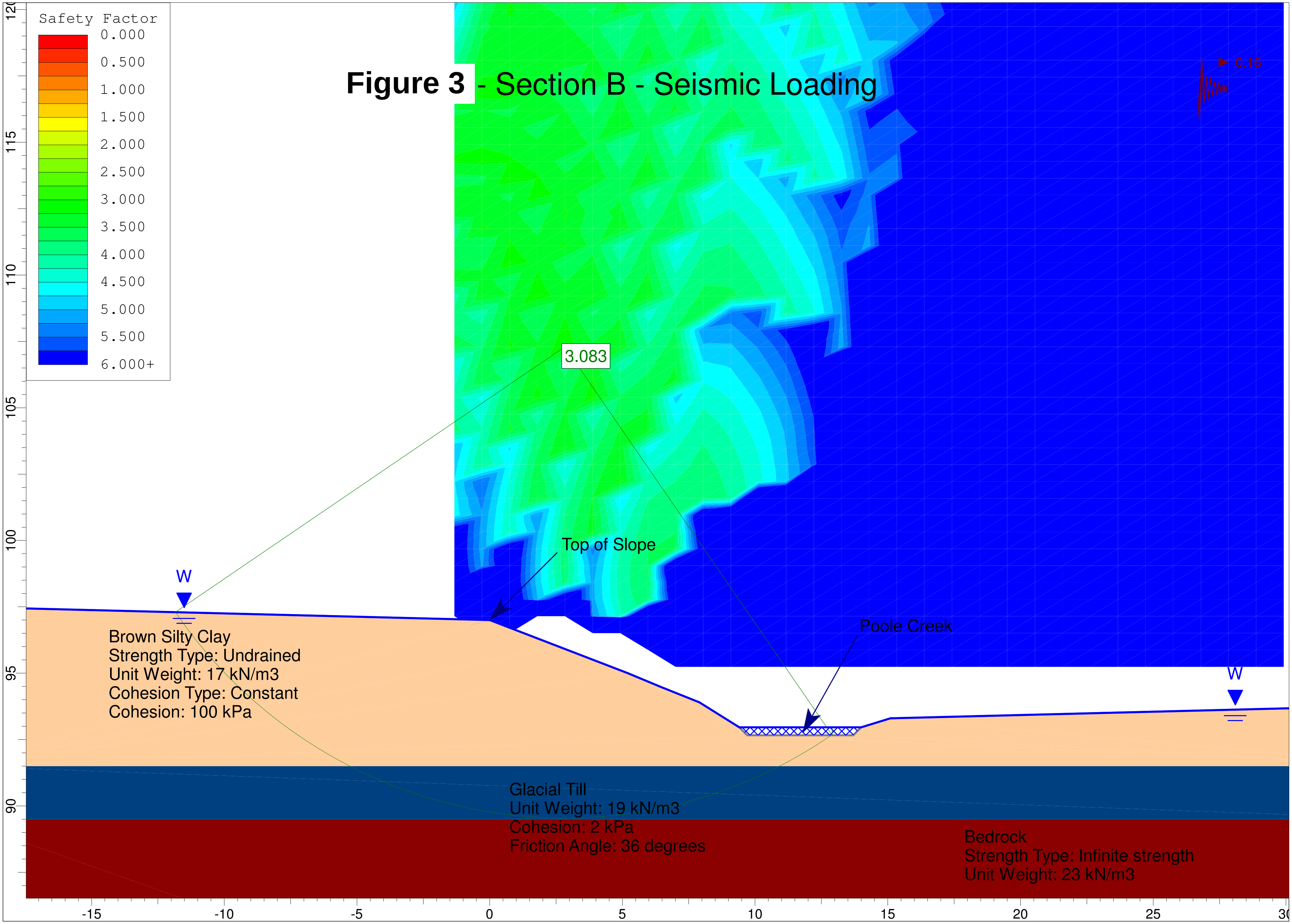

Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Figure 2 - Section B - Static Conditions
Limit of Hazard Lands
13.2 m
Erosion Access Allowance
6.0 m
Toe Erosion Allowance
5.0 m
Stable Slope Allowance
2.2 m
1.486
Top of Slope
W
Poole Creek
Brown silty clay crust
Strength Type: Mohr-Coulomb
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Glacial Till
Unit Weight: 19 kN/m3
Cohesion: 2 kPa
Friction Angle: 36 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 23 kN/m3
120
115
110
105
100
95
90
85
80
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40

Figure 3 - Section B - Seismic Loading



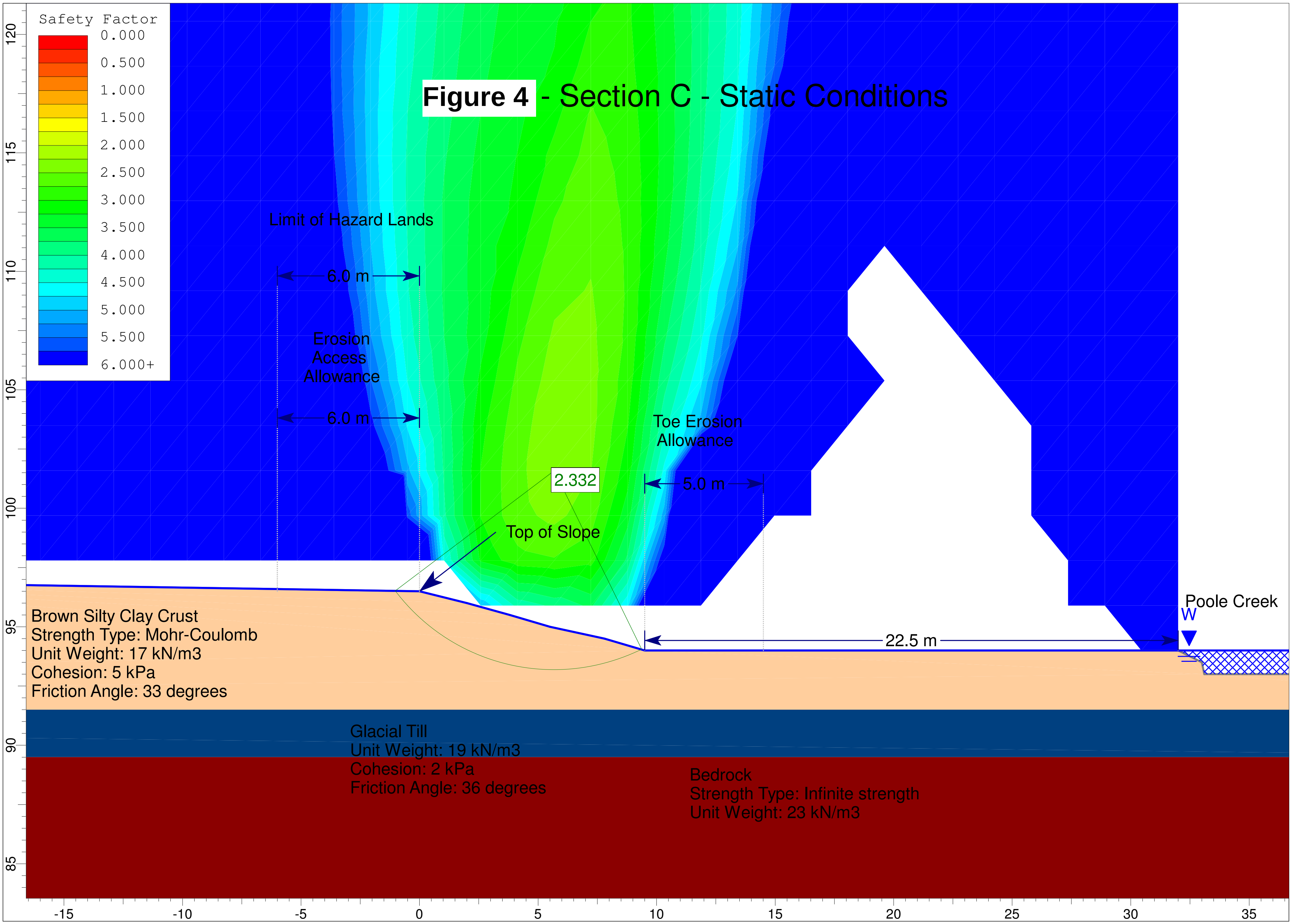


Figure 4 - Section C - Static Conditions
Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
Limit of Hazard Lands
6.0 m
Erosion Access Allowance
6.0 m
Toe Erosion Allowance
2.332
5.0 m
Top of Slope
Poole Creek
W
22.5 m
Brown Silty Clay Crust
Strength Type: Mohr-Coulomb
Unit Weight: 17 kN/m3
Cohesion: 5 kPa
Friction Angle: 33 degrees
Glacial Till
Unit Weight: 19 kN/m3
Cohesion: 2 kPa
Friction Angle: 36 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 23 kN/m3
120
115
110
105
100
95
90
85
-15
-10
-5
0
5
10
15
20
25
30
35

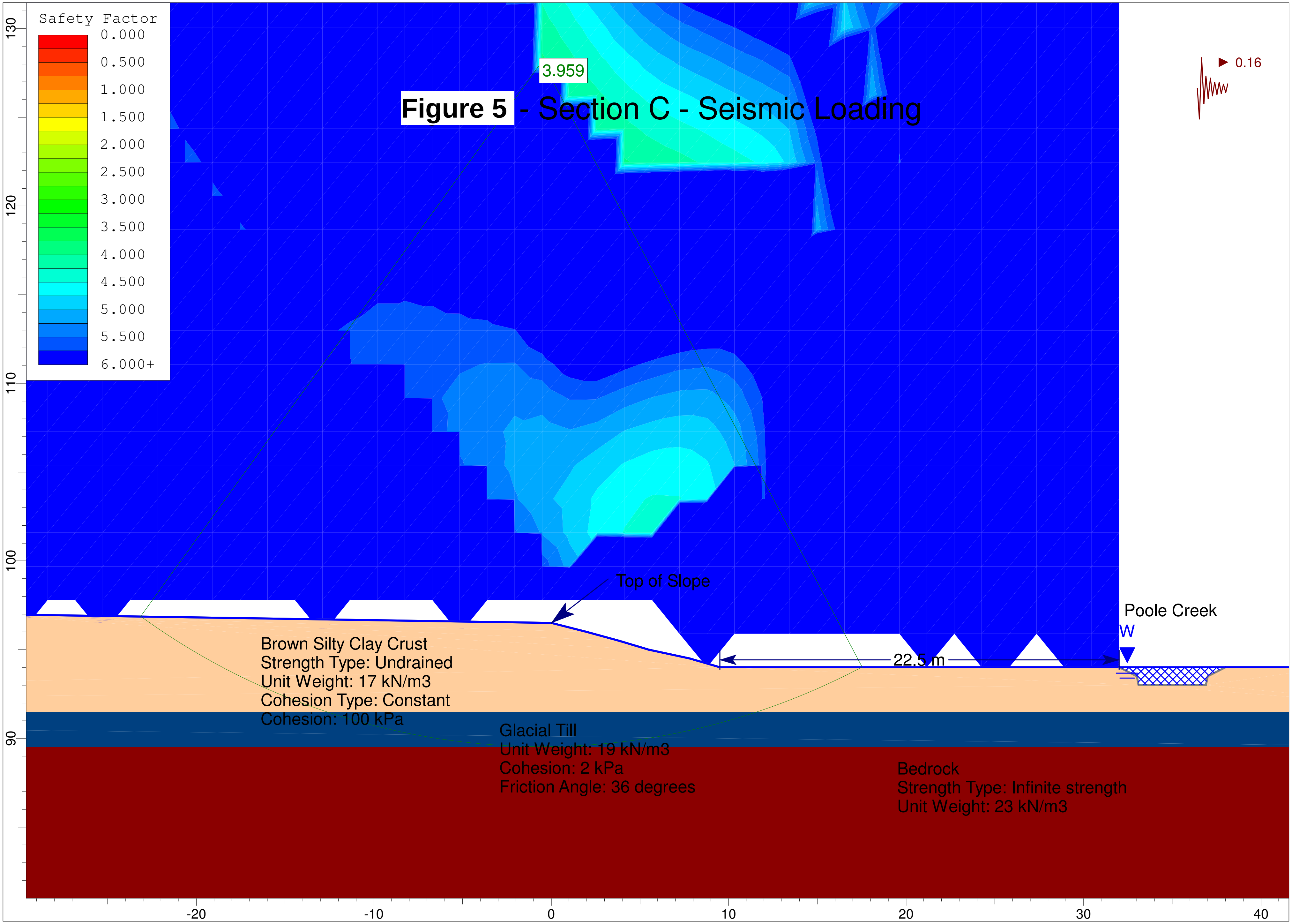
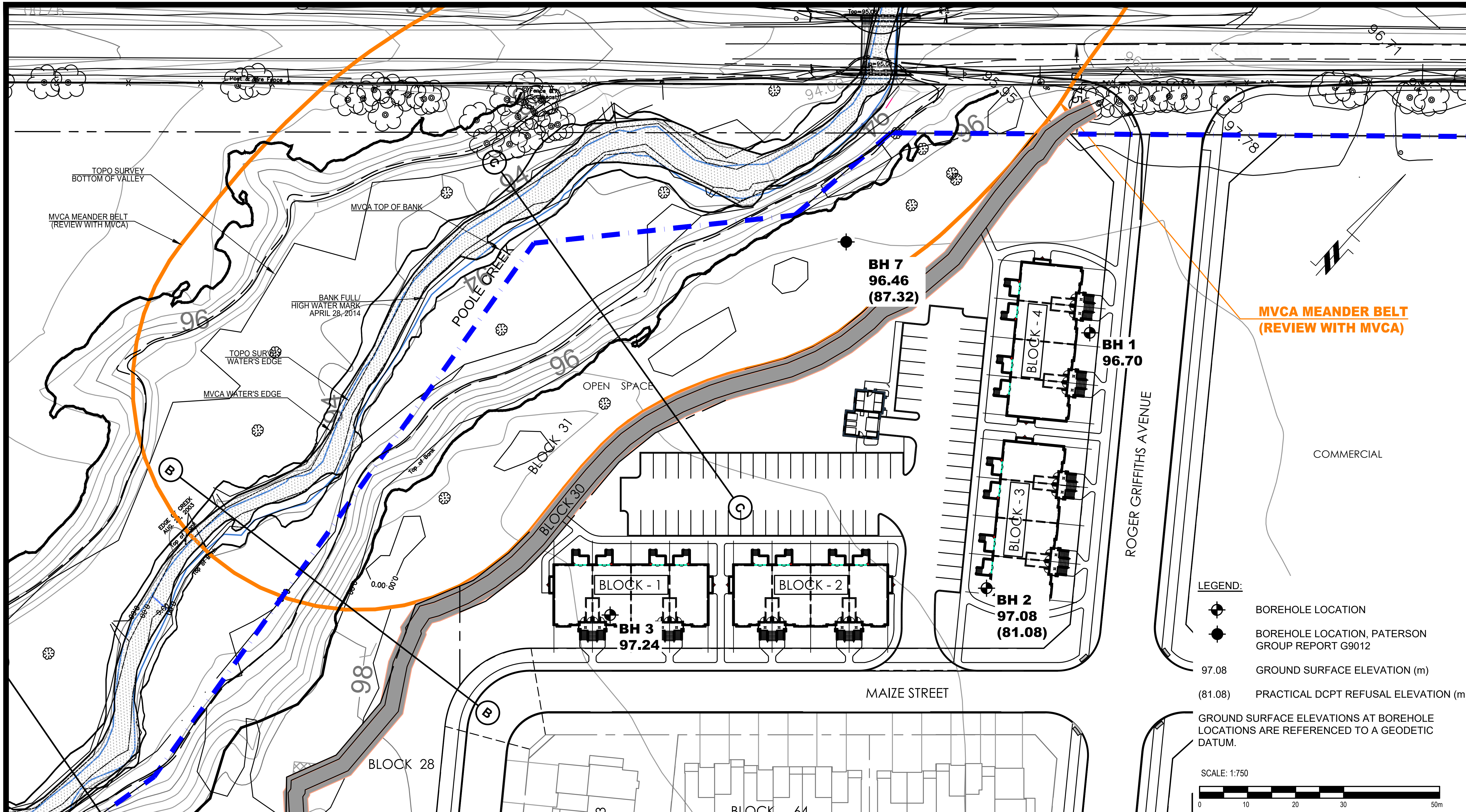


Figure 5 - Section C - Seismic Loading
Safety Factor
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
5.500
6.000+
3.959
0.16
Top of Slope
Poole Creek
W
22.5 m
Brown Silty Clay Crust
Strength Type: Undrained
Unit Weight: 17 kN/m3
Cohesion Type: Constant
Cohesion: 100 kPa
Glacial Till
Unit Weight: 19 kN/m3
Cohesion: 2 kPa
Friction Angle: 36 degrees
Bedrock
Strength Type: Infinite strength
Unit Weight: 23 kN/m3
130
120
110
100
90
-20
-10
0
10
20
30
40



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

NO.	REVISIONS	DATE	INITIAL
0			

RICHCRAFT HOMES
GEOTECHNICAL INVESTIGATION
KANATA WEST - BLOCK 29 - MAPLE GROVE ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 07/2020
Drawn by: MPG	Report No.: PG5398-1
Checked by: YT	Dwg. No.: PG5398-1
Approved by: SD	Revision No.:

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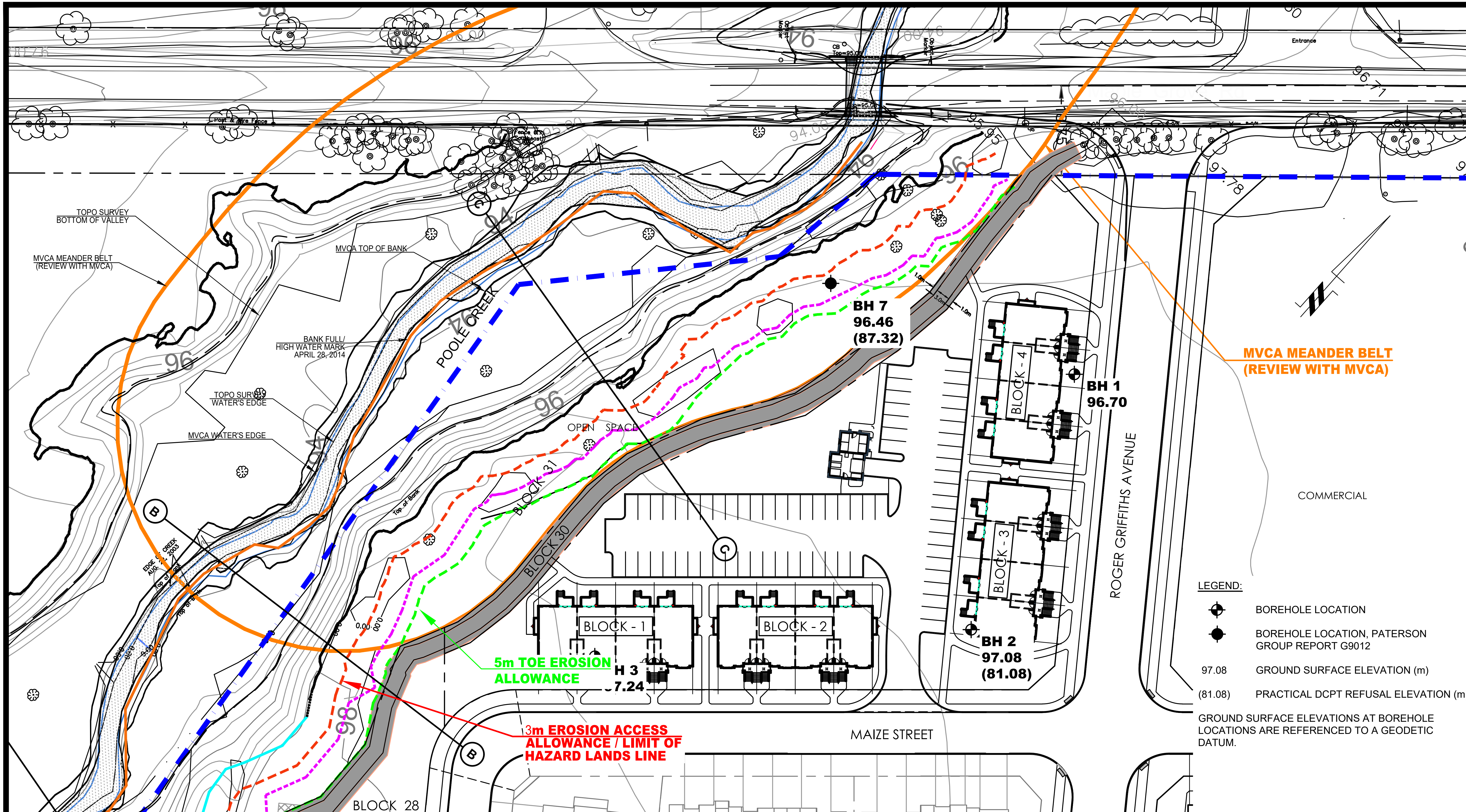
11x17



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

NO.	REVISIONS	DATE	INITIAL
0			

RICHCRAFT HOMES
GEOTECHNICAL INVESTIGATION
KANATA WEST - BLOCK 29 - MAPLE GROVE ROAD
OTTAWA, ONTARIO

Title:
LIMIT OF HAZARD LANDS

Scale: 1:750	Date: 07/2020
Drawn by: MPG	Report No.: PG5398-1
Checked by: YT	Dwg. No.: PG5398-2
Approved by: SD	Revision No.:

p:\autocad drawings\geotechnical\pg53xx\pg5398\pg5398-1 thlp.dwg

11x17

D.2 CONFIRMATION OF GRADE RAISE SUITABILITY BY PATERSON GROUP

From:	Scott Dennis
To:	Johnson, Warren
Cc:	Gillis, Sheridan
Subject:	RE: Kanata West Block 29
Date:	Monday, February 08, 2021 1:12:21 PM

Warren,

See below my responses in red:

- Can you confirm the required number of foundation drains for the proposed buildings?

In addition to 1 perimeter drain for each building, it is recommended to have 1 sub-slab drain running lengthwise through the center of each block.

- The site slightly exceeds the grade raise restriction of 1.5m in some areas by 0.3-0.5m. Please let me know if you see any issues based on the attached grading plan (existing grades in red) or if this variance will be acceptable.

This variance will be accepted. However, if the grade raises get any higher, lightweight fill will likely be required.

Regards,

Scott S. Dennis, P.Eng.

patersongroup

solution oriented engineering
over 60 years serving our clients

154 Colonnade Road South
Ottawa, Ontario, K2E 7J5
Tel: (613) 226-7381 Ext. 332

From: Johnson, Warren <Warren.Johnson@stantec.com>
Sent: February 5, 2021 11:52 AM
To: Scott Dennis <sdennis@Patersongroup.ca>
Cc: Gillis, Sheridan <Sheridan.Gillis@stantec.com>
Subject: Kanata West Block 29

Hi Scott,

See attached working drawings for Block 29. As we are working through the design there are two items we would appreciate your feedback on.

- Can you confirm the required number of foundation drains for the proposed buildings?
- The site slightly exceeds the grade raise restriction of 1.5m in some areas by 0.3-0.5m. Please let me know if you see any issues based on the attached grading plan (existing grades in red) or if this variance will be acceptable.

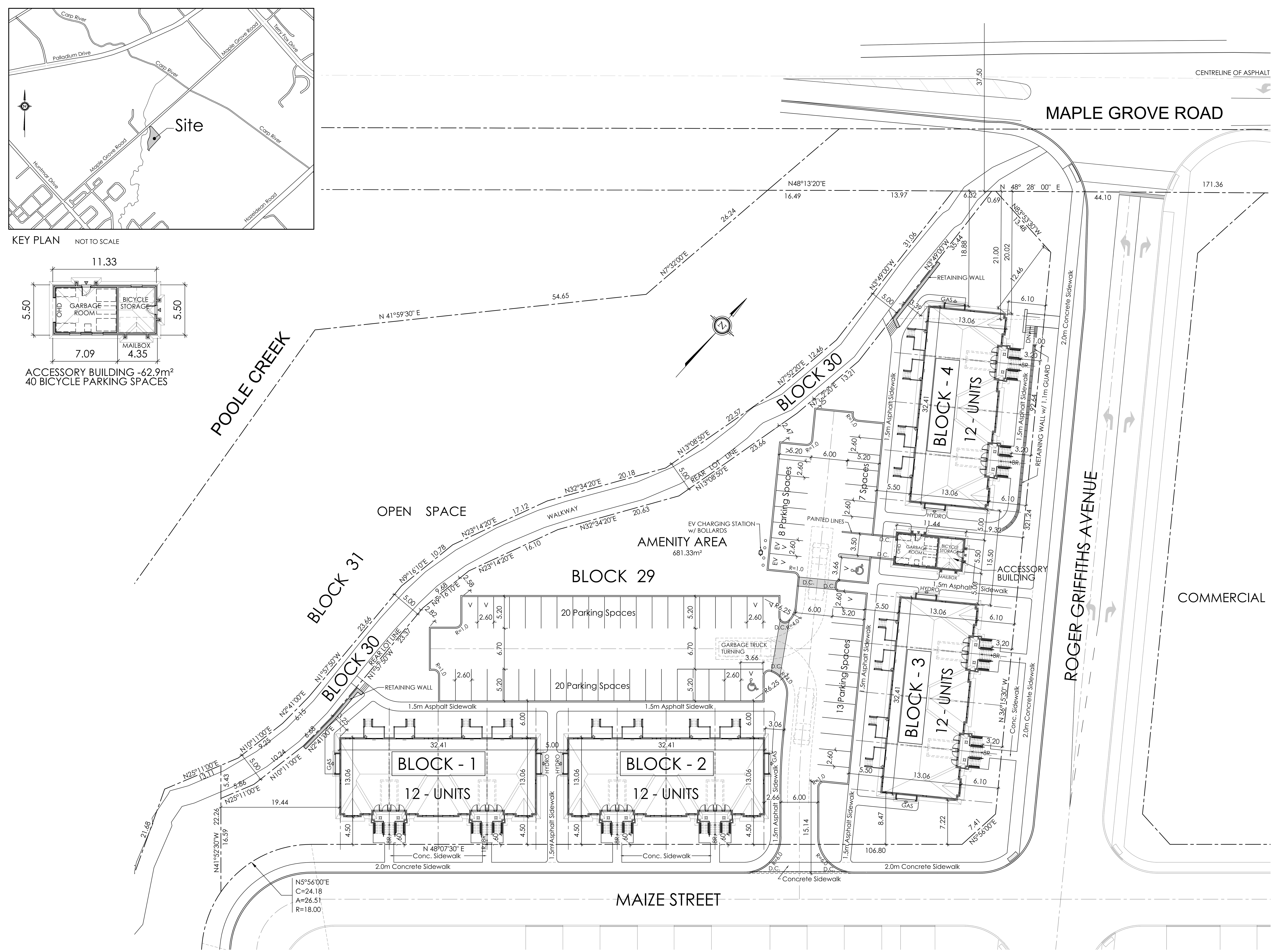
Thanks,

Warren Johnson C.E.T.
Civil Engineering Technologist

Direct: 613-784-2272
Mobile: 613-868-8692
warren.johnson@stantec.com

Stantec
400 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

Appendix E - PROPOSED SITE PLAN



SITE INFORMATION :

PROPOSED ZONING : R4Z - PERMITTED USES : - PLANNED UNIT DEVELOPMENT

SITE AREA : 7,373.84 m²
TOTAL BUILDING AREA : 1,710.90 m²

PROPOSED ZONING:	R4Z	PROVIDED:
LOT AREA (MIN.):	1,400.0 m²	7,373.84 m² (0.73 ha)
LOT WIDTH (MIN.):	n/a	0.69 m (Maple Grove Rd.)
FRONT YARD (MIN.) :	3.0 m	17.80 m
CORNER SIDE YARD (MIN.) :	3.0 m	4.50 m
INTERIOR SIDE YARD (MIN.) :		
Within 21m of Front Lot Line		
Bldg. Ht. Less Than 11m	1.5 m	3.39 m
Bldg. Ht. Greater Than 11m	3.0 m	
All Other	6.0 m	3.25 m
REAR YARD (MIN.) :	6.0 m - 4.5m [135]	4.50 m
BUILDING SPACING :		
BETWEEN BUILDING & PRIVATE WAY	1.8 m	2.86 m
BETWEEN GARAGE & PRIVATE WAY	5.2 m	n/a
BETWEEN BUILDINGS	1.2 m	5.00 m
MINIMUM LANDSCAPED AREA :	30.0%	51.5 % (3,801.5m²)
BUILDING HEIGHT (MAX.):	11.0 m	9.45 m
PORCH STAIR TO LOT LINE (SECTION 65)	0.60 m	1.60 m

TOTAL AMENITY AREA REQUIRED :
- STACKED DWELLING 6.0m² x 48 = 288.0 m²
COMMUNAL AMENITY AREA REQ'D. (MIN.):
50% of 288 m² = 144.0 m²

- PRIVATE AMENITY AREA -
(BALCONIES & PATIOS) 6.5m² x 48 = 312.0 m²
- COMMUNAL AMENITY AREA - 681.3 m²
TOTAL AMENITY AREA PROVIDED : 993.3 m²

ACCESSORY BUILDING	R4Z	PROVIDED:
BUILDING HEIGHT (MAX.):	4.5 m	3.6 m
FLOOR AREA (MAX.) :	200.0 m²	62.90 m²

TERRACE FLATS PARKING :
PARKING REQUIRED : 1.2 Spaces / (48) d.u. + 0.2 / (48) d.u. (Visitor) = 57.6 + 9.6 = 67.2 Spaces
PARKING PROVIDED : 58 Spaces + 10 Visitor Spaces = 68 Spaces

BICYCLE PARKING REQUIRED : 48 (0.5 / (96) d.u.) = 24.0 Spaces
BICYCLE PARKING PROVIDED : 40 Interior Spaces

TWO STOREY - THREE LEVEL TOWNHOMES :

BLOCK No. :		BUILDING AREA:	GROSS FLOOR AREA:	No. UNITS:
BLOCK 1 =	TERRACE FLATS	412.0 m²	1,219.0 m²	12 UNITS
BLOCK 2 =	TERRACE FLATS	412.0 m²	1,219.0 m²	12 UNITS
BLOCK 3 =	TERRACE FLATS	412.0 m²	1,219.0 m²	12 UNITS
BLOCK 4 =	TERRACE FLATS	412.0 m²	1,219.0 m²	12 UNITS
BICYCLE / GARBAGE =		62.9 m²		
TOTAL =		1,710.9 m²	4,876.0 m	48 UNITS

SNOW STORAGE : SNOW STORAGE WILL BE OFF SITE.

NOTE:
SITE PLAN TO BE READ IN CONJUNCTION WITH :
- SITE SERVICING PLAN PREPARED BY ______________.
- LANDSCAPING PLAN PREPARED BY ______________.
- BOUNDARIES DERIVED FROM: PLAN _M - ___ PART OF LOTS 28 AND 29 CONCESSION 12, GEOGRAPHIC TOWNSHIP OF GOULBOURN, CITY OF OTTAWA PLAN PREPARED BY ANNIS O'SULLIVAN VOLLEBEKK LTD. DATED JUNE 10, 2019 REV. No. 3.

LEGEND:
D.C. - DEPRESSED CURB
- WALL MOUNT LIGHT FIXTURE

M. David Blakely
Architect Inc.
2200 Prince of Wales Dr. - Suite 101
Ottawa, Ontario K2E 6Z9
Phone (613) 226-8811 Fax (613) 226-7942

GENERAL NOTES:
1. THE CONTRACTOR IS RESPONSIBLE FOR CHECKING AND VERIFYING ALL DIMENSIONS, ANY DISCREPANCY MUST BE REPORTED TO M. DAVID BLAKELY ARCHITECT INC.
2. ALL WORK AND MATERIALS TO BE IN COMPLIANCE WITH ALL CODES, REGULATIONS, AND BY-LAWS.
3. ADDITIONAL DRAWINGS MAY BE ISSUED FOR CLARIFICATION TO ASSIST THE PROPER EXECUTION OF WORK. SUCH DRAWINGS WILL HAVE THE SAME MEANING AND INTENT AS IF THEY WERE INCLUDED WITH THE PLANS IN CONTRACT DOCUMENTS.
4. DO NOT SCALE DRAWINGS.
5. THIS DRAWING SHALL NOT BE USED FOR PERMIT OR CONSTRUCTION UNLESS THE DRAWING BEARS THE ARCHITECT'S SEAL AND SIGNATURE.
6. THIS REPRODUCTION SHALL NOT BE ALTERED

CONSTRUCTION NORTH

SEAL

No.	DATE	DESCRIPTION	INIT.
10.			
9.			
8.	17/03/21	RELOCATED BLKS. 3, 4 & SIDEWALK	SM
7.	09/02/21	ACCESSORY BLDG. DETAIL ADDED	SM
6.	22/01/21	RELOCATED BLOCKS 3 & 4, 0.5m WEST	SM
5.	07/12/20	REVISED AS PER RICHCRAFT	SM
4.	05/11/20	REVISED AS PER RICHCRAFT	SM
3.	10/09/20	GATEWAY FEATURE ADDED	SM
2.	31/08/20	FOR REVIEW	SM
1.	26/05/20	FOR REVIEW	SM

REVISIONS

No.	DATE	DESCRIPTION	INIT.
20.			
19.			
18.			
17.			
16.			
15.			
14.			
13.			
12.			
11.			

REVISIONS

PROJECT
BLOCK 29, KANATA WEST
PLANNED UNIT DEVELOPMENT
OTTAWA, ONT.

CLIENT
RICHCRAFT
Group Of Companies

DRAWING TITLE
SITE PLAN

DATE: MAY., 2020.
SCALE: 1:300
SHEET No.: SP-1
DRAWN BY: SBM
CHECKED: MDB

Appendix F- DRAWINGS