Shell Canada Ltd.

Site Servicing Report
5 Orchard Drive, Ottawa

Hazeldean Road and Fringewood Drive

Prepared by:

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Date: April, 2020

Project #: 60593779

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Kelvin Wong
Shell Canada Ltd.
400 – 4 Avenue SW
Calgary, AB T2P 0J4

April 3, 2020

Project #
60593779

Dear Mr. Wong:

Subject: Site Servicing Report,
5 Orchard Drive, Ottaway
Hazeldean Road and Fringewood Drive

AECOM is pleased to submit herewith the servicing report supporting the site development application for the proposed Shell Station on the southwest corner of Hazeldean Road and Fringewood Drive in the west end of Ottawa.

The report has been prepared following a pre-application consultation with the City of Ottawa undertaken in July 2019. The design for gas station is outlined in within this report including the water servicing, sanitary and storm pipe sizing, stormwater management design and erosion and sediment control measures proposed for construction.

We trust the site servicing information provided is sufficient to support the Site Plan application. If you have any questions, comments or concerns, please do not hesitate to contact the undersigned at your convenience.

Sincerely,
AECOM Canada Ltd.

Rike Brown,P. Eng., LEED AP
Water Resources Engineer
rike.brown@aecom.com

RB:rb
Encl.
cc:

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This Statement of Qualifications and Limitations is attached to and forms part of the Report and any use of the Report is subject to the terms hereof.

AECOM: 2015-04-13

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1. Introduction

AECOM has been retained by Shell Canada Ltd. (Shell) to complete the grading, site servicing plans and memo and stormwater design in support of the development of a proposed gas station, convenience store, car wash and other associated amenities in the City of Ottawa. The site is located on the southwest corner of the intersection of Hazeldean Road and Fringewood Drive. Throughout this report, the proposed Shell gas station is referred to as the subject site.

1.1 Background Documents

The subject site is 0.306 ha (3065 m2) of a larger site plan by Campanale Homes at 5 Orchard Drive. The larger site plan has been presented to the City as part of the Draft Plan of Subdivision within the report entitled Functional Servicing and Stormwater Management Report for Campanale Homes 5 Orchard Drive (DSEL, March 2019). The 2019 Functional Servicing Report demonstrated that the servicing of the entire proposed development could be undertaken as per the City of Ottawa design standards. The overall site includes residential and commercial area and the subject site is part of the commercial area. The total commercial area is 1.82 ha with the subject site being one-sixth of that area. Shell will be leasing the property for the subject site from Campanale Homes and has agreed to submit an independent site servicing design to the City.

The following background documentation was referenced in the preparation of this memo:

- *Functional Servicing and Stormwater Management Report for Campanale Homes 5 Orchard Drive (DSEL, March 2019)*
- *Ottawa Sewer Design Guidelines (October 2012) and Technical Bulletins ISDTB-2014-01 (February 2014), PIEDTB-2016-01 (September 2016) and ISTB-2018-01 (March 2018)*
- *Ottawa Design Guidelines – Water Distribution and Technical Bulletins ISD-2010-2 (December 2010), ISDTB-2014-02 (May 2014) and ISDTB-2019-02 (March 2018)*
- *Stormwater Management Planning and Design Manual (Ontario Ministry of the Environment, 2003)*

1.2 Existing Conditions and Infrastructure

The subject site is located on the southwest corner of the intersection of Hazeldean Road and Fringewood Drive (see Figure 1.1). Under current conditions, the site drains from southwest to northeast toward the intersection. Runoff from the site is captured in one of two ditch inlet catch basins (DICBs) servicing the entire development area. These DICBs are connected into the existing Hazeldean Road storm sewer system. There are no existing water and sanitary services on the subject site itself. The 2019 Functional Servicing Report identified the existing sewer and watermain services within the municipal right-of-ways adjacent the subject site and the report is provided in **Appendix A**.

Figure 1.1: Existing Site Location

1.3 Consultation and Permits

Pre-consultation meeting was undertaken with the City of Ottawa in July 16, 2019. The meeting was coordinated with the review and consultation process associated with the entire development area and the 2019 Functional Servicing Report. The minutes from the pre-consultation meeting are included in **Appendix B**.

Development of the site may proceed subject to compliance and approval from the City of Ottawa Planning and Development Approvals process. Approval can be obtained following the submission of detailed engineering drawings, reports and calculations supporting the development of the subject site following review and the satisfaction of City of Ottawa staff.

A tree conservation memo has been produced for the subject site entitled *5 Orchard Drive = Tree Conservation Report Memo* (WSP, February 13, 2020). The memo outlines the inventory of tree to be impacted by the development of the subject site, mitigation measures and tree compensation suggestions. At this time, there is no anticipated need for a tree removal permit from the City. A copy of the memo accompanies the application package.

The Development Servicing Study Checklist in support of this servicing application is provided in **Appendix C**.

2. GEOTECHNICAL CONSIDERATIONS

A geotechnical investigation was undertaken for the subject site and documented in the report entitled *Draft Geotechnical Investigation Report Proposed Shell Service Station 5 Orchard Drive Ottawa Ontario* (Gemtec, June 2019). The recommendations from that report were incorporated into the design of the subject site. A copy of the report accompanies the application package.

3. DEVIATIONS

There are no deviations from the City of Ottawa guidelines as part of the design of the subject site.

4. Water Servicing

4.1 Design Criteria

The design criteria applied to the water servicing design followed the Ottawa Water Distribution Guidelines and associated Technical Bulletins. A summary of the applicable criteria is presented below:

- Commercial Space = 28,000 L/ha/d
- Commercial Maximum Daily Demand = 1.5 x average day L/gross ha/d
- Commercial Maximum Hour Demand = 1.8 x average day L/gross ha/d
- Minimum Watermain Size = 150 mm diameter
- Minimum Depth of Cover = 2.4 m from top of watermain to finished grade
- During normal operating conditions desired operating pressure is within the range of 350 kPa and 480 kPa
- During normal operating conditions pressure must not drop below 275 kPa
- During normal operating conditions pressure shall not exceed 552 kPa
- During fire flow operating pressure must not drop below 140 kPa

The anticipated water demand and required minimum and maximum water pressures are based on the Ottawa Water Distribution Guidelines. Boundary conditions for the subject site were provided by the City of Ottawa and obtained from the 2019 Functional Servicing Report (see **Appendix A**). A summary of the water demand information is provided below:

The consumption rate was first estimated based on the Ottawa Water Distribution Guidelines for Commercial Space, using 28,000 L/ha/d. The area of the subject site is 0.3065 Ha, giving an average daily flow rate of 8,582 L/day or 5.96 L/min. Applying the peak factors noted in Section 4.1, the Maximum Day and Maximum Hour demand are The above estimation is considered low for this site due the car wash which will be part of the development. The anticipated Maximum hour flow, based on similar car wash facilities, is in the range of 5,000 L/Hour, or 83L/min calculated as 8.94 L/min and 10.71 L/min, respectively.

Per the Ottawa Water Distribution Guidelines and Technical Bulletin **ISTB-2018-02**, the FUS method was used to estimate the required fire flow for the subject site. Below is a summary of the calculation:

F = 220C(A)^0.5

Where:
C = 0.8
A = 168 sq.m. + 112 sq.m. = 280 sq.m.
F = 220 (0.8) (280)^ 0.5
F = 3000 L/min
Add a separation charge of 30%
F = 4000 L/min (rounded to nearest 1000)

From the 2019 Functional Servicing Report, an assumed commercial fire flow requirement of 15,000 L/min was used for the entire commercial development, including the subject site. The report includes simulations performed using EPANET that show:

- During Peak Hour Demand, the operating pressure does not drop below 140 kPa, and
- During Maximum Day Demand plus Fire Flow, operating pressure does not drop below 140 kPa.

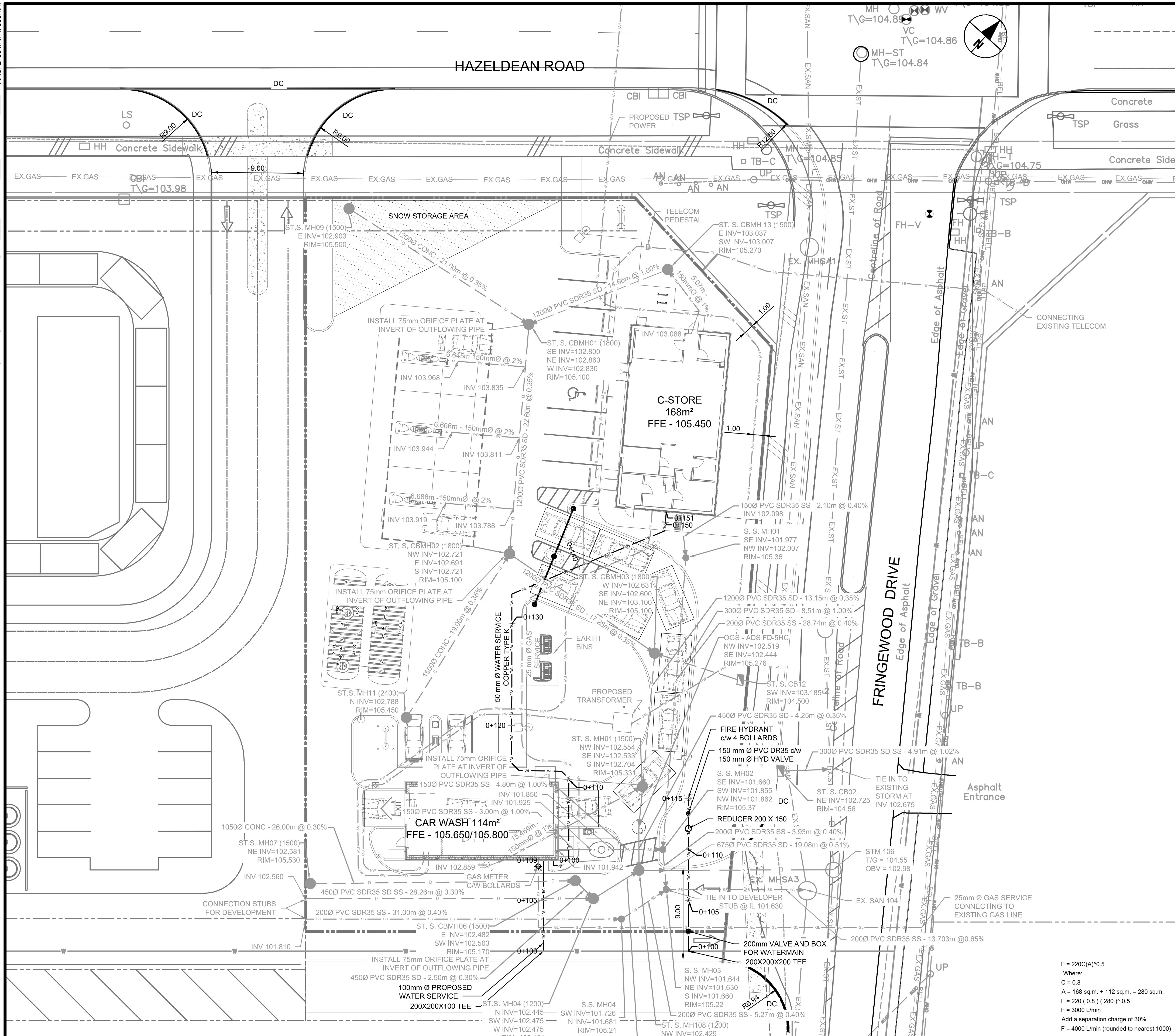
4.2 Proposed Servicing

The 2019 Functional Servicing Report identified the existing 203 mm diameter watermain located in Fringewood Drive as the water supply point for the proposed residential and commercial development, via a new looped (double connection) 200 mm diameter watermain. The subject site will be connected to the northern portion of this new watermain, a section of which will be constructed as part of the subject site. The existing watermain in Fringewood Drive is located within the City of Ottawa 3W pressure zone.

The proposed water service for the subject site is presented on **Drawing C801.0** (Figure 4.1). A portion of the proposed 200 mm diameter watermain for the residential and commercial area is proposed to be extended west from Fringewood Drive along the south portion of the subject site. The subject site is proposed to be serviced off this watermain via two (2) connections: a 200 mm diameter pipe extended to the north for a proposed fire hydrant; and a 100 mm diameter pipe to service the car wash and building The minimum pipe size on the site is 50 mm diameter.

4.3 Summary and Conclusions

From Sections 4.1 to 4.3, water service calculations provided in 2019 Functional Servicing Report (Appendix A) and Drawing C801.0, the water servicing design for the subject site meets the City of Ottawa requirements.



WATER SERVICING PLAN

Scale 1:200

NOTES

1. FOR GENERAL NOTES SEE DRAWING C001.0

LEGEND

- PROPOSED LEASE AND PROPERTY LINE
- PROPOSED STORMWATER / CATCH BASIN MANHOLE / MANHOLE — ST.S. CBMH / ST.S MH
- PROPOSED SANITARY MANHOLE — S.S. MH
- PROPOSED STORMWATER CATCH BASIN — ST.S. CB
- PROPOSED OGS ADS FD-5HC
- PROPOSED HYDRANT AND VALVE — HYD VB
- PROPOSED LIGHT STANDARD
- SANITARY PROCEPTOR
- PROPOSED GAS LINE — GL
- PROPOSED WATER LINE — WL
- PROPOSED RAIN WATER COLLECTOR — RW
- PROPOSED TELECOM — TE
- PROPOSED SANITARY SEWER — SS
- PROPOSED STORM SEWER — D
- PROPOSED POWER LINE — PW
- EXISTING STORM LINE — EX.ST
- EXISTING SANITARY LINE — EX.SAN
- EXISTING GAS LINE — EX.GAS
- EXISTING TELECOMMUNICATION — BELL
- EXISTING WATER LINE — W
- GAS METER
- WATER VALVE

PROPOSED 200mmØ WATERMAIN TABLE (TO FIRE HYDRANT)

STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100	105.156	102.000	CONNECTION 200 X 200 X 200 TEE
0+101.96	105.153	102.000	WATER VALVE WITH BOX
0+102.32	105.153	102.000	GAS LINE CROSSING
0+106.20	105.147	102.000	SANITARY LINE CROSSING
0+107.93	105.144	102.000	STORM LINE CROSSING
0+112.03	105.307	102.000	REDUCER 200 X 150
0+113.47	105.315	102.000	HYDRANT VALVE
0+115.00	105.309	102.000	FIRE HYDRANT

PROPOSED 100mmØ WATERMAIN TABLE (TO CAR WASH)

STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100.00	105.300	102.000	200 X 200 X100 TEE
0+102.290	105.239	102.000	GAS LINE CROSSING
0+103.150	105.218	102.000	SANITARY LINE CROSSING
0+105.150	105.178	102.000	STORM LINE CROSSING
0+106.680	105.175	102.000	STORM LINE CROSSING
0+109.00	105.800	102.000	CONNECTION TO CAR WASH

PROPOSED 50mmØ WATERMAIN TABLE (CAR WASH TO C-STORE)

STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100	105.800	102.000	CONNECTION FROM CAR WASH
0+100.50	105.532	102.000	90° HORIZONTAL BEND
0+101.29	105.535	102.000	45° HORIZONTAL BEND
0+102.29	105.552	102.000	45° HORIZONTAL BEND
0+103.08	105.590	102.000	SANITARY LINE CROSSING
0+110.10	105.620	102.000	45° HORIZONTAL BEND
0+111.11	105.609	102.000	45° HORIZONTAL BEND
0+115.30	105.629	102.000	45° HORIZONTAL BEND
0+116.310	105.465	102.000	45° HORIZONTAL BEND
0+121.210	105.343	102.000	POWER LINE CROSSING
0+132.91	105.158	102.000	45° HORIZONTAL BEND
0+136.750	105.197	102.000	STORM LINE CROSSING
0+140.650	105.252	102.000	POWER LINE CROSSING
0+147.620	105.382	102.000	GAS LINE CROSSING
0+149.100	105.414	102.000	45° HORIZONTAL BEND
0+151.00	105.450	102.000	CONNECTION TO C-STORE

$F = 220C(A)^{0.5}$

Where:

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$A = 168$ sq.m. + 112 sq.m. = 280 sq.m.

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F = 3000 L/min

Add a separation charge of 30%

F = 4000 L/min (rounded to nearest 1000)

0 5 10 m
1:200

AECOM

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REGISTRATION

LEGAL DESCRIPTION

PART OF BLOCK 21 OF DRAFT PLAN OF SUBDIVISION OF PARTS OF LOTS 26 AND 27 CONCESSION 11 GEOGRAPHIC TOWNSHIP OF GOULBOURN (CITY OF OTTAWA)

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A	2020-03-31	ISSUED FOR SERVICING REPORT

DRAWN BY

SG

KEY PLAN

HAZELDEAN ROAD
FRINGEWOOD DRIVE
CANOPY
C-STORE
CAR WASH
N

GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

WATER SERVICING PLAN

AECOM FILE NAME

C801.0-SSP-HZLX

SHEET NUMBER

C801.0

ANSI D 864mm x 559mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____

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5. Sanitary Servicing

5.1 Design Criteria

The following summarizes the parameters, as per the Ottawa Sewer Design Guidelines, used to design the sanitary sewer system for the subject site:

- Average Daily Demand = 280 L/d/per
- Peaking Factor = Harmon’s Peaking Factor Max 3.8 and Min 2.0
- Commercial Floor Space = 28,000 L/ha/d
- Infiltration and Inflow Allowance = 0.33L/s/ha
- Sanitary sewers sized using Manning’s Equation
- Commercial Peaking Factor = 1.50
- Minimum Sanitary Sewer Lateral = 135 mm diameter
- Minimum Manning’s n = 0.013
- Minimum Depth of Cover = 2.5 m from crown of sewer to grade
- Minimum Full Flowing Velocity = 0.6 m/s
- Maximum Full Flowing Velocity – 3.0 m/s

5.2 Proposed Sanitary Servicing and Calculations

The anticipated peak flow from the proposed subject site is provided below and supporting calculations are provide in **Appendix D**:

- Average Dry Weather Flow Rate = 0.44 L/s
- Peak Dry Weather Flow Rate = 0.89 L/s
- Peak Wet Weather Flow Rate = 1.49 L/s

The estimated sanitary flow based on the proposed subject site and associated uses is anticipated to be a wet weather flow of 1.49 L/s.

The sanitary sewer connection for the entire commercial site was proposed, in the 2019 Functional Servicing Report, to be serviced via a 250 mm diameter sanitary pipe in Fringewood Drive. This sanitary system was constructed in 2019. The Fringewood Drive sanitary sewer connects to the 675 mm diameter sanitary sewer within Hazeldean Road. The Hazeldean Road system is tributary to the Kanata West Pumping Station.

The proposed sanitary servicing of the subject site is presented in **Drawing C802.0** (Figure 5.1). The connection of the entire commercial site is proposed to be connected to the recently constructed (2019) 250 mm diameter sanitary system within Fringewood Drive. From this existing sanitary system, a 200 mm diameter pipe to service the entire commercial site is proposed to be extended west through the southern portion of the subject site. Service for the subject site is via a 200 mm diameter sanitary sewer extended north into the subject site. The smallest diameter sanitary pipe proposed on the subject site is a 150 mm diameter pipe. Calculations are presented on the attached Sanitary Sewer Design spreadsheet and were prepared using Manning's Equation. The sanitary sewer spreadsheet supporting the design of the subject site is presented in **Appendix D**.

5.3 Summary and Conclusions

From the above, calculations provided in **Appendix E** and **Drawing C802.0**, the sanitary sewer design for the subject site meets the City of Ottawa requirements.

ANSI D 864mm x 559mm

Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____

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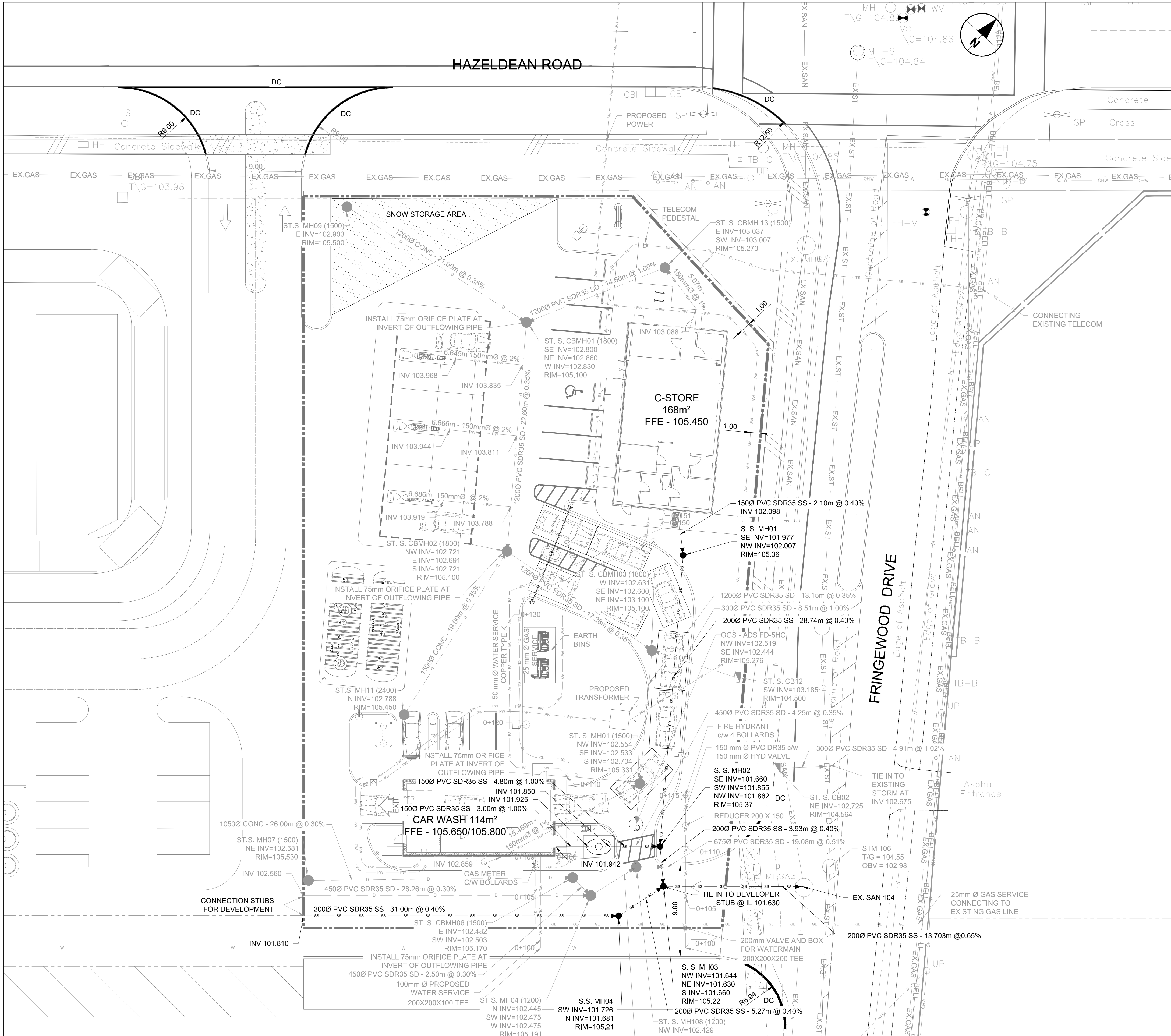

SANITARY SERVICING PLAN

Scale 1:200

NOTES

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PROPOSED WATER LINE	WL
PROPOSED RAIN WATER COLLECTOR	RW
PROPOSED TELECOM	TE TE
PROPOSED SANITARY SEWER	SS
PROPOSED STORM SEWER	D
PROPOSED POWER LINE	PW PW
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EXISTING SANITARY LINE	EX.SAN
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WATER VALVE	

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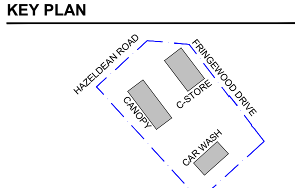

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SANITARY SERVICING PLAN

AECOM FILE NAME

C802.0-SSP-HZLX

SHEET NUMBER

C802.0

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6. Storm Servicing and Stormwater Management

6.1 Background

The subject site is tributary to the Hazeldean Road storm sewer which discharges into the interim Hazeldean Road Stormwater Facility. The facility ultimately discharges into the Carp River which is in the jurisdiction of the Mississippi Valley Conservation Authority (MVCA).

The runoff from the subject site is captured by one of two existing DICBs on the site that discharge into the existing Hazeldean Road storm sewer (see Existing Conditions and Existing Stormwater Drainage plans from the 2019 Functional Servicing Report in **Appendix A**). Existing topography of the site indicates that the drainage is from the south to the north, but also west to east toward the Fringewood Drive and Hazeldean Road intersection. Details of the site drainage for the subject site plus the surrounding related development (proposed commercial and south residential area) was presented in the 2019 Functional Servicing Report (**Appendix A**).

As part of the assessment of the entire development area referred to as 5 Orchard Drive and reported in the 2019 Functional Servicing Report, a hydraulic evaluation was undertaken of the downstream recipient storm sewer within Hazeldean Road. The purpose of the assessment was to establish the available capacity within the downstream system for the development site. Based on the results of the analysis, it was determined that the downstream system has only 251.9 l/s for the entire development site (5 Orchard Drive – total commercial and residential areas) during the 100-year storm event. From the 2019 Functional Servicing Report, 200 l/s was assigned for residential design and the remainder 51.9 l/s was assigned to the commercial area assuming a site runoff coefficient of C=0.9.

The total commercial area (including the subject site) is 1.82 ha and therefore the level of service unit rate is 28.5 l/s/ha. The level of service was confirmed by the developer and their engineer and the communication is provided in **Appendix E**.

6.2 Storm Servicing Strategy

The storm servicing strategy for the subject site includes:

- Site grading to maintain sufficient site lines and tie-in with surrounding right-of-way and areas;
- Minimize cut and fill earth operations;
- Reduce or eliminate retaining walls, where feasible;
- Minimize impact to abutting properties;
- Site grading to contain the runoff up to the 100-year storm event within the property boundaries;
- Enable gravity servicing outlets;
- Conveyance of runoff to catchbasins strategically located throughout the site;
- Grading of low points for ponding to a maximum of 0.3 m at catchbasin locations, where feasible;

- Storm sewers through site sized for the 5-year Rational Method flow;
- All site storm sewers will be conveyed to an oil grit separator for capture of any potential on-site spills;
- All roof drains directed to a storm sewer;
- On-site storage provided for all storm events up to, and including, the 100-year storm event;
- Emergency overflow route provided for the subject site out to Fringewood Drive;
- Storm sewer from subject site connected into existing storm sewer in Fringewood Drive.

6.3 Proposed Storm Servicing

6.3.1 Design Criteria (Minor and Major System)

The following Ottawa Sewer Design Guidelines (OSDG) criteria was applied to the subject site for storm servicing:

- Intensity Duration Frequency curves as per OSDG Section 5.4.2;
- Evaluation of the system for climate change (100-year storm event increased 20%);
- Runoff Coefficients as per OSDG Section 5.4.5.2.1 and Table 5.7 (discussed in further detail in Section 6.3.2);
- Time of concentration 10 minutes for most upstream drainage area of each sewer run;
- Storm sewer minor flow should be controlled to meeting the existing recipient sewer level of service or runoff from the design storm (2 or 5 year), whichever is less.
- On-site storage provided for all storm events up to, and including, the 100-year event;
- Water quality and quantity control provided by the Interim Hazeldean Stormwater Facility (capacity for subject site demonstrated in the 2019 Functional Servicing Report, Appendix A);
- Particle size distribution for oil grit separator sizing provided in Appendix E;
- Minimum inlet control device size is 83 mm (round) or a minimum flowrate of 6 l/s for a Vortex-type unit (as per the City Approved Sewer and Miscellaneous Products Listing);
- Stormwater quantity criteria should be consistent with the approved Servicing and Stormwater Report for the area;
- Storm sewer pipe size gravity capacity determined using Manning's equation;
- Manning's n value = 0.013;
- Minimum storm sewer pipe used is 250 mm diameter;
- Minimum storm sewer slope = 0.1%;
- Minimum storm sewer velocity = 0.8 m/s;
- Maximum storm sewer velocity = 3.0 m/s;

- Storm sewer diameter and minimum slope as per OSDG Table 6.1;
- Depth of cover (pipe obvert to finished grade) is 2.0 m, but no less than 1.0 m;
- Storm sewers match obvert to obvert where practical, lowered in some areas to increase cover;;
- On-site ponding depth not to exceed 350 mm under static or dynamic conditions; and,
- The emergency overflow spill elevation 300 mm below lowest building opening.

6.3.2 Runoff Coefficient

In support of the proposed storm sewer sizing and stormwater management design, the following runoff coefficients were utilized for different land uses found on the subject site:

Table 6.1: Summary of the Runoff Coefficients used for Storm Sewer Sizing of Subject Site

Proposed Land Use	Runoff Coefficient, C	Runoff Coefficient for 100 year Storm Event*
Pavement - Asphalt	0.9	1.0
Building Area	0.9	1.0
Grass – Vegetation	0.3	0.38

*Notes: * As per the Ottawa Sewer Design Guidelines, the runoff coefficient should be increased by 25% for the 100 year storm event up to a maximum runoff coefficient of 1.0.*

*** Based on the soil type (silt and some clay), the runoff coefficient of 0.3 was selected.*

6.3.3 Minor System and Allowable Capture Rate

The site outflow rate was established as part of the 2019 Functional Servicing Report and discussed in **Section 6.1**. As noted in **Section 6.1**, the developer and their engineer provided an allowable outflow rate of 28.5 l/s/ha (see email communication in **Appendix E**). Based on the subject site area of 0.306 ha plus 0.027 ha of external area (total 0.333 ha), the allowable storm outflow during the 100 year storm event is 9.5 l/s. Therefore, post-development runoff from the 100-year storm event is to be fully contained within the site prior to discharge at the outflow rate of 9.5 l/s.

The storm sewer system was sized using the 5 year Rational Method and the criteria noted in **Sections 6.3.1 and 6.3.2**. The supporting calculations are provided in **Appendix E**. The storm sewer system was designed with the dual purpose of conveying subject runoff to the recipient storm sewer in Fringewood Drive and providing underground or in-line storage for the subject site. Therefore, the some of the pipes proposed for the site are oversized. The size of storm pipes to service the site range from a 300 mm diameter PVC SDR35 to 1500 mm diameter concrete pipes. The storm sewer system is presented in **Drawing C803.0** (Figure 6.3) and the drainage area plan is presented in **Drawing C105.0** (Figure 6.2). Generally, the storm system captures flow via CBs and CBMH structures from the north portion of the subject site to the southwest corner. The majority of the storm system is directed to an oil grit separator prior to discharging to an east-west storm pipe (675 mm diameter) that connects into the existing storm system in Fringewood Drive. There is one branch of the storm sewer system servicing the subject site that connects directly into the sewer that will service the future commercial site (**Drawing C803.0,** MH07 to MH04).

The cover provided for the storm pipes throughout the site are between 1.0 m to 2.3 m. There is one instance where the cover is less than 1.0 m and it occurs along the downstream half of the 1500 mm diameter concrete pipe located between MH11 and CBMH02 (see **Drawing C803.0**). Insulation of along this pipe will be provided, as required.

To achieve the restricted outflow rate established for the site, inlet control devices (ICDs) are proposed in three locations throughout the site. The locations of these ICDs are indicated in **Drawing C803.0** (Figure 6.3). The ICDs proposed at all three locations are the Hydrovex® VHV Vertical Vortex Flow Regulator model 75 VHV-1. The units are listed on the City of Ottawa *Approved Sewer and Miscellaneous Products Listing* (MS-22.15 March 2019).

The dual purpose of the storm system to provide flow conveyance during frequent storm events and storage during less frequent storm events (100 year storm event), the storm sewer system was modeled hydraulically to confirm on-site storage (pipes and surface), site outflow targets, ICD function and the resultant hydraulic grade line. The details of the hydraulic assessment is presented in the *Shell Canada Stormwater Management Report 5 Orchard Drive, Stittsville, City of Ottawa* (AECOM, 2020) and included in **Appendix F**. This report is referred to herewith as the Site SWM Report.

Based on the resultant hydraulic modeling for the 2 to 100 year storm events, the storm sewers throughout the subject site are partially to totally submerged depending upon the event. Profiles of the main branch of the subject site storm sewer system for numerous storm events is presented in the Site SWM Report (**Appendix G**).

6.3.4 Major System

The design of the subject site has assumed all site runoff generated and not captured in the minor system will be stored on the surface or be conveyed to underground storage. It is anticipated that runoff up to, and including, the 100 year storm event will remain on-site. Available surface ponding area is discussed and documented in the Site SWM Report and included in **Appendix F**. The area and depth of available surface storage is provided in **Drawing C104.0** (Figure 6.1) and the depths range from a minimum of 0.08 m to a maximum of 0.13 m. Based on the hydraulic evaluation undertaken for the site, it was determined that the 100 year storage could be fully accommodated within the oversized storm sewer pipes and no surface storage will be utilized. Further details with respect to onsite storage utilization is provided in the Site SWM Report.

As part of the design of the stormwater design of the subject site, it was assumed that the future commercial area west of the site would be not discharge any major flow into the subject site. In other words, the proposed commercial site will contain all their runoff up to the 100 year storm event.

6.4 Stormwater Management

The stormwater management design was undertaken using the above storm servicing design and is documented in the Site SWM Report (entitled *Shell Canada Stormwater Management Report 5 Orchard Drive, Stittsville, City of Ottawa* (AECOM, 2020)) and included in **Appendix F**. The noted report details the evaluation undertaken in support of the stormwater measures proposed for the site, including surface and underground storage for quantity control and an oil grit separator for water quality control.

6.5 Temporary Works Required

To service the site during construction, a temporary diversion ditch is proposed to be installed around the perimeter of the site to collect and convey runoff to one of two storm outlets: catchbasins within Hazeldearn Road or a temporary connection to the storm sewer in Fringewood Drive. Further details related to the diversion ditch are provided in **Section 7** and on **Drawing C-101.0**.

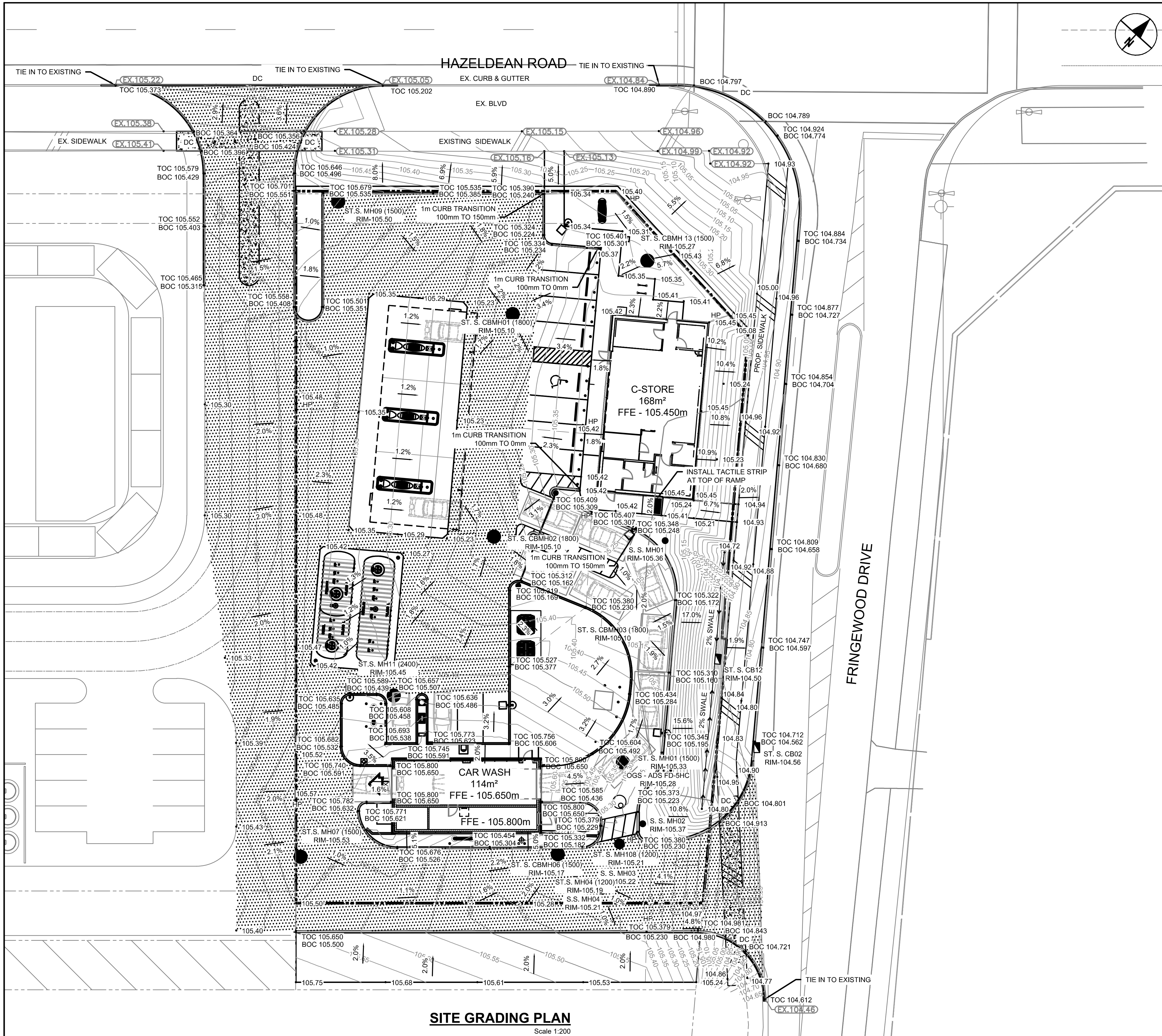
It is not anticipated that any of the temporary works would remain in place following construction of the subject site.

6.6 Summary and Conclusions

From **Sections 6.1 to 6.5**, the Site SWM Report (**Appendix F**), calculations supporting the storm pipe design (**Appendix E**) and **Drawings C104.0, C105.0 and C803.0**, the storm sewer and stormwater management design for the subject site meets the City of Ottawa requirements.

ANSI D 864mm x 559mm

Approved: ______ Checked: ______ Designer: ______ Project Management Initials: ______



SITE GRADING PLAN

Scale 1:200

0 5 10 m
1:200

NOTES

1. FOR GENERAL NOTES SEE DRAWING C001.0
2. CONTRACTOR IS RESPONSIBLE TO CONFIRM EXISTING GRADES IN FIELD.

LEGEND

PROPOSED GRADE	657.000
TOP OF CURB PROPOSED GRADE	TOC 657.150
BOTTOM OF CURB PROPOSED GRADE	BOC 657.000
EXISTING SURFACE GRADE	EX.105.15

PAVEMENT STRUCTURES:

LIGHT DUTY (NEW PAVEMENT)
50mm HL3 or SUPERPAVE 19.0 ASPHALTIC CONCRETE
150mm GRANULAR "A" BASE CRUSHED STONE
300mm GRANULAR "B" TYPE II SUBBASE
ASPHALT GRADE PG-58-34
*INSTALLED PER GEOTECHNICAL REPORT

HEAVY DUTY (NEW PAVEMENT)
40mm HL3 or SUPERPAVE 12.5 ASPHALTIC CONCRETE
50mm HL8 or SUPERPAVE 19.0 ASPHALTIC CONCRETE
150mm GRANULAR "A" BASE CRUSHED STONE
450mm GRANULAR "B" TYPE II SUBBASE
ASPHALT GRADE PG 58-34
* INSTALLED PER GEOTECHNICAL REPORT

PROPOSED LEASE AND PROPERTY LINE

ST.S. CBMH / ST.S MH — PROPOSED STORMWATER / CATCH BASIN MANHOLE / MANHOLE

S.S. MH — PROPOSED SANITARY MANHOLE

ST.S. CB — PROPOSED STORMWATER CATCH BASIN

PROPOSED OGS - ADS FD-5HC

HYD — PROPOSED FIRE HYDRANT

PROPOSED LIGHT STANDARD

EXISTING HYDRANT

DC — PROPOSED DEPRESSED CURB (AS PER SC7.1)

AECOM

PROJECT

Shell Canada Products
Hazeldean Road and
Fringewood Drive NTI

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REGISTRATION

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GEOGRAPHIC TOWNSHIP OF GOULBOURN
(CITY OF OTTAWA)

ISSUE/REVISION

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A	2020-03-31	ISSUED FOR SPA

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SG

KEY PLAN

GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

SITE GRADING PLAN

AECOM FILE NAME

C104.0-SGP-HZLX

SHEET NUMBER

C104.0

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STORM WATER MANAGEMENT PLAN

Scale 1:200

NOTES

1. ALL DIMENSIONS ARE IN METRES, UNLESS NOTED OTHERWISE.
2. FOR GENERAL NOTES SEE DRAWING C001.0.

LEGEND

- PROPOSED LEASE AND PROPERTY LINE
- ST.S. CBMH / ST.S MH — PROPOSED STORMWATER / CATCH BASIN MANHOLE / MANHOLE
- S.S. MH — PROPOSED SANITARY MANHOLE
- ST.S. CB — PROPOSED STORMWATER CATCH BASIN
- PROPOSED OGS - ADS-FD-5HC
- PROPOSED SUB-CATCHMENT BOUNDARIES
- A1 — POST - DEVELOPMENT AREA ID
- 0.058 Ha — POST - DEVELOPMENT DRAINAGE AREA (ha)
- 0.87 — 1:5 YEAR WEIGHTED RUNOFF COEFFICIENT
- UNCONTROLLED STORMWATER FLOW AREA
- EXTERNAL STORMWATER FLOW AREA

AREA STATEMENT (IN HECTARES)				
Sub Catchment	PAVED AREA (ha)	LANDSCAP AREA (ha)	ROOF TOP AREA (ha)	TOTAL AREA (ha)
A1	0.057	0.003		0.06
A2	0.078	0.008	0.02	0.106
A3	0.025	0.005		0.03
A4	0.006	0.007	0.017	0.03
A5	0.021	0.014		0.035
A6-1	0.001	0.006		0.007
A6-2	0.002	0.021		0.023
A7	0.004			0.004
A8 (External Area)	0.01	0.017		0.027
A9			0.011	0.011

0 5 10 m
1:200

AECOM

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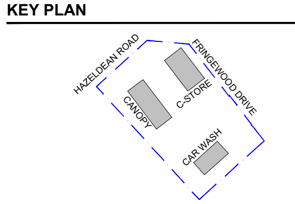
KEY PLAN



GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

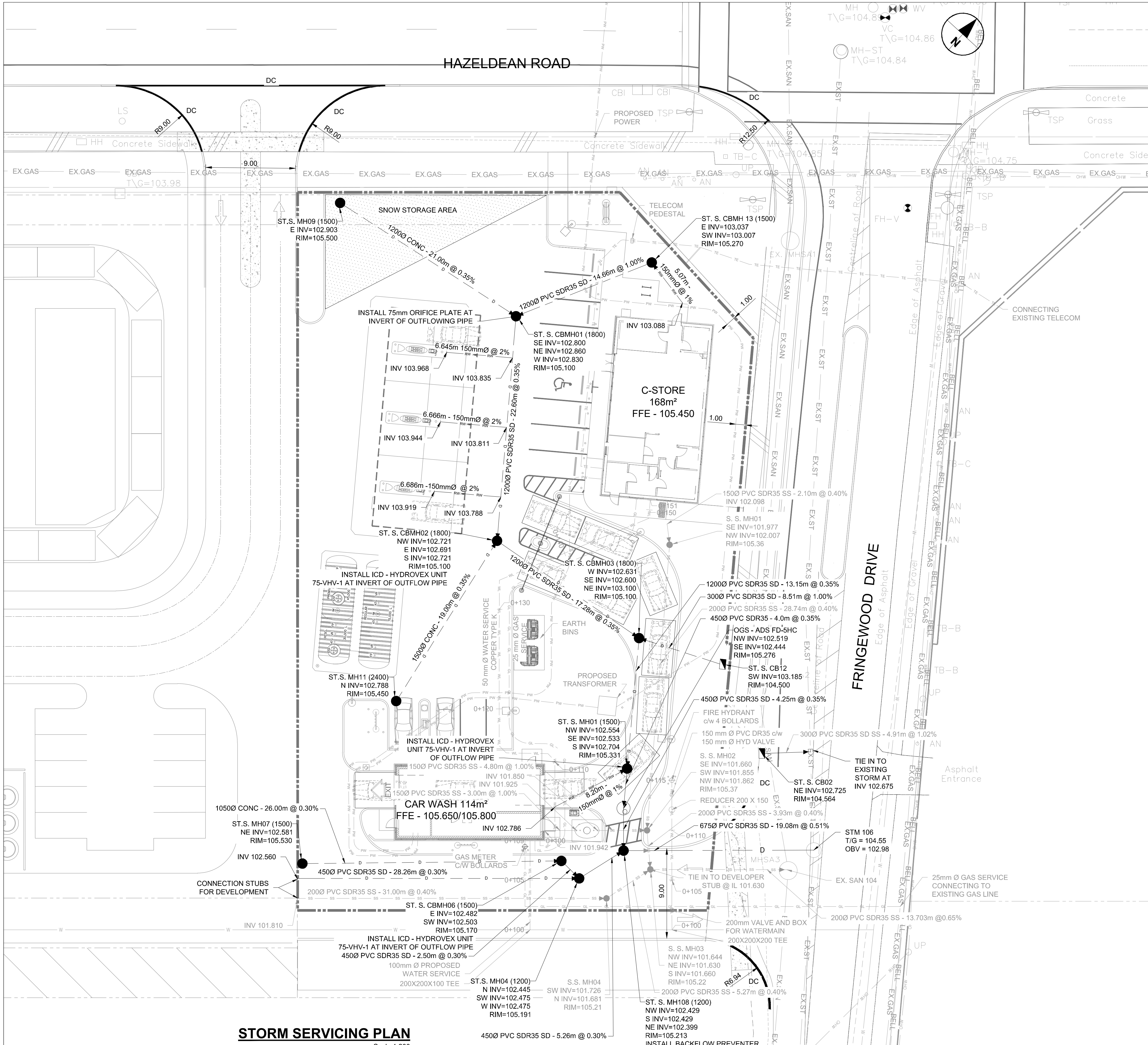
STORMWATER MANAGEMENT PLAN

AECOM FILE NAME

C105.0-SWM-HZLX

SHEET NUMBER

C105.0



NOTES

1. FOR GENERAL NOTES SEE DRAWING C001.0

LEGEND

PROPOSED LEASE AND PROPERTY LINE	
PROPOSED STORMWATER / CATCH BASIN MANHOLE / MANHOLE	ST.S. CBMH / ST.S MH
PROPOSED SANITARY MANHOLE	S.S. MH
PROPOSED STORMWATER CATCH BASIN	ST.S. CB
PROPOSED OGS ADS FD-5HC	
PROPOSED HYDRANT AND VALVE	HYD VB
PROPOSED LIGHT STANDARD	
SANITARY PROCEPTOR	
PROPOSED GAS LINE	GL
PROPOSED WATER LINE	WL
PROPOSED RAIN WATER COLLECTOR	RW
PROPOSED TELECOM	TE
PROPOSED SANITARY SEWER	SS
PROPOSED STORM SEWER	D
PROPOSED POWER LINE	PW
EXISTING STORM LINE	EX.ST
EXISTING SANITARY LINE	EX.SAN
EXISTING GAS LINE	EX.GAS
EXISTING TELECOMMUNICATION	BELL
EXISTING WATER LINE	W
GAS METER	
WATER VALVE	

PROPOSED 200mmØ WATERMAIN TABLE (TO FIRE HYDRANT)

STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100	105.156	102.000	CONNECTION 200 X 200 X 200 TEE
0+101.96	105.153	102.000	WATER VALVE WITH BOX
0+102.32	105.153	102.000	GAS LINE CROSSING
0+106.20	105.147	102.000	SANITARY LINE CROSSING
0+107.93	105.144	102.000	STORM LINE CROSSING
0+112.03	105.307	102.000	REDUCER 200 X 150
0+113.47	105.315	102.000	HYDRANT VALVE
0+115.00	105.309	102.000	FIRE HYDRANT

PROPOSED 100mmØ WATERMAIN TABLE (TO CAR WASH)

STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100.00	105.300	102.000	200 X 200 X100 TEE
0+102.290	105.239	102.000	GAS LINE CROSSING
0+103.150	105.218	102.000	SANITARY LINE CROSSING
0+105.150	105.178	102.000	STORM LINE CROSSING
0+106.680	105.175	102.000	STORM LINE CROSSING
0+109.00	105.800	102.000	CONNECTION TO CAR WASH

PROPOSED 50mmØ WATERMAIN TABLE (CAR WASH TO C-STORE)

STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100	105.800	102.000	CONNECTION FROM CAR WASH
0+100.50	105.532	102.000	90° HORIZONTAL BEND
0+101.29	105.535	102.000	45° HORIZONTAL BEND
0+102.29	105.552	102.000	45° HORIZONTAL BEND
0+103.08	105.590	102.000	SANITARY LINE CROSSING
0+110.10	105.620	102.000	45° HORIZONTAL BEND
0+111.11	105.609	102.000	45° HORIZONTAL BEND
0+115.30	105.629	102.000	45° HORIZONTAL BEND
0+116.310	105.465	102.000	45° HORIZONTAL BEND
0+121.210	105.343	102.000	POWER LINE CROSSING
0+132.91	105.158	102.000	45° HORIZONTAL BEND
0+136.750	105.197	102.000	STORM LINE CROSSING
0+140.650	105.252	102.000	POWER LINE CROSSING
0+147.620	105.382	102.000	GAS LINE CROSSING
0+149.100	105.414	102.000	45° HORIZONTAL BEND
0+151.00	105.450	102.000	CONNECTION TO C-STORE

0 5 10 m
1:200

AECOM

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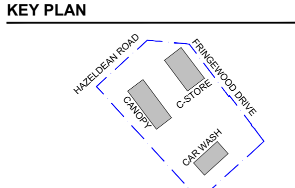
SG

KEY PLAN



GLOBAL PROJECT ID NUMBER
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SHEET TITLE
STORM SERVICING PLAN

AECOM FILE NAME
C803.0-SSP-HZLX

SHEET NUMBER
C803.0

ANSI D 864mm x 559mm
Approved: ____
Checked: ____
Designer: ____
Project Management Initials: ____
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7. Sediment and Erosion Control

Sediment and erosion control measures are presented on **Drawing C101.0** (Figure 7.2). The following mitigation measures are proposed during the construction phase of the subject site servicing:

- Heavy duty silt fence per OPSD 219.110 around the perimeter of the site;
- Diversion ditch around the perimeter of the site;
- Erosion control blankets within the diversion ditch;
- Rock flow check dams per OPSS Prov 804 at numerous locations in the diversion ditch;
- Sediment control mud mat at the designated construction entrance off Fringewood Drive;
- Temporary culvert crossing (500 mm diameter) at the mud mat;
- Filter fabric installed below grate on existing catchbasins within the area; and,
- Sediment traps within the diversion ditches per OPSD 219.220 at numerous locations.

To facilitate construction, access will be via Fringewood Drive. A diversion ditch is proposed around the perimeter of the site to capture and convey runoff from the site to either an existing catchbasin on Hazeldean Road or a temporary connection to the existing storm sewer manhole in Fringewood Drive. Rock check dams and sediment traps are located at strategic locations within the diversion ditch to retain and allow sediment deposition prior to discharging into either system. To protect the side slopes of the diversion ditch, erosion control blankets are proposed.

The sediment and erosion control plan presented will be used as a base for the contractor who is responsible for installation, maintenance and monitoring of the measures during construction. The contractor will be responsible to adjust and repair any sediment and erosion control measures as required. The contractor's installation, inspection and maintenance requirements are captured on the general notes and erosion and sediment control plan presented on **Drawings C001.0** (Figure 7.1) **and C101.0** (Figure 7.2), respectively.

GENERAL NOTES

1. COORDINATE AND SCHEDULE ALL WORK WITH OTHER TRADES AND CONTRACTORS.
2. THE POSITION OF ALL POLE LINES, CONDUITS, WATERMAINS, SEWERS AND OTHER UNDERGROUND AND OVERGROUND UTILITIES AND STRUCTURES IS NOT NECESSARILY SHOWN ON THE CONTRACT DRAWINGS, AND WHERE SHOWN, THE ACCURACY OF THE POSITION OF SUCH UTILITIES AND STRUCTURES IS NOT GUARANTEED. BEFORE STARTING WORK, DETERMINE THE EXACT LOCATION, SIZE, MATERIAL AND ELEVATION OF ALL EXISTING UTILITIES AND STRUCTURES AND ASSUME ALL LIABILITY FOR DAMAGE TO THEM.
3. ALL WORKS SHALL BE COMPLETED IN ACCORDANCE WITH THE CURRENT OCCUPATIONAL HEALTH AND SAFETY ACT AND REGULATIONS FOR CONSTRUCTION PROJECTS. THE GENERAL CONTRACTOR SHALL BE DEEMED TO BE THE CONSTRUCTOR AS DEFINED IN THE ACT.
4. THE CONTRACTOR AND SUB CONTRACTORS ARE RESPONSIBLE TO ENSURE THAT THEIR CONSTRUCTION MATERIALS AND PRACTICES CONFORM TO THE LATEST CITY/ REGION STANDARDS, SPECIFICATIONS AND DESIGN CRITERIA. IN THE ABSENCE OF CITY/REGIONAL SPECIFICATIONS, THE ONTARIO PROVINCIAL STANDARD SPECIFICATIONS (OPSS) SHALL APPLY.
5. OBTAIN ALL NECESSARY PERMITS AND APPROVALS FROM THE CITY OF OTTAWA BEFORE COMMENCING CONSTRUCTION.
6. THE CONTRACTOR IS RESPONSIBLE FOR DESIGNING AND IMPLEMENTING TEMPORARY TRAFFIC MANAGEMENT PLANS FOR CONSTRUCTION WITHIN THE CITY RIGHT OF WAY, ALL PLANS ARE TO FOLLOW THE REQUIREMENTS OF THE CITY AND PROVINCIAL STANDARDS (OTM BOOK 7).
7. BEFORE COMMENCING CONSTRUCTION OBTAIN AND PROVIDE PROOF OF COMPREHENSIVE, ALL RISK AND OPERATIONAL LIABILITY INSURANCE FOR $5,000,000.00. INSURANCE POLICY TO NAME OWNERS, ENGINEERS AND ARCHITECTS AS CO-INSURED.
8. THE CONTRACTOR, AT THEIR EXPENSE AND TO THE SATISFACTION OF THE CITY OF OTTAWA AND THE ENGINEER, SHALL BE RESPONSIBLE FOR THE RESTORATION AND THE REPAIR OF ALL AREAS DISTURBED DURING CONSTRUCTION ON-SITE AND OFF-SITE, INCLUDING TRENCHES AND EXISTING UTILITIES TO EXISTING CONDITIONS OR BETTER.
9. REMOVE FROM SITE ALL EXCESS EXCAVATED MATERIAL, ORGANIC MATERIAL AND DEBRIS UNLESS OTHERWISE INSTRUCTED BY ENGINEER. EXCAVATE AND REMOVE FROM SITE ANY CONTAMINATED MATERIAL. ALL CONTAMINATED MATERIAL SHALL BE DISPOSED OF AT A LICENSED LANDFILL FACILITY.
10. THE SUPPORT OF ALL UTILITIES SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE AUTHORITY HAVING JURISDICTION.
11. ALL BACKFILL FOR SEWERS, WATERMAINS AND UTILITIES ON THE ROAD ALLOWANCE MUST BE MECHANICALLY COMPACTED.
12. CONTRACTOR IS RESPONSIBLE FOR VERIFYING LOCATION AND ELEVATION OF ALL EXISTING UTILITIES AND CITY SERVICES (WATER, SANITARY & STORM) PRIOR TO CONSTRUCTION. ANY DISCREPANCIES MUST BE REPORTED TO AECOM LTD.
13. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE SPECIFIED.
14. ALL ELEVATIONS ARE GEODETIC.
15. REFER TO GEOTECHNICAL REPORT (PROJECT: 6399369 DATED JULY 3, 2019), PREPARED BY GEMTEC FOR SUBSURFACE CONDITIONS, CONSTRUCTION RECOMMENDATIONS, AND GEOTECHNICAL INSPECTION REQUIREMENTS. THE GEOTECHNICAL CONSULTANT IS TO REVIEW ON-SITE CONDITIONS AFTER EXCAVATION PRIOR TO PLACEMENT OF THE GRANULAR MATERIAL.
16. REFER TO ARCHITECT'S AND LANDSCAPE ARCHITECT'S DRAWINGS FOR BUILDING AND HARD SURFACE AREAS AND DIMENSIONS.
17. REFER TO STORMWATER MANAGEMENT REPORT PREPARED BY AECOM, DATED MARCH 27, 2020.
18. CONTRACTOR TO PROVIDE THE CONSULTANT WITH A GENERAL PLAN OF SERVICES INDICATING ALL SERVICING AS-BUILT INFORMATION SHOWN ON THIS PLAN. AS-BUILT INFORMATION MUST INCLUDE: PIPE MATERIAL, SIZES, LENGTHS, SLOPES, INVERT AND T/G ELEVATIONS, STRUCTURE LOCATIONS, VALVE AND HYDRANT LOCATIONS, T/WM ELEVATIONS AND ANY ALIGNMENT CHANGES, ETC.
19. A UTILITY CLEARANCE RADIUS OF 1.2M BETWEEN THE PROPOSED DRIVEWAY ENTRANCE CURB RETURN AND ALL ABOVE GROUND UTILITIES MUST BE MAINTAINED.
20. THE OWNER SHALL INDICATE IN THE AGREEMENT, IN WORDS SATISFACTORY TO BELL CANADA, THAT IT WILL GRANT TO BELL CANADA ANY EASEMENTS THAT MAY BE REQUIRED, WHICH MAY INCLUDE A BLANKET EASEMENT, FOR COMMUNICATION/TELECOMMUNICATION INFRASTRUCTURE. IN THE EVENT OF ANY CONFLICT WITH EXISTING BELL CANADA FACILITIES OR EASEMENTS, THE OWNER SHALL BE RESPONSIBLE FOR THE RELOCATION OF SUCH FACILITIES OR EASEMENTS.

GRADING NOTES

1. ALL CONSTRUCTION WORK FOR THIS PROJECT SHALL COMPLY WITH THE STANDARD DRAWINGS AND SPECIFICATIONS OF THE CITY OF OTTAWA, THE ONTARIO PROVINCIAL STANDARDS AND SPECIFICATIONS (O.P.S.S.) AND THE ONTARIO BUILDING CODE (O.B.C.)
2. ALL SURFACE DRAINAGE SHALL BE CONTAINED AT SITE, COLLECTED AND DISCHARGED AT A LOCATION TO BE APPROVED PRIOR TO THE ISSUANCE OF A BUILDING PERMIT. DRAINAGE OF ABUTTING PROPERTIES SHALL NOT BE ADVERSELY AFFECTED. UNLESS NOTED OTHERWISE.
3. ALL TOPSOIL, ORGANIC OR DELETERIOUS MATERIAL MUST BE ENTIRELY REMOVED FROM BENEATH THE PROPOSED PAVED AREAS AS DIRECTED BY THE SITE ENGINEER OR GEOTECHNICAL ENGINEER.
4. THE SUBGRADE SHALL BE COMPACTED TO AT LEAST 98% OF THE STANDARD PROCTOR MAXIMUM DRY DENSITY VALUE.
5. EXPOSED SUBGRADES IN PROPOSED PAVED AREAS SHOULD BE PROOF ROLLED WITH A LARGE STEEL DRUM ROLLER AND INSPECTED BY THE GEOTECHNICAL ENGINEER PRIOR TO THE PLACEMENT OF GRANULARS.
6. ANY SOFT AREAS EVIDENT FROM THE PROOF ROLLING SHOULD BE SUB-EXCAVATED AND REPLACED WITH SUITABLE MATERIAL THAT IS FROST RESISTANT AND COMPATIBLE WITH THE EXISTING SOILS AS RECOMMENDED BY THE GEOTECHNICAL ENGINEER.
7. MINIMUM OF 2% GRADE FOR ALL GRASS AREAS UNLESS OTHERWISE NOTED.
8. MAXIMUM TERRACING GRADE TO BE 3:1 UNLESS OTHERWISE NOTED.
9. ALL GRADES BY CURBS ARE EDGE OF PAVEMENT GRADES UNLESS OTHERWISE INDICATED.
10. REFER TO LANDSCAPE PLAN FOR PLANTING AND OTHER LANDSCAPE FEATURE DETAILS.
11. CONTRACTOR TO PROVIDE THE CONSULTANT WITH A GRADING PLAN INDICATING AS-BUILT ELEVATIONS OF ALL DESIGN GRADES SHOWN ON THIS PLAN.

SANITARY AND STORM SEWER NOTES

1. SUPPLY AND CONSTRUCT ALL SEWERS AND APPURTENANCES IN ACCORDANCE WITH THE MOST CURRENT STANDARDS AND SPECIFICATIONS OF CITY OF OTTAWA AND ONTARIO PROVINCIAL STANDARDS.
2. MAIN LINE PVC PIPE SHALL BE DR 35 AND SERVICE CONNECTION PVC PIPE SHALL BE DR 28.
3. SERVICES ARE TO BE CONSTRUCTED TO 1.0M FROM FACE OF BUILDING AT A MINIMUM SLOPE OF 1.0%.
4. BEDDING FOR FLEXIBLE PIPE SHALL BE AS PER OPSD 802.010, 802.013 OR 802.014.
5. PIPE BEDDING, COVER AND BACKFILL ARE TO BE COMPACTED TO MINIMUM 98% OF THE STANDARD PROCTOR MAXIMUM DRY DENSITY. THE USE OF CLEAR CRUSHED STONE AS A BEDDING LAYER SHALL NOT BE PERMITTED.
6. MAINTENANCE HOLES AS PER OPSD 701.010 (1200MM), 701.011 (1500MM) AND 701.012 (1800MM).
7. FRAME AND COVER AS PER OPSD 401.010 TYPE A CLOSED (SANITARY) AND 400.070 (STORM)
8. SANITARY MAINTENANCE HOLE SHALL HAVE WATERTIGHT FRAME AND COVER IN PONDING AREAS AS PER OPSD 401.030.
9. BENCHING SHALL BE AS PER OPSD 701.021.
10. TRENCH WIDTH (SEPARATE TRENCH) AT TOP OF THE PIPE SHALL BE TO CITY OF OTTAWA STANDARD S6. CONTRACTOR IS RESPONSIBLE FOR SUPPLYING ADDITIONAL BEDDING AND/OR STRONGER PIPE IF ACTUAL TRENCH WIDTHS EXCEED DESIGN WIDTHS.
11. ALL SEWERS CONSTRUCTED WITH GRADES 0.50% OR LESS, SHALL BE INSTALLED WITH LASER LEVEL AND CHECKED PRIOR TO BACKFILL AT THE CONTRACTOR'S EXPENSE.
12. ALL STORM AND SANITARY SERVICE LATERALS SHALL BE EQUIPPED WITH BACKFLOW PREVENTION DEVICES AS PER THE CITY OF OTTAWA STANDARD DETAILS S14 AND S14.1 OR S14.2.
13. INSULATE ALL PIPES THAT HAVE LESS THAN 1.5M COVER WITH HI-40 INSULATION PER INSULATION DETAIL FOR SHALLOW SEWERS. PROVIDE 150MM CLEARANCE BETWEEN PIPE AND INSULATION.
14. SERVICE CONNECTIONS AND UTILITY CUTS TO BE BACKFILLED WITH UNSHRINKABLE FILL.
15. STORM PIPE LENGTHS ARE TO BARREL OF MANHOLE AND DO NOT INCLUDE BENCHING.
16. FLEXIBLE CONNECTIONS ARE REQUIRED FOR CONNECTING PIPES TO MANHOLES (FOR EXAMPLE KOR-N-SEAL, PSX: POSITIVE SEAL AND DURASEAL). THE CONCRETE CRADLE FOR THE PIPE CAN BE ELIMINATED.
17. ALL STORM MANHOLES AND CATCHBASIN MANHOLES ARE TO HAVE 300MM SUMPS UNLESS OTHERWISE INDICATED. ALL CATCHBASINS ARE TO HAVE 600MM SUMPS UNLESS OTHERWISE INDICATED.
18. ALL CATCHBASINS, MANHOLES AND/OR CATCHBASIN MANHOLES THAT ARE TO HAVE ICD'S INSTALLED WITHIN THEM ARE TO HAVE 600MM SUMPS.
19. THE OWNER SHALL REQUIRE THAT THE SITE SERVICING CONTRACTOR PERFORM FIELD TESTS FOR QUALITY CONTROL OF ALL SANITARY SEWERS. LEAKAGE TESTING SHALL BE COMPLETED IN ACCORDANCE WITH OPSS 410.07.16, 410.07.16.04 AND 407.07.24. DYE TESTING IS TO BE COMPLETED ON ALL SANITARY SERVICES TO CONFIRM PROPER CONNECTION TO THE SANITARY SEWER MAIN. THE FIELD TESTS SHALL BE PERFORMED IN THE PRESENCE OF A CERTIFIED PROFESSIONAL ENGINEER WHO SHALL SUBMIT A CERTIFIED COPY OF THE TEST RESULTS.
20. CONTRACTOR TO TELEVISE (CCTV) ALL PROPOSED SEWERS, 200MMØ OR GREATER PRIOR TO BASE COURSE ASPHALT. UPON COMPLETION OF CONTRACT, THE CONTRACTOR IS RESPONSIBLE TO FLUSH AND CLEAN ALL SEWERS & APPURTENANCES.

WATERMAIN NOTES

1. SUPPLY AND CONSTRUCT ALL WATERMAINS AND APPURTENANCES IN ACCORDANCE WITH THE CITY OF OTTAWA STANDARDS AND SPECIFICATIONS.
2. CONNECTIONS AND SHUT-OFFS AT THE MAIN AND CHLORINATION OF THE WATER SYSTEM SHALL BE PERFORMED BY THE CONTRACTOR IN THE PRESENCE CITY OF OTTAWA REPRESENTATIVES.
3. PVC WATERMAINS SHALL BE MINIMUM DR 18 CLASS 235 (AWWA) C900-07
4. WATER SERVICE CONNECTIONS TO C-STORE SHALL BE 50MM Ø TYPE "K" SOFT COPPER AS PER OPSD 1104.01.AND CONFORM TO ASTM B88-03 (ASTM B88M-05 FOR METRIC SIZES) .
5. BEDDING SHALL BE AS PER CITY OF OTTAWA STANDARD DRAWING W17.
6. THERMAL INSULATION IN SHALLOW TRENCHES AND ADJACENT TO OPEN STRUCTURES SHALL BE AS PER CITY OF OTTAWA STANDARD DRAWING W22 AND W23.
7. MINIMUM COVER ON WATERMAINS SHALL BE 2.4 METRES.
8. PROVISIONS FOR FLUSHING THE WATER LINE PRIOR TO TESTING AND SO FORTH MUST BE PROVIDED WITH AT LEAST A 50 MM OUTLET ON 100 MM AND LARGER LINES AS PER OPSD1104.03-1. COPPER LINES ARE TO HAVE FLUSHING POINTS AT THE END, THE SAME SIZE AS THE LINE. ON FIRE LINES, FLUSHING OUTLET TO BE 100 MM DIAMETER MINIMUM OR A HYDRANT.
9. ALL TEES, PLUGS, HORIZONTAL, VERTICAL BENDS, REDUCERS AND HYDRANTS TO HAVE CONCRETE THRUST BLOCKS AS PER OPSD 1103.01 AND 1103.021.
10. PROPOSED WATER SERVICES ARE TO BE CONSTRUCTED TO WITHIN 1.0M OF FOUNDATION WALL AND CAPPED, UNLESS OTHERWISE INDICATED.
11. WATERMAINS MUST FOLLOW THE MINISTRY OF THE ENVIRONMENT PROCEDURES THAT GOVERN THE SEPARATION OF SEWERS AND WATERMAINS F-6-1. A MINIMUM VERTICAL CLEARANCE OF 0.30 METER OVER, 0.5 METER UNDER SEWERS AND ALL OTHER UTILITIES WHEN CROSSING. MUST ALSO MAINTAIN 2.5 METRES HORIZONTAL SEPARATION WITH SEWERS.
12. ALL PROPOSED WATER PIPING MUST BE ISOLATED FROM EXISTING LINES IN ORDER TO ALLOW INDEPENDENT PRESSURE TESTING AND CHLORINATING FROM THE EXISTING SYSTEM. FLUSHING, SWABBING AND TESTING OF WATERMAIN AS PER ONTARIO PROVINCIAL STANDARDS AND SPECIFICATIONS (OPSS), AS WELL AS CITY OF OTTAWA SPECIFICATION.
13. AFTER PASSING THE HYDROSTATIC PRESSURE TEST AND LEAKAGE TEST, CHLORINATION CAN PROCEED. SAMPLING OF THE NEW MAINS IS TO BE DONE AT THE REQUIRED LOCATIONS PRIOR TO CONNECTING TO THE CITY WATERMAIN SYSTEM. THE TEE FITTING IS TO BE CUT INTO THE EXISTING WATERMAIN TO MAKE THE CONNECTION. TO MAINTAIN THE PRESSURE IN THE NEW MAIN DURING INSTALLATION OF SERVICE, A 50MM BY-PASS WITH AN APPROVED PRESSURE DIFFERENTIAL BACKFLOW PREVENTER, MOUNTED ABOVE GROUND LEVEL IS TO BE INSTALLED AROUND THE CLOSED ISOLATING VALVE.
14. CITY IN-SERVICE WATER VALVES CAN ONLY BE OPERATED BY CITY OF OTTAWA WATER STAFF.
15. WATERMAINS TO BE INSTALLED TO GRADE AS SHOWN ON APPROVED PLANS, COPY OF GRADE SHEET MUST BE SUPPLIED TO INSPECTOR PRIOR TO COMMENCEMENT OF WORK, WHEN REQUESTED BY INSPECTOR.
16. VALVE IN BOXES SHALL BE INSTALLED AS PER CITY OF OTTAWA STD. MAINLINE VALVES TO BE RESTRAINED AS PER CITY OF OTTAWA STD.
17. THE CONTRACTOR SHALL COMPLETE THE NECESSARY WATER TESTING (I.E. PRESSURE TEST, FLUSHING, CHLORINATE, SAMPLING, ETC.)

CURB, SIDEWALK, AND PAVEMENT NOTES

1. ALL CURBS SHALL BE BARRIER CURB (150MM) UNLESS OTHERWISE NOTED AND CONSTRUCTED AS PER CITY OF OTTAWA STANDARDS (SC1.1). MOUNTABLE CURBS ARE TO BE PER CITY OF OTTAWA STANDARD (SC1.3).
2. THE GRANULAR SUB-BASE AND BASE SHALL BE COMPACTED TO AT LEAST 98% OF THE STANDARD PROCTOR MAXIMUM DRY DENSITY VALUE. ANY ADDITIONAL GRANULAR FILL USED BELOW THE PROPOSED PAVEMENT SHOULD BE COMPACTED TO AT LEAST 98% OF THE STANDARD PROCTOR MAXIMUM DRY DENSITY VALUE.
3. AT ALL ENTRANCES TO THE SITE THE ROAD CURB AND SIDEWALK WILL BE CONTINUOUS THROUGH THE DRIVEWAY, THE DRIVEWAY GRADE WILL BE COMPATIBLE WITH THE EXISTING SIDEWALK AND CURB DEPRESSION WILL BE PROVIDED FOR EACH ENTRANCE. ACCESS CONSTRUCTION AS PER O.P.S.D. 350.010.
4. SAW CUT AND KEY GRIND ASPHALT AT ALL ROAD CUTS AND ASPHALT TIE IN POINTS AS PER CITY OF OTTAWA STANDARDS (R10). WHERE THE NEW PAVEMENT WILL ABUT EXISTING PAVEMENT, THE DEPTHS OF THE GRANULAR MATERIALS SHOULD TAPER UP OR DOWN AT 5 HORIZONTAL TO 1 VERTICAL, OR FLATTER TO MATCH THE DEPTHS OF THE GRANULAR MATERIAL(S) EXPOSED IN THE EXISTING PAVEMENT.
5. THE PORTION OF THE DRIVEWAY WITHIN THE CITY BOULEVARD MUST BE PAVED TO THE LATEST CITY OF OTTAWA STANDARDS.
6. PROVIDE LINE/PARKING PAINTING.
7. PAVEMENT STRUCTURE FOR PARKING LOT AND ACCESS ROADWAYS (GEMTEC GEOTECHNICAL INVESTIGATION REPORT DATED JULY 3, 2019)

   90 MILLIMETRES ASPHALTIC CONCRETE,
   150 MILLIMETRES OF OPSS GRANULAR A BASE,
   450 MILLIMETRES OF OPSS GRANULAR B TYPE I OR II SUBBASE

   THE 90 MILLIMETRES ASPHALTIC CONCRETE SURFACE SHOULD CONSIST OF 40 MILLIMETRES OF SUPERPAVE 12.5 (TRAFFIC LEVEL B) OVER 50 MILLIMETRES OF SUPERPAVE 19.0 (TRAFFIC LEVEL B). PERFORMANCE GRADE PG58-34 ASPHALTIC CONCRETE SHOULD BE SPECIFIED.

GEOTECHNICAL NOTE

1. THE CONTRACTOR SHALL FOLLOW THE RECOMMENDATIONS OF THE GEOTECHINICAL INVESTIGATION REPORT PREPARED BY GEMTEC, DATES JULY 03,2019 AND OTHER AVAILABLE REPORTS SPECIFIC TO THE SUBJECT SITE.

AECOM

PROJECT

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REGISTRATION

LEGAL DESCRIPTION
PART OF BLOCK 21 OF DRAFT PLAN OF SUBDIVISION OF PARTS OF LOTS 26 AND 27
CONCESSION 11
GEOGRAPHIC TOWNSHIP OF GOULBOURN
(CITY OF OTTAWA)

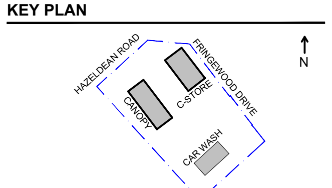
ISSUE/REVISION

I/R	DATE	DESCRIPTION
A	2020-03-31	ISSUED FOR SPA

DRAWN BY

SG

KEY PLAN



GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

GENERAL NOTES (CIVIL)

AECOM FILE NAME

C001.0-GEN-HZLX

SHEET NUMBER

C001.0

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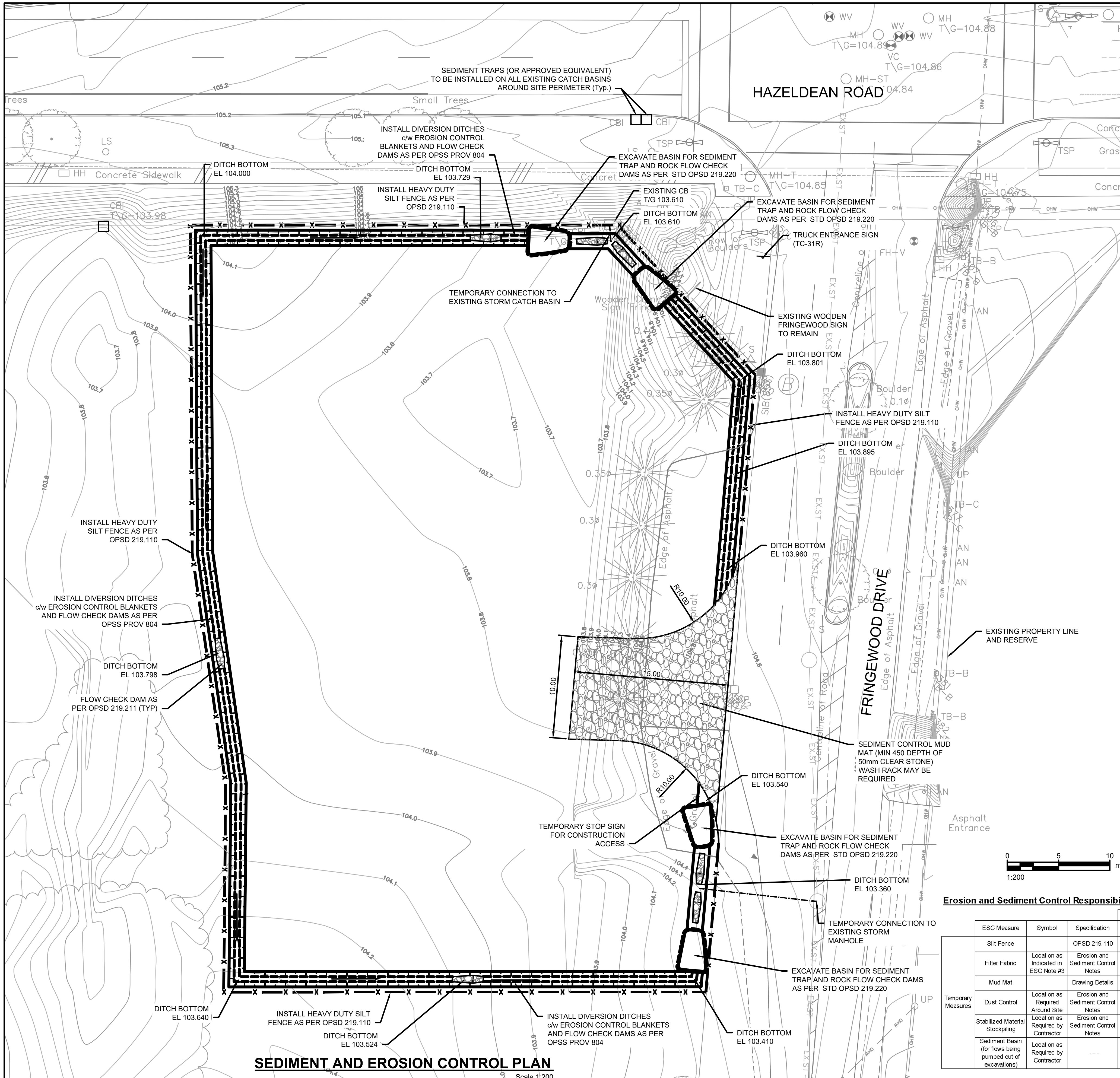

SEDIMENT AND EROSION CONTROL PLAN

Scale 1:200

NOTES:

1. FOR GENERAL NOTES SEE DRAWING C001.0.
2. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE PROPER INSTALLATION, MAINTENANCE, AND REMOVAL OF ALL TEMPORARY EROSION AND SEDIMENT CONTROL MEASURES DURING CONSTRUCTION AND AS DIRECTED BY THE ENGINEER.
3. ADDITIONAL EROSION AND SEDIMENT CONTROL (ESC) MEASURES MAY BE REQUIRED AND SHALL BE DETERMINED BY THE ENGINEER.
4. SILT CONTROL FENCING SHALL BE INSTALLED ACCORDING TO THIS DRAWING AND MAINTAINED UNTIL COMPLETION OF THE LANDSCAPING AND SITE STABILIZATION.
5. NO CONSTRUCTION ACTIVITY OR MACHINERY SHALL INTRUDE BEYOND THE SILT/SNOW FENCE OR LIMIT OF DEVELOPMENT. ALL CONSTRUCTION VEHICLES SHALL LEAVE THE SITE AT DESIGNATED LOCATIONS AS SHOWN ON THE PLANS. ALL MATERIALS AND EQUIPMENT SHALL BE STORED ON SITE IN A DESIGNATED AREA. NO MATERIAL OR EQUIPMENT SHALL BE STORED ON THE MUNICIPAL RIGHT OF WAY. NO CONSTRUCTION VEHICLES WILL PARK ON THE MUNICIPAL ROADS.
6. STOCKPILES SHALL BE SET BACK FROM ANY WATERCOURSE AND STABILIZED AGAINST EROSION AS SOON AS POSSIBLE. A SETBACK OF AT LEAST 15m FROM ANY TOP OF BANK OR WATERCOURSE IS REQUIRED. ALL EXPOSED SOILS SHALL BE IMMEDIATELY STABILIZED WITH A SEED AND MULCH APPLICATION AS DIRECTED BY THE ENGINEER.
7. SERVICING OF CONSTRUCTION EQUIPMENT ON-SITE IS PROHIBITED.
8. CLEANING OF EXISTING ROAD(S) AT SITE ACCESS POINTS SHALL BE DONE DAILY DURING CONSTRUCTION OR AS NECESSARY THROUGH REGULAR INSPECTION OR AS DIRECTED BY THE ENGINEER.
9. DUST CONTROL TO BE REVIEWED DAILY. WATER TRUCK TO BE PROVIDED ON-SITE AND ALL HAUL ROAD / WORKING AREAS TO BE SPRAYED WITH WATER AS REQUIRED TO ENSURE DUST IS CONTROLLED ON-SITE.
10. ALL RE-GRADED AREAS WITHIN THE SITE WHICH ARE NOT OCCUPIED BY BUILDINGS, ROADWAYS, SIDEWALKS OR DRIVEWAYS SHALL BE TOP-SOILED AND SODDED / SEEDED IMMEDIATELY AFTER COMPLETION OF FINAL GRADING OPERATIONS OR AS DIRECTED BY THE ENGINEER.
11. SEDIMENT TRAPS (OR APPROVED EQUIVALENT) ARE TO BE INSTALLED AT ALL CATCHBASINS AND CATCHBASIN MANHOLE LOCATIONS UPON COMPLETION OF SERVICING.
12. THE ESC STRATEGIES ON THESE PLANS ARE NOT STATIC AND MAY NEED TO BE UPGRADED / AMENDED AS SITE CONDITION CHANGES TO PREVENT SEDIMENT RELEASE TO THE NATURAL ENVIRONMENT. FAILED ESC MEASURE MUST BE REPAIRED IMMEDIATELY.
13. MATERIALS TO REPAIR DAMAGED EROSION AND SEDIMENT CONTROL MEASURES MUST BE KEPT ON-SITE AT ALL TIMES.
14. INSPECTION OF THE PROPOSED EROSION AND SEDIMENT CONTROL MEASURES WILL OCCUR ON A WEEKLY BASIS, AFTER SIGNIFICANT RAINFALL OR SNOW MELT EVENTS AND DAILY DURING EXTENDED RAIN OR SNOW MELT PERIODS.
15. SEDIMENT / SILT SHALL BE REMOVED FROM THE SEDIMENT CONTROL DEVICE AND THE CATCHBASIN BUFFERS AFTER STORM EVENTS AND DISPOSED OF IN AREAS AS APPROVED BY THE ENGINEER.
16. ALL LITTER AND DEBRIS SHALL BE MONITORED AND DISPOSED OF DAILY OR AS NECESSARY THROUGH REGULAR INSPECTION.
17. ROCK CHECK DAMS ARE TO BE CLEANED OF ALL ACCUMULATED SEDIMENT AS SOON AS SEDIMENT HAS ACCUMULATED TO DEPTH GREATER THAN 50% OF THE UPSTREAM CHECK DAM.
18. THE SILT FENCE MUST BE INSPECTED WEEKLY AND IMMEDIATELY AFTER RAINFALL OR SIGNIFICANT SNOW MELT EVENTS FOR RIPS AND TEARS, BROKEN STAKES, BLOW OUTS (STRUCTURAL FAILURE) AND ACCUMULATION OF SEDIMENT. THE SILT FENCE MUST BE FIXED AND / OR REPLACED IMMEDIATELY WHEN DAMAGED. ACCUMULATED SEDIMENT MUST BE REMOVED FROM THE SILT FENCE WHEN ACCUMULATION REACHES 50% OF THE HEIGHT OF THE FENCE.

MUD MAT

STONE SIZE - USE CLEAR CRUSHED 50mm STONE.

THICKNESS - NOT LESS THAN 300mm

LENGTH - AS REQUIRED

WIDTH - 10m MINIMUM, BUT NOT LESS THAN THE WIDTH AT POINTS WHERE INGRESS OR EGRESS OCCURS.

FILTER CLOTH - NON-WOVEN GEOTEXTILE WILL BE PLACED OVER THE ENTIRE AREA PRIOR TO PLACING STONE.

MAINTENANCE - THE ENTRANCE SHALL BE MAINTAINED IN A CONDITION WHICH WILL PREVENT TRACKING OR FLOWING OF SEDIMENT ONTO PUBLIC RIGHT-OF-WAY. THIS MAY REQUIRE PERIODIC TOP DRESSING WITH ADDITIONAL STONE AS CONDITIONS DEMAND AND REPAIR AND/ OR CLEANOUT OF ANY MEASURES USED TO TRAP SEDIMENTS. ALL SEDIMENTS SPILLED, DROPPED, WASHED OR TRACKED ONTO PUBLIC RIGHT-OF-WAY MUST BE REMOVED IMMEDIATELY.

TIRE WASH STATION - WHEELS SHALL BE CLEANED TO REMOVE SEDIMENT PRIOR TO ENTRANCE ONTO PUBLIC RIGHT-OF-WAY. WHEN WASHING IS REQUIRED IT SHALL BE DONE ON A DESIGNATED AREA STABILIZED WITH STONE AND WHICH DRAINS INTO AN APPROVED SEDIMENT TRAPPING DEVICE.

INSPECTION AND REQUIRED - INSPECTION AND MAINTENANCE MAINTENANCE SHALL BE PROVIDED PERIODCALLY AND AFTER AFTER SIGNIFICANT RAINFALL AND SNOWMELT.

SEDIMENT CONTROL CONSTRUCTION SCHEDULE:

1. INSTALL PERIMETER ENVIRONMENTAL FENCE AND CONSTRUCTION VEHICLE ACCESS.
2. EXCAVATE PERIMETER SWALES AND INSTALL CHECK DAMS.
3. STRIP SITE OF TOPSOIL AND REMOVE OFF SITE.
4. INSTALL MINOR STORM SEWER SYSTEM ALONG WITH OTHER SERVICES.
5. INSTALL CATCH BASIN FILTRATION ON ALL CATCH BASINS AND CATCH BASIN MANHOLES.
6. SEDIMENT CONTROL MEASURES ARE TO BE MAINTAINED UNTIL ALL AREAS OF THE SITE HAVE BEEN STABILIZED WITH SOD OR ASPHALT.

LEGEND:

HEAVY DUTY SILT FENCE

SEDIMENT TRAP

MUD MAT CLEAR STONE

MUD MAT LIMESTONE

0 5 10 m
1:200

Erosion and Sediment Control Responsibilities:

				During Construction			After Construction Prior to Final Acceptance		After Final Acceptance
	ESC Measure	Symbol	Specification	Installation Responsibility	Inspection/Maintenance Responsibility	Inspection Frequency	Approval to Remove	Removal Responsibility	Inspection/Maintenance Responsibility
Temporary Measures	Silt Fence		OPSD 219.110	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Consultant	Developer's Contractor	N/A
	Filter Fabric	Location as Indicated in ESC Note #3	Erosion and Sediment Control Notes	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Consultant	Developer's Contractor	N/A
	Mud Mat		Drawing Details	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Developer's Contractor	Developer's Contractor	N/A
	Dust Control	Location as Required Around Site	Erosion and Sediment Control Notes	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Consultant	Developer's Contractor	N/A
	Stabilized Material Stockpiling	Location as Required by Contractor	Erosion and Sediment Control Notes	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Developer's Contractor	Developer's Contractor	N/A
	Sediment Basin (for flows being pumped out of excavations)	Location as Required by Contractor	- - -	Developer's Contractor	Developer's Contractor	After Every Rainstorm	Developer's Contractor	Developer's Contractor	N/A

AECOM

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REGISTRATION

LEGAL DESCRIPTION
PART OF BLOCK 21 OF DRAFT PLAN OF SUBDIVISION OF PARTS OF LOTS 26 AND 27 CONCESSION 11 GEOGRAPHIC TOWNSHIP OF GOULBOURN (CITY OF OTTAWA)

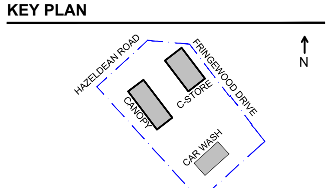
ISSUE/REVISION

I/R	DATE	DESCRIPTION
A	2020-03-31	ISSUED FOR SPA

DRAWN BY

SG

KEY PLAN



GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

SEDIMENT AND EROSION CONTROL PLAN

AECOM FILE NAME

C101.0-SDC-HZLX

SHEET NUMBER

C101.0

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8. Approval and Permit Requirements

The following is a list of the approval and permits required to develop the subject site:

- Ontario Ministry of Environment, Conservation and Parks – Environmental Compliance Approval (ECA) under the Ontario Water Resources Act for water service, sanitary and storm sewer pipes and stormwater management measures;
- Mississippi Valley Conservation Authority – Review of site servicing to provide agreement to the site development in support of the ECA application submission;
- City of Ottawa – Approvals for the development application, municipal sign-off on the ECA application and all other applicable permits required for connection of services to the ROW and site development.

AECOM Imagine it. Delivered.

Appendix A

Functional Servicing and Stormwater Management Report for Campanale Homes 5 Orchard Drive, City of Ottawa (DSEL, March 2019)

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FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

FOR

CAMPANALE HOMES
5 ORCHARD DRIVE

CITY OF OTTAWA

PROJECT NO.: 18-1006

MARCH 2019 – REV. 3

FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT FOR CAMPANALE HOMES 5 ORCHARD DRIVE

MARCH 2019 – REV. 3

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**FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT
FOR
CAMPANALE HOMES
5 ORCHARD DRIVE**

CITY OF OTTAWA

**MARCH 2019 – REV. 3
PROJECT NO.: 18-1006**

1.0 INTRODUCTION

David Schaeffer Engineering Ltd. (DSEL) has been retained by Campanale Homes to prepare a Functional Servicing and Stormwater Management Report in support of the Draft Plan of Subdivision (DPS) for the proposed development at 5 Orchard Drive.

The subject property is located within the City of Ottawa urban boundary, in the Stittsville ward. As illustrated in ***Figure 1***, the subject property is bounded by Hazeldean Road to the north, Fringewood Drive to the east, an existing restaurant to the west and existing residential development to the south. The subject property measures approximately ***3.97 ha*** and is designated Arterial Mainstreet (AM9) under the current City of Ottawa zoning by-law.

Figure 1: Site Location

The proposed development consists of ***1.82 ha*** of commercial space and ***2.13 ha*** of residential land: comprised of 65 townhouse units; 2 semi-detached units; and 7 single home units.

The objective of this report is to support the application for Draft Plan of Subdivision by providing sufficient detail demonstrating that the proposed development is supported by existing and proposed municipal servicing infrastructure. Additionally, this report will demonstrate that the site design conforms to current City of Ottawa design standards.

1.1 Existing Conditions

The subject site is currently undeveloped. Two existing parallel ditches run from the south side of the property toward two ditch-inlet catch basins (DICBs) at the north edge of the property along Hazeldean Road. The existing DICBs outlet into the existing 675 mm diameter stormwater on Hazeldean Road. There is also a ditch along the southern property line which collects storm water runoff from the existing residential units on the adjacent property and outlets into the western most ditch of the two previously mentioned ditches. Note that in existing conditions there is a drop in elevation between the gravel shoulder and the subject property, to the north of the subject site, along Hazeldean Road. Sewer system and watermain distribution mapping collected from the City of Ottawa indicate that the following services exist across the property frontages, within the adjacent municipal right-of-ways:

Hazeldean Road:

- 762 mm watermain;
- 675 mm storm sewer;
- 450 mm storm sewer;
- 150 mm sanitary sewer at northwest corner of site; and
- 675 mm sanitary sewer northeast of site.

Fringewood Drive:

- 200 mm watermain.

1.2 Required Permits / Approvals

Development of the site is subject to the City of Ottawa Planning and Development Approvals process. The City of Ottawa must approve detailed engineering design drawings and reports prepared to support the proposed development plan before issuing approval.

The subject property contains existing trees. Development, which may require removal of existing trees, may be subject to the City of Ottawa Urban Tree Conservation By-law No. 2009-200.

1.3 Pre-consultation

Pre-consultation correspondence and the servicing guidelines checklist are located in ***Appendix A***.

Further pre-consultation with City Staff has been completed via email. Associated correspondence is located in ***Appendix A***.

2.0 GUIDELINES, PREVIOUS STUDIES, AND REPORTS

2.1 Existing Studies, Guidelines, and Reports

The following studies were utilized in the preparation of this report:

- **Ottawa Sewer Design Guidelines,**
  City of Ottawa, October 2012.
  (City Standards)

  - **Technical Bulletin ISDTB-2014-01**
    City of Ottawa, February 5, 2014.
    (ITSB-2014-01)

  - **Technical Bulletin PIEDTB-2016-01**
    City of Ottawa, September 6, 2016.
    (PIEDTB-2016-01)

  - **Technical Bulletin ISTB-2018-01**
    City of Ottawa, March 21, 2018.
    (ISTB-2018-01)

- **Ottawa Design Guidelines – Water Distribution**
  City of Ottawa, July 2010.
  (Water Supply Guidelines)

  - **Technical Bulletin ISD-2010-2**
    City of Ottawa, December 15, 2010.
    (ISDTB-2010-2)

  - **Technical Bulletin ISDTB-2014-02**
    City of Ottawa, May 27, 2014.
    (ISDTB-2014-02)

  - **Technical Bulletin ISDTB-2018-02**
    City of Ottawa, March 21, 2018.
    (ISDTB-2018-02)

- **Stormwater Planning and Design Manual,**
  Ministry of the Environment, March 2003.
  (SWMP Design Manual)

- **Ontario Building Code Compendium**
  Ministry of Municipal Affairs and Housing Building Development Branch, January 1, 2010 Update.
  (OBC)

- **West End Pumping Stations Decommissioning & By-Pass Sewers Fringewood Drive By-Pass Sewer Design**
  Novatech, May 2018.
  (Fringewood By-Pass Sewer Design)

- **Hunting Properties Development / Proposed Realignment of Channel on 2 and 3 Iber Road**
  JF Sabourin and Associates Inc., March 2017.
  (JFSA Channel Realignment)

- **Hazeldean Road Widening Poole Creek to Terry Fox Drive Stormwater Management**
  IBI Group, November 2009
  (Hazeldean SWM Report)

- **5 Orchard External Stormwater Management – Cost Implications**
  DSEL, March 2019
  (External SWM Cost Implications)

- **5 Orchard Drive – Stormwater Functional Servicing Analysis**
  JF Sabourin and Associates Inc., March 2019
  (5 Orchard JFSA Memo)

- **Kanata West Master Servicing Study**
  Stantec Consultin Ltd., June 2006
  (Kanata West Master Servicing Plan)

3.0 WATER SUPPLY SERVICING

3.1 Existing Water Supply Services

The subject property lies within the City of Ottawa 3W pressure zone, as shown by the Pressure Zone map in ***Appendix B.*** Watermains exist within Hazeldean Road and Fringewood Drive.

3.2 Water Supply Servicing Design

The subject property is proposed to be serviced through two connections to the existing 203 mm watermain within Fringewood Drive.

Table 1, below, summarizes the ***Water Supply Guidelines*** employed in the preparation of the water demand estimate.

Table 1
Water Supply Design Criteria

Design Parameter	Value
Commercial-Floor space	2.5 L/m²/d
Single Family House	3.4 P/unit
Semi-Detached House	2.7 P/unit
Townhouse	2.7 P/unit
Average Daily Demand	280 L/d/per
Residential Maximum Daily Demand	3.6 x Average Daily *
Residential Maximum Hourly	5.4 x Average Daily *
Commercial Maximum Daily Demand	1.5 x avg. day L/gross ha/d
Commercial Maximum Hour Demand	1.8 x avg. day L/gross ha/d
Minimum Watermain Size	150 mm diameter
Minimum Depth of Cover	2.4 m from top of watermain to finished grade
During normal operating conditions desired operating pressure is within	350 kPa and 480 kPa
During normal operating conditions pressure must not drop below	275 kPa
During normal operating conditions pressure shall not exceed	552 kPa
During fire flow operating pressure must not drop below	140 kPa
** Residential Max. Daily and Max. Hourly peaking factors per MOE Guidelines for Drinking-Water Systems Table 3-3 for 0 to 500 persons.* *** Table updated to reflect ISD-2010-2*	

Table 2, below, summarizes the anticipated water demand and boundary conditions for the proposed development; calculated using the ***Water Supply Guidelines.*** The City provided both the anticipated minimum and maximum water pressures, as well as, the estimated water pressure during fire flow as indicated by the correspondence located in ***Appendix A***.

Table 2
Proposed Water Demand

Design Parameter	Anticipated Demand[1] (L/min)	Boundary Conditions[2] Fringewood Dr. (South of valve) (m H_2O / kPa)	Boundary Conditions[2] Fringewood Drive (North of valve) (m H_2O / kPa)
Average Daily Demand	71.2	56.4 / 553.7	56.0 / 549.3
Max Day + Fire Flow (@10,000L/min)	190.9+10,000 = 10,190.9	40.8 / 400.6	53.3 / 522.8
Max Day + Fire Flow (@15,000L/min)	190.9+15,000 = 15,190.9	26.1 / 256.4	52.4 / 513.9
Peak Hour	300.3	52.6 / 516.4	52.7 / 516.9

1) Water demand calculation per ***Water Supply Guidelines***. See ***Appendix B*** for detailed calculations.
2) Boundary conditions supplied by the City of Ottawa for the demands indicated in the correspondence; assumed ground elevation 104.56m for connection 1 and 105.01m for connection 2 to the municipal watermain. See ***Appendix A.***

The residential component of the development is contemplated to meet the criteria for the ***10,000 L/min*** maximum fire flow cap, as per ***ISDTB-2014-02.*** As the commercial component is considered a future development and details have not yet been established, maximum fire flow for the commercial component was assumed to be ***15,000 L/min***, as per ***ISDTB-2014-02.***

3.3 Watermain Modelling

EPANet was utilized to model the proposed watermain system during peak hour, average day and max daily water demand, plus fire flow scenarios. The model was developed to assess pipe sizing.

EPANET uses pipe length, pipe diameter, elevation and friction loss factors based on pipe diameter obtained from ***Water Supply Guidelines, Table 4.4***. Minor loss coefficients based on bends, valves and tees in the pipe were also utilized in the model. EPANet calculated pressure drop using the Hazen-Williams equation and is used to assess the pressure that is being provided to each node.

To model the maximum daily flow scenario, ***10,000L/min*** was applied to each of the proposed hydrants for the residential part of the site and ***15,000L/min*** at the connection to the future commercial component of the property.

Table 3, below, summarizes pressures reported during average day, peak hour and maximum daily plus fire flow scenarios for nodes at points of interest.

Table 3
Model Simulation Output Summary

Node ID	Average Day (kPa)	Peak Hour (kPa)	Max Day + Fire Flow (10,000L/min) (kPa)	Max Day + Fire Flow (15,000L/min) (kPa)
10	553.3	516.4	399.6	255.4
12	551.8	516.7	401.3	252.0
14	552.0	516.6	395.3	251.1
15	552.4	517.0	330.5	232.1
17	551.5	516.8	409.5	253.2
18	552.2	516.8	381.3	247.2
19	551.6	516.8	396.0	175.1
20	552.4	517.2	303.3	203.9
21	552.6	517.3	269.8	214.2
23	552.8	517.5	284.8	209.8
25	552.1	516.4	395.9	251.7

The pressures modeled in average day scenario are either near or exceed the maximum allowable, per ***Table 2***. Pressures which exceed the desired operation pressure in the peak hour scenario, however, do not exceed the maximum allowable pressure. It is recommended a pressure check is performed during construction to determine if pressure reducing valves are required.

The pressures during maximum daily plus fire flow scenarios as well as peak hour scenarios fall within the required pressure range outlined in ***Table 2***. For the residential area, the node yielding the lowest pressure during fire flow scenario at ***10,000L/min*** is node 21. For the commercial area of the development, the fire flow scenario of ***15,000 L/min*** was modeled through node 19. The pressure at both of these critical nodes fall above the minimum required pressure indicated in ***Table 1***.

Model output reports, as well as, figures for each model scenario are found in ***Appendix B***.

3.4 Water Supply Conclusion

It is proposed to service the development from two connections to the existing 203 mm watermain within Fringewood Drive.

The contemplated development was analyzed using 10,000 L/min max fire flow for the residential components and assuming 15,000 L/min maximum fire flows for the future commercial component.

Water modeling was completed to confirm that adequate pressure is available to service the ultimate proposed development based on boundary conditions received from the ***City of Ottawa***. Fire flow scenario pressures fall within the guidelines outline in ***Table 2***, however, pressure check should be completed during construction to determine if pressure reducing valves will be required. The municipal system is capable of delivering water within the ***Water Supply Guidelines*** pressure range.

The design of the water distribution system conforms to all relevant City Guidelines and Policies.

4.0 WASTEWATER SERVICING

4.1 Existing Wastewater Services

The subject property lies within the future Kanata West Pump Station catchment area, per the ***Kanata West Master Servicing Plan***.

There is an existing 675 mm diameter sanitary sewer within Hazeldean Road. Currently there is no sanitary sewer services within Fringewood Drive, on the section of the road directly adjacent to the subject property.

Pre-consultation with the City of Ottawa indicates that the Hazeldean Road sanitary sewer has been sized to convey additional flows from the proposed subdivision, upon completion of the Kanata West Pumping Station (KWPS), which is slated for completion in the summer of 2019. It is anticipated the contemplated development will proceed after the completion of the KWPS, therefore, the downstream system will have capacity to convey flow from the subject property.

4.2 Wastewater Design

The proposed development will be serviced via a connection to the existing 675 mm diameter sanitary sewer within Hazeldean Road through a future 250 mm diameter sanitary sewer within Fringewood Drive, running along the east end of the property.

Table 4, below, summarizes the ***City Standards*** employed in the calculation of wastewater flow rates for the proposed development.

Table 4
Wastewater Design Criteria

Design Parameter	Value
Average Daily Demand	280 L/d/per
Single Family House	3.4 P/unit
Semi-Detached House	2.7 P/unit
Townhouse	2.7 P/unit
Peaking Factor	Harmon's Peaking Factor. Max 3.8, Min 2.0
Commercial Floor Space	28,000 L/ha/d
Infiltration and Inflow Allowance	0.33 L/s/ha
Sanitary sewers are to be sized employing the Manning's Equation	$Q = \frac{1}{n} AR^{2/3} S^{1/2}$
Commercial Peaking Factor	1.50 per City of Ottawa Sewer Design Guidelines Appendix 4B
Minimum Sanitary Sewer Lateral	135 mm diameter
Minimum Manning's 'n'	0.013
Minimum Depth of Cover	2.5 m from crown of sewer to grade
Minimum Full Flowing Velocity	0.6 m/s
Maximum Full Flowing Velocity	3.0 m/s
Extracted from Sections 4 and 6 of the City of Ottawa Sewer Design Guidelines, October 2012 updated per **ISTB-2018-01**	

Table 5, below, demonstrates the anticipated peak flow from the proposed development. See ***Appendix C*** for associated calculations.

Table 5
Summary of Proposed Wastewater Flows

Design Parameter	Anticipated Sanitary Flow (L/s)
Average Dry Weather Flow Rate	**1.26**
Peak Dry Weather Flow Rate	**3.24**
Peak Wet Weather Flow Rate	**4.51**

The estimated sanitary flow for the contemplated development anticipates a peak wet weather flow of ***4.51 L/s***.

A future sanitary sewer is contemplated to be constructed within Fringewood Drive starting in May 2019. A gravity sanitary connection from the existing subdivision to the north will by-pass the existing Fringewood Pump Station, thus directing wastewater flows from the proposed development to the existing 675 mm sanitary sewer within Hazeldean Road.

In the design of the bypass sewer, the subject property was estimated to have a total anticipated peak flow equal to ***6.22 L/s*** as indicated in the ***Fringewood By-Pass Sewer Design (FBPSD)***, calculation shown in ***Appendix C***. The contemplated development results in a reduction of ***1.71L/s*** flow to the future sanitary sewer than that anticipated in the ***(FBPSD)***, therefore, the future sewer has sufficient capacity to convey the wastewater flow from the subject site. Refer to ***Appendix C*** for a copy of ***FSPSD,*** including future sanitary design sheets and sanitary drainage figure.

4.3 Wastewater Servicing Conclusions

The site is tributary to the existing sanitary sewer within Hazeldean Road.

A future sanitary sewer is contemplated to be constructed adjacent to the subject property within Fringewood Drive. The proposed development results in a decrease in wastewater flow of ***1.71L/s*** to the future sanitary sewer contemplated in the ***Fringewood By-Pass Sewer Design.*** The proposed future Fringewood Drive sanitary sewer has sufficient capacity to convey wastewater flow from the subject property to the existing sanitary sewer with Hazeldean Road.

The proposed wastewater design conforms to all relevant ***City Standards***.

5.0 STORMWATER MANAGEMENT

5.1 Existing Stormwater Services

Stormwater runoff from the subject property is tributary to the Carp River sub-watershed via Poole Creek and City of Ottawa storm sewer system and is therefore, reviewed by the Mississippi Valley Conservation Authority (MVCA). Runoff from the subject site is collected and conveyed by storm sewers within Hazeldean Road to an interim stormwater wetland located on Hazeldean Road, east of the intersection of Hazeldean Road and Huntmar Drive. The interim wetland discharges to a ditch that conveys flow along the north edge of the existing commercial development on Hazeldean, eventually discharging to the Carp River.

Two parallel ditches currently exist on the subject property that lead to two existing DICBs; refer to ***DICB 1*** and ***DICB 2*** on drawing ***EX-SWM-1,*** accompanying this report. The majority of the flow from the subject site is picked up by the ditch draining to ***DICB 1,*** with flow from the east portion of the site directed to ***DICB 2***. A portion of flow from the west of the site is directed to Poole Creek, denoted as ***P1*** on the drawing ***EX-SWM-1.***

Based on the topographic survey of Hazeldean Road, adjacent to the site, major overland flow is directed east and south down Fringewood Drive. The Major overland flow route for this area, 100-year subtract 10-year storm event, shown as ***MH400, MH405 & MH413*** on drawing ***EX-SWM-1***, would enter the site and be captured by ***DICB 2***.

The runoff from the rear yards of the Cloverloft Court properties that bound the south edge of the subject property, shown as ***EX2*** and ***EX3*** in ***EX-SWM-1***, flow into a rear yard ditch that runs along the south property line of the subject property. Drainage area ***EX2*** drains to the ***DICB 1,*** whereas, ***EX3*** drains to ***DICB 2***.

Drainage from the existing subdivision to the south of the subject property drains east towards the intersection of Fringewood Drive and Cloverloft Court. Note that based on field inspection completed by DSEL in May 2018, a culvert crossing Fringewood Drive at Cloverloft Court is perched and would not accept flow from ***EX5***, thus it is assumed all ***EX5*** drainage by-passes this culvert and is directed north to ***DICB 2.*** Further investigation will be conducted in the Spring 2019, when a survey will be completed to determine the ditch and culvert inverts.

Both ***DICB 1*** and ***DICB 2*** discharge to the existing 675 mm diameter storm sewer within Hazeldean Road. The stormwater discharge is conveyed through the existing storm sewer within Hazeldean road to ditches north of Hazeldean Road, and east of Huntmar Drive which convey directly to the Carp River.

Drainage from the existing restaurant located west of the subject property drains to the existing storm sewer within Hazeldean Road through existing catch basins, denoted as ***EX6*** on ***EX-SWM-1***.

The estimated pre-development peak flows from the subject site and external areas for the 2, 5, and 100-year events are summarized in ***Table 6*** and ***Table 7,*** below***:***

Table 6
Summary of Existing Peak Storm Flow Rates from Subject Property

City of Ottawa Design Storm	Estimated Peak Flow Rate to DICB1 (3.14 Ha) (L/s)	Estimated Peak Flow Rate to DICB2 (0.78 Ha) (L/s)	Estimate Peak Flow to Poole Creek (0.05 Ha) (L/s)
2-year	72.1	15.6	3.4
5-year	96.9	21.0	4.6
100-year	206.0	44.6	9.9

Table 7
Summary of Existing Peak Storm Flow Rates from External Area

City of Ottawa Design Storm	External Peak Flow Rate to DICB1 (EX2 0.422 Ha) (L/s)	Estimated Peak Flow Rate to DICB2 (MH400, MH405, MH413*, EX3, EX4, EX5 4.104 Ha) (L/s)
2-year	30.9	182.3
5-year	41.9	245.1
100-year	89.8	457.9
* Only Major System Contributions from MH400, MH405 & MH413 (100-Year – 10-Year)		

Based on field investigation by DSEL in May 2018, no stormwater management controls for flow attenuation exist on-site.

A capacity analysis of the existing DICB capture rate and DICB leads was completed to determine if the existing DICB are capable of capturing the 100-year storm in the 100-year storm event. DICB elevation, head and capture rate are summarized in ***Table 8,*** below:

Table 8
Summary of Existing DICB Capture Rate

Parameter	DICB 1	DICB 2
DICB Grate Invert Elevation (m)	103.98	103.65
DICB Lead Invert (m)	102.94	102.71
Ponding Level[1] (m)	104.49	104.49
Assumed Downstream HGL[2] (m)	103.08	102.77
Total Head[3] (m)	1.41	1.72
DICB Grate Capture Rate[4] (L/s)	660	660
375mm DICB Lead Capture[5] (L/s)	354	391
1) Spill Elevation across Fringewood Drive per topographic survey 2) Downstream HGL assumed equal to obvert of Ex. 675mm Storm within Hazeldean Road 3) Total Head equal to Ponding Level less the downstream HGL 4) DICB capture rate determined from Design Chart 4.20 from the MTO Drainage Management Manual, 1997 using 0.51m of ponding, capture rate multiplied by1.2 to account for 1200mm x 600mm grate and then by 0.5 to account for blockages. DICB2 has a higher ponding than DICB1 so the capture rate for DICB1 was used for both DICBs conservatively. 5) Orifice equation used per the ***City Standards***, refer to ***Appendix D*** for orifice equation		

Per the above, the flow through the DICB lead will restrict flow to ***354 L/s*** and ***391 L/s*** to ***DICB 1*** and ***DICB 2***, respectively. Based on the total flows summarized in ***Table 6 & 7, DICB 1*** is capable of conveying the 100-year flow form areas ***DICB 1*** and ***EX 2***. Flow to ***DICB 2*** exceeds ***391 L/s*** in the 100-year storm event. Ponding will occur in the existing condition up to the elevation of 104.49 where spill will occur across Fringewood Drive to the adjacent property. The spill is conveyed through a tributary of the carp river, currently the adjacent property is proposed to be re-developed and the tributary re-aligned. The spill from the subject property has been accounted for in the design of the re-aligned tributary and downstream culverts, described in ***JFSA Channel Alignment***.

A design sheet has been prepared by DSEL in lieu of the design information from the City of Ottawa for the Hazeldean storm sewer in the existing condition, located in ***Appendix D***. The design sheet indicates that storm sewers are surcharged in the existing condition. A hydraulic grade line (HGL) analysis was complete for the existing storm sewer, by JFSA, and outlined in the ***5 Orchard JFSA Memo.*** The results of the HGL analysis conclude that spill to the surface occurs in the existing condition at manholes 405, 413, 421 ,426 and 13. Refer to drawing ***EX-SWM-1,*** for drainage area IDs and ***Appendix D*** for HGL results prepared by JFSA.

5.2 Post-development Stormwater Management Target

Based on City of Ottawa standards, stormwater management requirements for the proposed development are as follows:

- The release rate for the subject property is limited by the capacity of the existing storm sewers within Hazeldean Road. A hydraulic grade line analysis was completed for the existing sewers to determine the maximum available capacity of the sewers. To ensure that the hydraulic grade line in the proposed condition does not impact the proposed development or have negative impact on the downstream system, the allowable release rate for the subject property has been determined to be ***251.9 L/s***;
- As stormwater quality control is constrained on the residential portion of the subject site, a larger portion of the allowable release rate is allocated to the residential block of ***200 L/s,*** with the remaining ***51.9 L/s*** to be the release rate for the commercial block;
- Uncontrolled Flow to Poole Creek is less than during the existing condition in the 5-Year and 100-Year event;
- All storms, up to and including the City of Ottawa 100-year design event, are to be attenuated on site; and
- Quality controls are required, as per correspondence with the MVCA, 70% TSS removal will be necessary. Refer to ***Appendix A*** for correspondence. However, the quality control that will be provided will be 80% TSS removal.

5.3 Proposed Stormwater Management System

It is proposed that the stormwater for the development will be serviced by the existing 675 mm diameter storm sewer on Hazeldean Road via a new storm sewer extended south on Fringewood Drive.

It is proposed to service the residential component of the development with a proposed 450 mm diameter storm sewer that would connect to a proposed 675 mm diameter storm sewer within Fringewood Drive. The commercial component of the site would connect independently to the proposed storm sewer within Fringewood Drive. The existing swale along Fringewood Drive would be regraded to flow towards the existing ***DICB 2***.

It is contemplated to re-grade the existing roadside ditch south of the subject property to re-direct flow from ***EX5*** to the Hazeldean Tributary on the 2 Iber Road lands, located on the east side of Fringewood Drive. Refer to drawing ***SWM-1,*** accompanying this report, for storm servicing and stormwater management details.

Drainage to existing ***DICB 2*** would include major system flow only (100-Year – 10-Year Flow) from a portion of Hazeldean Road (Area ***MH400, MH405, MH413***) and major and minor system flow from Fringewood Drive (Area ***EX4***). A 100-year flow rate of ***105.5 L/s*** is contemplated to continue to discharge to ***DICB 2***.

5.4 Proposed Quantity Controls

The release rate for the proposed development is restricted to ensure the hydraulic grade line allows for gravity drainage for the majority of residential units. A sewer analysis was completed for the downstream Hazeldean storm sewer system in the post-development condition to ensure no negative impacts, refer to ***Appendix D*** for HGL analysis in the proposed condition. To provide gravity drainage for the proposed units and improve the downstream condition, a release rate of ***251.9 L/s*** was selected as described in ***Section 5.1.*** Refer to the sewer analysis included in ***Appendix D***.

Table 9, below, summarizes post-development flow rates and anticipated storage for the development of the property.

Table 9
Stormwater Flowrate and Storage Summary

Control Area	5-Year Release Rate	5-Year Storage	100-Year Release Rate	100-Year Storage
	(L/s)	(m^3)	(L/s)	(m^3)
Unattenuated Areas to Poole Creek	0.6	0.0	1.2	0.0
Residential Areas	116.7	169.5	200.0	416.9
Commercial Areas	30.3	434.9	51.9	843.1
Total Comm + Res to Hazeldean*	**147.0**	**604.4**	**251.9**	**1260.0**
* Total Flow does not include Flow to Poole Creek				

It is anticipated that ***416.9 m³*** of storage will be required for the residential development and ***843.1 m³*** of storage will be needed for the future commercial development in order to attenuate flows to the target flow rate of ***251.9 L/s*** in the 100-year storm event***.*** Refer to storage calculations that are contained within ***Appendix D***.

To achieve the allowable release rate, the proposed residential portion of the development will employ a combination of Low Impact Development (LID) practice infiltration chambers located in the 8 m easement between the commercial and residential properties, as well as, take advantage of surface ponding on the streets***.*** Proposed surface ponding will be designed in accordance with ***City Standards***. The commercial block is contemplated to use similar stormwater management techniques to attenuate to the allowable release rate.

An HGL analysis was completed for the proposed condition, summarized in the ***5 Orchard JFSA Memo***, for the downstream Hazeldean storm sewer network. The analysis concluded that adequate freeboard is provided from the 100-year HGL to the proposed Underside of Footing (USF) of the development and that the HGL is lowered in the proposed condition compared to the existing condition within the existing storm sewer system. Spill will continue to occur within the Hazeldean storm sewer system during the 100-year storm event, however, the spill is less than in the existing condition. Only road drainage and the subject property are connected to the storm system, so the resulting spill presents no risk of surcharging into existing foundation drains.

A preliminary stormwater analysis was completed by JFSA, summarized in the ***5 Orchard JFSA Memo***, which reviewed the impacts of the development on the water levels within the Carp River and the tributary to the Carp River using the City of Ottawa's PCSWMM model of the Carp River. Based on the results from the ***5 Orchard JFSA Memo***, the tributary to the Carp River has sufficient capacity to convey stormwater in the 100-year storm event. Sheer stress was also analyzed from the existing to proposed condition, during detailed design, and it was concluded that a geomorphological review will be required to determine if erosion control measures are necessary for the proposed condition. At the outlet to the Carp River, the analysis concluded that there are no impacts to the 100-year water levels within the Carp River in the proposed condition, refer to ***Appendix D*** for ***5 Orchard JFSA Memo***.

A detailed hydrologic model will be completed during the detailed design phase to confirm the conclusions from the ***5 Orchard JFSA Memo*** and confirm storage requirements. During detailed design, efforts will be made to reduce the LID infiltration chambers maximize surface ponding within the right-of-way.

The unattenuated area directed to Poole Creek, U1 on drawing ***SWM-1***, is less than the flow to Poole Creek in the pre-development condition shown in ***Table 7*** for the 5 and 100-year storm events. The drainage area consists of rear yard area, which is considered clean water, therefore, quality controls are not anticipated for the uncontrolled area draining to Poole Creek.

Due to the depth of the existing storm sewer within Hazeldean Road, the proposed four blocks of townhomes units closest to Fringewood Drive will be required to use sump pumps, discharging to the surface to service the foundation drains, refer to ***CSP-1,*** accompanying this report for applicable units.

5.5 Proposed Quality Control

Quality controls are proposed to be provided by the interim Wetland located approximately 380 m north-east of the intersection of Huntmar Drive and Hazeldean Road. As discussed in ***Section 5.1***, a portion of the 5 Orchard site was contemplated to drain to the interim Wetland. Per the ***Hazeldean SWM Report,*** a total of ***3.84 Ha*** of External Drainage and ***3.51 Ha*** of Hazeldean Road runoff was contemplated to drain to the interim Wetland, for a total of ***7.35 Ha***. ***3.08 Ha*** of the subject property at 5 Orchard Drive was allocated to drain to the interim Wetland.

The total proposed drainage area to the interim pond includes ***3.94 Ha*** from the subject site; ***0.87 Ha*** of external drainage from Fringewood Drive, Existing Residential and an Existing Restaurant on Hazeldean Road and ***3.91 Ha*** of Hazeldean Road widening for a total of ***8.72 Ha***. This results in an increase in ***1.37 Ha*** compared to the contemplated drainage in the ***Hazeldean SWM Report***.

The pond sizing was reviewed to confirm if it can accommodate the additional site drainage and external flow not contemplated in the ***Hazeldean SWM Report.*** Interim Westland Quality Control is summarized in ***Table 10,*** below, refer to ***Appendix D*** for quality control calculations.

Table 10
Interim Wetland Quality Control

	Area (Ha)	Impervious (%)	Required Extended Detention (m^3)	Required Permanent Pool (m^3)
Per ***Hazeldean SWM Report***	7.35	77%	294	331
Per 5 Orchard FSR	8.72	71%	349	401
Provided Volumes in Interim SWM Pond per ***Hazeldean SWM Report***			406	432

The interim Wetland facility has sufficient permanent pool and extended detention volume to treat the drainage area from the development and external area to the required ***80% TSS Removal***.

Upon the decommissioning of the Hazeldean Road interim Wetland, it is proposed to achieve the quality control of 80% TSS removal through the implementation of an Oil/Grit Separator (OGS). The proposed OGS would be installed downstream of the interim wetland and will discharge to the existing ditch as shown on figure 1 provided in ***Appendix D***. The OGS has been sized to treat all drainage areas that are directed in the interim to the Wetland. Detailed description of cost and reasonability is included in a separate memo, ***External SWM Cost Implications***, included in ***Appendix D*** of this report. Sizing report and shop drawings for the proposed OGS are also included in ***Appendix D***.

5.6 Stormwater Management Conclusions

Post development stormwater runoff will be required to be restricted to the allowable target release rate for storm events up to and including the 100-year storm, in accordance with City of Ottawa, ***City Standards***. The post-development allowable release rate to the sewer within Hazeldean Road was calculated to be ***251.9 L/s***; with an estimated ***416.9 m^3*** of storage required for the residential development and ***843.1 m^3*** of storage required in the future commercial development in order to meet this release rate.

Four blocks of townhomes will be required to be sump pumped due to the shallow connection to the existing storm sewer within Hazeldean Road.

Please refer to ***5 Orchard JFSA Memo*** and the ***External SWM Cost Implications,*** both located in ***Appendix D,*** for further information on Quality and Quantity controls in the existing and proposed conditions.

The proposed stormwater design conforms to all relevant ***City Standards*** and Policies for approval.

6.0 UTILITIES

Utility servicing will be coordinated with the individual utility companies prior to site development.

7.0 EROSION AND SEDIMENT CONTROL

Soil erosion occurs naturally and is a function of soil type, climate and topography. The extent of erosion losses is exaggerated during construction where vegetation has been removed and the top layer of soil becomes agitated.

Prior to topsoil stripping, earthworks or underground construction, erosion and sediment controls will be implemented and will be maintained throughout construction.

Silt fence will be installed around the perimeter of the site and will be cleaned and maintained throughout construction. Silt fence will remain in place until the working areas have been stabilized and re-vegetated.

Catch basins will have SILTSACKs installed under the grate during construction to protect from silt entering the storm sewer system.

A mud mat will be installed at the construction access in order to prevent mud tracking onto adjacent roads.

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

- Limit extent of exposed soils at any given time;
- Re-vegetate exposed areas as soon as possible;
- Minimize the area to be cleared and grubbed;
- Protect exposed slopes with plastic or synthetic mulches;
- Install silt fence to prevent sediment from entering existing ditches;
- No refueling or cleaning of equipment near existing watercourses;
- Provide sediment traps and basins during dewatering;
- Install filter cloth between catch basins and frames;
- Plan construction at proper time to avoid flooding; and
- Establish material stockpiles away from watercourses, so that barriers and filters may be installed.

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

- Verification that water is not flowing under silt barriers; and
- Clean and change filter cloth at catch basins.

8.0 CONCLUSION AND RECOMMENDATIONS

David Schaeffer Engineering Ltd. (DSEL) has been retained by Campanale Homes to prepare a Functional Servicing and Stormwater Management report in support of the application for Draft Plan of Subdivision for the proposed development at 5 Orchard Drive. The preceding report outlines the following:

- Based on boundary conditions provided by the City the existing municipal water infrastructure is capable of providing the proposed development with water within the City's required pressure range. Pressure check will need to be completed during construction to determine if pressure reducing valves will be required;
- The proposed development is anticipated to have a peak wet weather flow of ***4.51* L/s** directed to the Stittsville Trunk Sewer, the property has been contemplated in the sizing of the future sewer to be installed within Fringewood Drive;
- The proposed development will be required to attenuate post development flows to an equivalent release rate of ***251.9 L/s*** to the sewer within Hazeldean Road, for all storms up to and including the 100-year storm event;
- It is anticipated that ***416.9 m³*** of storage will be required for the residential development and ***843.1 m³*** of storage will be needed for the future commercial development to attenuate stormwater to the allowable release rate to the storm sewer within Hazeldean Road; and
- Utility services would need to be coordinated with utility companies prior to development.

Prepared by,
David Schaeffer Engineering Ltd.

JBB# 18-1006

Per: Steven L. Merrick, P.Eng

Reviewed by,
David Schaeffer Engineering Ltd.

Per: Stephen Pichette, P.Eng.

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LEGEND:

- EXISTING WATERMAIN
- EXISTING SANITARY SEWER
- EXISTING STORM SEWER
- EXISTING BELL LINE
- EXISTING GAS LINE
- EXISTING HYDRO LINE
- EXISTING OVERHEAD HYDRO LINE
- PROPERTY LINE
- EXISTING OVERLAND FLOW DIRECTION
- FUTURE DEVELOPMENT BY OTHERS
- SPILL ELEVATION
- SPILL ELEVATION
- SPILL ELEVATION

DSEL
david schaeffer engineering ltd

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

EXISTING CONDITIONS
5 ORCHARD DRIVE

PROJECT No.:	18-1006
SCALE:	1:500
DATE:	MARCH 2019
DRAWING No.	EX-1
SHEET NO.	1 OF 7

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LEGEND:

EXISTING DRAINAGE AREA
PROPERTY LINE
DICB 1 / 2.957 / 0.24 — AREA ID / AREA (HA) / RUNOFF COEFFICIENT
EXISTING OVERLAND FLOW DIRECTION
FUTURE DEVELOPMENT BY OTHERS
SPILL ELEVATION
SPILL ELEVATION

DSEL
david schaeffer engineering ltd

120 Iber Road, Unit 103
Stittsville, ON K2S 1E9
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Fax. (613) 836-7183
www.DSEL.ca

EXISTING STORMWATER DRAINAGE PLAN
5 ORCHARD DRIVE

PROJECT No.:	18-1006
SCALE:	AS NOTED
DATE:	MARCH 2019
DRAWING No.	EX-SWM-1
SHEET NO.	5 OF 7

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Appendix B

5 Orchard Drive Pre-Application Consultation Meeting Notes (July 22, 2019)

File Number: PC2019-0169
July 22, 2019

5 Orchard Drive
Pre-application Consultation Meeting Notes

Location: Room 5105E, City Hall
Date: July 16, 2019

Attendees: Colette Gorni, Planner, City of Ottawa
Laurel McCreight, Planner, City of Ottawa
Samantha Gatchene, Student Planner, City of Ottawa
Rosanna Baggs, Project Manager (Transportation), City of Ottawa
Lino Paoloni, Shell
Kerry K. Morrison, Shell
Bikram Arora, Shell
Tony Batten, AECOM
Cody Campanale, Campanale Homes
Nadia De Santi, WSP
Michael Hanifi, WSP
Sarah MacDonald, WSP

Comments from the Applicant

Campanale Homes:

1. Campanale Homes has applied for a Plan of Subdivision and a Zoning By-law Amendment for the lands municipally known as 5 Orchard Drive. There is intended to be both residential and commercial uses on the property. These applications are pending.
2. Residential development will occur in the rear portion of the property. A mix of townhomes, semi- and single-detached dwellings along a cul-de-sac is proposed.
3. A future commercial block is planned along Hazeldean Road. However, Campanale Homes has not submitted an application with City for this portion of the site.
4. Campanale Homes has an agreement with Shell to lease lands in the north eastern portion of the site for use as a gas station.
5. There are two blocks that are being dedicated to the City of Ottawa as a part of the Plan of Subdivision application. An 8-metre block is being dedicated for storm water tanks and a watermain to service the residential block. The other block being dedicated is identified as a pedestrian pathway.

WSP/Shell:

6. This is the first shell site in Ottawa that WSP is working on. There will likely be many more.
7. The applicant is proposing a gas station use on the leased portion of the site. There will also be associated gas pumps, car wash, and convenience store.
8. There is an interest in proceeding with the Shell gas station ahead of the rest of the Plan of Subdivision.
9. The conceptual site plan layout was designed based on the queuing line placement and fuel delivery routes within the site.

Planning Comments

1. This is a formal pre-application consultation meeting for a Site Plan Control Application - Standard. Application form, timeline and fees can be found here.
2. Please confirm the number of parking spaces provided. A total of 7 spaces is required under the Zoning by-law for the proposed convenience store use (3.4 per 100m^2).
3. Please provide some bicycle parking on the site for the customers of the convenience store. Based on the size of the proposed retail building, the Zoning By-law requires 0.8 spaces be provided, which should be rounded up.
4. Please refer to Section 112 - Provisions for Drive-Through Operations when designing the car wash facility on the site.
5. Registration of the associated subdivision is required before a building permit can be obtained. However, the applicant is encouraged to submit a site plan control application in advance of registration to begin the process.
6. Please reach out to the applicable Ward Councillor and set up a meeting to present plans for the site.

Urban Design Comments

1. The City prefers for drive through queuing lines be internal to the site and not adjacent to roadways.
2. Please provide landscaping along Hazeldean Road, and along the rear of the property. Coniferous trees would be a good option to provide year-round green.
3. Consider moving the convenience store building closer to Hazeldean Road.

4. Please note that the City of has 'Urban Design Guidelines for Gas Stations'.

Transportation Comments

1. Follow Traffic Impact Assessment Guidelines
    - Traffic Impact Assessment will be required.
    - Start this process asap.
    - Applicant advised that their application will not be deemed complete until the submission of the draft step 1-4, including the functional draft RMA package (if applicable) and/or monitoring report (if applicable).
2. ROW protection on Hazeldean is 37.5m even.
3. Corner triangles as per OP Annex 1 - Road Classification and Rights-of-Way at the following locations on the final plan will be required:
    - Local Road to Arterial Road: 5 metre x 5 metres
4. Noise Impact Studies required for the following:
    - Stationary (if there will be any exposed mechanical equipment due to the proximity to neighbouring noise sensitive land uses)
5. The curb line on Fringewood will be required to be adjusted so that the through lane is reduce to 3.5m in width.
6. It is recommended that the path that the WB-20 takes to service the fuel storage tanks be plan in a way to minimize the access widths; i.e. make use of the entire site for turning movements, this can be accomplished by the entering by the future full movement access at the west end of the site. Otherwise, make use of truck turning aprons to reduce the access widths.
7. The current configuration of the drive thru car was queue may lead to congestion at the pumps. Recommended to relocate the drive-thru entrance.
8. On site plan:
    - Show all details of the roads abutting the site up to and including the opposite curb; include such items as pavement markings, accesses and/or sidewalks.
    - Turning templates will be required for all accesses showing the largest vehicle to access the site; required for internal movements and at all access (entering and exiting and going in both directions). Provide on a separate drawing.

- Show all curb radii measurements; ensure that all curb radii are reduced as much as possible
- Show lane/aisle widths.
- Sidewalk is to be constructed as per City Specification 7.1.
- Grey out any area that will not be impacted by this application. Private access minimum distance to signalized intersection as per TAC design;
    - i. On Hazeldean 70m
    - ii. On Fringewood 15m
- Clear throat length for the commercial block as per TAC design will be dependent on the use of the entire site of the site. The RIRO should expect a throat length of a minimum 15-25m.

Engineering Comments

1. The Servicing Study Guidelines for Development Applications are available at the following link: https://ottawa.ca/en/city-hall/planning-and-development/information-developers/development-application-review-process/development-application-submission/guide-preparing-studies-and-plans
2. Record drawings and utility plans are available for purchase from the City's Information Centre. Contact the City's Information Centre by email at informationcentre@ottawa.ca or by phone at (613) 580-2424 x44455
3. Stormwater quantity control criteria – be consistent with the quantity control criteria that will be specified in the approved subdivision Servicing and Stormwater Management Report
4. Stormwater quality control – Consult with the Conservation Authority (MVCA) for their requirements. Include the correspondence with the MVCA in the stormwater/site servicing report.
5. Oil and Grit separator is required for the proposed use (gas station)
6. MECP ECA is required (Industrial sewage works-direct submission)
7. Sanitary quantity control criteria - be consistent with the quantity control criteria that will be specified in the approved subdivision Servicing and Stormwater Management Report

8. When calculating the composite runoff coefficient (C) for the site (post development), please provide a drawing showing the individual drainage area and its runoff coefficient.

9. When using the modified rational method to calculate the storage requirements for the site, the underground storage should not be included in the overall available storage. The modified rational method assumes that the restricted flow rate is constant throughout the storm which, in this case, underestimates the storage requirement prior to the 1:100 year head elevation being reached. Alternately, if you wish to include the underground storage, you may use an assumed average release rate equal to 50% of the peak allowable rate. Otherwise, disregard the underground storage as available storage or provide modeling to support the design.

10. Engineering plans are to be submitted on standard A1 size (594mm x 841mm) sheets.

11. Phase 1 ESA and Phase 2 ESA must conform to clause 4.8.4 of the Official Plan that requires that development applications conform to Ontario Regulation 153/04.

12. Provide the following information for water main boundary conditions:

    - Location map with water service connection location
    - Average daily demand (l/s)
    - Maximum daily demand (l/s)
    - Maximum hourly demand (l/s)
    - Fire flow demand (provide fire detailed flow calculations based on the fire underwriters survey method)
    - If you are proposing any exterior light fixtures, all must be included and approved as part of the site plan approval. Therefore, the lights must be clearly identified by make, model and part number. All external light fixtures must meet the criteria for full cut-off classification as recognized by the Illuminating Engineering Society of North America (IESNA or IES) and must result in minimal light spillage onto adjacent properties (as a guideline, 0.5 fc is normally the maximum allowable spillage). In order to satisfy these criteria, the applicant must provide certification from an acceptable professional engineer. The location of all exterior fixtures, a table showing the fixture types (including make, model, part number), and the mounting heights must be included on a plan.

File Number: PC2019-0169
July 22, 2019

Forestry Comments

1. If there are trees on site, a Tree Conservation Report (TCR) will be required.
2. A tree permit is required for the removal of trees.

TCR Requirements:

3. a Tree Conservation Report (TCR) must be supplied for review along with the various other plans/reports required by the City; an approved TCR is a requirement for Site Plan approval
4. any removal of privately-owned trees 10cm or larger in diameter requires a tree permit issued under the Urban Tree Conservation Bylaw; the permit is based on the approved TCR
5. the removal of City-owned trees will require the permission of Forestry Services who will also review the submitted TCR
6. the TCR may be combined with the Landscape Plan
7. the TCR must list all trees greater than 10cm in diameter by species, diameter and health condition;
8. the TCR must address all trees with a critical root zone that extends into the developable area – all trees that could be impacted by the construction that are outside the developable area need to be addressed.
9. Trees with a trunk that crosses/touches a property line are considered co-owned by both property owners; permission from the adjoining property owner must be obtained prior to the removal of co-owned trees
10. If trees are to be removed, the TCR must clearly show where they are, and document the reason they can not be retained – please provide a plan showing retained and removed treed areas
11. All retained trees must be shown and all retained trees within the area impacted by the development process must be protected as per City guidelines listed on Ottawa.ca
12. Please ensure newly planted trees have an adequate soil volume for their size at maturity. The following is a table of recommended minimum soil volumes:

Tree Type/Size	Single Tree Soil Volume (m3)	Multiple Tree Soil Volume (m3/tree)
Ornamental	15	9
Columnar	15	9
Small	20	12
Medium	25	15
Large	30	18
Conifer	25	15

13. The City requests that all efforts are made to retain trees – trees should be healthy, and of a size and species that can grow into the site and contribute to Ottawa’s urban forest canopy
14. For more information on the TCR process or help with tree retention options, contact Mark Richardson mark.richardson@ottawa.ca

MVCA

1. The commercial component of the site should connect independently to the proposed storm sewer within Fringewood Drive.
2. The total release rate for the entire commercial section of the site is 51.9 L/s (100yr). A total of 843.1 m3 of storage has been estimated to be needed for the commercial portion of the site which needs to be considered in the proposed development as well. It’s been mentioned that the commercial block is contemplated to use LID SWM techniques to attenuate to the allowable release rate.

Sincerely,

Colette Gorni
Planner I
Development Review - West

APPLICANT'S STUDY AND PLAN IDENTIFICATION LIST

Legend: **S** indicates that the study or plan is required with application submission.
A indicates that the study or plan may be required to satisfy a condition of approval/draft approval.

For information and guidance on preparing required studies and plans refer here:

S/A	Number of copies	ENGINEERING		S/A	Number of copies
S	15	1. Site Servicing Plan	2. Site Servicing Study	**S**	3
S	15	3. Grade Control and Drainage Plan	4. Geotechnical Study / Slope Stability Study	**S**	3
		5. Composite Utility Plan	6. Groundwater Impact Study		
		7. Servicing Options Report	8. Wellhead Protection Study		
S	9	9. Transportation Impact Assessment (TIA)	10.Erosion and Sediment Control Plan / Brief	**S**	3
S	3	11.Storm water Management Report / Brief	12.Hydro geological and Terrain Analysis		
		13.Hydraulic Water main Analysis	14.Noise / Vibration Study	**S**	3
	PDF only	15.Roadway Modification Functional Design	16.Confederation Line Proximity Study		

S/A	Number of copies	PLANNING / DESIGN / SURVEY		S/A	Number of copies
		17.Draft Plan of Subdivision	18.Plan Showing Layout of Parking Garage		
		19.Draft Plan of Condominium	20.Planning Rationale	**S**	3
S	15	21.Site Plan	22.Minimum Distance Separation (MDS)		
		23.Concept Plan Showing Proposed Land Uses and Landscaping	24.Agrology and Soil Capability Study		
		25.Concept Plan Showing Ultimate Use of Land	26.Cultural Heritage Impact Statement		
S	15	27.Landscape Plan	28.Archaeological Resource Assessment Requirements: **S** (site plan) **A** (subdivision, condo)		
S	2	29.Survey Plan	30.Shadow Analysis		
S	3	31.Architectural Building Elevation Drawings (dimensioned)	32.Design Brief (Included in Planning Rationale)	**S**	Available online
		33.Wind Analysis			

S/A	Number of copies	ENVIRONMENTAL		S/A	Number of copies
S	3	34.Phase 1 Environmental Site Assessment	35.Impact Assessment of Adjacent Waste Disposal/Former Landfill Site		
S	3	36.Phase 2 Environmental Site Assessment (depends on the outcome of Phase 1)	37.Assessment of Landform Features		
		38.Record of Site Condition	39.Mineral Resource Impact Assessment		
S	3	40.Tree Conservation Report	41.Environmental Impact Statement / Impact Assessment of Endangered Species		
		42.Mine Hazard Study / Abandoned Pit or Quarry Study	43.Integrated Environmental Review (Draft, as part of Planning Rationale)		

S/A	Number of copies	ADDITIONAL REQUIREMENTS		S/A	Number of copies
S	1	44. Applicant's Public Consultation Strategy (may be provided as part of the Planning Rationale)	45.		

Meeting Date: July 16, 2019 | Application Type: *Site Plan Control*

File Lead (Assigned Planner): Colette Gorni | Infrastructure Approvals Project Manager: Santhosh Kuruvilla

Site Address (Municipal Address): 5 Orchard Drive | *Preliminary Assessment: 1☐ 2☒ 3☐ 4☐ 5

*One (1) indicates that considerable major revisions are required before a planning application is submitted, while five (5) suggests that proposal appears to meet the City's key land use policies and guidelines. **This assessment is purely advisory and does not consider technical aspects of the proposal or in any way guarantee application approval.**

Please note that PDF versions of all the listed requirements must be submitted with the application, stored in a USB drive or CD

It is important to note that the need for additional studies and plans may result during application review. If following the submission of your application, it is determined that material that is not identified in this checklist is required to achieve complete application status, in accordance with the Planning Act and Official Plan requirements, the Planning, Infrastructure and Economic Development Department *will notify you of outstanding material required within the required 30 day period. Mandatory pre-application consultation will not shorten the City's standard processing timelines, or guarantee that an application will be approved. It is intended to help educate and inform the applicant about submission requirements as well as municipal processes, policies, and key issues in advance of submitting a formal development application. This list is valid for one year following the meeting date. If the application is not submitted within this timeframe the applicant must again pre-consult with the Planning, Infrastructure and Economic Development Department.*

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Appendix C

City of Ottawa – Development Servicing Study Checklist

Servicing study guidelines for development applications

4. Development Servicing Study Checklist

The following section describes the checklist of the required content of servicing studies. It is expected that the proponent will address each one of the following items for the study to be deemed complete and ready for review by City of Ottawa Infrastructure Approvals staff.

The level of required detail in the Servicing Study will increase depending on the type of application. For example, for Official Plan amendments and re-zoning applications, the main issues will be to determine the capacity requirements for the proposed change in land use and confirm this against the existing capacity constraint, and to define the solutions, phasing of works and the financing of works to address the capacity constraint. For subdivisions and site plans, the above will be required with additional detailed information supporting the servicing within the development boundary.

4.1 General Content

- ☐ Executive Summary (for larger reports only).
- ☒ Date and revision number of the report.
- ☒ Location map and plan showing municipal address, boundary, and layout of proposed development.
- ☒ Plan showing the site and location of all existing services.
- ☒ Development statistics, land use, density, adherence to zoning and official plan, and reference to applicable subwatershed and watershed plans that provide context to which individual developments must adhere.
- ☒ Summary of Pre-consultation Meetings with City and other approval agencies.
- ☒ Reference and confirm conformance to higher level studies and reports (Master Servicing Studies, Environmental Assessments, Community Design Plans), or in the case where it is not in conformance, the proponent must provide justification and develop a defendable design criteria.
- ☒ Statement of objectives and servicing criteria.
- ☒ Identification of existing and proposed infrastructure available in the immediate area.
- ☐ Identification of Environmentally Significant Areas, watercourses and Municipal Drains potentially impacted by the proposed development (Reference can be made to the Natural Heritage Studies, if available).
- ☒ Concept level master grading plan to confirm existing and proposed grades in the development. This is required to confirm the feasibility of proposed stormwater management and drainage, soil removal and fill constraints, and potential impacts to neighbouring properties. This is also required to confirm that the proposed grading will not impede existing major system flow paths.
- ☐ Identification of potential impacts of proposed piped services on private services (such as wells and septic fields on adjacent lands) and mitigation required to address potential impacts.
- ☐ Proposed phasing of the development, if applicable.

☒ Reference to geotechnical studies and recommendations concerning servicing.

☒ All preliminary and formal site plan submissions should have the following information:

- Metric scale
- North arrow (including construction North)
- Key plan
- Name and contact information of applicant and property owner
- Property limits including bearings and dimensions
- Existing and proposed structures and parking areas
- Easements, road widening and rights-of-way
- Adjacent street names

4.2 Development Servicing Report: Water

☐ Confirm consistency with Master Servicing Study, if available

☒ Availability of public infrastructure to service proposed development

☐ Identification of system constraints

☐ Identify boundary conditions

☐ Confirmation of adequate domestic supply and pressure

☐ Confirmation of adequate fire flow protection and confirmation that fire flow is calculated as per the Fire Underwriter's Survey. Output should show available fire flow at locations throughout the development.

☐ Provide a check of high pressures. If pressure is found to be high, an assessment is required to confirm the application of pressure reducing valves.

☐ Definition of phasing constraints. Hydraulic modeling is required to confirm servicing for all defined phases of the project including the ultimate design

☐ Address reliability requirements such as appropriate location of shut-off valves

☐ Check on the necessity of a pressure zone boundary modification.

☐ Reference to water supply analysis to show that major infrastructure is capable of delivering sufficient water for the proposed land use. This includes data that shows that the expected demands under average day, peak hour and fire flow conditions provide water within the required pressure range

- ☐ Description of the proposed water distribution network, including locations of proposed connections to the existing system, provisions for necessary looping, and appurtenances (valves, pressure reducing valves, valve chambers, and fire hydrants) including special metering provisions.
- ☐ Description of off-site required feedermains, booster pumping stations, and other water infrastructure that will be ultimately required to service proposed development, including financing, interim facilities, and timing of implementation.
- ☐ Confirmation that water demands are calculated based on the City of Ottawa Design Guidelines.
- ☐ Provision of a model schematic showing the boundary conditions locations, streets, parcels, and building locations for reference.

4.3 Development Servicing Report: Wastewater

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

4.4 Development Servicing Report: Stormwater Checklist

- ☒ Description of drainage outlets and downstream constraints including legality of outlets (i.e. municipal drain, right-of-way, watercourse, or private property)
- ☒ Analysis of available capacity in existing public infrastructure.
- ☐ A drawing showing the subject lands, its surroundings, the receiving watercourse, existing drainage patterns, and proposed drainage pattern.
- ☒ Water quantity control objective (e.g. controlling post-development peak flows to pre-development level for storm events ranging from the 2 or 5 year event (dependent on the receiving sewer design) to 100 year return period); if other objectives are being applied, a rationale must be included with reference to hydrologic analyses of the potentially affected subwatersheds, taking into account long-term cumulative effects.
- ☒ Water Quality control objective (basic, normal or enhanced level of protection based on the sensitivities of the receiving watercourse) and storage requirements.
- ☒ Description of the stormwater management concept with facility locations and descriptions with references and supporting information.
- ☐ Set-back from private sewage disposal systems.
- ☐ Watercourse and hazard lands setbacks.
- ☐ Record of pre-consultation with the Ontario Ministry of Environment and the Conservation Authority that has jurisdiction on the affected watershed.
- ☒ Confirm consistency with sub-watershed and Master Servicing Study, if applicable study exists.
- ☒ Storage requirements (complete with calculations) and conveyance capacity for minor events (1:5 year return period) and major events (1:100 year return period).
- ☐ Identification of watercourses within the proposed development and how watercourses will be protected, or, if necessary, altered by the proposed development with applicable approvals.
- ☒ Calculate pre and post development peak flow rates including a description of existing site conditions and proposed impervious areas and drainage catchments in comparison to existing conditions.
- ☐ Any proposed diversion of drainage catchment areas from one outlet to another.
- ☒ Proposed minor and major systems including locations and sizes of stormwater trunk sewers, and stormwater management facilities.
- ☐ If quantity control is not proposed, demonstration that downstream system has adequate capacity for the post-development flows up to and including the 100 year return period storm event.
- ☐ Identification of potential impacts to receiving watercourses
- ☐ Identification of municipal drains and related approval requirements.
- ☒ Descriptions of how the conveyance and storage capacity will be achieved for the development.
- ☒ 100 year flood levels and major flow routing to protect proposed development from flooding for establishing minimum building elevations (MBE) and overall grading.

- ☒ Inclusion of hydraulic analysis including hydraulic grade line elevations.
- ☒ Description of approach to erosion and sediment control during construction for the protection of receiving watercourse or drainage corridors.
- ☐ Identification of floodplains – proponent to obtain relevant floodplain information from the appropriate Conservation Authority. The proponent may be required to delineate floodplain elevations to the satisfaction of the Conservation Authority if such information is not available or if information does not match current conditions.
- ☐ Identification of fill constraints related to floodplain and geotechnical investigation.

4.5 Approval and Permit Requirements: Checklist

The Servicing Study shall provide a list of applicable permits and regulatory approvals necessary for the proposed development as well as the relevant issues affecting each approval. The approval and permitting shall include but not be limited to the following:

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

4.6 Conclusion Checklist

- ☒ Clearly stated conclusions and recommendations
- ☐ Comments received from review agencies including the City of Ottawa and information on how the comments were addressed. Final sign-off from the responsible reviewing agency.
- ☒ All draft and final reports shall be signed and stamped by a professional Engineer registered in Ontario

Appendix D

Sanitary Sewer Calculations

Sanitary Sewer Design Sheet				
Site location (City)	5 Orchard Drive, Stittsville, ON	n= 0.013	Checking Date (yyyy/mm/dd)	2020-03-24
Ref#			Reviewer	Qasim Shafi

Commercial flow: 28000L/ha/D

Infiltration: 0.33 (L/s/ha)

q = *average* daily per capita flow (L/cap.d)

I = Unit of peak extraneous flow (L/s/ha)

Q(p) = peak population flow (L/s)

Q(I) = peak extraneous flow (L/s)

Q(d) = peak design flow (L/s)

Peaking Factor: 1.5

Q(p) = (P/1000)qM/86.4 (L/s)

Q(I) =IA (L/s); where A= Area in hectares

Q(d) = Q(p) + Q(I) (L/s)

Manning Equation:

Qcap. = (D/1000)^2 . 6 67*(S/100)^0.5/(3.211*n)*1000 (L/s)

D: pipe size (mm)

S: slope (grade) of pipe(%)

n: roughness coefficient

Location			Inlet Flow								Pipe										
Node ID					Cumulative		Cumulative	Peak Flow			Length							Velocity			
Upper	Lower	Link ID	Site Area (ha)	Cum Area (ha)	Avg. Daily Q(p) (l/s)	Peaking Factor	Infiltration Q(I) (l/s)	Q(p) DWF (l/s)	Q(d) WWF (l/s)	m	Dia of Pipe, mm	Grade of Pipe	Q Capacity (l/s)	Velocity	Qdesign/ Qcap	Capacity (%)	Actual Flow (m/s)	Type of Pipe	U.S. Invert	D.S. Invert
SSMH1	SSMH2	C1	0.11	0.11	0.03	1.50	0.04	0.05	0.09	28.74	200	0.40%	20.72	0.66	0.00	0.42%	0.21	PVC	101.977	101.862
Proceptor	SSMH2	C2	0.20	0.20	0.06	1.50	0.07	0.10	0.16	6.02	150	1.00%	15.21	0.86	0.01	1.07%	0.34	PVC	101.903	101.855
SSMH2	SSMH3	C3	0.00	0.31	0.10	1.50	0.10	0.15	0.25	3.93	200	1.00%	32.75	1.04	0.01	0.76%	0.33	PVC	101.660	101.644
Connection	SSMH4	C4	1.52	1.52	0.49	1.50	0.50	0.74	1.24	31.00	200	0.40%	20.72	0.66	0.06	5.99%	0.36	PVC	101.810	101.726
SSMH4	SSMH3	C5	0.00	1.52	0.49	1.50	0.50	0.74	1.24	5.27	200	0.40%	20.72	0.66	0.06	5.99%	0.36	PVC	101.681	101.660
SSMH3	EX. SAN 104	C6	0.00	1.83	0.59	1.50	0.60	0.89	1.49	13.70	200	0.65%	26.41	0.84	0.06	5.65%	0.46	PVC	101.630	101.541

Appendix E

Supporting Storm System Information and Calculations

- Email Communication from Campanale Group regarding Restricted Outflow Rate (November 2019)
- Particle Size Distribution for Oil Grit Separator Sizing
- Rational Method Spreadsheet Calculations for Site Storm Sewer System

Brown, Rikke

From:	Ronne, Joel
Sent:	Monday, November 25, 2019 6:25 PM
To:	Shafi, Qasim
Cc:	Reid, Jason; Patterson, Al
Subject:	Shell - Hazeldean Dr & Fringewood - SWM Design requirements
Attachments:	SITE PLAN-Parsons (003)-Model.pdf

Hi Qasim,
Any issues with the Second item below?

Joel Ronne, PEng
Feasibility Manager, Shell Program
D: 604.444.6542; C: 778.928.7519
joel.ronne@aecom.com

From: Cody Campanale <Cody@campanale.com>
Sent: November-25-19 2:51 PM
To: Ronne, Joel <joel.ronne@aecom.com>
Subject: Transportation Plan - 5 Orchard Drive

Hi Joel,

First:
It took me long enough, but here is the Site Plan you should use to incorporate your Site Plan into to run the transportation study. Our idea, confirmed by our Transportation Consultant is to highlight the major access points into and out of the lands, as well as a legend detailing the maximum amount of units/commercial space that will be built on the lands.

Second:
In regards to Stormwater Management, as I let Lino know via email, we are fine with Shell maintaining their own Stormwater Management providing that our engineer is able to review/approve the designs being submitted to the City, and that Shell stick to a release rate of 9.8 L/s during a 100-year storm event. See below comment from DSEL:

> *"The commercial block was allotted a total release rate of 51.9 L/s (1.82 ha, C=0.90). It was estimated that approximately 843.1 m3 of storage would be required to meet this release rate. Based on what was allotted for the total commercial area, a flow rate of 28.5 L/s/ha is estimated for the lands. The last Site plan provided estimated an area of 0.343 ha for the Shell lands. Therefore the release rate for Shell was estimated to be 9.8 L/s during a 100-year storm event."*

Let me know if you have any questions.

Thanks,
Cody

Cody Campanale
Land Development
Campanale Group
1187 Bank St., Suite 200

Direct Line: 613-247-3089

ADS Canada

ADS OGS Sizing Summary

Project Name:	Shell, 5 Orchard Drive
Consulting Engineer:	AECOM
Location:	Ottawa, ON
Sizing Completed By:	C. Neath
Email:	cody.neath@ads-pipe.com

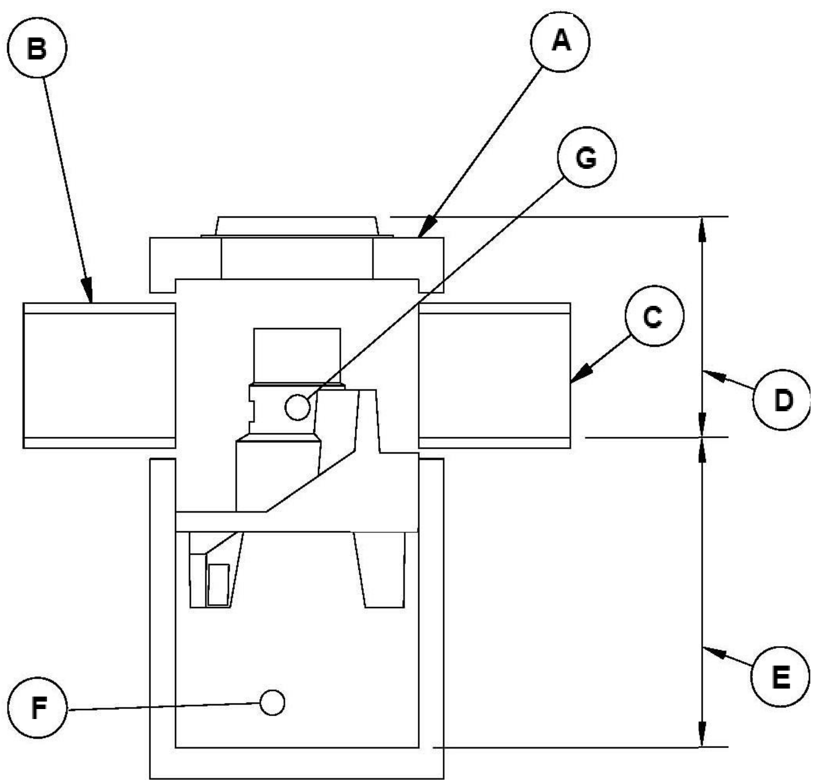
Treatment Requirements		
Treatment Goal:	Enhanced (MOE)	
Selected Parameters:	80% TSS	90% Volume
Selected Unit:	**ADS FD-5HC**	

Site Details	
Site Area:	0.266 ha
% Impervious:	---
Rational C:	0.92
Rainfall Station:	Ottawa, ONT
Particle Size Distribution:	Fine
Peak Flowrate:	--- L/s

Summary of Results

Model	TSS Removal	Volume Treated
ADS FD-4HC	94.1%	99.9%
ADS FD-5HC	96.3%	99.9%
ADS FD-6HC	97.4%	99.9%
ADS FD-8HC	98.6%	99.9%

ADS FD-5HC Specification	
Unit Diameter (A):	1,500 mm
Inlet Pipe Diameter (B):	300 mm
Outlet Pipe Diameter (C):	450 mm
Height,T/G to Outlet Invert (D):	1200 mm
Height, Outlet Invert to Sump (E):	1,500 mm
Sediment Storage Capacity (F):	0.84 m³
Oil Storage Capacity (G):	1,135 L
Max. Pipe Diameter:	600 mm
Peak Flow Capacity:	566 L/s



Site Elevations:	
Rim Elevation:	100.00
Inlet Pipe Elevation:	98.80
Outlet Pipe Elevation:	98.80

Notes:

Removal efficiencies are based on NJDEP Test Protocols and independently verified.

All units supplied by ADS have numerous local, provincial, and international certifications (copies of which can be provided upon request). The design engineer is responsible for ensuring compliance with applicable regulations.

Nikfarjam, Toktam

From: Haider Nasrullah <Haider.Nasrullah@ads-pipe.com>
Sent: Thursday, April 2, 2020 8:14 AM
To: Nikfarjam, Toktam; Cody Neath
Cc: Michael Reid
Subject: [EXTERNAL] RE: OGS Sizing, City of Ottawa

Hi Toktam,

The Particle Size Distribution (PSD) used to size the unit was the Fine PSD. Please see below.

Particle Size (micron)	Distribution (%)
20	20
60	20
150	20
400	20
2000	20

Regards,

Haider Nasrullah, P.Eng.
Engineered Products Manager

Storm Sewer Hydraulic Design Sheet

Site location (City)	5 Orchard Drive, Stittsville, ON	n= 0.013	Checking Date (yyyy/mm/dd)	2020-03-24
Ref#			Reviewer	Qasim Shafi

Design Storm: The 5 **Year Storm Event**

Rational Formula: Q = CIA
Where: Q: peak flow (m^3/s)
C: runoff coefficient
I: rainfall intensity (mm/h)
A: area (ha)

Concentration time: tc =ti+ tf (minute)
Where: ti: inlet time before pipe (minute)
tf: time of flow in pipe (minute) tf = L/(60V) (minute)

Manning Equation:
Qcap. $=1/_{(100n)}*A*R^{2/3}*S^{1/2}$
D: pipe size (m)
S: slope (grade) of pipe(%)
n: roughness coefficient

Qcap. = (D/1000)"2.667*(S/100)"0.5/(3.211*n)*1000 (Lis)

Rainfall intensity formula
$I = A\ T^{C}$

Location			Runoff							Pipe											
Pipe ID	Upper Node	Lower Node	Area (ha)	Runoff Coefficient (C)	Sec AC (ha)	Accum AC (ha)	Conc. Time (min)	Rainfall Intensity I (mm/hr)	Peak Flow Q (m^3/s)	Dia. (m)	Grade (%)	Length (m)	Q_{full} (m^3/s)	V_{full} (m/s)	Travel Time in Pipe (min)	Q/Qfull	Capacity (%)	Velocity Act. Flow (m/s)	Type of Pipe	U.S. Invert	D.S. Invert
P1	CBMH13	CBMH01	0.030	0.80	0.024	0.024	10.00	104.193	0.007	1.200	1.00	14.66	3.899	3.447	0.222	0.002	0.18%	1.101	PVC	103.007	102.860
P7	MH09	CBMH01	0.000	0.00	0.000	0.000	10.00	104.193	0.000	1.200	0.35	21.00							Concrete	102.903	102.830
P2	CBMH01	CBMH02	0.060	0.87	0.052	0.076	10.22	103.035	0.022	1.200	0.34	22.60	2.273	2.010	0.587	0.010	0.96%	0.642	PVC	102.800	102.721
P8	MH11	CBMH02	0.000	0.00	0.000	0.000	10.00	104.193	0.000	1.500	0.35	19.00							Concrete	102.788	102.721
P3	CBMH02	CBMH03	0.106	0.85	0.091	0.167	10.81	100.107	0.046	1.200	0.34	17.28	2.273	2.010	0.364	0.020	2.04%	0.791	PVC	102.691	102.631
P15	CB12	CBMH03	0.023	0.66	0.015	0.015	10.00	104.193	0.004	0.300	1.00	8.51	0.097	1.368	0.199	0.045	4.54%	0.713	PVC	103.185	103.100
P4	CBMH03	MH01	0.035	0.66	0.023	0.205	11.17	98.382	0.056	1.200	0.34	13.15	2.273	2.010	0.277	0.025	2.47%	0.791	PVC	102.600	102.554
P5	MH01	OGS	0.012	0.90	0.011	0.216	11.45	97.112	0.058	0.450	0.35	4.00	0.169	1.061	0.069	0.345	34.53%	0.967	PVC	102.553	102.519
P6	OGS	MH108	0.000	0.00	0.000	0.216	11.52	96.801	0.058	0.450	0.35	4.25	0.169	1.061	0.074	0.344	34.42%	0.959	PVC	102.444	102.429
P10	MH07	CBMH06	0.000	0.00	0.000	0.000	10.00	104.193	0.000	1.050	0.30	26.00							Concrete	102.581	102.503
P11	CBMH06	MH04	0.056	0.66	0.037	0.037	10.00	104.193	0.011	0.450	0.30	2.50	0.156	0.982	0.074	0.069	6.89%	0.565	PVC	102.482	102.475
P12	Connection	MH04					10.00		0.025	0.450	0.30	28.26	0.156	0.982	0.655	0.161	16.14%	0.719	PVC	102.560	102.475
P13	MH04	MH108	0.000	0.00	0.000	0.037	10.65	100.857	0.010	0.450	0.34	5.26	0.166	1.045	0.153	0.063	6.27%	0.575	PVC	102.445	102.429
P14	MH108	STM106	0.000	0.00	0.000	0.253	11.59	96.472	0.068	0.675	0.51	19.08	0.600	1.678	0.289	0.113	11.30%	1.102	PVC	102.399	102.980

5 Year storm IDF data	A	998.071	B	0.814	C	6.053

Runoff Co-efficient

Node	Catchment Area (ha)				Cummulative Runoff Coefficient, C	Drainage Area ID (Drwg C105.0)
	Roof	Paved	Grass/ Landscape	Total		
	0.9	0.9	0.3			
CBMH13	0.018	0.007	0.005	0.030	0.80	A4
MH09	0.000	0.000	0.000	0.000	0.00	
CBMH01	0.000	0.057	0.003	0.060	0.87	A1
CBMH02	0.020	0.078	0.008	0.106	0.85	A2
MH11	0.000	0.000	0.000	0.000	0.00	
CBMH03	0.000	0.021	0.014	0.035	0.66	A5
CB12	0.000	0.002	0.021	0.023	0.35	A6-2
MH01	0.012	0.000	0.000	0.012	0.90	A9
OGS	0.000	0.000	0.000	0.000	0.00	
MH108	0.000	0.000	0.000	0.000	0.00	
MH07	0.000	0.000	0.000	0.000	0.00	
CBMH06	0.000	0.034	0.022	0.056	0.66	A3 & A8
MH04	0.000	0.000	0.000	0.000	0.00	
			Total Area (ha)	**0.322**		

Note: Flow from connection to adjacent commercial development west of site is calculated to be 25.2 L/s, based on a 5-year release rate of 30.3 L/s per 1.83 ha shown in the "Functional Servicing and Stormwater Management Report for Campanale Homes 5 Orchard Drive" by David Schaeffer Engineering Ltd., March 2019.
The following calculation was used: 30.3L/s X (1.83 ha - 0.3065 ha) / (1.83 ha) = 25.2 L/s

Appendix F

Shell Canada Stormwater Management Report 5 Orchard Drive, Stittsville, City of Ottawa (AECOM, 2020)

DRAFT

AECOM Imagine it. Delivered.

Shell Canada

Stormwater Management Report

5 Orchard Drive, Stittsville, City of Ottawa

Prepared by:

AECOM Canada Ltd.
105 Commerce Valley Drive West, 7th Floor
Markham, ON L3T 7W3
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T: 905.886.7022
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400 – 4 Avenue SW
Calgary, AB T2P 0J4

Date: March, 2020

Project #: 60593779

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Revision History

Rev #	Date	Revised By:	Revision Description

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- represents AECOM's professional judgement in light of the Limitations and industry standards for the preparation of similar reports;
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- was prepared for the specific purposes described in the Report and the Agreement; and
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AECOM: 2015-04-13

Authors

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Toktam Nikfarjam, P.Eng
Water Engineer

Report Reviewed By:

Ken Pun, P.Eng
Senior Water Resource Engineer

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Appendices

1. Introduction

Shell Canada has retained the AECOM Canada Ltd. to complete Stormwater Management designs and prepare a report for the proposed Shell Site at 5 Orchard Drive, Stittsville, in Ottawa, Ontario. In 2019, Davis Shaeffer Engineering Ltd (DSEL), prepared Functional Servicing and Stormwater Management Report for the Campanale Homes, proposed development at 5 Orchard Drive, in Ottawa, Ontario. This site is bounded by Hazeldean Road to the north, Fringewood Drive to the east, an existing restaurant to the west and existing residential development to the south. The future development consists of 1.82 ha of commercial space and 2.13 ha of residential land. The Shell site development has a total area of 0.306 ha in part of the commercial block with 0.027ha external area.

This stormwater management report (Report) addresses the stormwater management option for the Shell Site located at part of commercial space (Refer to **Figure 1**). The report summarized the stormwater management (SWM) design requirements and proposed works to address stormwater flows arising from the site under post-development conditions and identify any storm water servicing concerns. The report describes any measures to be taken during construction to minimize erosion and sedimentation for the proposed Shell Site development located at 5 Orchard Drive, Ottawa, Ontario.

Figure 1: Site Location

Governing design criteria from the; City of Ottawa's (City) Sewer Design Guidelines, Second Edition (SDG002), October 2012, Technical Bulletin Piedtb – 2016-01, Revisions to Ottawa Design Guidelines – Sewer, the Mississippi Valley Conservation Authority (MVCA), and Ministry of Environment and Conservation and Parks (MECP), Stormwater Management Planning and Design Manual (March 2003) are applied in order to control the storm discharge quantity and quality from the site to meet pre-construction flow conditions. This can include, but is not limited to, combination of absorbent landscaping, Oil and Grit Separator and onsite oversized pipe as well as depression surface areas for storage.

The site has a total area of 0.306 ha and is currently undeveloped. The proposed works will consist of a 168 square meter convenience store, a 97 square meter carwash, a pump island on a 240 square meter concrete apron with a 198 square meter canopy, access roadway and parking areas, and two (2) underground fuel storage tanks.

The following materials were reviewed in the preparation of this report:

- Geotechnical Investigation Report, Proposed Shell Service Station, 5 Orchard Drive, Ottawa, Ontario, prepared by GEMTEC Consulting Engineers and Scientists, July 2019,
- Functional Servicing and Stormwater Management Report for Campanale Homes Development, 5 Orchard Drive, City of Ottawa, Prepared by David Schaeffer Engineering Ltd (DSEL), March 2019,
- Sewer Design Guidelines, Second Edition (SDG002), City of Ottawa, October 2012,
- Technical Bulletin PIEDTB – 2016-01, Revisions to Ottawa Design Guidelines – Sewer, September 2016,
- Stormwater Management Planning and Design Manual, Ministry of Environment and Conservation and Parks (MECP), March 2003, and
- Pre-application Consultation Meeting Notes, 5 Orchard Drive, July 2019.

2. Analysis Methodology

- The Rational Method is used to calculate the stormwater flow rates based on: 2-year, 5-year, 10-year, 25- year, 50-year, and 100-year storm intensity.

$$Q = 0.0028 \times Ca * C * I * A$$

where:

Q – discharge flow rate in cubic metres per second

Ca – antecedent coefficient for storm intensities meeting City of Ottawa requirements

C – surface runoff coefficient, as outlined in **Table 1**: Coefficient (C) Values

I – storm intensity in mm/hour

A – site area in hectares (ha)

- The site is categorized into three main sub-areas including; buildings, green areas (grass/vegetation), and asphalt surfacing.
- With reference to the Geotechnical Report prepared by GEMTEC Consulting Engineers and Scientists, the surface grade at the borehole locations consists of dark brown clayey silt topsoil and a deposit of brown silt with some clay and trace sand was encountered below the topsoil (Refer to **Appendix A**).
- Runoff coefficient value was selected as below for the above-mentioned land use for the existing and proposed conditions:

Table 1: Coefficient (C) Values from City's SDG002

Description	C
Building	0.9
Pavement – Asphalt	0.9
Grass – Vegetation*	0.3

*Based on the type of the soil (silt and some clay), the Runoff Coefficient of 0.3 was selected for the grass/vegetation areas (Refer to **Table 2**)

Table 2: Runoff Coefficients for Various Soil Conditions

Topography and Vegetation		Soil Texture		
		Open Sandy Loam	Clay and Silt Loam	Tight Clay
Woodland				
Flat	0-5 % Slope	0.10	0.30	0.40
Rolling	5-10% Slope	0.25	0.35	0.50
Hilly	10-30% Slope	0.3	0.50	0.60
Pasture				
Flat	0-5 % Slope	0.10	0.30	0.40
Rolling	5-10% Slope	0.16	0.36	0.55
Hilly	10-30% Slope	0.22	0.42	0.60
Cultivated				
Flat	0-5 % Slope	0.30	0.50	0.60
Rolling	5-10% Slope	0.40	0.60	0.70
Hilly	10-30% Slope	0.53	0.72	0.82

Note: Reference to the Table 5.7 of City of Ottawa SDG002.

- Time of Concentration (Tc): The City's SDG002 stipulated that minimum initial time of concentration is to be 10 minutes. This minimum time was calculated and checked for the subject site using the Airport Formula method while the weighted runoff coefficient is less than 0.4 (Refer to **Appendix B** for the calculation). The time of concentration using Airport Formula is calculated as 26.37 minutes. However, since this calculated time is bigger than the City's requirement, the Time of Concentration of 10 minutes considered for the subject site.
- The rainfall intensity for the site was calculated using the following equation:

$$I = \frac{\mathrm{A}}{(\mathrm{Tc} + \mathrm{C})\hat{}\mathrm{B}}$$

where:

I = intensity of rainfall in mm/hour

Tc = time of concentration in 10 minutes

Table 3: Rainfall Intensity (mm/hr)

Parameter	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr
A=	732.951	998.071	1174.184	1402.884	1569.58	1735.688
B=	0.81	0.814	0.816	0.819	0.82	0.82
C=	6.199	6.053	6.014	6.018	6.014	6.014
i=	76.81	104.19	122.14	144.69	161.47	178.56

Note: The numbers for A, B and C values have taken from Section 5.4.2 of City of Ottawa SDG002.

3. Design Standards and Criteria Review

The following discusses a review of the applicable stormwater management (SWM) criteria for the subject development.

3.1 City of Ottawa – Sewer Design Guidelines (Second Edition, October 2012) and Technical Bulletin PIEDTB – 2016-01, Revisions to Ottawa Design Guidelines – Sewer (September 2016)

The City's Sewer Design Guideline and Technical Bulletin PIEDTB are the governing document with respect to drainage and stormwater management in the City. The manual outlines requirements for quantity control, quality control and runoff volume reduction, as well as discharge criteria to municipal infrastructure. These requirements are typically translated into a need to either detain (i.e., attenuate and gradually release flows) or retain (i.e., reduce volume of) stormwater runoff.

3.1.1 Water Quantity

The following were considered to determine quantity control requirements:

- In existing separated areas, flow in the minor system must be controlled to meet the existing level of service of the existing receiving system, or the minor system must be designed to accommodate the runoff from a storm with the following return periods, whichever is less.
- For collector roads, the minor system shall be designed as a minimum for a 5-year return period under free flow conditions.
- Stormwater quantity control criteria must be consistent with the approved subdivision Servicing and Stormwater Management report.
- The minimum orifice opening for plate or plug type ICDs shall be 75 mm (round) or 67 mm x 67 mm (square or diamond). Vortech-type ICDs with a minimum of 6 l/s also can be acceptable.
- In order to account for the increase in runoff due to saturation of the catchment surface that would occur for larger, less frequent storms, the adjustment factor as shown below table shall be used:

Table 4: Adjustment Factor to Calculate the Flow Rate

Return Period	Adjustment Factor
10-Year	1.0
25-Year	1.1
50-Year	1.2
100-Year	1.25

Note: The numbers have taken from Section 5.4.5.2.1 of City of Ottawa SDG002.

3.1.2 Erosion and Sediment Control Criteria

Regardless of the size of the development site, temporary erosion and sediment control during construction must be provided to eliminate the opportunity for water borne sediments to be washed on to the adjacent properties and to delineate the environmental protection zones for trees and vegetation around the perimeter of the site. For small infill sites and site plans less than 5 hectares the minimum erosion control requirements is runoff reduction from the site through infiltration, evapotranspiration and reuse of a minimum 5 mm of rainfall depth across all impervious surfaces as a general practice.

3.1.3 Water Quality

The water quality target is the long-term average removal of 70% Total Suspended Solids (TSS) on an annual loading basis as per correspondence with the MVCA (Refer to **Appendix C**).

4. Existing Conditions

4.1 General

The total site area is 0.306 ha under existing conditions undeveloped land. There is no onsite water retention and/or detention is observed. Drawing C131.0 (Refer to **Appendix F**) provides a lot drainage configuration for the existing conditions. As it can be shown in this drawing most of the runoff generated from site is sheet flow to the adjacent properties and City road right-of-way. The surface runoff is generally split in two directions consisting of the:

- Catchment Area 1 (0.276 ha) draining to adjacent landscape area to the north toward Hazeldean Road.
- Catchment Area 2 (0.030 ha) drainage to adjacent landscape area to the east toward Fringewood Drive.

The calculated peak flow rates for the existing condition of the site for 2 to 100-year storm events are summarized in **Table 5**. Detailed calculations are provided in **Appendix B**.

Table 5: Existing Peak Flow

Return Period (Years)	Rainfall Intensity (mm/hr)	Peak Flow (L/s)
2	76.81	22.42
5	104.19	30.41
10	122.14	35.65
25	144.69	42.24
50	161.47	47.13
100	178.56	52.12

4.2 Site Soil Condition

A geotechnical investigation was conducted to obtain information on the existing subsurface conditions by means of three boreholes. The results show that the existing soil is predominantly dark brown clayey sill topsoil. The thickness of the topsoil soil is about 150 and 200 millimeters at the borehole locations. A deposit of brown silt with some clay and trace sand was encountered below the topsoil at all of the borehole locations. The silt has a thickness of about 0.8 meters and extends to a depth of about 0.9 meters below surface grade at the borehole locations. Glacial till was encountered below the silt at all of the borehole locations at a depth of about 0.9 meters below ground surface. In addition, below the glacial till, fractured and weathered bedrock was encountered at depths of 2.8 to 3.3 meters below ground surface. Groundwater was encountered at a depth of 1.7 m below ground surface of one of borehole (Borehole MW19-1, Refer to **Appendix A**).

5. Proposed Conditions

5.1 Site Developments and Site Grading

The proposed site development will include a 168 square meter convenience store, a 97 square meter carwash, a pump island on a 240 square meter concrete apron with a 198 square meter canopy, access roadway and parking areas, and two (2) underground fuel storage tanks (Refer to **Drawing C104.0** in **Appendix I**).

All grading will be completed in a manner to satisfy the following goals:

- Achieve proper road gradients to maintain sufficient site lines,
- Minimize cut to fill earth operations,
- Enable gravity servicing outlets,
- Reduce or eliminate the need for retaining walls, where feasible,
- Provide minimal impact to abutting properties,
- Achieve stormwater management and environmental objectives required for the proposed development, and
- Provide 15 cm of vertical clearance between the spill elevation on street and the ground elevation at the building envelope that is in the proximity of the flow route or ponding area.

The proposed grading design for the site will consist of grading through the concrete surfaced area with water ponding above the catchbasins in north and central side of the site. With this type of design, the majority of runoff will be collected in the storm sewers. For storm events larger than the minor system event, storm runoff will pond above the catchbasins and convey via the oversized pipe system. The maximum depth of ponding on the proposed site area will be 0.10 to 0.13 m above the catchbasin grates before spill. Dedicated stormwater systems will be constructed to collect all runoff and control from the site except Catchment Areas 6-1 and 7 are uncontrolled and direct discharges to the Fringewood Drive. Refer to **Drawing C103** and **Drawing C803** in **Appendix H** as well as **Section 5.2.2** for an overview of the proposed development.

5.2 Water Quantity

5.2.1 Total Allowable Release Flow Rate

Flow attenuation is required to ensure there are no adverse impacts on downstream system at the Fringewood Drive and Hazeldean Road. With respect to the Functional Servicing and Stormwater Management Report for Campanale Homes Development, the release rate for the commercial block for total area of 1.82 ha was calculated as 30.26 L/s and 51.85 L/s for the 5-year and 100-year storm event respectively. As a result, the release rate for the subject site should be 5.54 L/s and 9.5 L/s for 5-year storm event and 100-year storm event respectively (Refer to **Table 6**).

Table 6: Allowable Release Flow Rate

Remark	Area (ha)	5-year	100-year
Calculated Flow Rate* (L/s)	1.82	30.26	51.85
Allowable Release Rate (L/s/ha)	---	16.62	28.50
Allowable Release Rate for Shell Site (L/s)	0.333	5.54	9.50

**The value extracted from Functional Servicing and Stormwater Management Report, prepared by DESL, 2019*.

5.2.2 Control and Uncontrolled Area

The proposed development will consist of about a 0.048 ha building, and 0.204 ha will be asphalt surfaced. All remaining areas will be grassed/landscaped areas. For the purposes of this storm water management design, the site has been divided into uncontrolled and controlled areas as outlined on **Drawing C105.0** (Refer to **Appendix G**). As it shown, runoff from Catchment Area A6-1 (0.007 ha) and Catchment Area A7 (0.004 ha) are uncontrolled and sheet flow towards Fringewood Drive. Catchment Areas A1 through A5 as well as Catchment Areas A8 and A9, the runoff will be captured by the proposed catchbasin (CB) /catchbasin manhole (CBMH) and convey through new sewer system, which eventually directed to the existing 600 mm CONC STM located at Fringewood Drive. In order to capture the flow generated from Catchment Area A6-2, CBs and swale are proposed in this area which collect the runoff and convey the flow via a 300 mm diameter storm sewer pipe which connect to the CBMH-03.

5.2.3 External Area

As it shown in the **Drawing C131.0** (Refer to **Appendix F**), the foot print of the site for the existing condition is 0.306 ha. In the proposed condition Catchment Area of A8 with a 0.027 ha area which located outside of the lease line draining onto the site, as a result, total catchment area for the site will be 0.333 ha in proposed conditions.

5.2.4 Proposed Quantity Control and Post Development Restricted Flow

The following options were considered for quantity control on site:

Surface Storage – Surface ponding could be provided in depressions surface area at CBs or CBMHs to attenuate the flow. Surface storage is acceptable for storm events greater than the 5-year design storm.

Underground Storage – A restrictor could be installed inside the manhole at downstream of out flow storm pipe to control the release rate with surplus storage provided in the proposed oversize pipe systems, as necessary. For this subject site, excess runoff from the 5-year design storm event, the runoff is to be stored underground to meet the target release rate.

The calculated peak flow rates for the proposed condition during 5 to 100-year storm events are summarized in **Table 7**. Detailed calculations are provided in **Appendix B**. It should be noted that, to calculate the peak flow for different storm events, the required adjustment factors as indicated in **Section 3.1** have applied in order to account for the increase in runoff due to saturation of the catchment surface. As indicated in previous section, the flow from Catchment Areas A6-1 and A7 will be uncontrolled and discharge via sheet flow towards Fringewood Drive.

Table 7: Uncontrolled Proposed Peak Flow Rate

Catchment #	Area (ha)	Peak Flow Considering Adjustment Factor (L/s)					
		2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
Catchment Area 1	0.060	11.15	15.12	17.72	23.10	28.12	29.78
Catchment Area 2	0.106	19.34	26.24	30.76	40.09	48.80	52.62
Catchment Area 3	0.029	5.12	6.95	8.15	10.62	12.93	14.89
Catchment Area 4	0.030	4.87	6.60	7.74	10.09	12.28	14.15
Catchment Area 5	0.035	4.93	6.69	7.84	10.22	12.44	14.33
Catchment Area 6-1	0.007	0.58	0.78	0.92	1.19	1.45	1.68
Catchment Area 6-2	0.023	1.73	2.35	2.75	3.58	4.36	5.03
Catchment Area 7	0.004	0.77	1.04	1.22	1.59	1.94	1.99
Catchment Area 8	0.027	3.01	4.08	4.79	6.24	7.60	8.75
Catchment Area 9	0.012	2.11	2.87	3.36	4.38	5.33	5.46
Total	0.333	53.61	72.73	85.26	111.10	135.26	148.67

A comparison of existing and post-development peak flow rates at the site is presented in **Table 8**.

Table 8: Comparison of Peak Flow for the Existing and Proposed Conditions

Return Period	Peak Flow Existing (L/s)	Un-Controlled Peak Flow Proposed (L/s)	Target Release Rate (L/s)
2-Year	22.42	53.61	----
5-Year	30.41	72.73	5.54
10-Year	35.65	85.26	----
25-Year	42.24	111.10	----
50-Year	47.13	135.26	----
100-Year	52.12	148.67	9.5

As shown, the uncontrolled post-development peak flow rates are higher than existing peak flow, it is required 2 to 100-year post-development peak flows are controlled to target release flow rate as indicated in approved Functional Servicing and Stormwater Management Report for Campanale Homes Development prepared by DSEL. **Table 8** compares the calculated peak flows for the existing and proposed conditions as well as target release rate for 5-year and 100-year storm.

As indicated earlier, on the proposed runoff of 0.011 ha of the site (flow from Catchment Areas 6-1 and 7) are uncontrolled via sheet flow to Fringewood Drive. However, External Catchment Area A8 (0.027 ha) which is directing to the site by sheet flow will be controlled with the proposed sewer systems. In order to meet the allowable release rates, both options of surface storage and oversized pipe storage are considered for the site.

Various design options have been developed and evaluated to determine the best available option to meet the requirement and control the flow generated from the site as much as possible. It is identified that control the flow in different locations upstream of the site by using Vortech-type ICDs with underground oversize pipe storage would be the best effort to control the flow for proposed condition. An inlet control device (ICD) located at the outlet pipe of catch basin/manhole CBMH02, MH01, and CBMH07 will control the release of stormwater from subject site. The ICDs will restrict the flow and store the stormwater to underground storage pipes. The ICDs shall be a Hydrovex

"VHV Vertical Vortex Flow Regulator" and developed by the manufacturer for a discharge rate of 6 L/s at 2.48 m head. With this approach, three (3) – Vortech-type orifices (Unit 75VHV-1, Refer to **Appendix K**) have been proposed to control the flow and minimize the negative impact to the downstream outfall (Refer to **Drawing C105.0** in **Appendix G**).

The required storage volume is 134m3 to control the 100-year post-development flow rate to the target release rate (9.5 L/s) has been calculated utilizing the Modified Rational Method as shown in **Appendix B**. The proposed site area has been graded generally flat to distribute the surface storage over the site area. The total surface storage volume for the site has been calculated between the lowest catch basin top elevations and the overland spill elevations for the surface. Oversized pipes are proposed in the design to provide additional storage capacity as well as meet the acceptable release rate as much as possible. **Table 9** provides a summary of the available storage volume based on the proposed grading and storm sewer design and Vortech ICDs controls (Refer to **Appendix B** for detailed calculation).

Table 9: Potential Available Storage Within the Site

Potential Storage	Storage Volume (m^3)
Underground Storage (Oversized Pipe)	160.84
Surface Depressions	9.92
Total	170.76

A PCSWMM model was used to determine the volume and assess the hydraulic grade line and release rate at the site. Based on the proposed grading plan (Refer to **Drawing C104.0** in **Appendix I**) and result of PCSWMM model (Refer to **Appendix D** and **Table 10**), the release flow rate is 6 L/s and 10 L/s for 5-year and 100-year storm event respectively which closely meet the required target release rate. The storage depth on the surface will be in a range of 0.1 m to 0.13 m during a 100-year storm event. There will be no storage on the surface storage during a 5-year storm event.

In addition, in order to prevent the flood waters backing up into the storm sewer system from external, backflow preventer measures on ST.S.MH108 will be implemented.

Table 10: Comparison of Peak Flow for the Target Release Rate and Controlled Peak Flow under Proposed Condition

Return Period	Controlled Peak Flow Proposed (L/s)	Target Release Flow Rate (L/s)
2-Year	5	----
5-Year	6	5.54
10-Year	6	----
25-Year	7	----
50-Year	9	----
100-Year	10	9.5

5.3 Storm Sewer System

A hydrodynamic model (PCSWMM) was set up for the simulation of hydraulic capacity of the storm drainage system and assessment of the hydraulic grade line (HGL) for the proposed condition. PCSWMM utilizes the EPA SWMM5 engine and offers full dynamic modelling of conveyance systems. The key objective of the model is to assess the hydraulic performance of the proposed storm sewer system that could potentially be impacted by the proposed site development and proposed oversize pipe storage at the site.

5.3.1 Modelling Approach

The approach involved the following major milestones:

- The following data utilized in the development of the model: Detailed survey information, DEM, Aerial photo, existing site plan, as-built/ record drawing of properties surrounding the proposed site.
- Sub-catchments were delineated on a manhole-to-manhole basis based on topography and proposed drainage boundary.
- The model set up to simulate Chicago 3 hr and 6 hr using City's IDF information, to ensure that a 5-year level of protection is provided with no surcharge, also to ensure that the sewer system is protected against critical surcharging during the 100-year storm event without overtopping the manhole.
- There is at least 15 cm of vertical clearance between the spill elevation on street and the ground elevation at the building envelope that is in the proximity of the flow route or ponding area.
- The pipe roughness coefficient of 0.013 was used in the Manning Formula.
- The adjusted factor identified in **Table 4** has applied in model in order to size the oversize pipes to provide storage capacity.
- Rating curve for Vertical Vortex Flow Regulators (Unit 75VHV-1) generated on model (Refer to **Appendix K**).

Figure 2 to **Figure 9** show the 5-year and 100-year HGL profiles, respectively, considering underground storage pipes as well as surface storage, as simulated in PCSWMM. As shown, a combination of 1500 mm, 1200 mm, and 1050 mm PVC and/or concrete pipes with depression area provide sufficiently storage volume and the HGL remains below proposed surface grade.

Table 11: Proposed Storm Sewer Size

Pipe ID	Upstream Manhole ID	Downstream Manhole ID	Pipe Diameter (mm)	Roughness	Pipe Length (m)	Velocity* (m/s)	Flow * (L/s)
P1	CBMH13	CBMH01	1200	0.013	14.66	0.47	15
P2	CBMH01	CBMH02	1200	0.013	22.6	0.21	18
P3	CBMH02	CBMH03	1200	0.013	17.28	0.38	17
P4	CBMH03	MH01	1200	0.013	13.15	0.34	18
P5	MH01	OGS	450	0.013	4	0.54	5
P6	OGS	MH108	450	0.013	4.25	0.54	5
P7	MH09	CBMH01	1200	0.013	21	0.04	17
P8	MH11	CBMH02	1500	0.013	19	0.07	38
P9	CBMH07	CBMH06	1050	0.013	26	0.04	19
P10	Connection	CBMH04	450	0.013	28.26	0	0
P11	CBMH06	CBMH04	450	0.013	2.55	0.55	5
P12	CBMH04	MH108	450	0.013	5.72	0.53	5
P13	MH108	Ext.STM106	675	0.013	19.08	0.64	10

**The velocity and flow simulated with PCSWMM for 100 Year Chicago using City of Ottawa IDF*

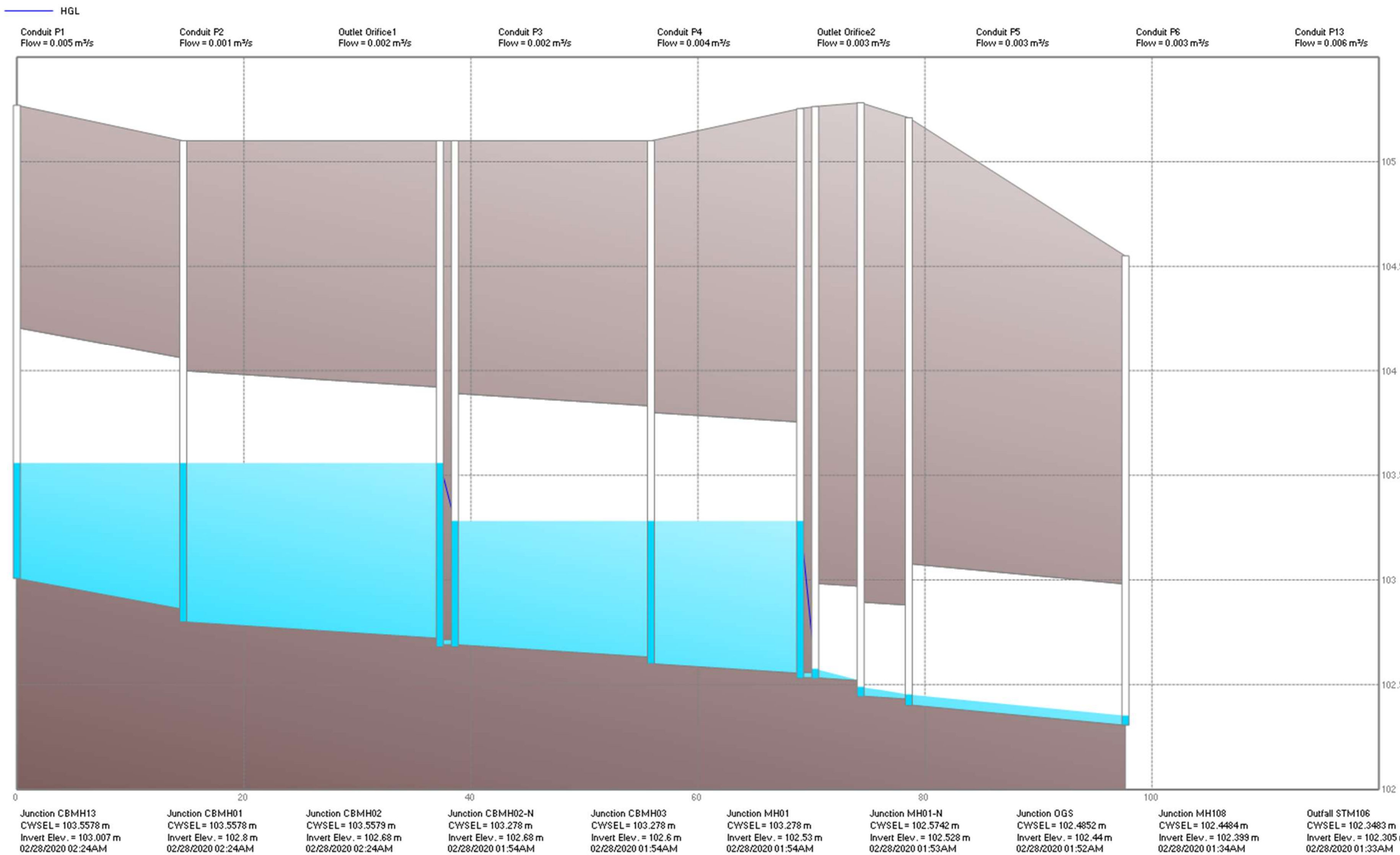

Figure 2: Proposed 5 - Year Storm HGL (From CBMH13 to Ex.STM106) – With SWM Control

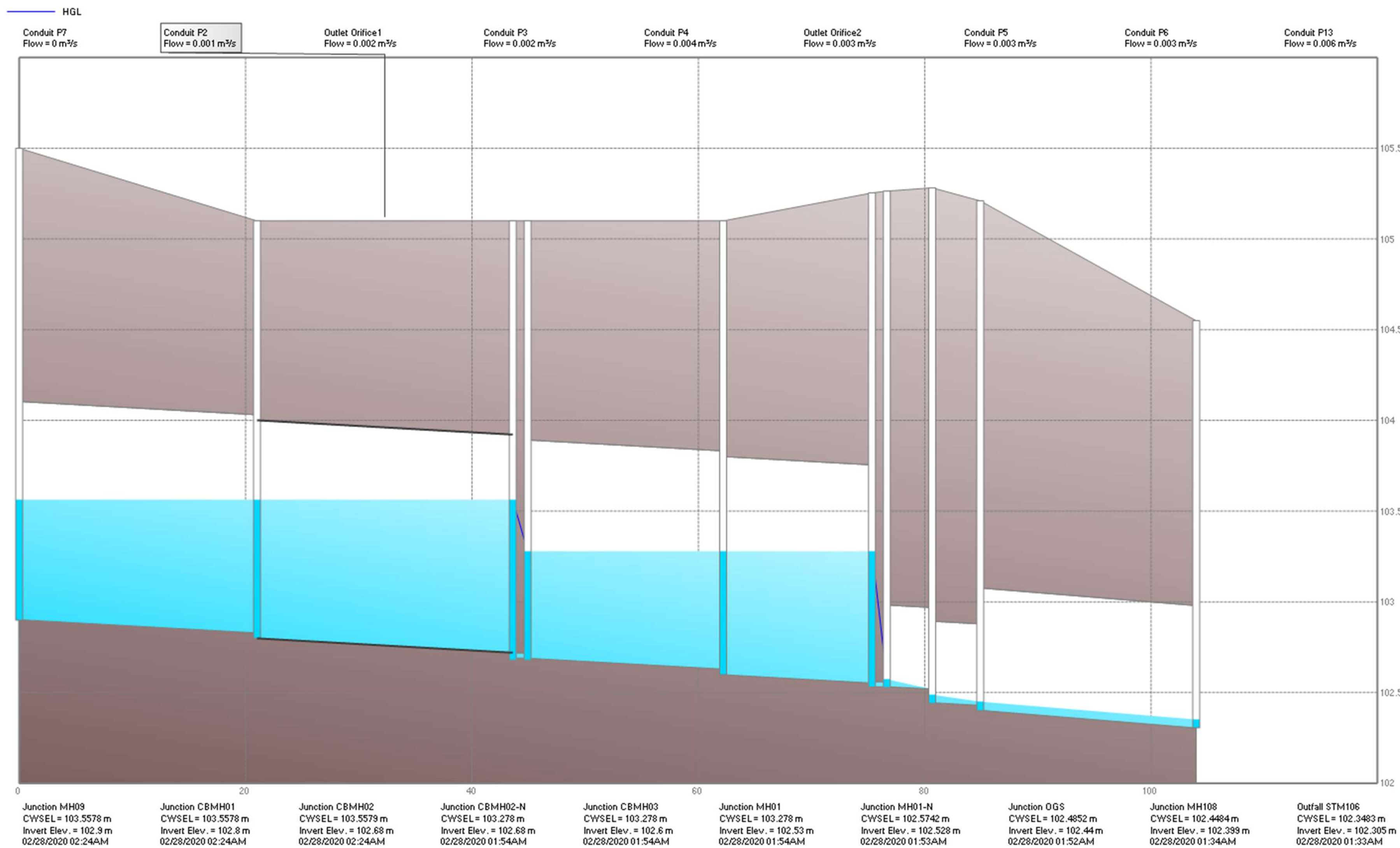

Figure 3: Proposed 5 - Year Storm HGL (From MH09 to Ex.STM106) – With SWM Control

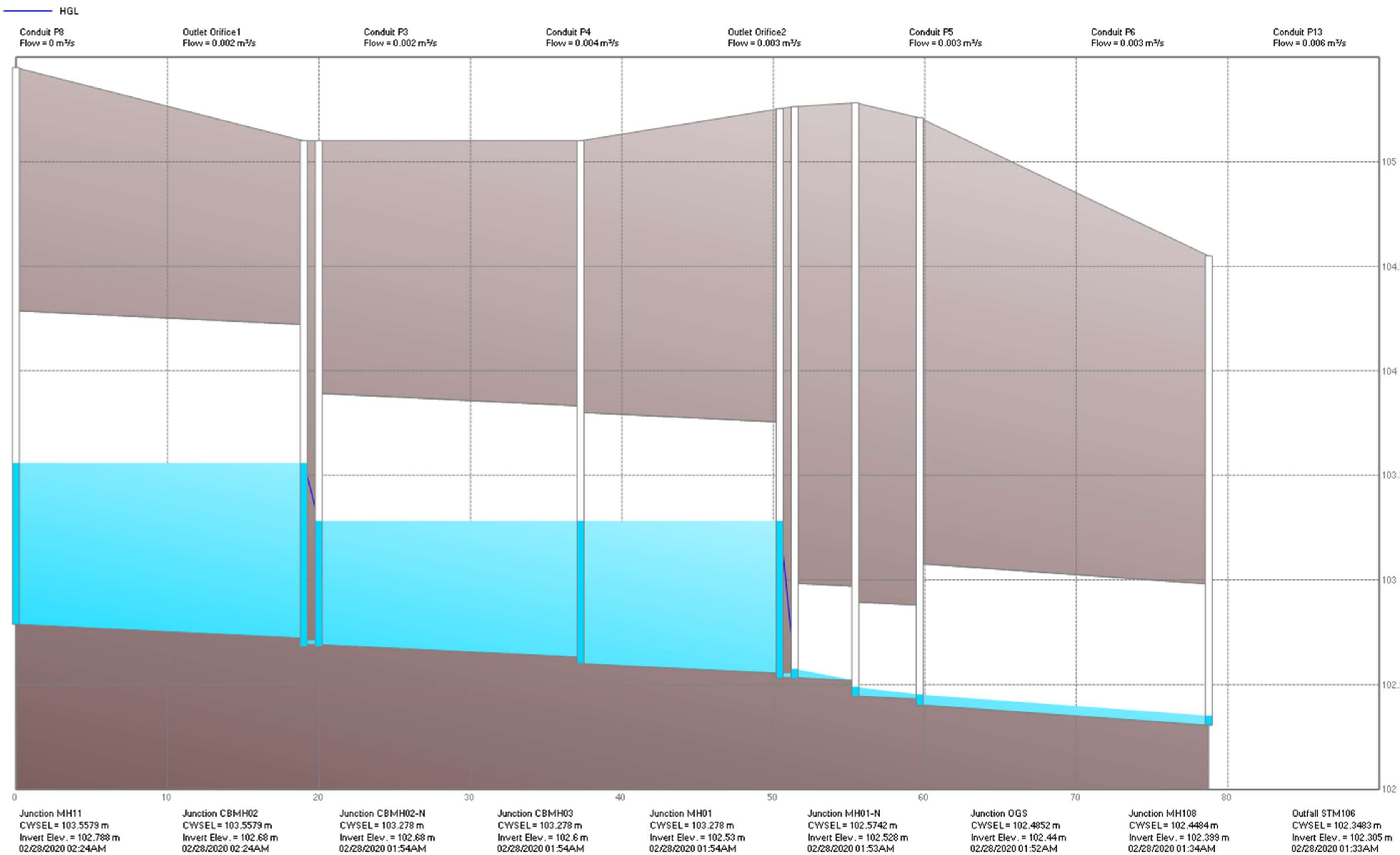

Figure 4: Proposed 5 - Year Storm HGL (From MH11 to Ex.STM106) – With SWM Control

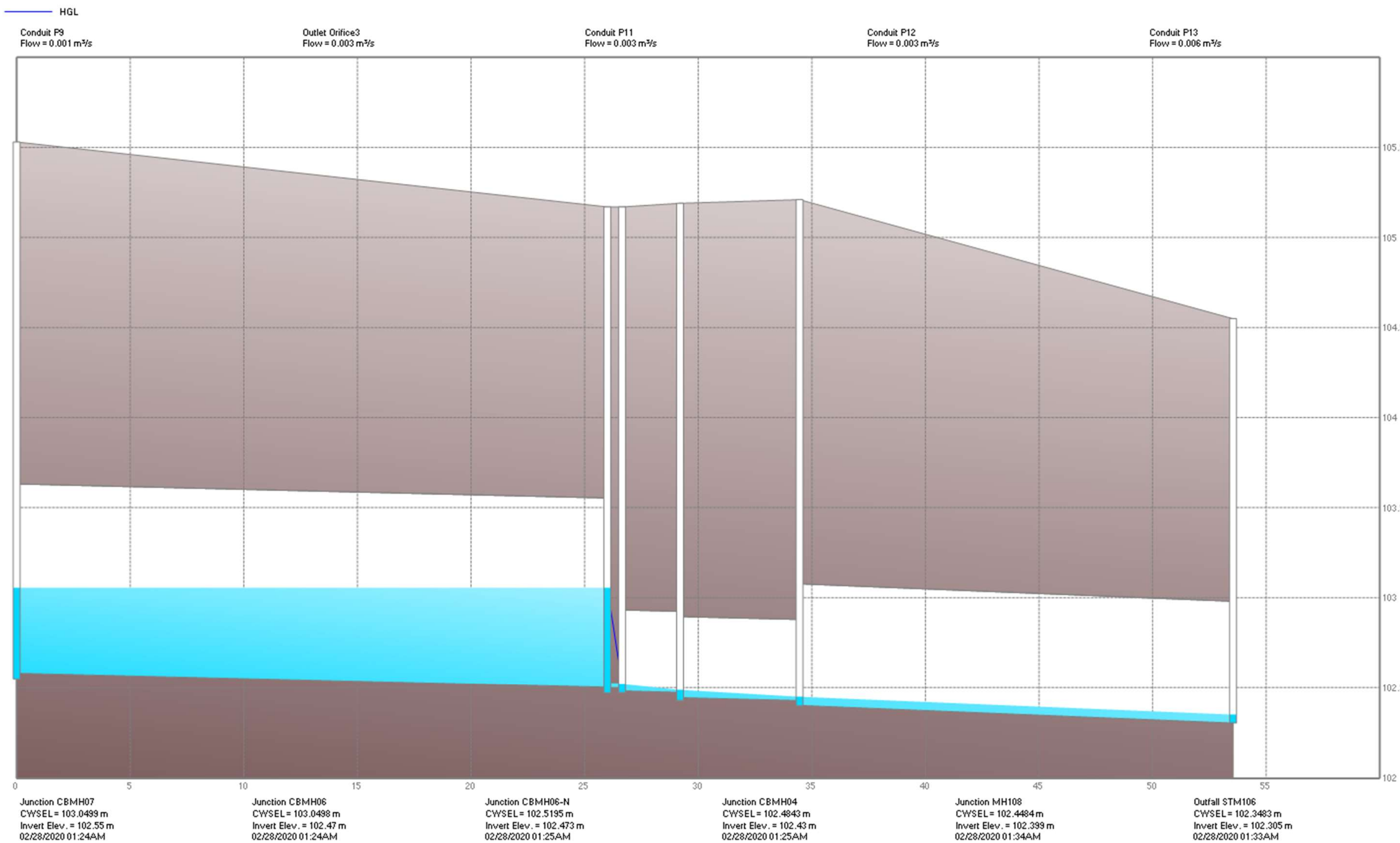

Figure 5: Proposed 5 - Year Storm HGL (From CBMH7 to Ex.STM106) – With SWM Control

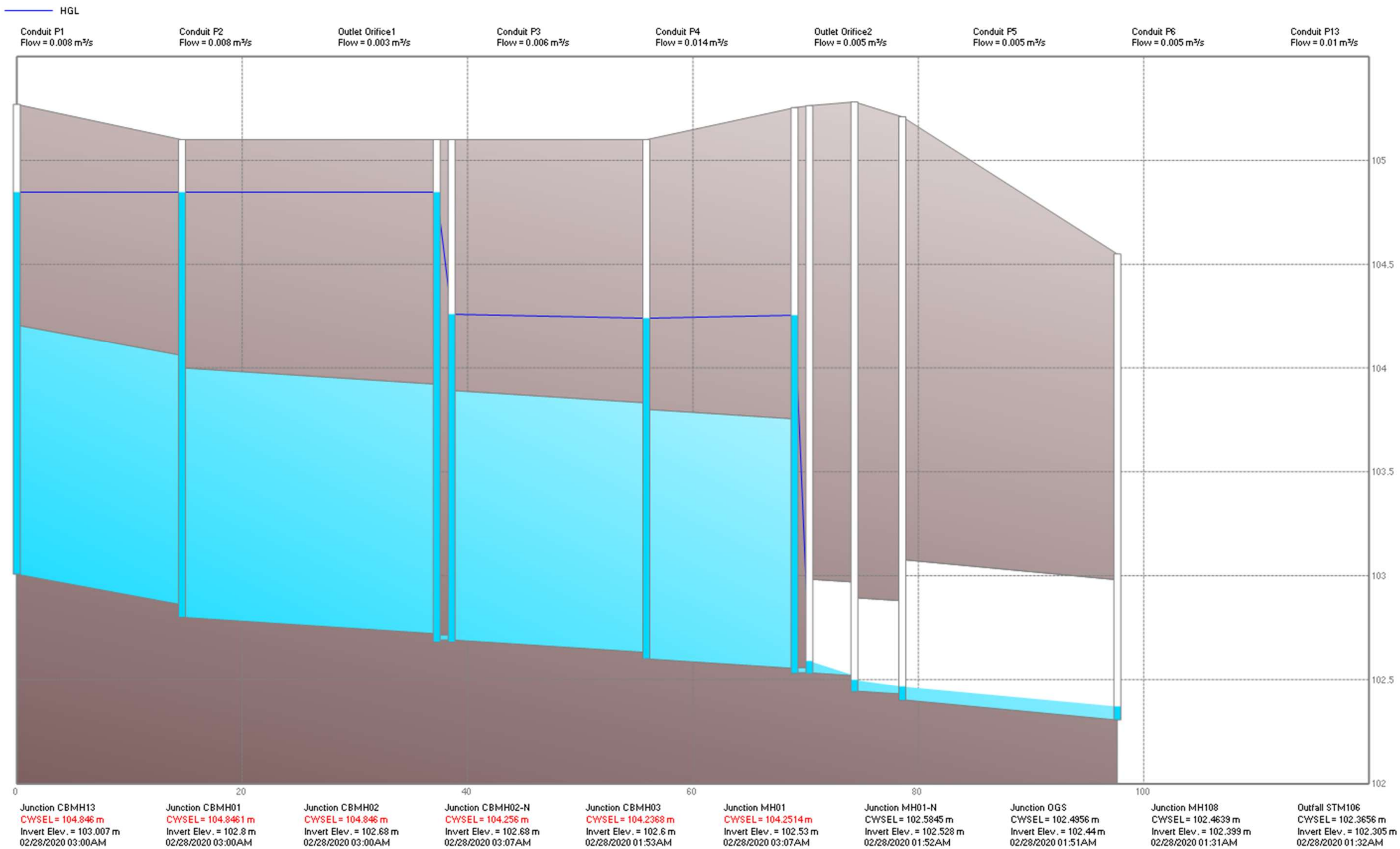

Figure 6: Proposed 100 - Year Storm HGL (From CBMH13 to Ex.STM106) – With SWM Control

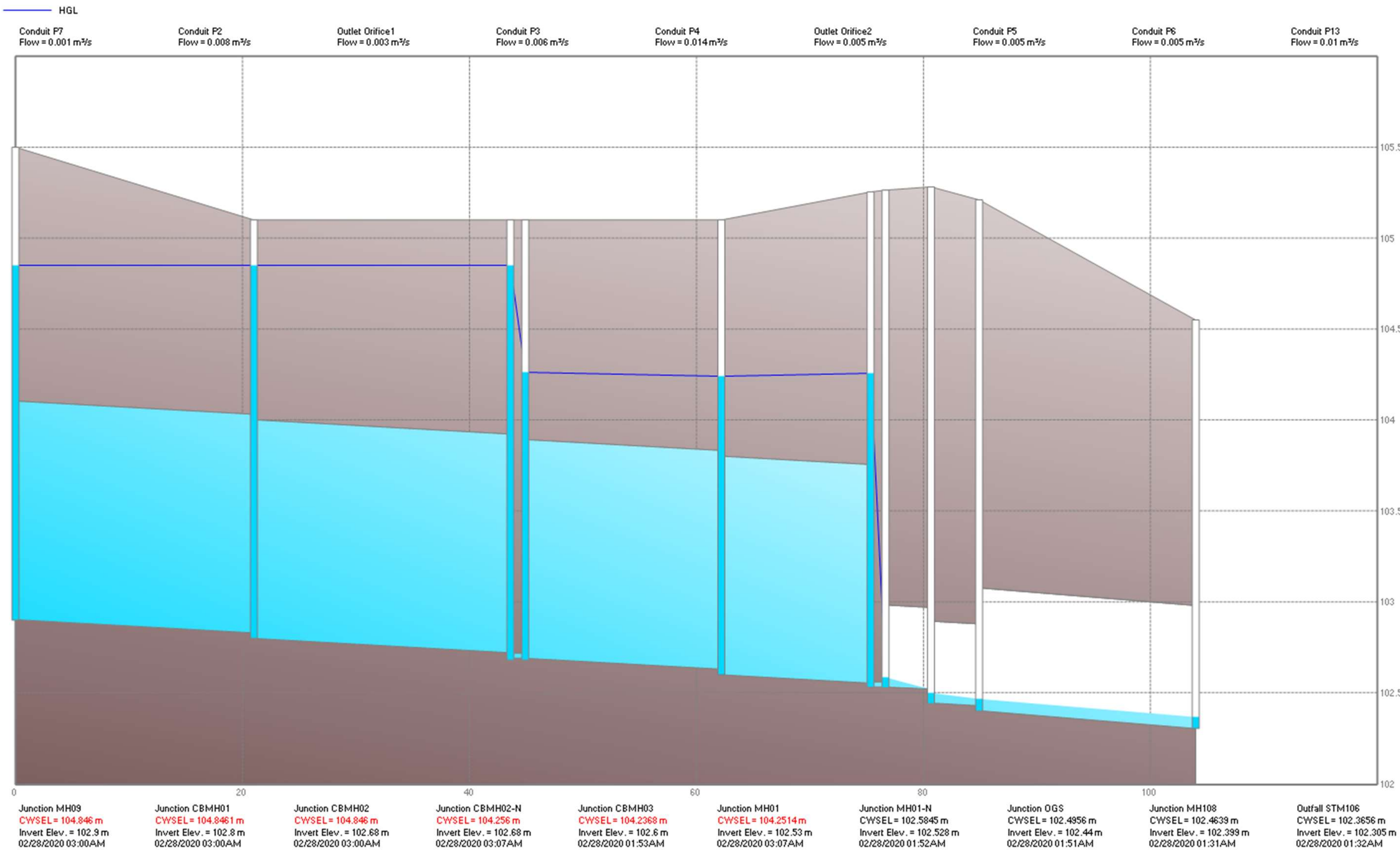

Figure 7: Proposed 100 - Year Storm HGL (From MH09 to Ex.STM106) – With SWM Control

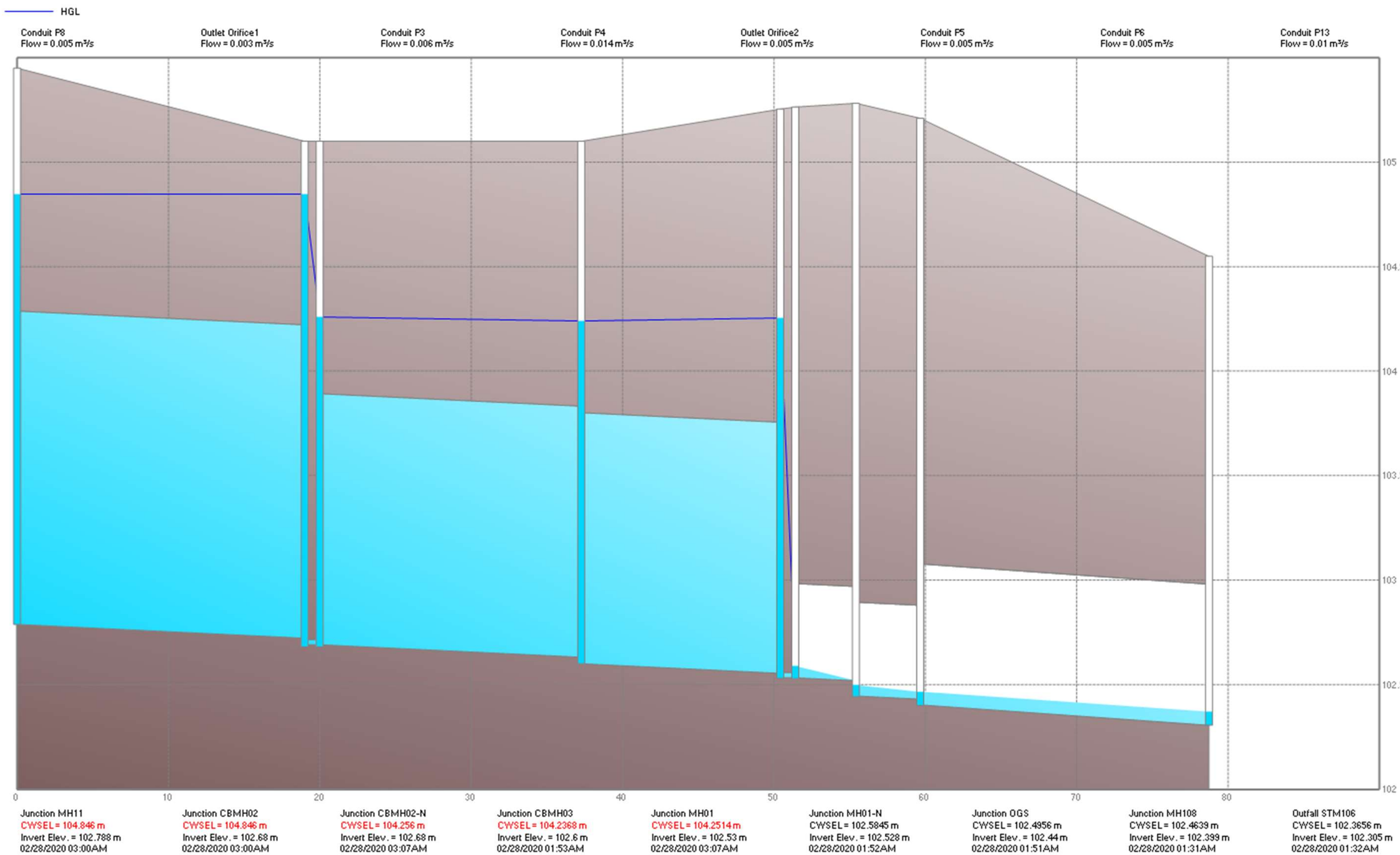

Figure 8: Proposed 100 - Year Storm HGL (From MH11 to Ex.STM106) – With SWM Control

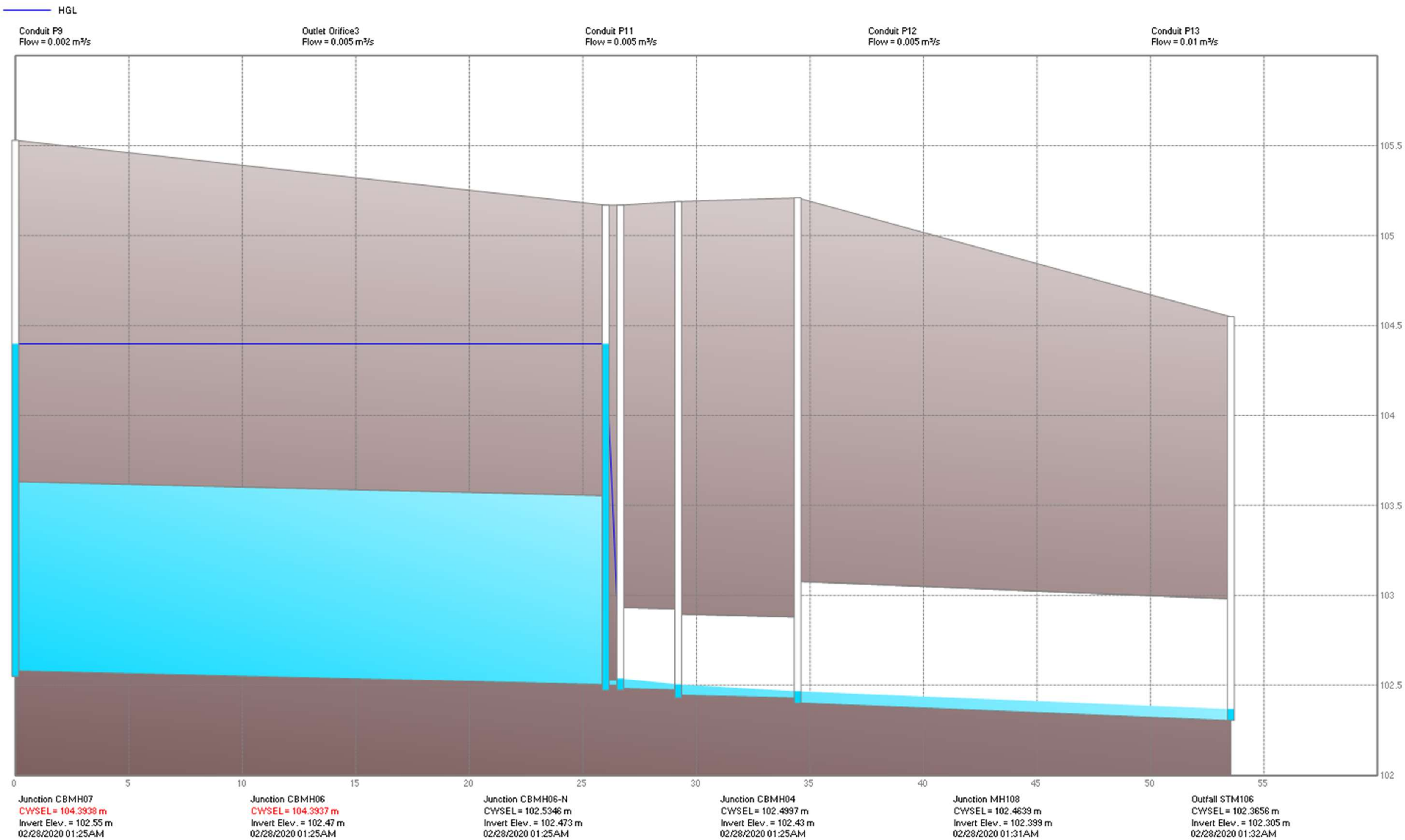

Figure 9: Proposed 100 - Year Storm HGL (From CBMH07 to Ex.STM106) – With SWM Control

5.4 Water Balance

Water balance for the subject site will be achieved by incorporating open landscape areas, where practicable. For the subject site, given space constraints, the feasible LID measures only include absorbent landscape features. The City of Ottawa has not stipulated any requirements for the Water Balance. In the absence of any guideline for the Mississippi Valley Conservation Authority (Refer to **Appendix C**) in terms of SWM criteria, the general practice of a minimum rainfall depth of 5 mm from site surfaces be retained on site through infiltration, evapotranspiration, water harvesting and/or re-use. Given the fact that the total site area for the proposed condition is 0.333 ha (footprint of the site), the minimum water balance requirement for the overall development is 16.65 m^3, required for the site. The proposed absorbent landscape area within the site as summarized in **Table 12** can closely meet that requirement in terms of water balance.

Table 12: On-Site Stormwater Retention Plan within the site

Water Balance Measure	Retention Depth (mm)	Area (ha)	Volume (m^3)
Proposed Condition			
Landscape Area (Absorbent)	25	0.064	16

5.5 Water Quality

The water quality target for the project is the long-term average removal of 70% Total Suspended Solids (TSS) on an annual loading basis from runoff leaving the site as per correspondence with the MVCA (Refer to **Appendix C**). As shown in **Drawing C 103.0** (Refer to **Appendix H**). An Oil Grit Separator model ADS FD-5HC (or approved equivalent) which provide a volume of 1,135 L for Oil storage has been designed to provide Enhanced Level of Treatment for all upstream areas where there is a potential for contamination (Refer to **Appendix E**). This proposed water quality treatment unit will be located downstream of MH01. The runoff of Catchment Areas A3, A6-1 and A7 as well as External Area A8 (mostly green area) are directing to outlet (STM 106) without any treatment. However, in general, the Normal Level of Treatment will be provided for the total area of the site (Refer to **Table 13**). This will be achieved through a “treatment train” approach where feasible, or by using standalone measures such as oil grit separator (OGS) unit.

With respect to the Functional Servicing and Stormwater Management Report for Campanale Homes Development prepared by DSEL, it indicates the runoff outlet from the site will eventually further treated with proposed wetland facility located approximately 380 m north-east of the intersection of Huntmar Drive and Hazeldean Road to provide the Enhanced Level of treatment.

Table 13: Summary of Water Quality Control Plan

Description	Area (ha)	Treatment Train (Assumed TTS Removal Efficiency)			% TSS Removal	% Treatment of Total Area
		Clean Water	Infiltration/ Water Balance	OGS		
Areas Directing to OGS*						
Landscape Area (Absorbent)	0.053		100		100	20.0
Asphalt and Roof Areas (Building)	0.212			80	80	64.0
Areas Directing to Outfall without Treatment**						
Landscape Area (Absorbent)	0.011		100		100	16.2
Green Area	0.017	80			80	20.0
Asphalt Area	0.039	20			20	11.8
Total Site Area	**0.333**					**76.64**

**These areas include flow from Catchment Areas A1, A2, A4, A5, A6-2 and A9*
***These areas include flow from Catchment Areas A3, A6-1, A7, and A8*

Table 14 presents a summary of the water quality control plan being proposed for the proposed site. As shown, the average annual TSS removal efficiency achieved for the proposed site is 80%.

Table 14: Summary of Proposed Oil and Grit Separator

Item	Specification
Model	ADS FD-5HC
Net Annual TSS Removal Efficiency	80%
Sediment Capacity (L)	840
Oil Capacity (L)	1,135
Total Holding Capacity (L)	1,975
Diameter of Outlet Pipe (mm)	450
Rated Treatment Flow Rate (L/s)	566

5.6 Erosion Control

For erosion control, the minimum requirement is to retain the runoff from a 5 mm rainfall event on site. This will be easily achieved as demonstrated in **Table 12**.

5.7 Erosion and Sediment Control

Construction activities such as grading, excavations, building and other activities have the potential to result in sediment transport to the existing nearby watercourses or municipal sewers. An erosion and sediment control plan (ESCP) will be required to ensure construction activities do not adverse impact downstream receivers or lead to siltation of the municipal sewers that could compromise water quality and available hydraulic capacity. Erosion and sediment controls (ECS) have been provided on the Erosion and Sediment Control Plan (Refer to **Drawing C101** in **Appendix J**, including notes pertaining to the maintenance of the control works).

6. Conclusion

This report has demonstrated that the proposed site drainage and SWM measures conform to the requirements of the City. The findings of this study are summarized as follows:

- The storm sewer will be designed to convey the 5-year post development flows. The proposed internal sewer system will connect the existing STM 106 on Fringewood Drive.
- The 2 to 100-year post-development flows from the site will be controlled to meets the target release rates with on-site water quantity controls are required.
- Dedicated stormwater systems will be constructed to collect all runoff from the site for 2-yr to 100-yr storm events except Catchment Areas 6-1 and 7 where the uncontrolled runoff is directly sheet flow to Fringewood Drive.
- Provide three (3) – Vortech-type ICDs (Unit 75VHV-1) to control 5-year and 100-year post development flows to meet the target release rate 5.56 L/s and 9.5 L/ as identified in **Table 10**. The required storage will be provided in the oversized underground storm sewer system.
- As simulated in PCSWMM, it indicates proposed oversize pipes provide sufficiently storage volume required and the HGL remains below manhole rim.
- The maximum depth of ponding on the proposed site area will be 0.10m to 0.13 m above the catch basin grates.
- An OGS model ADS FD-5HC (or approved equivalent) will provide enhance level of treatment for impervious for water quality. It will provide 80.0% of TSS removal (Refer to **Appendix E**). In general, the Normal Level of Treatment will be provided for the total area of the site through a “treatment train” approach.
- The proposed absorbent landscape area within the site can meet the common practice requirement of water balance.
- Erosion and sediment control measures have been proposed to minimize impacts on the surrounding environment during construction.

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Appendix A

Geotechnical Report

www.gemtec.ca

Geotechnical Investigation Report
Proposed Shell Service Station
5 Orchard Drive
Ottawa, Ontario

www.gemtec.ca

Submitted to:

AECOM Canada Ltd.
3292 Production Way
Burnaby, BC
V5A 4R4

Geotechnical Investigation Report
Proposed Shell Service Station
5 Orchard Drive
Ottawa, Ontario

July 3, 2019
Project: 63993.69

experience • knowledge • integrity

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1.0 INTRODUCTION

1.1 General

This report presents the results of a geotechnical investigation carried out for the design and construction of a new Shell service station to be located at 5 Orchard Drive in Ottawa, Ontario (refer to Borehole Location Plan, Figure 1). The purpose of the geotechnical investigation was to identify the general subsurface conditions at the site by means of a limited number of boreholes, and based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

1.2 Project and Site Description

Plans are being prepared to develop a vacant parcel of land located at the southwest corner of Hazeldean Road and Fringewood Drive in Ottawa (Stittsville), Ontario. Based on available property information from the City of Ottawa, the civic address for the proposed Shell site is 5 Orchard Drive, Ottawa.

Based on the information provided to us, the proposed structures will include a 168 square metre convenience store, a 97 square metre carwash, a pump island on a 240 square metre concrete apron with a 198 square metre canopy, access roadway and parking areas, and two (2) underground fuel storage tanks. It is anticipated that all of the structures will be of slab on grade (i.e. basementless) construction. The founding depth of the fuel storage tanks were not provided to us; however, based on our past experience, it is anticipated that the tanks will be founded at about 4.5 metres below finished grade. Similarly, it is anticipated that the pad footings for the canopy may be founded at depths between 2.5 and 4.5 metres.

2.0 SUBSURFACE INVESTIGATION

The fieldwork for this investigation was carried out on June 4th, 2019. At that time, three (3) boreholes were advanced across the property. The boreholes were advanced using a track mounted drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville-Sur-La-Rouge, Quebec. Details of the boreholes are provided below:

- Borehole BH19-1 was advanced to practical refusal of the auger at a depth of about 3.4 metres below ground surface in the area of the convenience store and car wash.

- Borehole BH19-2 was advanced to practical refusal of the auger at a depth of about 3.7 metres below ground surface in the area of the pump island and canopy. The bedrock was then cored from the bottom of the borehole to a depth of about 5.3 metres below ground surface using HQ size coring equipment.

- Borehole MW19-1 was advanced to practical refusal of the auger at a depth of about 2.9 metres below ground surface in the area of the underground fuel storage tanks. The bedrock was then cored from the bottom of the borehole to a depth of about 5.4 metres below ground surface using HQ size coring equipment. A well screen was installed in the borehole to facilitate hydraulic conductivity testing and to measure the stabilized groundwater level.

As part of Shell's health and safety policy, the following precautions were undertaken prior to advancing the boreholes at the site:

- The boreholes were daylighted to depths of about 1.5 and 2.0 metres below ground surface prior to starting the drilling operation.

The fieldwork was observed by members of our engineering staff who directed the drilling and hydro-vacuuming operations, observed the in situ testing and logged the samples and boreholes. Standard penetration tests were carried out within the overburden deposits and samples of the soils encountered were recovered using drive open sampling equipment. At boreholes MW19-1 and BH19-2, the encountered bedrock was cored using HQ size bedrock coring equipment. A well screen was sealed in the bedrock at the location of MW19-1.

A sample of the soil recovered from borehole BH19-1 was sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

Following the borehole drilling work, the soil and bedrock samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content and grain size distribution. A sample of the bedrock was tested for unconfined compressive strength. A hydraulic conductivity test was undertaken within the well screen installed in MW19-1 on June 13, 2019.

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix A. The results of the laboratory classification testing on the soil are also provided in Appendix A. A photo of the bedrock core samples recovered is provided on Figure B1 in Appendix B. The results of the hydraulic testing are provided in Appendix C. The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 1.

The borehole locations were selected by AECOM Canada Ltd. (AECOM) and GEMTEC Consulting Engineers and Scientists Limited (GEMTEC), and positioned at the site by GEMTEC personnel relative to existing site features. Elevations were measured using our Trimble R10 GPS equipment and are referenced to geodetic datum CGVD28.

3.0 SUBSURFACE CONDITIONS

3.1 General

As previously indicated, the subsurface conditions identified in the boreholes are given on the Record of Borehole sheets in Appendix A. The logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of drilling, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the test locations may vary from the conditions encountered in the boreholes. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities in the area.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and GEMTEC does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The following presents an overview of the subsurface conditions encountered in the boreholes advanced during this investigation.

3.2 Topsoil

The surface grade at the borehole locations consists of dark brown clayey silt topsoil. The thickness of the topsoil soil is about 150 and 200 millimetres at the borehole locations.

The moisture content of the topsoil samples from boreholes BH19-1 and BH19-2 are 31 and 34 percent, respectively.

3.3 Silt

A deposit of brown silt with some clay and trace sand was encountered below the topsoil at all of the borehole locations. The silt has a thickness of about 0.8 metres and extends to a depth of about 0.9 metres below surface grade at the borehole locations.

The SPT N values recorded within the silt range from 3 to 5 blows per 0.3 metres of penetration, which reflects a very loose to loose relative density.

The results of a grain size distribution test on a sample of the silt from borehole BH19-1 are provided on the Soils Grading Chart in Appendix A and summarized in Table 3.1.

Table 3.1 – Summary of Grain Size Distribution Testing (Silt)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH19-1	1B	0.3 – 0.6	0	8	72	20

The moisture content of the silt samples from boreholes BH19-1 and BH19-2 range from 26 to 28 percent.

3.4 Glacial Till

Glacial till was encountered below the silt at all of the borehole locations at a depth of about 0.9 metres below ground surface. The thickness of the glacial till ranges from about 1.9 to 2.4 metres.

Glacial till is a heterogeneous mixture of all grain sizes. At this site, the glacial till is described as brown to grey brown gravelly silty sand with trace clay, cobbles and boulders.

The SPT N values recorded within the glacial generally range from 7 to 33 blows per 0.3 metres of penetration, which reflects a loose to dense relative density. The SPT tests that encountered practical refusal (i.e. less than 0.3 metres of penetration) reflect the presence of cobbles in the glacial till or a very dense relative density.

The results of a grain size distribution test on a sample of the glacial till from borehole MW19-1 are provided on the Soils Grading Chart in Appendix A and summarized in Table 3.2.

Table 3.2 – Summary of Grain Size Distribution Testing (Glacial Till)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
MW19-1	3	1.2 – 1.8	21	48	23	8

The moisture content of the glacial till samples from all of the boreholes range from 10 to 31 percent.

3.5 Bedrock

Below the glacial till, fractured and weathered bedrock was encountered at depths of 2.8 to 3.3 metres below ground surface. At boreholes BH19-1 and BH19-2, the bedrock was penetrated 0.1 and 0.9 metres, respectively, with the augering equipment. Auger refusal was encountered on or within the bedrock at all of the borehole locations at depths ranging from about 2.9 to 3.7 metres below ground surface.

At boreholes MW19-1 and BH19-2, the bedrock was cored using HQ sized coring equipment. Borehole MW19-1 was cored from 2.9 to 5.4 metres below ground surface, and borehole BH19-2 was cored from 3.7 to 5.3 metres below ground surface.

The bedrock consists of moderately fractured, slightly weathered, limestone bedrock banded with shale. The solid core recovery (SCR) values range from 59 to 80 percent, and the rock quality designation (RQD) values range from 44 to 80 percent. Based on the RQD values, the bedrock quality is poor, becoming good with depth. Photographs of the collected rock cores are provided in Appendix B.

One (1) bedrock core sample was tested for unconfined compressive strength and the result is summarized in Table 3.3 below.

Table 3.3 – Unconfined Compressive Strength of Bedrock Core – Borehole 19-102

Borehole	Sample No.	Depth (metres)	Unconfined Compressive Strength (MPa)
MW19-1	RC5	3.2 – 3.4	146

Based on the unconfined compressive strength test results presented in Table 3.3, the bedrock strength may be classified as very strong.

3.6 Groundwater Levels

The groundwater level was measured in the well screen at MW19-1 on June 10, 2019, and is summarized in Table 3.4.

Table 3.4 – Groundwater Level – June 10, 2019

Monitoring Well	Ground Surface Elevation (Metres, Geodetic)	Groundwater Depth (metres)	Groundwater Elevation (metres, geodetic datum)
MW19-1	104.0	1.7	102.3

It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation.

3.7 Soil Chemistry Relating to Corrosion

The chemical testing results of a soil sample recovered from borehole BH19-1 are provided in Appendix D and summarized in Table 3.5.

Table 3.5 – Summary of Corrosion Testing - Soil

Parameters	Borehole BH19-1 SA3
Chloride Content (µg/g dry)	34
Resistivity (Ohm.m)	61.9
pH	7.88
Sulphate Content (µg/g dry)	7

4.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

4.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from off-site sources are outside the terms of reference for this report.

4.2 Overburden Excavation

It is anticipated that the excavation for the proposed building, fuel storage tanks, and pump island canopy will be carried out through the topsoil, and native deposits of silt, glacial till, and bedrock. The sides of the excavations within overburden soils should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the soils at this site can be classified as Type 3 soils. Therefore, for design purposes, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation side slopes in the overburden.

4.3 Bedrock Excavation

Based on the results of the boreholes, limestone bedrock interbedded with shale may be encountered during the excavation of the fuel storage tanks and pump island canopy.

Localized bedrock removal at this site could be carried out using hoe ramming techniques in conjunction with line drilling on close centres. The vibration effects of hoe ramming are usually minor and localized.

It is noted, based on observations during drilling and local experience, that the bedrock may contain horizontal bedding planes and near vertical joints. Therefore, some horizontal and vertical overbreak should be expected. Allowance should be made for additional granular material below the fuel storage tanks and footings for the pump island canopy.

4.4 Groundwater Pumping

Based on the grain size distribution results for the glacial till, groundwater inflow from the overburden soil for the construction of the convenience store, car wash and pump island canopy should be controlled by pumping from filtered sumps within the excavation. Suitable detention and filtration will be required before discharging the water to any sewers.

A hydraulic conductivity (falling head) test was undertaken in the monitoring well installed in borehole MW19-1 on June 19, 2019. The well screen is sealed within the bedrock and as such, the testing provided information on the permeability of the bedrock. The results of the hydraulic conductivity testing, which are provided in Appendix C, indicate that there was insufficient recovery of the groundwater level during the test to calculate a hydraulic conductivity value (about 3 centimetres over 30 minutes), which indicates that the bedrock in the area of MW19-1 has low permeability. Therefore, significant groundwater inflow from the bedrock during the construction of the underground fuel storage tanks is not anticipated. Any groundwater inflow from the soil and bedrock should be controlled by pumping from filtered sumps within the excavation.

4.5 Site Grade Raise Restrictions

The subsurface conditions at this site consist of very loose to loose silt overlying compact to dense glacial till. Based on this information, there are no grade raise restrictions for the proposed development, from a geotechnical perspective.

4.6 Foundation Design

Based on the results of the subsurface investigation, the proposed structures could be founded on spread and pad footings bearing on undisturbed native soil. All topsoil, loose or water-softened soils encountered should be removed from the footing areas.

In areas where the underside of footing level is above the level of the native soil, or where subexcavation is required, the grade below the proposed footing could be raised with granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type I or Type II. The granular material should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. To provide adequate spread of load below the footings, the granular material should extend at least 0.3 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

The spread footing foundations should be sized using the bearing pressures provided in Table 4.1.

Table 4.1 – Foundation Bearing Pressures

Subgrade Material	Geotechnical Reaction at Serviceability Limit State (kilopascals)	Factored Geotechnical Resistance at Ultimate Limit State (kilopascals)
Native undisturbed silt, or on a pad of engineered fill above native undisturbed silt	100[1]	275
Native undisturbed glacial till, or on a pad of engineered fill above native undisturbed glacial till	250[1]	500
Competent bedrock	n/a[2]	1,000[3]

Notes:

1. Provided that the subgrade surface and engineered fill are prepared as described in this report, the post construction total and differential settlement of the footings at SLS should be less than 25 and 15 millimetres, respectively.
2. The geotechnical reaction at SLS for 25 millimetres of settlement will be greater than the factored resistance at ULS; as such, ULS conditions will govern for footings founded directly on the competent bedrock surface.
3. The above bearing pressure assumes that all soil, and disturbed or loosened bedrock is removed from the bearing surface. Allowance should be made in the contract for concrete fill below the foundations due to vertical overbreak of the bedrock.

4.7 Frost Protection of the Foundations

All exterior footings in heated areas of the structure should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) footings that are located in areas that are to be cleared of snow should be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation, similarly to the insulation currently in place along the existing structure. An insulation detail could be provided upon request.

If the new foundation and\or concrete slab on grade is insulated in a way that reduces heat loss towards the surrounding soil, the required earth cover over the footings should conform to that of an unheated structure (i.e. 1.8 metres).

4.8 Foundation Backfill and Drainage

The native deposits at this site are considered frost susceptible and should not be used as backfill against foundation walls. To avoid frost adhesion and possible heaving, the following options are provided for foundation backfilling:

- Backfill the foundations with imported, free-draining, non-frost susceptible granular material such as that meeting OPSS Granular A or Granular B Type I or II requirements, or

- Provide a suitable bond break to the surfaces of all the foundations and backfill using the fill or native soils. A suitable bond break could consist of at least 2 layers of 6-mil polyethylene sheeting.

Where the backfill will ultimately support areas of hard surfacing (roadways or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Where future landscaped areas will exist next to the proposed structures and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.

Where areas of hard surfacing (pavement or pathways, etc.) abut the proposed structures, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible material to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from the underside of footing level to the underside of the granular subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

Perimeter foundation drainage is not considered necessary for a slab on grade structure at this site, provided that the floor slab level is above the finished exterior ground surface level.

4.9 Slab on Grade Support (Heated Areas Only)

For predictable performance of the slab on grade for the proposed structures, the area should be stripped of topsoil to expose the underlying native soil. The subgrade surface should then be proof rolled with a 10 tonne steel drum roller (without vibration) under dry conditions. Any soft areas that are evident from the proof rolling should be subexcavated and replaced with granular material meeting OPSS Granular B Type I or II. The subgrade surfaces and the proof rolling should be observed throughout by geotechnical personnel.

The grade within the proposed building could be raised, where necessary, with granular material meeting OPSS requirements for Granular B Type I or II. The use of Granular B Type II material is preferred under wet conditions. The granular base for the proposed slab on grade should consist of at least 150 millimetres of OPSS Granular A.

The granular materials should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment.

4.10 Seismic Design of Proposed Structures

Based on the results of the subsurface investigation, the proposed structures will be founded on or within silt and/or glacial till deposits having a very loose to dense relative density. In accordance with the Ontario Building Code (OBC), Site Class C could be used for the seismic design of the proposed building.

In our opinion, the potential for liquefaction of the overburden soils at this site is negligible.

5.0 PROPOSED UNDERGROUND FUEL STORAGE TANKS

5.1 Excavation and Groundwater Pumping

It is understood that the service station will contain two (2) underground fuel storage tanks located within the northeast corner of the site.

Based on the investigation results, the excavation for the proposed underground storage tanks will be carried out through topsoil and native deposits of silt and glacial till, and possibly bedrock. Our comments on overburden excavation, bedrock excavation, and groundwater pumping provided in Sections 4.2 to 4.4 apply equally to the fuel storage tanks.

5.2 Bedding

The subbedding and bedding should conform to the tank manufacturer's recommendations for grain size distribution and compaction requirements. All of the topsoil, disturbed soil, and soft or deleterious materials should be removed from the tank footprint.

In areas where subexcavation is required, the grade below the proposed footing could be raised with granular material meeting OPSS requirements for Granular B Type II. The granular material should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. To provide adequate spread of load below the tanks, the granular material should extend at least 0.3 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

5.3 Backfill

To prevent frost adhesion and possible heaving, the tanks should be backfilled with a free-draining, non-frost susceptible granular material such as OPSS Granular A, or Granular B Type II. It should be noted that the tank manufacturer's specifications for backfill material supersedes our recommendations.

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.

Where future landscaped areas will exist next to the proposed tanks and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.

Where areas of hard surfacing (concrete, sidewalk, pavement, etc.) abut the proposed tanks, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible soil to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from the maximum depth of frost penetration (i.e. 1.8 metres below ground surface). The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

For design purposes, the earth pressure parameters provided in Table 5.1 could be used to calculate the lateral earth pressure on the underground fuel storage tank.

Table 5.1 – Backfill Earth Pressure Parameters

Parameter	OPSS Granular A, Granular B Type II
Material Unit Weight, γ (kN/m^3)	22
Estimated Friction Angle (degrees)	36
"Active" Earth Pressure Coefficient, K_a, assuming horizontal backfill behind the structure	0.26
"Passive" Earth Pressure Coefficient, K_p, assuming horizontal backfill behind the structure	3.85
"At Rest" Earth Pressure Coefficient, K_o, assuming horizontal backfill behind the structure	0.41

The lateral pressures due to compaction should be considered in the design. The magnitude of the compaction surcharge pressure depends on the mass and type of compaction equipment. For light, hand operated compaction equipment having a mass of approximately 400 kilograms, the surcharge pressure can be taken as 16 kilopascals. The surcharge pressure should be increased if heavier equipment is used.

5.4 Buoyant Uplift of Tanks

The groundwater levels could be higher than those measured during our investigation due to both seasonal fluctuations and surface water seepage into the granular backfill material, therefore, the design and installation of the tanks should consider the tank manufacturer's recommendations for managing hydrostatic pressures and buoyant uplift. As a conservative design approach, we recommend that the ground water level be assumed near ground surface for buoyancy computations.

6.0 SITE SERVICES

6.1 Overburden Excavation

Based on the investigation results, it is anticipated that the excavation for services will be carried out through topsoil and native deposits of silt and glacial till. The planned depth of the services was not known at the time the report was written.

In the overburden, the excavation for flexible service pipes should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.010 for Type 3 Soil. The excavation for rigid service pipes should be in accordance with OPSD 802.031 for Type 3 soil.

The excavations for the services should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the soils at this site can be classified as Type 3 and allowance should be made for 1 horizontal to 1 vertical side slopes extending upwards from the base of the excavation. Alternatively, the excavations could be carried out near vertically within a tightly fitting, braced steel trench box designed specifically for this purpose.

Additional comments on overburden excavation are provided in Sections 4.2.

6.2 Bedrock Excavation

Depending on the invert of the new sewer and watermain, excavation of the bedrock may be required.

In bedrock, the excavation for flexible service pipes should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.013 for bedrock. The excavation for rigid service pipes should be in accordance with OPSD 802.033 for bedrock.

Our comments on bedrock excavation provided in Section 4.3 apply equally to the excavation for site services.

6.3 Groundwater Pumping and Management

Groundwater pumping and management guidelines are provided in Section 4.4 of this report. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services.

6.4 Pipe Bedding

The bedding for the new sewers should be in accordance with OPSD 802.010 and 802.013 for flexible pipes in earth and bedrock excavation, respectively, and OPSD 802.031 and OPSD 802.033 for rigid pipes in earth and bedrock excavation, respectively. The pipe bedding material should consist of at least 150 millimetres of well graded crushed stone meeting OPSS requirements for Granular A. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A material. Since the source of recycled material cannot be determined, it is suggested that any granular materials used in the service trench be composed of virgin (i.e., not recycled) material only.

In areas where the subgrade is disturbed or where unsuitable material (such as existing fill material) exists below the pipe subgrade level, the disturbed/unsuitable material should be removed and replaced with a subbedding layer of compacted granular material, such as that meeting OPSS Granular B Type I or Type II (50 or 100 millimetre minus crushed stone). To provide adequate support for the pipes in the long term in areas where subexcavation of overburden material is required below design subgrade level, the excavations should be sized to allow a 1 horizontal to 2 vertical spread of granular material down and out from the bottom of the pipe.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The subbedding, bedding and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value.

6.5 Trench Backfill

In areas where the service trench will be located below or in close proximity to existing or future areas of hard surfacing (pavement, sidewalk, etc.), acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetration in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent hard surfaced area. The depth of frost penetration in exposed areas can normally be taken as 1.8 metres below finished grade. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I or Type II.

It is anticipated that most of the inorganic overburden materials encountered during the subsurface investigation will be acceptable for reuse as trench backfill. Any topsoil or organic soil should be wasted from the trench.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, sidewalks, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density. The specified density may be reduced to 90 percent of the standard Proctor dry density in areas where the trench backfill is not located below or in close proximity to existing or future roadways, parking areas, sidewalks, etc. (i.e. in landscaped areas) and provided that some settlement above the trench is acceptable.

Depending on the weather conditions at the time of construction, some wetting of materials could occur. As such, the specified densities may not be possible to achieve and, consequently, some settlement of these backfill materials should be expected. Consideration could be given to implementing one or a combination of the following measures to reduce post construction settlement above the trenches, depending on the weather conditions encountered during the construction:

- Allow the overburden materials to dry prior to compaction;
- Reuse any wet materials in the lower part of the trenches and make provision to defer final placement of the final lift of the asphaltic concrete for 3 months, or longer, to allow some of the trench backfill settlement to occur and thereby improve the final pavement appearance.
- Avoid reusing any wet material within the trench. If additional material is required for trench backfill, consideration could be given to using imported relatively dry earth fill material, or imported OPSS Select Subgrade Material below the zone of frost penetration.

6.6 Seepage Barriers

To prevent the granular bedding in the services trench from acting as a “French Drain” and thereby promoting migration of potential contaminants off the property, seepage barriers should be installed along the service trenches just inside the property lines. The seepage barriers should begin at subgrade level and extend vertically through the granular pipe bedding and granular surround to within the native backfill materials, and horizontally across the full width of the service trench excavation. The seepage barriers could consist of 1.5 metre wide dykes of compacted weathered silty clay. The weathered silty clay should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. It is noted that silty clay will need to be imported to site. Alternatively, consideration could be given to installing an anti-seep collar or mixing OPSS Granular A with bentonite (as per OPSS 1205). The locations of the seepage barriers could be provided at the final design stage.

7.0 ACCESS ROADWAY AND PARKING AREAS

7.1 Subgrade Preparation

In preparation for the construction of the access roadway and parking areas at this site, all surficial topsoil, and any loose/soft, wet, organic or deleterious materials should be removed from the proposed subgrade surface. Any subexcavated areas could be filled with compacted earth borrow or imported granular material. The Granular B Type I, II, Select Subgrade Material or earth borrow should be placed in maximum 300 millimetres thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment.

The subgrade surfaces should be proof rolled with a large steel drum roller (under dry conditions) and shaped and crowned to promote drainage of the granular materials.

7.2 Flexible Pavement Structures for the Parking Areas and Access Roadway

It is suggested that parking and roadway areas be constructed using the following minimum pavement structure:

- 90 millimetres asphaltic concrete, over
- 150 millimetres of OPSS Granular A base, over
- 450 millimetres of OPSS Granular B Type I or II subbase

The 90 millimetres asphaltic concrete surface should consist of 40 millimetres of Superpave 12.5 (Traffic Level B) over 50 millimetres of Superpave 12.5 (Traffic Level B). Performance grade PG 58-34 asphaltic concrete should be specified.

This pavement structure is suitable for both light and heavy-duty vehicle access. If required, a pavement structure suitable for light-duty areas only (e.g., parking areas that will not be used by heavy trucks) could be provided as the design progresses.

Where the new pavement will abut existing pavement, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter to match the depths of the granular material(s) exposed in the existing pavement.

If the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the subbase material, install a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or a combination of both, to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

7.3 Compaction Requirements

All imported granular materials should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 98 percent of the standard Proctor dry density value using suitable vibratory compaction equipment.

8.0 ADDITIONAL CONSIDERATIONS

8.1 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the soil sample from borehole BH19-1 is 7 micrograms per gram. According to Canadian Standards Association (CSA) "Concrete Materials and Methods of Concrete Construction", the concentration of sulphate in the soil can be classified as low. For low exposure conditions, any concrete that will be in contact with the native soil or groundwater should be batched with General Use (formerly Type 10) cement. The design of any concrete should take into consideration freeze thaw effects and the presence of chlorides.

Based on the resistivity and pH of the soil samples, the soil can be classified as non-aggressive towards unprotected steel. The manufacturer of any buried steel elements that will be in contact with the soil and groundwater should be consulted to determine the durability of the product used. It is noted that the corrosivity of the soil and groundwater could vary throughout the year due to the application sodium chloride for de-icing.

8.2 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, hoe ramming, etc.) will cause ground vibration on and off of the site. The vibrations will attenuate with distance from the source, but may be felt at nearby structures. We recommend that preconstruction surveys be carried out on the adjacent structures and that vibration monitoring be carried out during the construction so that any damage claims can be addressed in a fair manner.

8.3 Winter Construction

In the event that construction is required during freezing temperatures, the soil below the proposed foundations and slabs should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

Any open excavations should be opened for as short a time as practicable. The materials on the sides of the excavation should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

Provision must be made to prevent freezing of any soil below the level of any existing structures or services. Freezing of the soil could result in damage to structures or services.

8.4 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed.

8.5 Design Review and Construction Observation

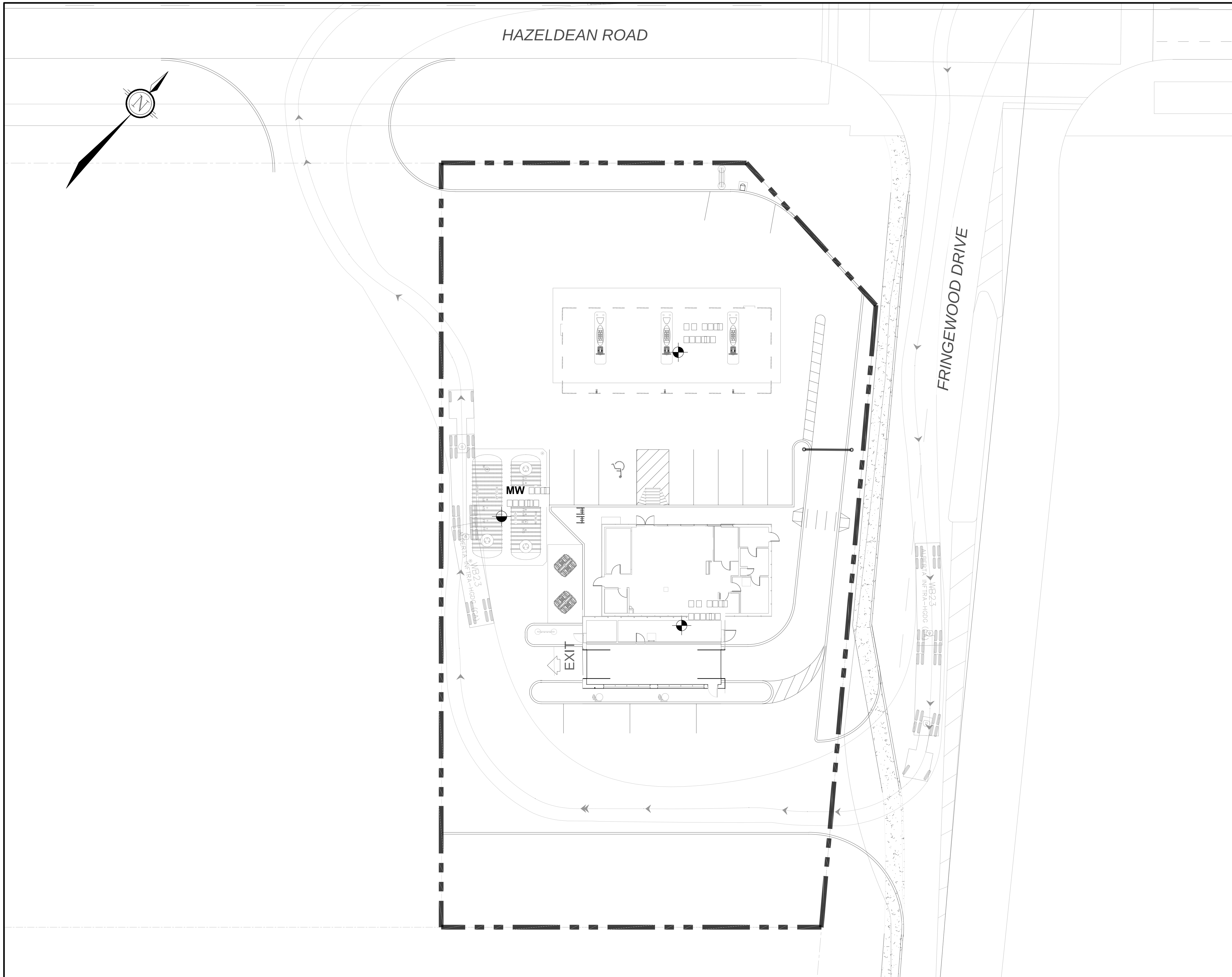
The details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the final design drawings be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

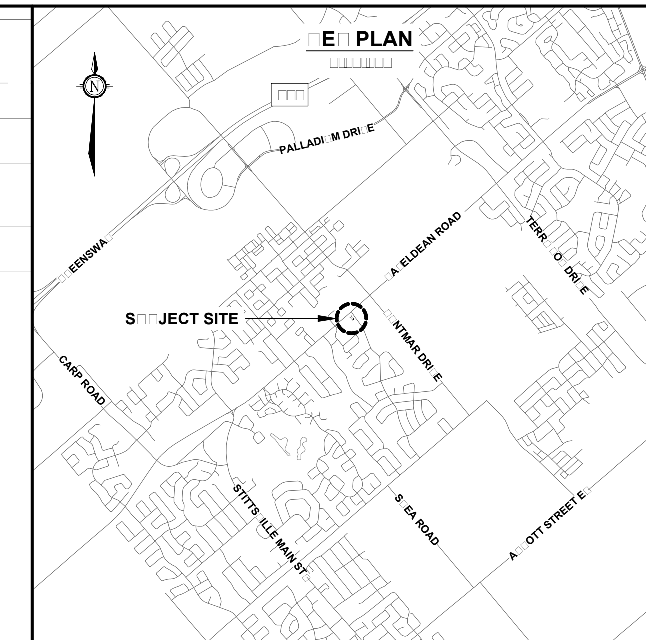
The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for the site services and roadways should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Luc Bouchard, P.Eng., ing.

Johnathan A. Cholewa, Ph.D., P.Eng.





LEGEND

BOREHOLE LOCATION IN PLAN
(current investigation by GEMTEC)

MONITORING WELL LOCATION IN PLAN
(current investigation by GEMTEC)

MW — BOREHOLE/MONITORING WELL ID
GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

NOTE:
THIS PLAN WAS PREPARED BY USING THE PROPOSED SITE PLAN BY OTHERS

Scale 1:400
0 8 16 24m

GEMTEC
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Ottawa, ON K2K 2A9
Tel: (613) 836-1422
www.gemtec.ca
ottawa@gemtec.ca

Client	AECOM CANADA LTD.	Project	63993.69
Location	PROPOSED SHELL SERVICE STATION 5 ORCHARD DRIVE, OTTAWA, ON		
Drwn by P.C.	Chkd by L.B.	BOREHOLE LOCATION PLAN	
Date	JULY 2019	Rev. 0	FIGURE

APPENDIX A

Record of Borehole Sheets
Results of Laboratory Classification Testing
List of Abbreviations and Terminology
Lithological and Geotechnical Rock Description Terminology

RECORD OF BOREHOLE 19-1

CLIENT: AECOM Canada Ltd.
PROJECT: Geotechnical Investigation
JOB#: 63993.69
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 4 2019

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface	103.95						
	Hydrovacuum	Dark brown clayey silt (TOPSOIL)	103.80 / 0.15	1a	SS	350	4		Backfilled with auger cuttings
	Hydrovacuum	Very loose, brown SILT, some clay, trace sand		1b	SS	350	4	MH	
1	Hydrovacuum	Compact, gravelly silty sand, trace clay, cobbles and boulders (GLACIAL TILL)	103.05 / 0.90	2	SS	450	21		
	Power Auger, Hollow Stem Auger (210mm OD)			3	SS	450	15		
2	Power Auger, Hollow Stem Auger (210mm OD)			4	SS	400	26		
3	Power Auger, Hollow Stem Auger (210mm OD)			5	SS	50	50 for 75mm		
		Fractured and weathered BEDROCK	100.67						
		Auger Refusal End of Borehole	100.59 / 3.36						

PENETRATION RESISTANCE (N), BLOWS/0.3m
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA: + NATURAL ⊕ REMOULDED
WATER CONTENT, %: W_P — W — W_L
10 20 30 40 50 60 70 80 90

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LOGGED: ML
CHECKED: LB

RECORD OF BOREHOLE 19-2

CLIENT: AECOM Canada Ltd.
PROJECT: Geotechnical Investigation
JOB#: 63993.69
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 4 2019

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface	103.81					
	Hydrovacuum	Dark brown clayey silt (TOPSOIL)	103.61 / 0.20	1	SS	350	3	
		Very loose to compact, brown SILT, some clay, trace sand	102.91 / 0.90	1	SS	350	3	
				2	SS	450	14	
1	Power Auger, Hollow Stem Auger (210mm OD)	Loose to dense, brown, gravelly silty sand, trace clay, cobbles and boulders (GLACIAL TILL)	101.01 / 2.80	3	SS	450	7	
2				4	SS	400	33	Backfilled with auger cuttings
3		Fractured and weathered BEDROCK	100.10 / 3.71					
4	Diamond Rotary Core, HQ (89mm OD)	Moderately fractured, slightly weathered LIMESTONE BEDROCK, banded with shale	98.47 / 5.34	5	RC	1630	TCR= 84%, SCR= 63%, RQD= 45%	
5		End of borehole						
6								

PENETRATION RESISTANCE (N), BLOWS/0.3m
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA + NATURAL ⊕ REMOULDED
WATER CONTENT, % W_P W W_L
10 20 30 40 50 60 70 80 90
ADDITIONAL LAB. TESTING

GEO - BOREHOLE LOG 63993.69_GINT LOGS BOREHOLES.GPJ GEMTEC 2018.GDT 28/6/19

GEMTEC Consulting Engineers and Scientists

LOGGED: ML
CHECKED: LB

RECORD OF BOREHOLE MW19-1

CLIENT: AECOM Canada Ltd.
PROJECT: Geotechnical Investigation
JOB#: 63993.69
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 4 2019

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
0	Power Auger / Hydrovacuum	Ground Surface	103.98					
		Dark brown clayey silt (TOPSOIL)	103.78 / 0.20					
		Loose to compact, brown SILT, some clay, trace sand		1	SS	350	5	
1	Power Auger / Hollow Stem Auger (210mm OD)	Compact, brown gravelly silty sand, trace clay, cobbles and boulders (GLACIAL TILL)	103.08 / 0.90	2	SS	350	16	
				3	SS	450	23	MH
2				~~5~~	~~SS~~	~~25~~	~~50 for 25mm~~	
3	Diamond Rotary Core / HQ (89mm OD)	Moderately fractured, slightly weathered LIMESTONE BEDROCK, banded with shale	101.08 / 2.90	5	RC	1370	TCR= 89%, SCR= 59%, RQD= 44%	UC= 146 MPa
4–5				6	RC	1140	TCR= 84%, SCR= 80%, RQD= 80%	
		End of borehole	98.57 / 5.41					

PENETRATION RESISTANCE (N), BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA + NATURAL ⊕ REMOULDED
DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
WATER CONTENT, % W_P W W_L
10 20 30 40 50 60 70 80 90

PIEZOMETER OR STANDPIPE INSTALLATION

Bentonite

Filter sand

50mm diameter well screen, 1.5m long

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
19/06/10	1.7 ▽	102.3

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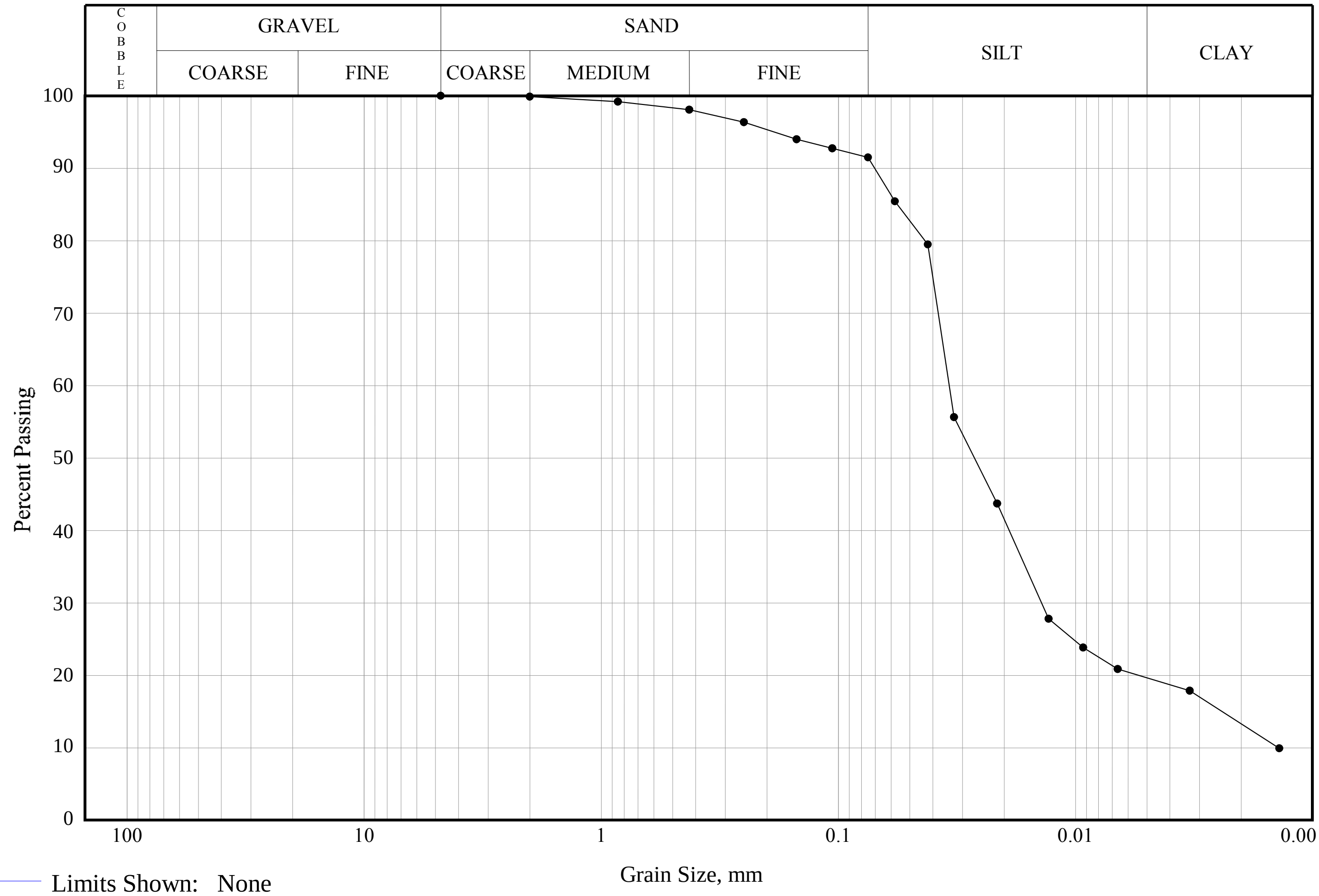
LOGGED: ML
CHECKED: LB

GEMTEC Consulting Engineers and Scientists	Client: AECOM Canada Ltd	Soils Grading Chart
	Project: Geotechnical Investigation - 5 Orchard Drive, Ottawa	
	Project #: 6399369	

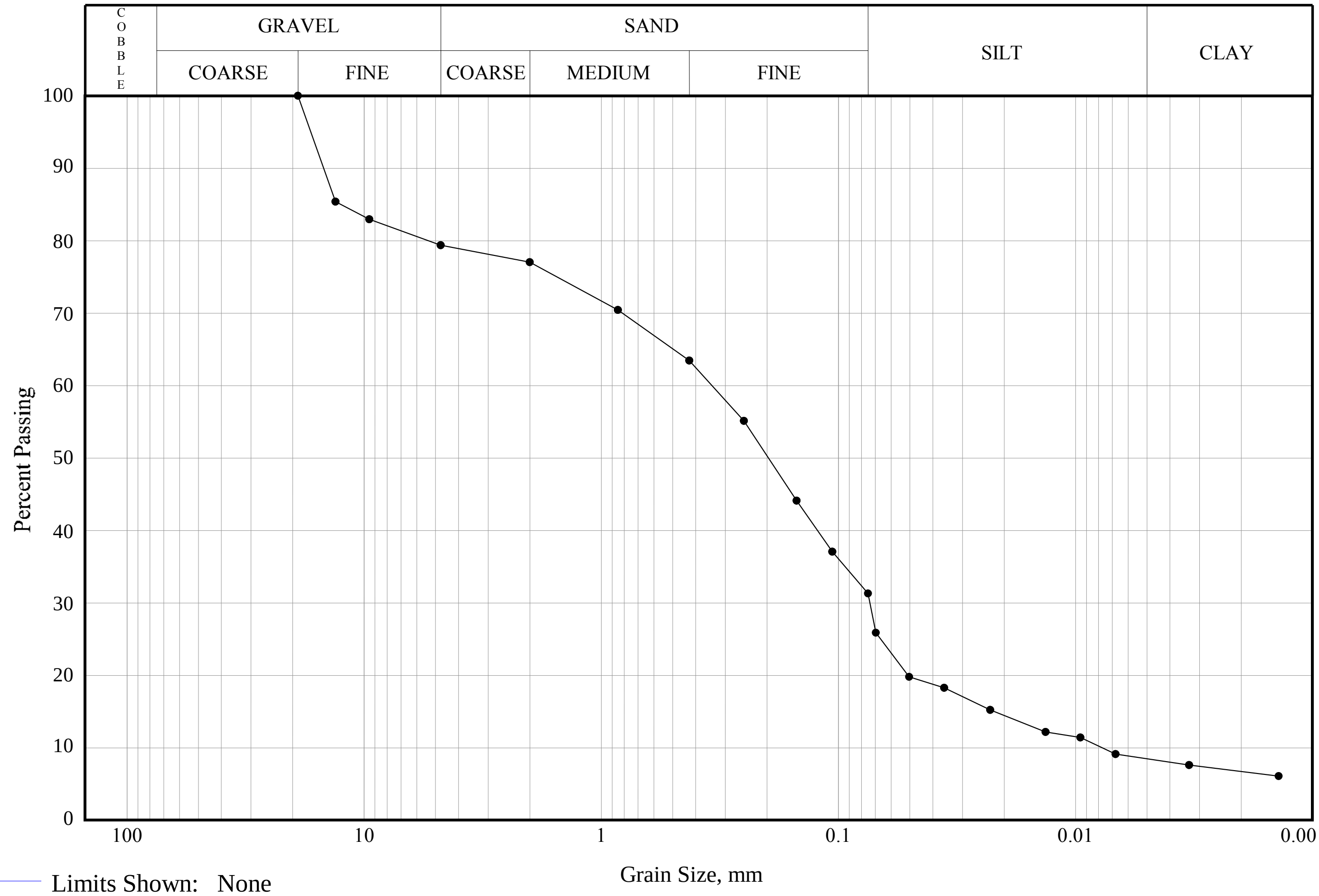


Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	Silt	19-1	01b	0.30-0.61	0.0	8.5	71.8	19.7

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—	Silt , some clay , trace sand	N/A	0.00	0.00	0.01	0.03	0.03	0.06	71.8

GEMTEC Consulting Engineers and Scientists	Client: AECOM Canada Ltd	Soils Grading Chart
	Project: Geotechnical Investigation - 5 Orchard Drive, Ottawa	
	Project #: 6399369	



Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	Glacial Till	19-1MW	03	1.22-1.82	20.6	48.1	22.8	8.5

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—	Gravelly silty sand , trace clay	N/A	0.01	0.02	0.07	0.20	0.34	12.53	22.8

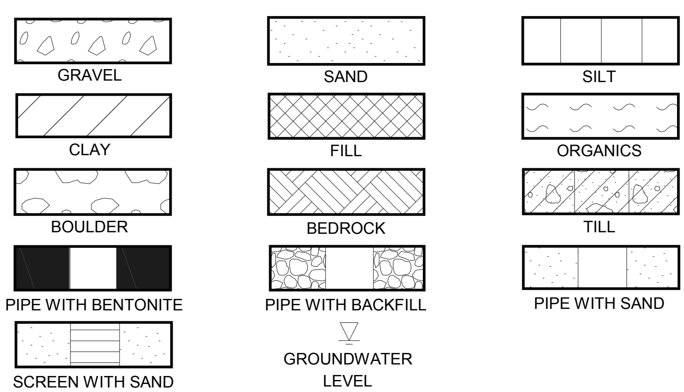
ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

SAMPLE TYPES	
AS	Auger sample
CA	Casing sample
CS	Chunk sample
BS	Borros piston sample
GS	Grab sample
MS	Manual sample
RC	Rock core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled open shelby tube
TP	Thin-walled piston shelby tube
WS	Wash sample

SOIL TESTS	
w	Water content
PL, w_p	Plastic limit
LL, w_L	Liquid limit
C	Consolidation (oedometer) test
D_R	Relative density
DS	Direct shear test
G_S	Specific gravity
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	Organic content test
UC	Unconfined compression test
γ	Unit weight

PENETRATION RESISTANCE	
Standard Penetration Resistance, N The number of blows by a 63.5 kg (140 lb) hammer dropped 760 millimetres (30 in.) required to drive a 50 mm split spoon sampler for a distance of 300 mm (12 in.). For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.	
Dynamic Penetration Resistance The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).	
WH	Sampler advanced by static weight of hammer and drill rods
WR	Sampler advanced by static weight of drill rods
PH	Sampler advanced by hydraulic pressure from drill rig
PM	Sampler advanced by manual pressure

COHESIONLESS SOIL Compactness		COHESIVE SOIL Consistency	
SPT N-Values	Description	Cu, kPa	Description
0-4	Very Loose	0-12	Very Soft
4-10	Loose	12-25	Soft
10-30	Compact	25-50	Firm
30-50	Dense	50-100	Stiff
>50	Very Dense	100-200	Very Stiff
		>200	Hard



GRAIN SIZE

0.01 – 0.1	0.1 – 1.0			1.0 – 10	10 – 100		100 – 1000mm
SILT CLAY	SAND				GRAVEL	COBBLE	BOULDER
	Fine	Medium	Coarse				
0.08	0.4	2	5		80	200	

DESCRIPTIVE TERMINOLOGY
(Based on the CANFEM 4th Edition)

0	10	20	35
TRACE	SOME	ADJECTIVE	noun > 35% and main fraction
trace clay, etc	some gravel, etc.	silty, etc.	sand and gravel, etc.

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE	
Fresh	No visible sign of rock material weathering
Faintly weathered	Weathering limited to the surface of major discontinuities
Slightly weathered	Penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material
Moderately weathered	Weathering extends throughout the rock mass but the rock material is not friable
Completely weathered	Rock is wholly decomposed and in a friable condition but the rock and structure are preserved

CORE CONDITION
Total Core Recovery (TCR) The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run
Solid Core Recovery (SCR) The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.
Rock Quality Designation (RQD) The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completed broken core to 100% for core in solid segments.

BEDDING THICKNESS	
Description	**Thickness**
Thinly laminated	< 6 mm
Laminated	6 - 20 mm
Very thinly bedded	20 - 60 mm
Thinly bedded	60 - 200 mm
Medium bedded	200 - 600 mm
Thickly bedded	600 - 2000 mm
Very thickly bedded	2000 - 6000 mm

DISCONTINUITY SPACING	
Description	**Spacing**
Very close	20 - 60 mm
Close	60 - 200 mm
Moderate	200 - 600 mm
Wide	600 -2000 mm
Very wide	2000 - 6000 mm

ROCK QUALITY	
RQD	**Overall Quality**
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

ROCK COMPRESSIVE STRENGTH	
Comp. Strength, MPa	**Description**
1 - 5	Very weak
5 - 25	Weak
25 - 50	Moderate
50 - 100	Strong
100 - 250	Very strong

APPENDIX B

Rock Core Photo – Figure B1

Report to: AECOM Canada Ltd.
Project: 63993.69 (July 3, 2019)

MW□□□
□ORING DATE□J□□□ □□□□□□
DEPT□□□□□□ □□ □□□□ □ □□□

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Project

GEOTECHNICAL INVESTIGATION
PROPOSED SHELL SERVICE STATION
5 ORCHARD DRIVE, OTTAWA, ON

□IG□RE □□

File No.
63993.69

ROCK CORE PHOTOGRAPHS

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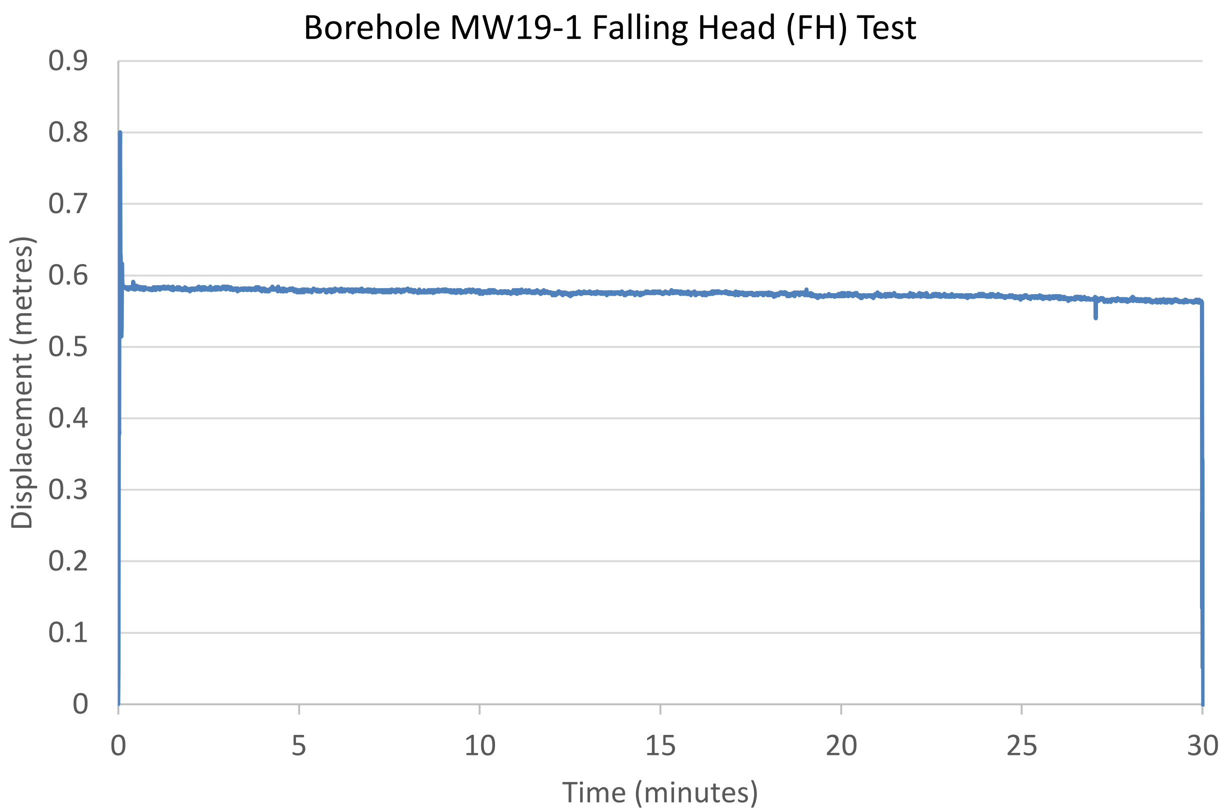
APPENDIX C

Hydraulic Testing Results

Report to: AECOM Canada Ltd.
Project: 63993.69 (July 3, 2019)

Hydraulic Testing

FIGURE C1



Notes:

1. Static water level 1.8 metres below ground surface as measured on June 13, 2019.
2. Insufficient recovery, hydraulic conductivity not calculated.

<u>Well Data:</u>

Displacement observed (slug size): 0.79 metres (0.60 m)
Well Depth: 5.10 metres
Screen Length: 1.52 metres
Well Radius: 0.085 metres

<u>Aquifer Data</u>

Saturated Thickness: 3.3 metres
Anisotropy Ratio (Kz/Kr): 1
Aquifer Model: Confined
Static Water Level: 1.8 metres bgs

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

Date: June 2019

Project: 63993.69

APPENDIX D

Chemical Test Results on Soil Sample
Corrosion of Buried Concrete and Steel
Paracel Laboratories Ltd. Order No. 1924207

PARACEL LABORATORIES LTD.

Order #: 1924207

Certificate of Analysis

Client: GEMTEC Consulting Engineers and Scientists Limited

Client PO:

Report Date: 17-Jun-2019

Order Date: 11-Jun-2019

Project Description: 63993.69

	Client ID:	19-1 SA3	-	-	-
	Sample Date:	04-Jun-19 09:00	-	-	-
	Sample ID:	1924207-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	88.3	-	-	-
General Inorganics					
pH	0.05 pH Units	7.88	-	-	-
Resistivity	0.10 Ohm.m	61.9	-	-	-
Anions					
Chloride	5 ug/g dry	34	-	-	-
Sulphate	5 ug/g dry	7	-	-	-

experience • knowledge • integrity

civil
geotechnical
environmental
field services
materials testing

civil
géotechnique
environnementale
surveillance de chantier
service de laboratoire des matériaux

expérience • connaissance • intégrité

Appendix B

Stormwater Calculation

Time of Concentration:

Airport Equation C<0.4

Basin Length (L)=	72.29	m
Slope%	0.506	%
Basin Area (A)=	0.333	ha

T_C = (3.26 (1.1-C)*L^0.5)/S^0.33

T_C= 26.37 min

Modified Rational Method

Project Name :	Shell - Hazeldean & Fringewood NTI
	Control 100 Year Post to Target Release Rate
Project No. :	

Area =	**0.333**	ha
"C" =	**0.90**	
AC=	**0.298593**	
Tc =	**10.0**	min
Time Increment =	**5.0**	min
Release Rate =	**9.5**	l/s
Max.Storage =	**134**	m3

One Hundred Year

a=	1735.688
b=	6.014
c=	0.820

Time (min)	Rainfall Intensity (mm/hr)	Storm Runoff (l/s)	Runoff Volume (m3)	Released Volume (m3)	Storage Volume (m3)	
10.0	178.6	148.22	88.9	5.7	83.2	
15.0	142.9	118.61	106.8	8.5	98.2	
20.0	120.0	99.57	119.5	11.4	108.1	
25.0	103.8	86.20	129.3	14.2	115.1	
30.0	91.9	76.26	137.3	17.1	120.2	
35.0	82.6	68.55	144.0	19.9	124.0	
40.0	75.1	62.38	149.7	22.8	126.9	
45.0	69.1	57.32	154.8	25.6	129.1	
50.0	64.0	53.09	159.3	28.5	130.8	
55.0	59.6	49.49	163.3	31.3	132.0	
60.0	55.9	46.40	167.0	34.2	132.9	
65.0	52.6	43.70	170.4	37.0	133.4	
70.0	49.8	41.33	173.6	39.8	133.7	
75.0	47.3	39.23	176.5	42.7	133.8	<<<<
80.0	45.0	37.35	179.3	45.5	133.7	
85.0	43.0	35.66	181.8	48.4	133.5	
90.0	41.1	34.13	184.3	51.2	133.1	
95.0	39.4	32.73	186.6	54.1	132.5	
100.0	37.9	31.46	188.8	56.9	131.9	
105.0	36.5	30.30	190.9	59.8	131.1	
110.0	35.2	29.22	192.9	62.6	130.2	
115.0	34.0	28.23	194.8	65.5	129.3	
120.0	32.9	27.31	196.6	68.3	128.3	
125.0	31.9	26.45	198.4	71.2	127.2	

The volume is calculated based on the assumption that all sub-catchments will be directed to the outlet.

Detailed Calculation for Peak Flow - Existing Condition (2 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	C Asphalt/Concrete	C Green Area	C Weighted	I (mm/hr)	Q (l/s)
Catchment Area 1	0.2760	0	0.276	0.90	0.30	0.30	76.805	17.679
Catchment Area 2	0.0300	0.022	0	0.90	0.30	0.74	76.805	4.740
Total Area	0.3060	0.022	0.284	0.90	0.30	0.34	76.805	22.419

Detailed Calculation for Peak Flow - Existing Condition (5 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	C Asphalt/Concrete	C Green Area	C Weighted	I (mm/hr)	Q (l/s)
Catchment Area 1	0.2760	0.000	0.276	0.90	0.30	0.30	104.193	23.984
Catchment Area 2	0.0300	0.022	0	0.90	0.30	0.74	104.193	6.430
Total Area	0.3060	0.022	0.284	0.90	0.30	0.34	104.193	30.414

Detailed Calculation for Peak Flow - Existing Condition (10 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	C Asphalt/Concrete	C Green Area	C Weighted	I (mm/hr)	Q (l/s)
Catchment Area 1	0.2760	0	0.276	0.90	0.30	0.30	122.142	28.115
Catchment Area 2	0.0300	0.022	0	0.90	0.30	0.74	122.142	7.538
Total Area	0.3060	0.022	0.284	0.90	0.30	0.34	122.142	35.653

Detailed Calculation for Peak Flow - Existing Condition (25 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	C Asphalt/Concrete	C Green Area	C Weighted	I (mm/hr)	Q (l/s)
Catchment Area 1	0.2760	0.000	0.276	0.90	0.30	0.30	144.693	33.306
Catchment Area 2	0.0300	0.022	0.008	0.90	0.30	0.74	144.693	8.930
Total Area	0.3060	0.022	0.284	0.90	0.30	0.34	144.693	42.236

Detailed Calculation for Peak Flow - Existing Condition (50 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	C Asphalt/Concrete	C Green Area	C Weighted	I (mm/hr)	Q (l/s)
Catchment Area 1	0.2760	0.000	0.276	0.90	0.30	0.30	161.471	37.168
Catchment Area 2	0.0300	0.022	0.008	0.90	0.30	0.74	161.471	9.965
Total Area	0.3060	0.022	0.284	0.90	0.30	0.34	161.471	47.133

Detailed Calculation for Peak Flow - Existing Condition (100 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	C Asphalt/Concrete	C Green Area	C Weighted	I (mm/hr)	Q (l/s)
Catchment Area 1	0.2760	0.000	0.276	0.90	0.30	0.30	178.559	41.101
Catchment Area 2	0.0300	0.022	0.008	0.90	0.30	0.74	178.559	11.020
Total Area	0.3060	0.022	0.284	0.90	0.30	0.34	178.559	52.121

Detailed Calculation for Peak Flow - Proposed Condition (2 Year)

Catchment Area	Area (ha)	Asphalt/Concert Area (ha)	Green Area (ha)	Building Area (ha)	C Asphalt/Concrete/Building	C Green Area	C Weighted	I (mm/hr)	Q (l/s)
Catchment A1	0.060	0.0570	0.003		0.90	0.3	0.87	76.805	11.15
Catchment A2	0.106	0.0780	0.008	0.020	0.90	0.3	0.85	76.805	19.34
Catchment A3	0.030	0.0250	0.005		0.90	0.3	0.80	76.805	5.12
Catchment A4	0.030	0.0060	0.007	0.017	0.90	0.3	0.76	76.805	4.87
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	76.805	4.93
Catchment A6-1	0.007	0.0010	0.006		0.90	0.3	0.39	76.805	0.58
Catchment A6-2	0.023	0.0020	0.021		0.90	0.3	0.35	76.805	1.73
Catchment A7	0.004	0.0040			0.90	0.3	0.90	76.805	0.77
Catchment A8	0.027	0.0100	0.017		0.90	0.3	0.52	76.805	3.01
Catchment A9	0.011			0.011	0.90	0.3	0.90	76.805	2.11
Total Area	0.333	0.2040	0.081	0.048	0.90	0.3	0.75	76.805	53.61

Detailed Calculation for Peak Flow - Proposed Condition (5 Year)

Catchment Area	Area (ha)	Asphalt/concrete Area (ha)	Green Area (ha)	Building Area (ha)	C Asphalt/Concrete/Building	C Green Area	C Weighted	I (mm/hr)	Q (l/s)
Catchment A1	0.060	0.0570	0.003		0.90	0.3	0.87	104.193	15.12
Catchment A2	0.106	0.0780	0.008	0.020	0.90	0.3	0.85	104.193	26.24
Catchment A3	0.030	0.0250	0.005		0.90	0.3	0.80	104.193	6.95
Catchment A4	0.030	0.0060	0.007	0.017	0.90	0.3	0.76	104.193	6.60
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	104.193	6.69
Catchment A6-1	0.007	0.0010	0.006		0.90	0.3	0.39	104.193	0.78
Catchment A6-2	0.023	0.0020	0.021		0.90	0.3	0.35	104.193	2.35
Catchment A7	0.004	0.0040			0.90	0.3	0.90	104.193	1.04
Catchment A8	0.027	0.0100	0.017		0.90	0.3	0.52	104.193	4.08
Catchment A9	0.011			0.011	0.90	0.3	0.90	104.193	2.87
Total Area	0.333	0.2040	0.081	0.048	0.90	0.3	0.75	104.193	72.73

Detailed Calculation for Peak Flow - Proposed Condition (10 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	Building Area (ha)	C Asphalt/Concrete/Building	C Green Area	C Weighted	I (mm/hr)	Q (l/s)
Catchment A1	0.060	0.0570	0.003		0.90	0.3	0.87	122.142	17.72
Catchment A2	0.106	0.0780	0.008	0.020	0.90	0.3	0.85	122.142	30.76
Catchment A3	0.030	0.0250	0.005		0.90	0.3	0.80	122.142	8.15
Catchment A4	0.030	0.0060	0.007	0.017	0.90	0.3	0.76	122.142	7.74
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	122.142	7.84
Catchment A6-1	0.007	0.0010	0.006		0.90	0.3	0.39	122.142	0.92
Catchment A6-2	0.023	0.0020	0.021		0.90	0.3	0.35	122.142	2.75
Catchment A7	0.004	0.0040			0.90	0.3	0.90	122.142	1.22
Catchment A8	0.027	0.0100	0.017		0.90	0.3	0.52	122.142	4.79
Catchment A9	0.011			0.011	0.90	0.3	0.90	122.142	3.36
Total Area	0.333	0.2040	0.081	0.048	0.90	0.3	0.75	122.142	85.26

Detailed Calculation for Peak Flow - Proposed Condition (25 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	Building Area (ha)	C Asphalt/Concrete/Building	C Green Area	C Weighted	C Weighted (Including Add 10% Value)	I (mm/hr)	Q (l/s)
Catchment A1	0.060	0.0570	0.003		0.90	0.3	0.87	0.96	144.693	23.10
Catchment A2	0.106	0.0780	0.008	0.020	0.90	0.3	0.85	0.94	144.693	40.09
Catchment A3	0.030	0.0250	0.005		0.90	0.3	0.80	0.88	144.693	10.62
Catchment A4	0.030	0.0060	0.007	0.017	0.90	0.3	0.76	0.84	144.693	10.09
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	0.73	144.693	10.22
Catchment A6-1	0.007	0.0010	0.006		0.90	0.3	0.39	0.42	144.693	1.19
Catchment A6-2	0.023	0.0020	0.021		0.90	0.3	0.35	0.39	144.693	3.58
Catchment A7	0.004	0.0040			0.90	0.3	0.90	0.99	144.693	1.59
Catchment A8	0.027	0.0100	0.017		0.90	0.3	0.52	0.57	144.693	6.24
Catchment A9	0.011			0.011	0.90	0.3	0.90	0.99	144.693	4.38
Total Area	0.333	0.2040	0.081	0.048	0.90	0.3	0.75	0.83	144.693	111.10

Note:
C value for the 100-year storm is increased by 10%, to a maximum of 1.0 per Ottawa Sewer Design Guidelines.

Detailed Calculation for Peak Flow - Proposed Condition (50 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	Building Area (ha)	C Asphalt/Concrete/ Building	C Green Area	C Weighted	New C Weighted (Including Add 20% Value)	I (mm/hr)	Q (l/s)
Catchment A1	0.060	0.0570	0.003		0.90	0.3	0.87	1.00	161.471	28.12
Catchment A2	0.106	0.0780	0.008	0.020	0.90	0.3	0.85	1.00	161.471	48.80
Catchment A3	0.030	0.0250	0.005		0.90	0.3	0.80	0.96	161.471	12.93
Catchment A4	0.030	0.0060	0.007	0.017	0.90	0.3	0.76	0.91	161.471	12.28
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	0.79	161.471	12.44
Catchment A6-1	0.007	0.0010	0.006		0.90	0.3	0.39	0.46	161.471	1.45
Catchment A6-2	0.023	0.0020	0.021		0.90	0.3	0.35	0.42	161.471	4.36
Catchment A7	0.004	0.0040			0.90	0.3	0.90	1.00	161.471	1.94
Catchment A8	0.027	0.0100	0.017		0.90	0.3	0.52	0.63	161.471	7.60
Catchment A9	0.011			0.011	0.90	0.3	0.90	1.00	161.471	5.33
Total Area	0.333	0.2040	0.081	0.048	0.90	0.3	0.75	0.88	161.471	135.26

Note:
C value for the 100-year storm is increased by 20%, to a maximum of 1.0 per Ottawa Sewer Design Guidelines.

Detailed Calculation for Peak Flow - Proposed Condition (100 Year)

Catchment Area	Area (ha)	Asphalt/Concrete Area (ha)	Green Area (ha)	Building Area (ha)	C Asphalt/Concrete/Building	C Green Area	C Weighted	New C Weighted (Including Add 25% Value)	I (mm/hr)	Q (l/s)
Catchment A1	0.060	0.0570	0.003		0.90	0.3	0.87	1.00	178.559	29.78
Catchment A2	0.106	0.0780	0.008	0.020	0.90	0.3	0.85	1.00	178.559	52.62
Catchment A3	0.030	0.0250	0.005		0.90	0.3	0.80	1.00	178.559	14.89
Catchment A4	0.030	0.0060	0.007	0.017	0.90	0.3	0.76	0.95	178.559	14.15
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	0.83	178.559	14.33
Catchment A6-1	0.007	0.0010	0.006		0.90	0.3	0.39	0.48	178.559	1.68
Catchment A6-2	0.023	0.0020	0.021		0.90	0.3	0.35	0.44	178.559	5.03
Catchment A7	0.004	0.0040			0.90	0.3	0.90	1.00	178.559	1.99
Catchment A8	0.027	0.0100	0.017		0.90	0.3	0.52	0.65	178.559	8.75
Catchment A9	0.011			0.011	0.90	0.3	0.90	1.00	178.559	5.46
Total Area	0.333	0.2040	0.081	0.048	0.90	0.3	0.75	0.90	178.559	148.67

Note:
C value for the 100-year storm is increased by 25%, to a maximum of 1.0 per Ottawa Sewer Design Guidelines.

Detailed Calculation for Available Storage Within the Site:

Storage calculations in the surface area are based on the following formula for volume of a cone:

V = (A x d)/3

where:

V = volume in cu.m.
A = ponding area in sq.m.
d = ponding depth in meters

Underground Storage		
Pipe Dia (mm)	Length (m)	Storage (m3)
1200	92.69	104.78
1500	19	33.56
1050	26	22.5
Total		160.84

Surface Storage			
Area#	Ponding Depth (m)	Area (m^2)	Storage (m^3)
Catchment A1	0.13	67.55	8.7815
Catchment A2	0.1	98.84	9.884
Catchment A3	0.1	59.64	5.964
Catchment A4	0.08	13.49	1.0792
Catchment A5	0.1	40.66	4.066
Total			29.77/3=9.92 m^3

The actual total storage within the site is 170.76 m^3.

Appendix C

Pre - Consultation

Genavieve Melatti

From: Nader Nakhaei <NNakhaei@mvc.on.ca>
Sent: Tuesday, June 5, 2018 9:32 AM
To: Genavieve Melatti
Cc: Steve Merrick
Subject: RE: 5 Orchard Drive

Hi Genavieve,

The stormwater quality target for the Carp River is a 'Normal' Level of Protection (i.e. 70% TSS removal). Please let me know if you have any further question or concern.

Cheers,

Nader Nakhaei, Ph.D. | Postdoctoral Felllow / Water Resources Engineer (EIT) | Mississippi Valley Conservation Authority
www.mvc.on.ca | t. 613 253 0006 ext. 259 | f. 613 253 0122 | NNakhaei@mvc.on.ca

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From: Genavieve Melatti [mailto:GMelatti@dsel.ca]
Sent: Tuesday, June 5, 2018 9:14 AM
To: Nader Nakhaei <NNakhaei@mvc.on.ca>
Cc: Steve Merrick <SMerrick@dsel.ca>
Subject: 5 Orchard Drive

Good morning Nader,

We wanted to touch base with you regarding 5 Orchard Drive.

The development proposes a residential component consisting of 65 townhomes, 2 semi-detached homes and 7 single family residences. It also contemplates a future commercial component. The development will discharge stormwater into the existing 675 mm diameter storm sewer within Hazeldean Road. Stormwater collected form site travels approximately 0.7 km before discharging into a pond on the north side of Hazeldean Road show below. Discharge from the pond travels an additional 0.97m through an open ditch to Carp River.

Nikfarjam, Toktam

From: Matt Craig <mcraig@mvc.on.ca>
Sent: Wednesday, March 4, 2020 12:54 PM
To: Nikfarjam, Toktam
Subject: FW: Guideline for Stormwater Management for the site

Hi Toktam

We don't have published guidelines but use watershed plans or master servicing studies for reference to determine water quality objectives.

Regards

Matt Craig | Manager of Planning and Regulations | Mississippi Valley Conservation Authority

www.mvc.on.ca |t. 613 253 0006 ext. 226| f. 613 253 0122 | mcraig@mvc.on.ca

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From: Info
Sent: March 4, 2020 11:29 AM
To: Matt Craig <mcraig@mvc.on.ca>
Subject: FW: Guideline for Stormwater Management for the site

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From: Nikfarjam, Toktam <toktam.nikfarjam@aecom.com>
Sent: Wednesday, March 4, 2020 9:48 AM
To: Info <info@mvc.on.ca>
Subject: Guideline for Stormwater Management for the site

Hi Sir/Madam,

We have a site which is located in City of Ottawa and jurisdiction of Mississippi Valley Conservation Authority. I was searching in the website to find the guideline for the requirements of the MVCA in terms of Water Quality, Water Quantity and Erosion Control. But unfortunately I was not able to find the MVCA guideline. I would be appreciated if you send me the link for the guideline.

Thanks,

Toktam Nikfarjam, P.Eng
Water Engineer, Water
D +1-905-747-1696
M +1-519-580-6251
toktam.nikfarjam@aecom.com

File Number: PC2019-0169
July 22, 2019

<u>5 Orchard Drive</u>
<u>Pre-application Consultation Meeting Notes</u>

Location: Room 5105E, City Hall
Date: July 16, 2019

Attendees: Colette Gorni, Planner, City of Ottawa
Laurel McCreight, Planner, City of Ottawa
Samantha Gatchene, Student Planner, City of Ottawa
Rosanna Baggs, Project Manager (Transportation), City of Ottawa
Lino Paoloni, Shell
Kerry K. Morrison, Shell
Bikram Arora, Shell
Tony Batten, AECOM
Cody Campanale, Campanale Homes
Nadia De Santi, WSP
Michael Hanifi, WSP
Sarah MacDonald, WSP

<u>Comments from the Applicant</u>

Campanale Homes:

1. Campanale Homes has applied for a Plan of Subdivision and a Zoning By-law Amendment for the lands municipally known as 5 Orchard Drive. There is intended to be both residential and commercial uses on the property. These applications are pending.
2. Residential development will occur in the rear portion of the property. A mix of townhomes, semi- and single-detached dwellings along a cul-de-sac is proposed.
3. A future commercial block is planned along Hazeldean Road. However, Campanale Homes has not submitted an application with City for this portion of the site.
4. Campanale Homes has an agreement with Shell to lease lands in the north eastern portion of the site for use as a gas station.
5. There are two blocks that are being dedicated to the City of Ottawa as a part of the Plan of Subdivision application. An 8-metre block is being dedicated for storm water tanks and a watermain to service the residential block. The other block being dedicated is identified as a pedestrian pathway.

WSP/Shell:

6. This is the first shell site in Ottawa that WSP is working on. There will likely be many more.
7. The applicant is proposing a gas station use on the leased portion of the site. There will also be associated gas pumps, car wash, and convenience store.
8. There is an interest in proceeding with the Shell gas station ahead of the rest of the Plan of Subdivision.
9. The conceptual site plan layout was designed based on the queuing line placement and fuel delivery routes within the site.

Planning Comments

1. This is a formal pre-application consultation meeting for a Site Plan Control Application - Standard. Application form, timeline and fees can be found here.
2. Please confirm the number of parking spaces provided. A total of 7 spaces is required under the Zoning by-law for the proposed convenience store use (3.4 per $100m^2$).
3. Please provide some bicycle parking on the site for the customers of the convenience store. Based on the size of the proposed retail building, the Zoning By-law requires 0.8 spaces be provided, which should be rounded up.
4. Please refer to Section 112 - Provisions for Drive-Through Operations when designing the car wash facility on the site.
5. Registration of the associated subdivision is required before a building permit can be obtained. However, the applicant is encouraged to submit a site plan control application in advance of registration to begin the process.
6. Please reach out to the applicable Ward Councillor and set up a meeting to present plans for the site.

Urban Design Comments

1. The City prefers for drive through queuing lines be internal to the site and not adjacent to roadways.
2. Please provide landscaping along Hazeldean Road, and along the rear of the property. Coniferous trees would be a good option to provide year-round green.
3. Consider moving the convenience store building closer to Hazeldean Road.

4. Please note that the City of has '[Urban Design Guidelines for Gas Stations]'.

Transportation Comments

1. Follow Traffic Impact Assessment Guidelines
   - Traffic Impact Assessment will be required.
   - Start this process asap.
   - Applicant advised that their application will not be deemed complete until the submission of the draft step 1-4, including the functional draft RMA package (if applicable) and/or monitoring report (if applicable).
2. ROW protection on Hazeldean is 37.5m even.
3. Corner triangles as per OP Annex 1 - Road Classification and Rights-of-Way at the following locations on the final plan will be required:
   - Local Road to Arterial Road: 5 metre x 5 metres
4. Noise Impact Studies required for the following:
   - Stationary (if there will be any exposed mechanical equipment due to the proximity to neighbouring noise sensitive land uses)
5. The curb line on Fringewood will be required to be adjusted so that the through lane is reduce to 3.5m in width.
6. It is recommended that the path that the WB-20 takes to service the fuel storage tanks be plan in a way to minimize the access widths; i.e. make use of the entire site for turning movements, this can be accomplished by the entering by the future full movement access at the west end of the site. Otherwise, make use of truck turning aprons to reduce the access widths.
7. The current configuration of the drive thru car was queue may lead to congestion at the pumps. Recommended to relocate the drive-thru entrance.
8. On site plan:
   - Show all details of the roads abutting the site up to and including the opposite curb; include such items as pavement markings, accesses and/or sidewalks.
   - Turning templates will be required for all accesses showing the largest vehicle to access the site; required for internal movements and at all access (entering and exiting and going in both directions). Provide on a separate drawing.

- Show all curb radii measurements; ensure that all curb radii are reduced as much as possible
- Show lane/aisle widths.
- Sidewalk is to be constructed as per City Specification 7.1.
- Grey out any area that will not be impacted by this application. Private access minimum distance to signalized intersection as per TAC design;
    i. On Hazeldean 70m
    ii. On Fringewood 15m
- Clear throat length for the commercial block as per TAC design will be dependent on the use of the entire site of the site. The RIRO should expect a throat length of a minimum 15-25m.

Engineering Comments

1. The Servicing Study Guidelines for Development Applications are available at the following link: https://ottawa.ca/en/city-hall/planning-and-development/information-developers/development-application-review-process/development-application-submission/guide-preparing-studies-and-plans
2. Record drawings and utility plans are available for purchase from the City's Information Centre. Contact the City's Information Centre by email at informationcentre@ottawa.ca or by phone at (613) 580-2424 x44455
3. Stormwater quantity control criteria – be consistent with the quantity control criteria that will be specified in the approved subdivision Servicing and Stormwater Management Report
4. Stormwater quality control – Consult with the Conservation Authority (MVCA) for their requirements. Include the correspondence with the MVCA in the stormwater/site servicing report.
5. Oil and Grit separator is required for the proposed use (gas station)
6. MECP ECA is required (Industrial sewage works-direct submission)
7. Sanitary quantity control criteria - be consistent with the quantity control criteria that will be specified in the approved subdivision Servicing and Stormwater Management Report

8. When calculating the composite runoff coefficient (C) for the site (post development), please provide a drawing showing the individual drainage area and its runoff coefficient.

9. When using the modified rational method to calculate the storage requirements for the site, the underground storage should not be included in the overall available storage. The modified rational method assumes that the restricted flow rate is constant throughout the storm which, in this case, underestimates the storage requirement prior to the 1:100 year head elevation being reached. Alternately, if you wish to include the underground storage, you may use an assumed average release rate equal to 50% of the peak allowable rate. Otherwise, disregard the underground storage as available storage or provide modeling to support the design.

10. Engineering plans are to be submitted on standard A1 size (594mm x 841mm) sheets.

11. Phase 1 ESA and Phase 2 ESA must conform to clause 4.8.4 of the Official Plan that requires that development applications conform to Ontario Regulation 153/04.

12. Provide the following information for water main boundary conditions:

    - Location map with water service connection location
    - Average daily demand (l/s)
    - Maximum daily demand (l/s)
    - Maximum hourly demand (l/s)
    - Fire flow demand (provide fire detailed flow calculations based on the fire underwriters survey method)
    - If you are proposing any exterior light fixtures, all must be included and approved as part of the site plan approval. Therefore, the lights must be clearly identified by make, model and part number. All external light fixtures must meet the criteria for full cut-off classification as recognized by the Illuminating Engineering Society of North America (IESNA or IES) and must result in minimal light spillage onto adjacent properties (as a guideline, 0.5 fc is normally the maximum allowable spillage). In order to satisfy these criteria, the applicant must provide certification from an acceptable professional engineer. The location of all exterior fixtures, a table showing the fixture types (including make, model, part number), and the mounting heights must be included on a plan.

<u>Forestry Comments</u>

1. If there are trees on site, a Tree Conservation Report (TCR) will be required.
2. A tree permit is required for the removal of trees.

TCR Requirements:

3. a Tree Conservation Report (TCR) must be supplied for review along with the various other plans/reports required by the City; an approved TCR is a requirement for Site Plan approval
4. any removal of privately-owned trees 10cm or larger in diameter requires a tree permit issued under the Urban Tree Conservation Bylaw; the permit is based on the approved TCR
5. the removal of City-owned trees will require the permission of Forestry Services who will also review the submitted TCR
6. the TCR may be combined with the Landscape Plan
7. the TCR must list all trees greater than 10cm in diameter by species, diameter and health condition;
8. the TCR must address all trees with a critical root zone that extends into the developable area – all trees that could be impacted by the construction that are outside the developable area need to be addressed.
9. Trees with a trunk that crosses/touches a property line are considered co-owned by both property owners; permission from the adjoining property owner must be obtained prior to the removal of co-owned trees
10. If trees are to be removed, the TCR must clearly show where they are, and document the reason they can not be retained – please provide a plan showing retained and removed treed areas
11. All retained trees must be shown and all retained trees within the area impacted by the development process must be protected as per City guidelines listed on Ottawa.ca
12. Please ensure newly planted trees have an adequate soil volume for their size at maturity. The following is a table of recommended minimum soil volumes:

Tree Type/Size	Single Tree Soil Volume (m3)	Multiple Tree Soil Volume (m3/tree)
Ornamental	15	9
Columnar	15	9
Small	20	12
Medium	25	15
Large	30	18
Conifer	25	15

13. The City requests that all efforts are made to retain trees – trees should be healthy, and of a size and species that can grow into the site and contribute to Ottawa's urban forest canopy

14. For more information on the TCR process or help with tree retention options, contact Mark Richardson mark.richardson@ottawa.ca

MVCA

1. The commercial component of the site should connect independently to the proposed storm sewer within Fringewood Drive.

2. The total release rate for the entire commercial section of the site is 51.9 L/s (100yr). A total of 843.1 m3 of storage has been estimated to be needed for the commercial portion of the site which needs to be considered in the proposed development as well. It's been mentioned that the commercial block is contemplated to use LID SWM techniques to attenuate to the allowable release rate.

Sincerely,

Colette Gorni
Planner I
Development Review - West

APPLICANT'S STUDY AND PLAN IDENTIFICATION LIST

Legend: **S** indicates that the study or plan is required with application submission.
A indicates that the study or plan may be required to satisfy a condition of approval/draft approval.

For information and guidance on preparing required studies and plans refer here:

S/A	Number of copies	ENGINEERING		S/A	Number of copies
S	15	1. Site Servicing Plan	2. Site Servicing Study	**S**	3
S	15	3. Grade Control and Drainage Plan	4. Geotechnical Study / Slope Stability Study	**S**	3
		5. Composite Utility Plan	6. Groundwater Impact Study		
		7. Servicing Options Report	8. Wellhead Protection Study		
S	9	9. Transportation Impact Assessment (TIA)	10.Erosion and Sediment Control Plan / Brief	**S**	3
S	3	11.Storm water Management Report / Brief	12.Hydro geological and Terrain Analysis		
		13.Hydraulic Water main Analysis	14.Noise / Vibration Study	**S**	3
	PDF only	15.Roadway Modification Functional Design	16.Confederation Line Proximity Study		

S/A	Number of copies	PLANNING / DESIGN / SURVEY		S/A	Number of copies
		17.Draft Plan of Subdivision	18.Plan Showing Layout of Parking Garage		
		19.Draft Plan of Condominium	20.Planning Rationale	**S**	3
S	15	21.Site Plan	22.Minimum Distance Separation (MDS)		
		23.Concept Plan Showing Proposed Land Uses and Landscaping	24.Agrology and Soil Capability Study		
		25.Concept Plan Showing Ultimate Use of Land	26.Cultural Heritage Impact Statement		
S	15	27.Landscape Plan	28.Archaeological Resource Assessment Requirements: **S** (site plan) **A** (subdivision, condo)		
S	2	29.Survey Plan	30.Shadow Analysis		
S	3	31.Architectural Building Elevation Drawings (dimensioned)	32.Design Brief (Included in Planning Rationale)	**S**	Available online
		33.Wind Analysis			

S/A	Number of copies	ENVIRONMENTAL		S/A	Number of copies
S	3	34.Phase 1 Environmental Site Assessment	35.Impact Assessment of Adjacent Waste Disposal/Former Landfill Site		
S	3	36.Phase 2 Environmental Site Assessment (depends on the outcome of Phase 1)	37.Assessment of Landform Features		
		38.Record of Site Condition	39.Mineral Resource Impact Assessment		
S	3	40.Tree Conservation Report	41.Environmental Impact Statement / Impact Assessment of Endangered Species		
		42.Mine Hazard Study / Abandoned Pit or Quarry Study	43.Integrated Environmental Review (Draft, as part of Planning Rationale)		

S/A	Number of copies	ADDITIONAL REQUIREMENTS		S/A	Number of copies
S	1	44. Applicant's Public Consultation Strategy (may be provided as part of the Planning Rationale)	45.		

Meeting Date: July 16, 2019

Application Type: *Site Plan Control*

File Lead (Assigned Planner): Colette Gorni

Infrastructure Approvals Project Manager: Santhosh Kuruvilla

Site Address (Municipal Address): 5 Orchard Drive

*Preliminary Assessment: 1 ☐ 2 ☒ 3 ☐ 4 ☐ 5

*One (1) indicates that considerable major revisions are required before a planning application is submitted, while five (5) suggests that proposal appears to meet the City's key land use policies and guidelines. **This assessment is purely advisory and does not consider technical aspects of the proposal or in any way guarantee application approval.**

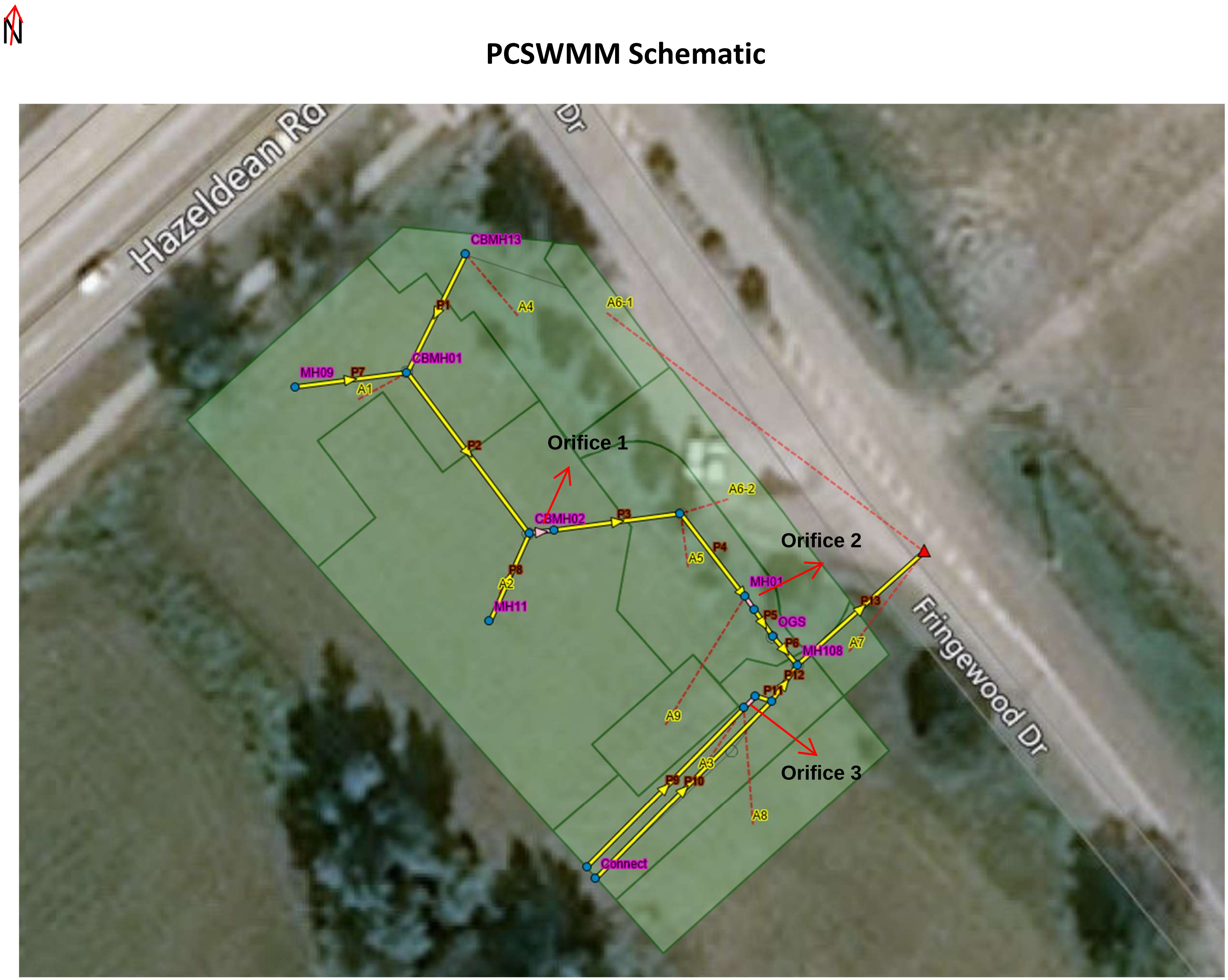
Please note that PDF versions of all the listed requirements must be submitted with the application, stored in a USB drive or CD

It is important to note that the need for additional studies and plans may result during application review. If following the submission of your application, it is determined that material that is not identified in this checklist is required to achieve complete application status, in accordance with the Planning Act and Official Plan requirements, the Planning, Infrastructure and Economic Development Department *will notify you of outstanding material required within the required 30 day period. Mandatory pre-application consultation will not shorten the City's standard processing timelines, or guarantee that an application will be approved. It is intended to help educate and inform the applicant about submission requirements as well as municipal processes, policies, and key issues in advance of submitting a formal development application. This list is valid for one year following the meeting date. If the application is not submitted within this timeframe the applicant must again pre-consult with the Planning, Infrastructure and Economic Development Department.*

Appendix D

PCWMM Output

PCSWMM Schematic



```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.012)
--------------------------------------------------------------


*************
Element Count
*************
Number of rain gages ...... 18
Number of subcatchments ... 10
Number of nodes ........... 17
Number of links ........... 16
Number of pollutants ...... 0
Number of land uses ....... 0


****************
Raingage Summary
****************
                                                              Data       Recording
Name                 Data Source                              Type       Interval
-----------------------------------------------------------------------------------
Chicago_3h_100Y      Chicago_3h_100Y                          INTENSITY    10 min.
Chicago_3h_100Y_1.25 Chicago_3h_100Y_1.25                     INTENSITY    10 min.
Chicago_3h_10Y       Chicago_3h_10Y                           INTENSITY    10 min.
Chicago_3h_25Y       Chicago_3h_25Y                           INTENSITY    10 min.
Chicago_3h_25Y_1.1   Chicago_3h_25Y_1.1                       INTENSITY    10 min.
Chicago_3h_2Y        Chicago_3h_2Y                            INTENSITY    10 min.
Chicago_3h_50Y       Chicago_3h_50Y                           INTENSITY    10 min.
Chicago_3h_50Y_1.2   Chicago_3h_50Y-1.2                       INTENSITY    10 min.
Chicago_3h_5Y        Chicago_3h_5Y                            INTENSITY    10 min.
Chicago_6h_100Y      Chicago_6h_100Y                          INTENSITY    10 min.
Chicago_6h_100Y_1.25 Chicago_6h_100Y_1.25                     INTENSITY    10 min.
Chicago_6h_10Y       Chicago_6h_10Y                           INTENSITY    10 min.
Chicago_6h_25Y       Chicago_6h_25Y                           INTENSITY    10 min.
Chicago_6h_25Y_1.1   Chicago_6h_25Y_1.1                       INTENSITY    10 min.
Chicago_6h_2Y        Chicago_6h_2Y                            INTENSITY    10 min.
Chicago_6h_50Y       Chicago_6h_50Y                           INTENSITY    10 min.
Chicago_6h_50Y_1.2   Chicago_6h_50Y_1.2                       INTENSITY    10 min.
Chicago_6h_5Y        Chicago_6h_5Y                            INTENSITY    10 min.


********************
Subcatchment Summary
********************
Name                        Area     Width   %Imperv    %Slope Rain Gage            Outlet
--------------------------------------------------------------------------------------------
A1                          0.06     25.00     87.00    1.2200 Chicago_3h_5Y        CBMH01
A2                          0.11     30.29     85.00    0.5500 Chicago_3h_5Y        CBMH02
A3                          0.03     13.81     80.00    0.5000 Chicago_3h_5Y        CBMH06
A4                          0.03     50.00     76.00    2.4300 Chicago_3h_5Y        CBMH13
A5                          0.04     31.82     66.00    1.7500 Chicago_3h_5Y        CBMH03
A6-1                        0.01      3.89     36.00    7.0000 Chicago_3h_5Y        STM106
A6-2                        0.02      6.76     35.00    7.0000 Chicago_3h_5Y        CBMH03
A7                          0.00      5.71     90.00    1.5100 Chicago_3h_5Y        STM106
A8                          0.03     11.74     52.00    1.0350 Chicago_3h_5Y        CBMH06
A9                          0.01      8.00     90.00    0.1000 Chicago_3h_5Y        MH01


************
Node Summary
************
                                         Invert      Max.    Ponded    External
Name                 Type                 Elev.     Depth      Area    Inflow
```

CBMH01	JUNCTION	102.80	2.30	67.5
CBMH02	JUNCTION	102.68	2.42	98.8
CBMH02-N	JUNCTION	102.68	2.42	0.0
CBMH03	JUNCTION	102.60	2.50	40.7
CBMH04	JUNCTION	102.43	2.76	59.6
CBMH06	JUNCTION	102.47	2.70	0.0
CBMH06-N	JUNCTION	102.47	2.70	0.0
CBMH07	JUNCTION	102.55	2.98	0.0
CBMH13	JUNCTION	103.01	2.26	0.0
Connect	JUNCTION	102.55	2.95	0.0
MH01	JUNCTION	102.53	2.72	0.0
MH01-N	JUNCTION	102.53	2.74	0.0
MH09	JUNCTION	102.90	2.60	0.0
MH108	JUNCTION	102.40	2.81	0.0
MH11	JUNCTION	102.79	2.66	0.0
OGS	JUNCTION	102.44	2.84	0.0
STM106	OUTFALL	102.31	0.68	0.0

****
Link Summary
****

Name	From Node	To Node	Type	Length	%Slope	Roughness
P1	CBMH13	CBMH01	CONDUIT	14.7	1.0028	0.0130
P10	Connect	CBMH04	CONDUIT	28.3	0.3008	0.0130
P11	CBMH06-N	CBMH04	CONDUIT	2.5	0.3137	0.0130
P12	CBMH04	MH108	CONDUIT	5.3	0.3042	0.0130
P13	MH108	STM106	CONDUIT	19.1	0.5084	0.0130
P2	CBMH01	CBMH02	CONDUIT	22.6	0.3496	0.0130
P3	CBMH02-N	CBMH03	CONDUIT	17.3	0.3472	0.0130
P4	CBMH03	MH01	CONDUIT	13.1	0.3498	0.0130
P5	MH01-N	OGS	CONDUIT	4.0	0.3500	0.0130
P6	OGS	MH108	CONDUIT	4.3	0.3529	0.0130
P7	MH09	CBMH01	CONDUIT	21.0	0.3476	0.0130
P8	MH11	CBMH02	CONDUIT	19.0	0.3474	0.0130
P9	CBMH07	CBMH06	CONDUIT	26.0	0.3000	0.0130
Orifice1	CBMH02	CBMH02-N	OUTLET			
Orifice2	MH01	MH01-N	OUTLET			
Orifice3	CBMH06	CBMH06-N	OUTLET			

****
Cross Section Summary
****

Conduit	Shape	Full Depth	Full Area	Hyd. Rad.	Max. Width	No. of Barrels	Full Flow
P1	CIRCULAR	1.20	1.13	0.30	1.20	1	3.90
P10	CIRCULAR	0.45	0.16	0.11	0.45	1	0.16
P11	CIRCULAR	0.45	0.16	0.11	0.45	1	0.16
P12	CIRCULAR	0.45	0.16	0.11	0.45	1	0.16
P13	CIRCULAR	0.68	0.36	0.17	0.68	1	0.60
P2	CIRCULAR	1.20	1.13	0.30	1.20	1	2.31
P3	CIRCULAR	1.20	1.13	0.30	1.20	1	2.30
P4	CIRCULAR	1.20	1.13	0.30	1.20	1	2.31
P5	CIRCULAR	0.45	0.16	0.11	0.45	1	0.17
P6	CIRCULAR	0.45	0.16	0.11	0.45	1	0.17
P7	CIRCULAR	1.20	1.13	0.30	1.20	1	2.30
P8	CIRCULAR	1.50	1.77	0.38	1.50	1	4.17
P9	CIRCULAR	1.05	0.87	0.26	1.05	1	1.50

```
*********************************************************
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 ............... CMS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ YES
  Water Quality .......... NO
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date ............ 02/28/2020 00:00:00
Ending Date .............. 02/29/2020 00:00:00
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 ........ 5.00 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
**************************     ---------       -------
Total Precipitation ......         0.014        42.540
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         0.001         2.232
Surface Runoff ...........         0.013        40.308
Final Storage ............         0.000         0.000
Continuity Error (%) .....         0.000


**************************        Volume        Volume
Flow Routing Continuity        hectare-m     10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.013         0.134
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         0.013         0.134
Flooding Loss ............         0.000         0.000
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.000         0.000
Continuity Error (%) .....         0.000


***************************
Time-Step Critical Elements
```

****
None

****
Highest Flow Instability Indexes
****
Link P2 (1)
Link P7 (1)

****
Routing Time Step Summary
****
Minimum Time Step : 4.50 sec
Average Time Step : 5.00 sec
Maximum Time Step : 5.00 sec
Percent in Steady State : 0.00
Average Iterations per Step : 2.00
Percent Not Converging : 0.00

****
Subcatchment Runoff Summary
****

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Pea Runof CM
A1	42.54	0.00	0.00	1.17	41.37	0.02	0.0
A2	42.54	0.00	0.00	1.35	41.19	0.04	0.0
A3	42.54	0.00	0.00	1.80	40.74	0.01	0.0
A4	42.54	0.00	0.00	2.16	40.38	0.01	0.0
A5	42.54	0.00	0.00	3.06	39.48	0.01	0.0
A6-1	42.54	0.00	0.00	5.76	36.78	0.00	0.0
A6-2	42.54	0.00	0.00	5.85	36.69	0.01	0.0
A7	42.54	0.00	0.00	0.90	41.64	0.00	0.0
A8	42.54	0.00	0.00	4.32	38.22	0.01	0.0
A9	42.54	0.00	0.00	0.90	41.64	0.00	0.0

****
Node Depth Summary
****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters
CBMH01	JUNCTION	0.22	0.76	103.56	0 02:24	0.76
CBMH02	JUNCTION	0.29	0.88	103.56	0 02:24	0.88
CBMH02-N	JUNCTION	0.17	0.60	103.28	0 01:54	0.60
CBMH03	JUNCTION	0.21	0.68	103.28	0 01:54	0.68
CBMH04	JUNCTION	0.02	0.05	102.48	0 01:25	0.05
CBMH06	JUNCTION	0.07	0.58	103.05	0 01:24	0.58
CBMH06-N	JUNCTION	0.01	0.05	102.52	0 01:24	0.05
CBMH07	JUNCTION	0.06	0.50	103.05	0 01:24	0.50
CBMH13	JUNCTION	0.13	0.55	103.56	0 02:24	0.55
Connect	JUNCTION	0.00	0.00	102.55	0 00:00	0.00
MH01	JUNCTION	0.25	0.75	103.28	0 01:54	0.75
MH01-N	JUNCTION	0.02	0.05	102.57	0 01:54	0.05

MH09	JUNCTION	0.18	0.66	103.56	0	02:23	0.66
MH108	JUNCTION	0.02	0.05	102.45	0	01:34	0.05
MH11	JUNCTION	0.23	0.77	103.56	0	02:24	0.77
OGS	JUNCTION	0.02	0.05	102.49	0	01:54	0.05
STM106	OUTFALL	0.02	0.04	102.35	0	01:34	0.04

****
Node Inflow Summary
****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Fl Balan Err Perce
CBMH01	JUNCTION	0.017	0.028	0 01:07	0.0248	0.0481	0.0
CBMH02	JUNCTION	0.031	0.031	0 01:06	0.0437	0.094	0.0
CBMH02-N	JUNCTION	0.000	0.006	0 01:09	0	0.0827	0.0
CBMH03	JUNCTION	0.017	0.017	0 01:05	0.0223	0.105	-0.0
CBMH04	JUNCTION	0.000	0.003	0 01:24	0	0.022	0.0
CBMH06	JUNCTION	0.016	0.016	0 01:05	0.0221	0.0277	0.1
CBMH06-N	JUNCTION	0.000	0.003	0 01:24	0	0.022	0.0
CBMH07	JUNCTION	0.000	0.007	0 01:06	0	0.00564	0.8
CBMH13	JUNCTION	0.009	0.009	0 01:05	0.0121	0.0121	-0.2
Connect	JUNCTION	0.000	0.000	0 00:00	0	0	0.0
MH01	JUNCTION	0.003	0.007	0 01:10	0.005	0.108	0.0
MH01-N	JUNCTION	0.000	0.003	0 01:54	0	0.108	0.0
MH09	JUNCTION	0.000	0.009	0 01:09	0	0.0076	0.2
MH108	JUNCTION	0.000	0.006	0 01:34	0	0.13	0.0
MH11	JUNCTION	0.000	0.009	0 01:05	0	0.00981	0.1
OGS	JUNCTION	0.000	0.003	0 01:54	0	0.108	0.0
STM106	OUTFALL	0.003	0.008	0 01:10	0.00424	0.134	0.0

****
Node Surcharge Summary
****

No nodes were surcharged.

****
Node Flooding Summary
****

No nodes were flooded.

****
Outfall Loading Summary
****

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
STM106	57.72	0.003	0.008	0.134
System	57.72	0.003	0.008	0.134

****
Link Flow Summary
****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days	hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
P1	CONDUIT	0.005	0	01:07	0.45	0.00	0.52
P10	CONDUIT	0.000	0	00:00	0.00	0.00	0.01
P11	CONDUIT	0.003	0	01:24	0.45	0.02	0.08
P12	CONDUIT	0.003	0	01:25	0.43	0.02	0.08
P13	CONDUIT	0.006	0	01:34	0.55	0.01	0.07
P2	CONDUIT	0.006	0	01:09	0.20	0.00	0.66
P3	CONDUIT	0.004	0	01:05	0.38	0.00	0.51
P4	CONDUIT	0.004	0	01:10	0.32	0.00	0.58
P5	CONDUIT	0.003	0	01:54	0.46	0.02	0.09
P6	CONDUIT	0.003	0	01:54	0.46	0.02	0.09
P7	CONDUIT	0.009	0	01:09	0.05	0.00	0.58
P8	CONDUIT	0.009	0	01:05	0.03	0.00	0.54
P9	CONDUIT	0.007	0	01:06	0.05	0.00	0.48
Orifice1	DUMMY	0.002	0	03:18			
Orifice2	DUMMY	0.003	0	01:54			
Orifice3	DUMMY	0.003	0	01:24			

****
Flow Classification Summary
****

Conduit	Adjusted /Actual Length	Fraction of Time in Flow Class: Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl
P1	1.00	0.48	0.07	0.00	0.43	0.00	0.00	0.02	0.57	0.00
P10	1.00	0.91	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00
P11	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
P12	1.00	0.53	0.26	0.00	0.05	0.00	0.00	0.16	0.83	0.00
P13	1.00	0.01	0.00	0.00	0.78	0.21	0.00	0.00	0.00	0.00
P2	1.00	0.44	0.00	0.00	0.55	0.00	0.00	0.00	0.47	0.00
P3	1.00	0.00	0.00	0.00	0.55	0.00	0.00	0.45	0.04	0.00
P4	1.00	0.00	0.00	0.00	0.57	0.00	0.00	0.43	0.01	0.00
P5	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
P6	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
P7	1.00	0.49	0.03	0.00	0.48	0.00	0.00	0.00	0.50	0.00
P8	1.00	0.45	0.01	0.00	0.54	0.00	0.00	0.00	0.46	0.00
P9	1.00	0.83	0.03	0.00	0.14	0.00	0.00	0.00	0.85	0.00

****
Conduit Surcharge Summary
****

No conduits were surcharged.

Analysis begun on: Wed Apr 01 08:37:07 2020
Analysis ended on: Wed Apr 01 08:37:07 2020
Total elapsed time: < 1 sec

EPA STORM WATER MANAGEMENT MODEL - VERSION 5.1 (Build 5.1.012)

****
Element Count
****

Number of rain gages 18
Number of subcatchments ... 10
Number of nodes 17
Number of links 16
Number of pollutants 0
Number of land uses 0

****
Raingage Summary
****

Name	Data Source	Data Type	Recording Interval
Chicago_3h_100Y	Chicago_3h_100Y	INTENSITY	10 min.
Chicago_3h_100Y_1.25	Chicago_3h_100Y_1.25	INTENSITY	10 min.
Chicago_3h_10Y	Chicago_3h_10Y	INTENSITY	10 min.
Chicago_3h_25Y	Chicago_3h_25Y	INTENSITY	10 min.
Chicago_3h_25Y_1.1	Chicago_3h_25Y_1.1	INTENSITY	10 min.
Chicago_3h_2Y	Chicago_3h_2Y	INTENSITY	10 min.
Chicago_3h_50Y	Chicago_3h_50Y	INTENSITY	10 min.
Chicago_3h_50Y_1.2	Chicago_3h_50Y-1.2	INTENSITY	10 min.
Chicago_3h_5Y	Chicago_3h_5Y	INTENSITY	10 min.
Chicago_6h_100Y	Chicago_6h_100Y	INTENSITY	10 min.
Chicago_6h_100Y_1.25	Chicago_6h_100Y_1.25	INTENSITY	10 min.
Chicago_6h_10Y	Chicago_6h_10Y	INTENSITY	10 min.
Chicago_6h_25Y	Chicago_6h_25Y	INTENSITY	10 min.
Chicago_6h_25Y_1.1	Chicago_6h_25Y_1.1	INTENSITY	10 min.
Chicago_6h_2Y	Chicago_6h_2Y	INTENSITY	10 min.
Chicago_6h_50Y	Chicago_6h_50Y	INTENSITY	10 min.
Chicago_6h_50Y_1.2	Chicago_6h_50Y_1.2	INTENSITY	10 min.
Chicago_6h_5Y	Chicago_6h_5Y	INTENSITY	10 min.

****
Subcatchment Summary
****

Name	Area	Width	%Imperv	%Slope	Rain Gage	Outlet
A1	0.06	25.00	87.00	1.2200	Chicago_3h_100Y_1.25	CBMH01
A2	0.11	30.29	85.00	0.5500	Chicago_3h_100Y_1.25	CBMH02
A3	0.03	13.81	80.00	0.5000	Chicago_3h_100Y_1.25	CBMH06
A4	0.03	50.00	76.00	2.4300	Chicago_3h_100Y_1.25	CBMH13
A5	0.04	31.82	66.00	1.7500	Chicago_3h_100Y_1.25	CBMH03
A6-1	0.01	3.89	36.00	7.0000	Chicago_3h_100Y_1.25	STM106
A6-2	0.02	6.76	35.00	7.0000	Chicago_3h_100Y_1.25	CBMH03
A7	0.00	5.71	90.00	1.5100	Chicago_3h_100Y_1.25	STM106
A8	0.03	11.74	52.00	1.0350	Chicago_3h_100Y_1.25	CBMH06
A9	0.01	8.00	90.00	0.1000	Chicago_3h_100Y_1.25	MH01

****
Node Summary
****

Name	Type	Invert Elev.	Max. Depth	Ponded Area	External Inflow

CBMH01	JUNCTION	102.80	2.30	67.5
CBMH02	JUNCTION	102.68	2.42	98.8
CBMH02-N	JUNCTION	102.68	2.42	0.0
CBMH03	JUNCTION	102.60	2.50	40.7
CBMH04	JUNCTION	102.43	2.76	59.6
CBMH06	JUNCTION	102.47	2.70	0.0
CBMH06-N	JUNCTION	102.47	2.70	0.0
CBMH07	JUNCTION	102.55	2.98	0.0
CBMH13	JUNCTION	103.01	2.26	0.0
Connect	JUNCTION	102.55	2.95	0.0
MH01	JUNCTION	102.53	2.72	0.0
MH01-N	JUNCTION	102.53	2.74	0.0
MH09	JUNCTION	102.90	2.60	0.0
MH108	JUNCTION	102.40	2.81	0.0
MH11	JUNCTION	102.79	2.66	0.0
OGS	JUNCTION	102.44	2.84	0.0
STM106	OUTFALL	102.31	0.68	0.0

****
Link Summary
****

Name	From Node	To Node	Type	Length	%Slope	Roughness
P1	CBMH13	CBMH01	CONDUIT	14.7	1.0028	0.0130
P10	Connect	CBMH04	CONDUIT	28.3	0.3008	0.0130
P11	CBMH06-N	CBMH04	CONDUIT	2.5	0.3137	0.0130
P12	CBMH04	MH108	CONDUIT	5.3	0.3042	0.0130
P13	MH108	STM106	CONDUIT	19.1	0.5084	0.0130
P2	CBMH01	CBMH02	CONDUIT	22.6	0.3496	0.0130
P3	CBMH02-N	CBMH03	CONDUIT	17.3	0.3472	0.0130
P4	CBMH03	MH01	CONDUIT	13.1	0.3498	0.0130
P5	MH01-N	OGS	CONDUIT	4.0	0.3500	0.0130
P6	OGS	MH108	CONDUIT	4.3	0.3529	0.0130
P7	MH09	CBMH01	CONDUIT	21.0	0.3476	0.0130
P8	MH11	CBMH02	CONDUIT	19.0	0.3474	0.0130
P9	CBMH07	CBMH06	CONDUIT	26.0	0.3000	0.0130
Orifice1	CBMH02	CBMH02-N	OUTLET			
Orifice2	MH01	MH01-N	OUTLET			
Orifice3	CBMH06	CBMH06-N	OUTLET			

****
Cross Section Summary
****

Conduit	Shape	Full Depth	Full Area	Hyd. Rad.	Max. Width	No. of Barrels	Full Flow
P1	CIRCULAR	1.20	1.13	0.30	1.20	1	3.90
P10	CIRCULAR	0.45	0.16	0.11	0.45	1	0.16
P11	CIRCULAR	0.45	0.16	0.11	0.45	1	0.16
P12	CIRCULAR	0.45	0.16	0.11	0.45	1	0.16
P13	CIRCULAR	0.68	0.36	0.17	0.68	1	0.60
P2	CIRCULAR	1.20	1.13	0.30	1.20	1	2.31
P3	CIRCULAR	1.20	1.13	0.30	1.20	1	2.30
P4	CIRCULAR	1.20	1.13	0.30	1.20	1	2.31
P5	CIRCULAR	0.45	0.16	0.11	0.45	1	0.17
P6	CIRCULAR	0.45	0.16	0.11	0.45	1	0.17
P7	CIRCULAR	1.20	1.13	0.30	1.20	1	2.30
P8	CIRCULAR	1.50	1.77	0.38	1.50	1	4.17
P9	CIRCULAR	1.05	0.87	0.26	1.05	1	1.50

```
*****************************************************
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 ............... CMS
Process Models:
  Rainfall/Runoff ........ YES
  RDII ................... NO
  Snowmelt ............... NO
  Groundwater ............ NO
  Flow Routing ........... YES
  Ponding Allowed ........ YES
  Water Quality .......... NO
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date ............ 02/28/2020 00:00:00
Ending Date .............. 02/29/2020 00:00:00
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 ........ 5.00 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
**************************     ---------       -------
Total Precipitation ......         0.030        89.635
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         0.001         2.232
Surface Runoff ...........         0.029        87.402
Final Storage ............         0.000         0.000
Continuity Error (%) .....         0.000


**************************        Volume        Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.029         0.291
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         0.048         0.479
Flooding Loss ............         0.000         0.000
Evaporation Loss .........         0.000         0.000
Exfiltration Loss ........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         0.013         0.134
Continuity Error (%) .....      -110.657


*************************
Highest Continuity Errors
```

****
Node MH01 (-84.58%)
Node CBMH01 (40.04%)
Node CBMH02-N (32.73%)
Node CBMH03 (-31.42%)
Node CBMH02 (-22.01%)

****
Time-Step Critical Elements
****
None

****
Highest Flow Instability Indexes
****
Link P4 (143)
Link P3 (140)
Link P8 (106)
Link P2 (103)
Link P1 (97)

****
Routing Time Step Summary
****

Minimum Time Step	:	4.50 sec
Average Time Step	:	5.00 sec
Maximum Time Step	:	5.00 sec
Percent in Steady State	:	0.00
Average Iterations per Step	:	2.11
Percent Not Converging	:	0.89

****
Subcatchment Runoff Summary
****

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Pea Runof CM
A1	89.63	0.00	0.00	1.17	88.46	0.05	0.0
A2	89.63	0.00	0.00	1.35	88.28	0.09	0.0
A3	89.63	0.00	0.00	1.80	87.83	0.03	0.0
A4	89.63	0.00	0.00	2.16	87.47	0.03	0.0
A5	89.63	0.00	0.00	3.06	86.57	0.03	0.0
A6-1	89.63	0.00	0.00	5.76	83.87	0.01	0.0
A6-2	89.63	0.00	0.00	5.85	83.78	0.02	0.0
A7	89.63	0.00	0.00	0.90	88.73	0.00	0.0
A8	89.63	0.00	0.00	4.32	85.31	0.02	0.0
A9	89.63	0.00	0.00	0.90	88.73	0.01	0.0

****
Node Depth Summary
****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters

CBMH01	JUNCTION	1.57	2.05	104.85	0	03:00	2.05
CBMH02	JUNCTION	1.69	2.17	104.85	0	03:00	2.17
CBMH02-N	JUNCTION	1.40	1.59	104.27	0	03:07	1.58
CBMH03	JUNCTION	1.48	1.67	104.27	0	03:07	1.64
CBMH04	JUNCTION	0.02	0.07	102.50	0	01:25	0.07
CBMH06	JUNCTION	0.18	1.92	104.39	0	01:24	1.92
CBMH06-N	JUNCTION	0.02	0.06	102.53	0	01:24	0.06
CBMH07	JUNCTION	0.17	1.84	104.39	0	01:24	1.84
CBMH13	JUNCTION	1.37	1.84	104.85	0	03:00	1.84
Connect	JUNCTION	0.00	0.00	102.55	0	00:00	0.00
MH01	JUNCTION	1.54	1.74	104.27	0	03:07	1.72
MH01-N	JUNCTION	0.05	0.06	102.58	0	01:53	0.06
MH09	JUNCTION	1.47	1.95	104.85	0	03:00	1.95
MH108	JUNCTION	0.05	0.06	102.46	0	01:28	0.06
MH11	JUNCTION	1.58	2.06	104.85	0	03:00	2.06
OGS	JUNCTION	0.05	0.06	102.50	0	01:53	0.06
STM106	OUTFALL	0.04	0.06	102.37	0	01:32	0.06

****
Node Inflow Summary
****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days	hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Fl Balan Err Perce
CBMH01	JUNCTION	0.037	0.058	0	01:06	0.0531	0.105	66.7
CBMH02	JUNCTION	0.066	0.066	0	01:06	0.0936	0.179	-18.0
CBMH02-N	JUNCTION	0.000	0.019	0	01:12	0	0.177	48.6
CBMH03	JUNCTION	0.036	0.036	0	01:05	0.0496	0.169	-23.9
CBMH04	JUNCTION	0.000	0.005	0	01:24	0	0.0539	0.0
CBMH06	JUNCTION	0.035	0.035	0	01:05	0.0485	0.0661	-3.6
CBMH06-N	JUNCTION	0.000	0.005	0	01:24	0	0.0539	0.0
CBMH07	JUNCTION	0.000	0.019	0	01:12	0	0.0147	-16.4
CBMH13	JUNCTION	0.019	0.021	0	01:14	0.0262	0.03	37.2
Connect	JUNCTION	0.000	0.000	0	00:00	0	0	0.0
MH01	JUNCTION	0.007	0.023	0	01:12	0.0106	0.225	-45.8
MH01-N	JUNCTION	0.000	0.005	0	01:53	0	0.416	0.0
MH09	JUNCTION	0.000	0.017	0	01:06	0	0.0216	163.1
MH108	JUNCTION	0.000	0.010	0	01:31	0	0.469	0.0
MH11	JUNCTION	0.000	0.038	0	09:29	0	0.0274	-43.4
OGS	JUNCTION	0.000	0.005	0	01:53	0	0.416	0.0
STM106	OUTFALL	0.007	0.014	0	01:10	0.00942	0.479	0.0

****
Node Surcharge Summary
****

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
CBMH01	JUNCTION	22.77	0.786	0.254
CBMH02	JUNCTION	22.74	0.624	0.254
CBMH02-N	JUNCTION	22.73	0.380	0.829
CBMH03	JUNCTION	22.75	0.439	0.830
CBMH06	JUNCTION	1.23	0.841	0.776

CBMH07	JUNCTION	1.13	0.763	1.136
CBMH13	JUNCTION	22.74	0.639	0.424
MH01	JUNCTION	22.77	0.511	0.989
MH09	JUNCTION	22.76	0.743	0.654
MH11	JUNCTION	22.73	0.558	0.604

****
Node Flooding Summary
****

No nodes were flooded.

****
Outfall Loading Summary
****

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
STM106	99.93	0.006	0.014	0.479
System	99.93	0.006	0.014	0.479

****
Link Flow Summary
****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days	hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
P1	CONDUIT	0.015	0	01:15	0.47	0.00	1.00
P10	CONDUIT	0.000	0	00:00	0.00	0.00	0.03
P11	CONDUIT	0.005	0	01:24	0.55	0.03	0.11
P12	CONDUIT	0.005	0	01:25	0.53	0.03	0.11
P13	CONDUIT	0.010	0	01:32	0.64	0.02	0.09
P2	CONDUIT	0.018	0	09:29	0.21	0.01	1.00
P3	CONDUIT	0.017	0	01:16	0.38	0.01	1.00
P4	CONDUIT	0.018	0	01:12	0.34	0.01	1.00
P5	CONDUIT	0.005	0	01:53	0.54	0.03	0.11
P6	CONDUIT	0.005	0	01:53	0.54	0.03	0.11
P7	CONDUIT	0.017	0	01:06	0.04	0.01	1.00
P8	CONDUIT	0.038	0	09:29	0.07	0.01	1.00
P9	CONDUIT	0.019	0	01:12	0.04	0.01	1.00
Orifice1	DUMMY	0.003	0	03:00			
Orifice2	DUMMY	0.005	0	01:53			
Orifice3	DUMMY	0.005	0	01:24			

****
Flow Classification Summary
****

Conduit	Adjusted /Actual Length	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl

(Columns Dry through Inlet Ctrl are grouped under: Fraction of Time in Flow Class)

P1	1.00	0.00	0.00	0.00	0.99	0.00	0.00	0.01	0.02	0.00
P10	1.00	0.83	0.17	0.00	0.00	0.00	0.00	0.00	0.00	0.00
P11	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
P12	1.00	0.00	0.00	0.00	0.79	0.00	0.00	0.21	0.06	0.00
P13	1.00	0.00	0.00	0.00	0.15	0.84	0.00	0.00	0.00	0.00
P2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.01	0.00
P3	1.00	0.00	0.00	0.00	0.99	0.00	0.00	0.00	0.01	0.00
P4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00
P5	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
P6	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
P7	1.00	0.01	0.01	0.00	0.98	0.00	0.00	0.00	0.00	0.00
P8	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00
P9	1.00	0.76	0.01	0.00	0.23	0.00	0.00	0.00	0.77	0.00

****
Conduit Surcharge Summary
****

Conduit	--------- Hours Full -------- Both Ends	Upstream	Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
P1	22.74	22.74	22.77	0.01	0.01
P2	22.79	22.79	22.81	0.01	0.01
P3	22.73	22.73	22.75	0.01	2.29
P4	22.76	22.76	22.77	0.01	2.96
P7	22.76	22.76	22.78	0.01	0.01
P8	22.73	22.73	22.74	0.01	0.01
P9	1.13	1.13	1.23	0.01	0.01

Analysis begun on: Wed Apr 01 08:49:17 2020
Analysis ended on: Wed Apr 01 08:49:18 2020
Total elapsed time: 00:00:01

Appendix E

Detailed OGS Sizing Report

ADS OGS Sizing Summary

Project Name: Shell, 5 Orchard Drive
Consulting Engineer: AECOM
Location: Ottawa, ON
Sizing Completed By: C. Neath **Email:** cody.neath@ads-pipe.com

Treatment Requirements		
Treatment Goal:	Enhanced (MOE)	
Selected Parameters:	80% TSS	90% Volume
Selected Unit:	**ADS FD-5HC**	

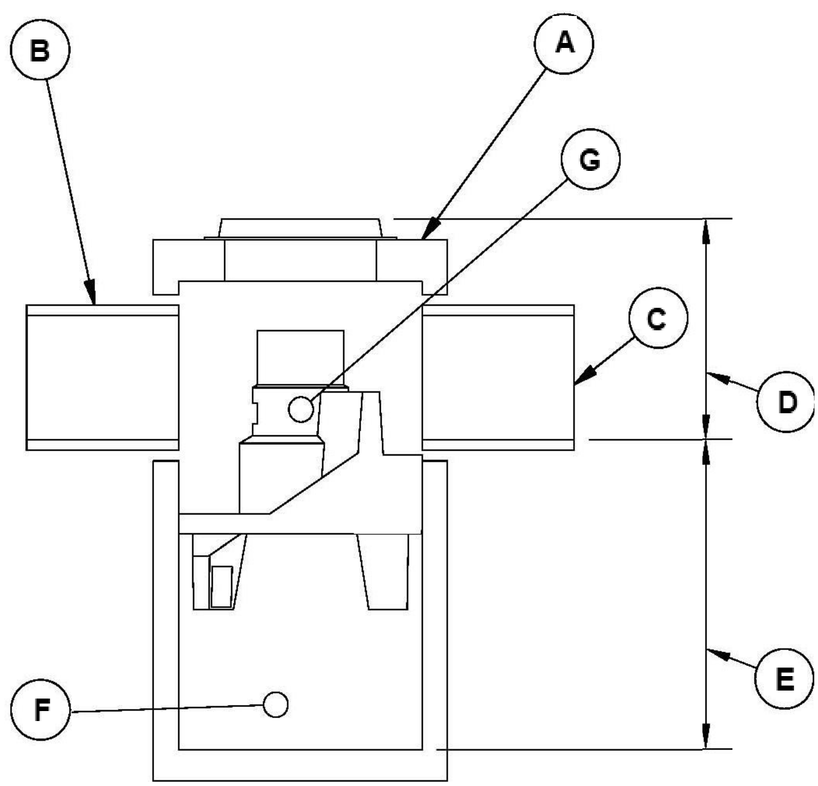
Site Details	
Site Area:	0.266 ha
% Impervious:	---
Rational C:	0.92
Rainfall Station:	Ottawa, ONT
Particle Size Distribution:	Fine
Peak Flowrate:	--- L/s

Summary of Results

Model	TSS Removal	Volume Treated
ADS FD-4HC	94.1%	99.9%
ADS FD-5HC	96.3%	99.9%
ADS FD-6HC	97.4%	99.9%
ADS FD-8HC	98.6%	99.9%

ADS FD-5HC Specification	
Unit Diameter (A):	1,500 mm
Inlet Pipe Diameter (B):	300 mm
Outlet Pipe Diameter (C):	450 mm
Height,T/G to Outlet Invert (D):	1200 mm
Height, Outlet Invert to Sump (E):	1,500 mm
Sediment Storage Capacity (F):	0.84 m³
Oil Storage Capacity (G):	1,135 L
Max. Pipe Diameter:	600 mm
Peak Flow Capacity:	566 L/s

Site Elevations:	
Rim Elevation:	100.00
Inlet Pipe Elevation:	98.80
Outlet Pipe Elevation:	98.80



Notes:

Removal efficiencies are based on NJDEP Test Protocols and independently verified.

All units supplied by ADS have numerous local, provincial, and international certifications (copies of which can be provided upon request). The design engineer is responsible for ensuring compliance with applicable regulations.

Project Name:	Shell, 5 Orchard Drive
Consulting Engineer:	AECOM
Location:	Ottawa, ON

Net Annual Removal Efficiency Summary: ADS FD-5HC

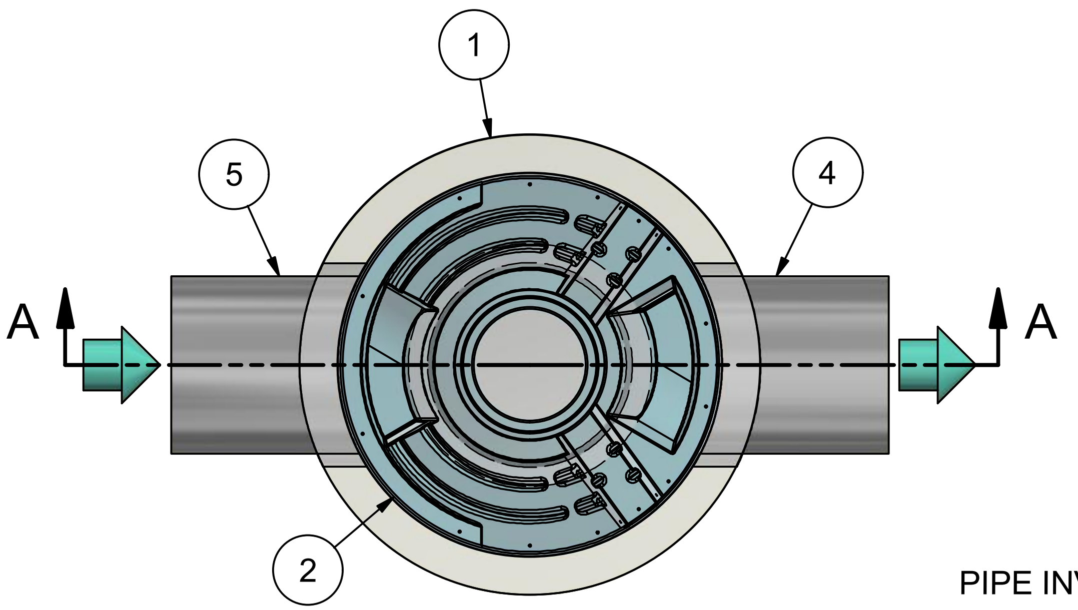
Rainfall Intensity[1]	Fraction of Rainfall[1]	ADS FD-5HC Removal Efficiency[2]	Weighted Net-Annual Removal Efficiency
mm/hr	%	%	%
0.50	0.1%	100.0%	0.1%
1.00	14.1%	100.0%	14.1%
1.50	14.2%	100.0%	14.2%
2.00	14.1%	100.0%	14.1%
2.50	4.2%	100.0%	4.2%
3.00	1.5%	100.0%	1.5%
3.50	8.5%	100.0%	8.5%
4.00	5.4%	99.2%	5.4%
4.50	1.2%	98.1%	1.1%
5.00	5.5%	97.2%	5.4%
6.00	4.3%	95.5%	4.1%
7.00	4.5%	94.2%	4.3%
8.00	3.1%	93.0%	2.9%
9.00	2.3%	92.0%	2.1%
10.00	2.6%	91.1%	2.3%
20.00	9.2%	85.4%	7.9%
30.00	2.6%	82.3%	2.2%
40.00	1.2%	80.1%	0.9%
50.00	0.5%	78.4%	0.4%
100.00	0.7%	73.6%	0.5%
150.00	0.1%	70.8%	0.0%
200.00	0.0%	69.0%	0.0%
		Total Net Annual Removal Efficiency:	96.3%
		Total Runoff Volume Treated:	99.9%

Notes:

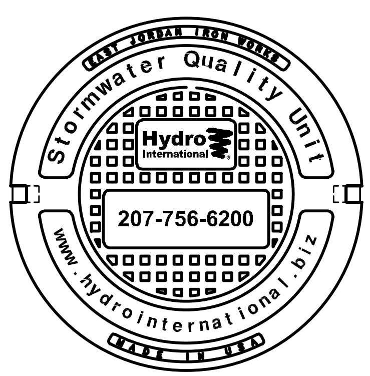
(1) Rainfall Data: 1960:2007, HLY03, Ottawa, ONT, 6105976 & 6105978.

(2) Based on third party verified data and appoximating the removal of a PSD similar to the STC Fine distribution

(3) Rainfall adjusted to 5 min peak intensity based on hourly average.

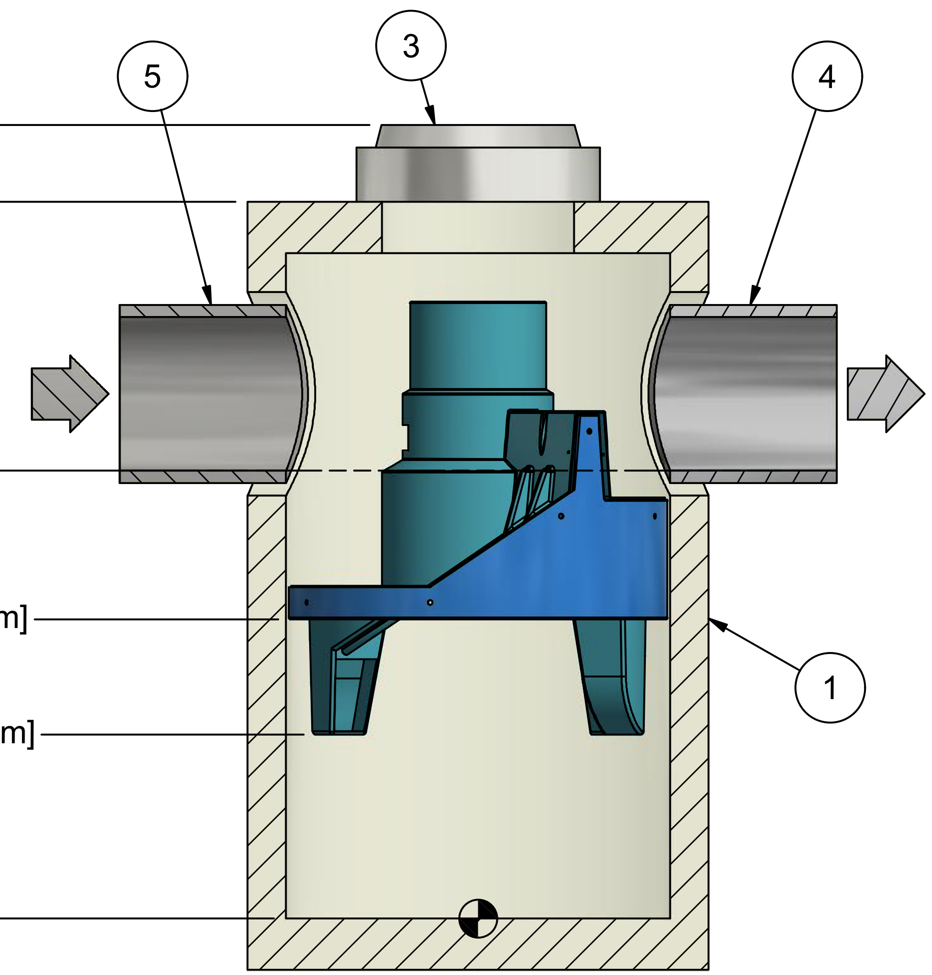


PLAN VIEW



HYDRO FRAME AND COVER (INCLUDED)

GRADE RINGS BY OTHERS AS REQUIRED



SECTION A-A

PRODUCT SPECIFICATION:

1. PEAK HYDRAULIC FLOW: 20.0 cfs (566 l/s)
2. MIN SEDIMENT STORAGE CAPACITY: 1.1 cu. yd. (0.8 cu. m.)
3. OIL STORAGE CAPACITY: 300 gal. (1135 liters)
4. MAXIMUM INLET/OUTLET PIPE DIAMETERS: 24 in. (600 mm)
5. THE TREATMENT SYSTEM SHALL USE AN INDUCED VORTEX TO SEPARATE POLLUTANTS FROM STORMWATER RUNOFF.
6. OK110 110 MICRON 80% TSS REMOVAL 2.34 cfs (66.2 l/s) (NJDEP TEST PROTOCOL)
7. NJDEP CERTIFIED 80% TSS REMOVAL AT 2.94 cfs (83.2 l/s)

GENERAL NOTES:
1. General Arrangement drawings only. Contact Hydro International for site specific drawings.
2. The diameter of the inlet and outlet pipes may be no more than 24".
3. Multiple inlet pipes possible (refer to project plan).
4. Inlet/outlet pipe angle can vary to align with drainage network (refer to project plan.s)
5. Peak flow rate and minimum height limited by available cover and pipe diameter.
6. Larger sediment storage capacity may be provided with a deeper sump depth.

PARTS LIST

ITEM	QTY	SIZE (in)	SIZE (mm)	DESCRIPTION
1	1	60	1500	I.D. PRECAST MANHOLE
2	1			INTERNAL COMPONENTS (PRE-INSTALLED)
3	1	30	750	FRAME AND COVER (ROUND)
4	1	24 (MAX)	600 (MAX)	OUTLET PIPE (BY OTHERS)
5	1	24 (MAX)	600 (MAX)	INLET PIPE (BY OTHERS)

DO NOT SCALE DRAWING

UNLESS OTHERWISE SPECIFIED, DIMENSIONS ARE IN INCHES. TOLERENCES ARE:

FRACTIONS ± 1/16
DECIMALS ± .06
ANGLES ± 1°

PROJECTION

COMMENTS:
1. MANHOLE WALL AND SLAB THICKNESSES ARE NOT TO SCALE.

2. CONTACT HYDRO INTERNATIONAL FOR A BOTTOM OF STRUCTURE ELEVATION PRIOR TO SETTING FIRST DEFENSE MANHOLE.

3. CONTRACTOR TO CONFIRM RIM, PIPE INVERTS, PIPE DIA. AND PIPE ORIENTATION PRIOR TO RELEASE OF UNIT TO FABRICATION.

DATE:	SCALE:	
6/11/2019	1:30	
DRAWN BY: ER	CHECKED BY: MRJ	APPROVED BY

Title
5-ft DIAMETER
FIRST DEFENSE HIGH CAPACITY

GENERAL ARRANGEMENT

Hydro International

94 Hutchins Drive
Portland, ME 04102
Tel: +1 (207) 756-6200
Fax: +1 (207) 756-6212
hydro-int.com

APPROX WEIGHT:	MATERIAL:	
N/A		
NEXT ASSEMBLY: -NEXT ASSY		
DRAWING NO.: -FDHC GA		
SHEET SIZE: B	SHEET: 1 OF 1	Rev: -

Nikfarjam, Toktam

From:	Haider Nasrullah <Haider.Nasrullah@ads-pipe.com>
Sent:	Thursday, April 2, 2020 8:14 AM
To:	Nikfarjam, Toktam; Cody Neath
Cc:	Michael Reid
Subject:	[EXTERNAL] RE: OGS Sizing, City of Ottawa

Hi Toktam,

The Particle Size Distribution (PSD) used to size the unit was the Fine PSD. Please see below.

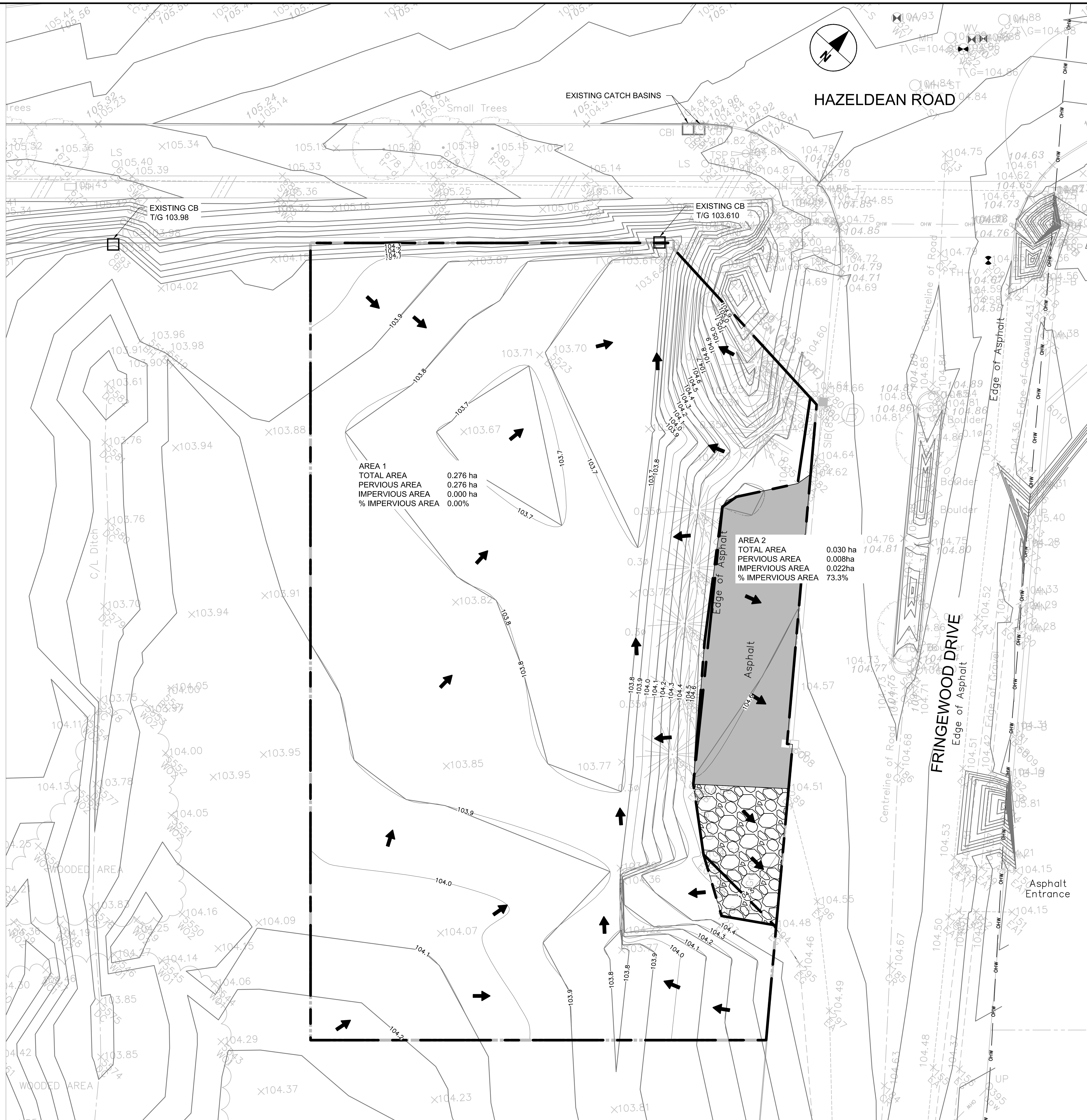
Particle Size (micron)	Distribution (%)
20	20
60	20
150	20
400	20
2000	20

Regards,

Haider Nasrullah, P.Eng.
Engineered Products Manager
ADS Canada

Appendix F

Existing Catchment Areas



LEGEND

OVERLAND FLOW	➡
EXISTING ASPHALT SURFACE	
EXISTING GRAVEL SURFACE	

0 5 10 m
1:200

AECOM

PROJECT

Shell Canada Products
Hazeldean Road and
Fringewood Drive NTI

5 Orchard Drive
Stittsville, Ontario

CLIENT

Shell Canada
400-4th Avenue SW
Calgary, AB T2P 0J4
403.252.4554 tel
www.shell.ca

CONSULTANT

AECOM Canada
4th Floor - 3292 Production Way
Burnaby, BC V5A 4R4
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www.aecom.com

REGISTRATION

LEGAL DESCRIPTION
PART OF BLOCK 21 OF DRAFT PLAN OF SUBDIVISION OF PARTS OF LOTS 26 AND 27
CONCESSION 11
GEOGRAPHIC TOWNSHIP OF GOULBOURN
(CITY OF OTTAWA)

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A	2020-03-31	ISSUED FOR REVIEW

DRAWN BY

AS

KEY PLAN

HAZELDEAN ROAD
FRINGEWOOD DRIVE
CANOPY
C-STORE
CAR WASH
N

GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

EXISTING STORMWATER DRAINAGE

AECOM FILE NAME

C131.0-SWM-HZLX

SHEET NUMBER

C131.0

ANSI D 864mm x 559mm
Project Management Initials: Designer: Checked: Approved:

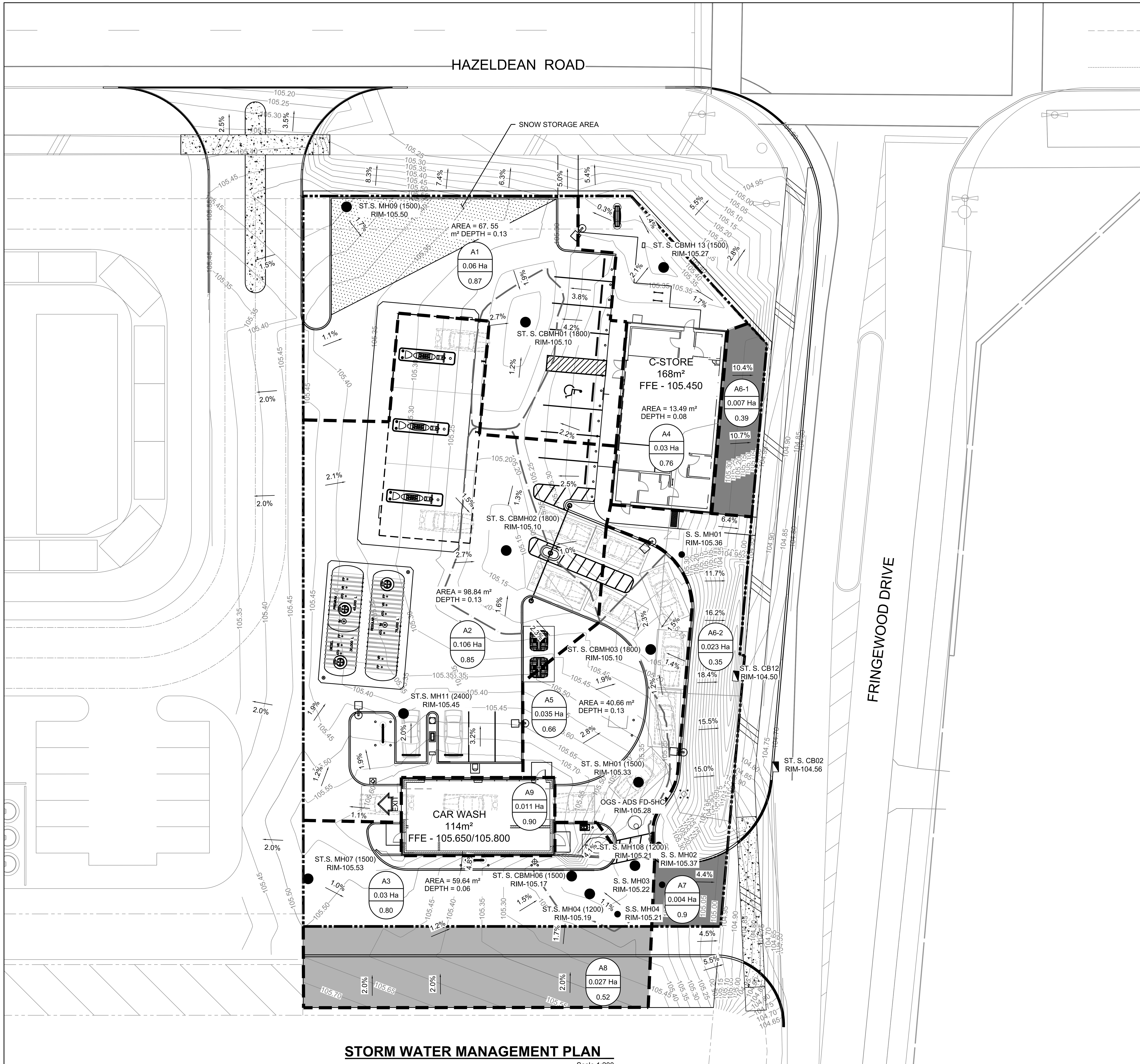
Last saved by: REIDJ2(2020-03-27) Last Plotted: 2020-03-27
Filename: C:\USERS\REIDJ2\DESKTOP\HAZELDEAN\20200326_CVL_SUBM_HZLX NTI\20200326_CVL_SUBM_HZLX NTI\03.CAD_(01)\HAZELDEAN_DP_2603\DP\ACAD-C131.0-SWM-HZLX.DWG

Printed on ___% Post-Consumer Recycled Content Paper

Appendix G

Stormwater Management Plan

Project Management Initials: Designer: ____ Checked: ____ Approved: ____ ANSI D 864mm x 559mm



STORM WATER MANAGEMENT PLAN

Scale 1:200

NOTES

1. ALL DIMENSIONS ARE IN METRES, UNLESS NOTED OTHERWISE.
2. FOR GENERAL NOTES SEE DRAWING C001.0.

LEGEND

	PROPOSED LEASE AND PROPERTY LINE
ST.S. CBMH / ST.S MH	PROPOSED STORMWATER / CATCH BASIN MANHOLE / MANHOLE
S.S. MH	PROPOSED SANITARY MANHOLE
ST.S. CB	PROPOSED STORMWATER CATCH BASIN
	PROPOSED OGS - ADS-FD-5HC
	PROPOSED SUB-CATCHMENT BOUNDARIES
A1	POST - DEVELOPMENT AREA ID
0.058 Ha	POST - DEVELOPMENT DRAINAGE AREA (Ha)
0.87	1:5 YEAR WEIGHTED RUNOFF COEFFICIENT
	UNCONTROLLED STORMWATER FLOW AREA
	EXTERNAL STORMWATER FLOW AREA

AREA STATEMENT (IN HECTARES)

Sub Catchment	PAVED AREA (ha)	LANDSCAP AREA (ha)	ROOF TOP AREA (ha)	TOTAL AREA (ha)
A1	0.057	0.003		0.06
A2	0.078	0.008	0.02	0.106
A3	0.025	0.005		0.03
A4	0.006	0.007	0.017	0.03
A5	0.021	0.014		0.035
A6-1	0.001	0.006		0.007
A6-2	0.002	0.021		0.023
A7	0.004			0.004
A8 (External Area)	0.01	0.017		0.027
A9			0.011	0.011

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1:200

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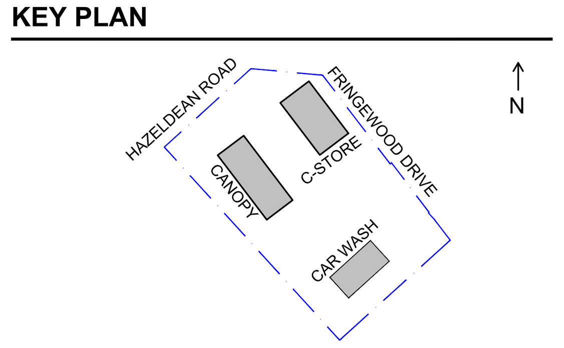
KEY PLAN



GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

STORMWATER MANAGEMENT PLAN

AECOM FILE NAME

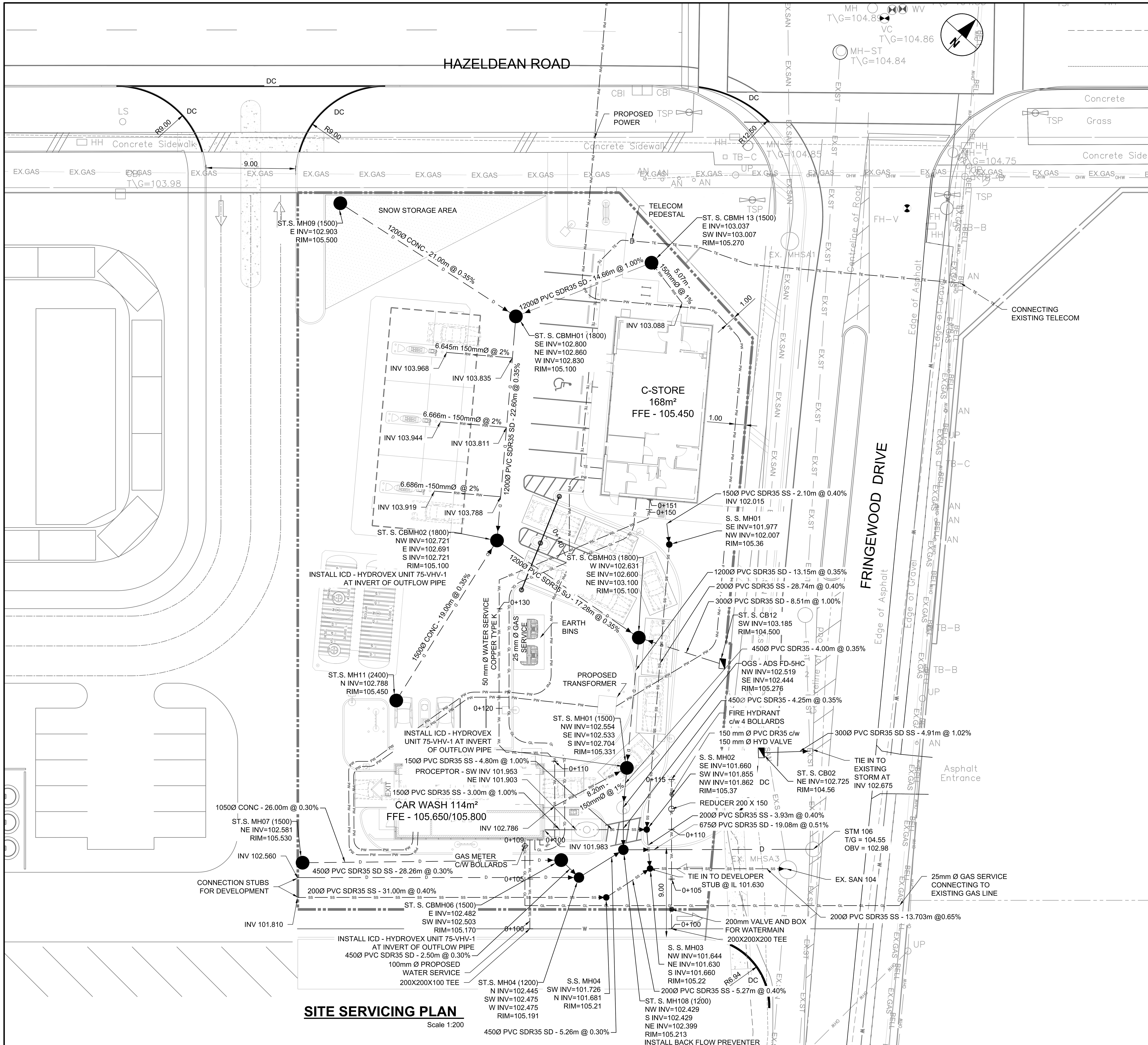
C105.0-SWM-HZLX

SHEET NUMBER

C105.0

Appendix H

Site Servicing Plan



SITE SERVICING PLAN

Scale 1:200

NOTES

1. FOR GENERAL NOTES SEE DRAWING C001.0

LEGEND

PROPOSED LEASE AND PROPERTY LINE	
PROPOSED STORMWATER / CATCH BASIN MANHOLE / MANHOLE	ST.S. CBMH / ST.S MH
PROPOSED SANITARY MANHOLE	S.S. MH
PROPOSED STORMWATER CATCH BASIN	ST.S. CB
PROPOSED OGS - ADS FD-5HC	
PROPOSED HYDRANT AND VALVE	HYD VB
PROPOSED LIGHT STANDARD	
SANITARY PROCEPTOR	
PROPOSED GAS LINE	GL GL
PROPOSED WATER LINE	WL
PROPOSED RAIN WATER COLLECTOR	RW
PROPOSED TELECOM	TE TE
PROPOSED SANITARY SEWER	SS
PROPOSED STORM SEWER	D
PROPOSED POWER LINE	PW PW
EXISTING STORM LINE	EX.ST
EXISTING SANITARY LINE	EX.SAN
EXISTING GAS LINE	EX.GAS
EXISTING TELECOMMUNICATION	BELL
EXISTING WATER LINE	W
GAS METER	
WATER VALVE	

PROPOSED 200mmØ WATERMAIN TABLE (TO FIRE HYDRANT)

STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100	105.156	102.000	CONNECTION 200 X 200 X 200 TEE
0+101.96	105.153	102.000	WATER VALVE WITH BOX
0+102.32	105.153	102.000	GAS LINE CROSSING
0+106.20	105.147	102.000	SANITARY LINE CROSSING
0+107.93	105.144	102.000	STORM LINE CROSSING
0+112.03	105.307	102.000	REDUCER 200 X 150
0+113.47	105.315	102.000	HYDRANT VALVE
0+115.00	105.309	102.000	FIRE HYDRANT

PROPOSED 100mmØ WATERMAIN TABLE (TO CAR WASH)

STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100.00	105.300	102.000	200 X 200 X100 TEE
0+102.290	105.239	102.000	GAS LINE CROSSING
0+103.150	105.218	102.000	SANITARY LINE CROSSING
0+105.150	105.178	102.000	STORM LINE CROSSING
0+106.680	105.175	102.000	STORM LINE CROSSING
0+109.00	105.800	102.000	CONNECTION TO CAR WASH

PROPOSED 50mmØ WATERMAIN TABLE (CAR WASH TO C-STORE)

STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100	105.800	102.000	CONNECTION FROM CAR WASH
0+100.50	105.532	102.000	90° HORIZONTAL BEND
0+101.29	105.535	102.000	45° HORIZONTAL BEND
0+102.29	105.552	102.000	45° HORIZONTAL BEND
0+103.08	105.590	102.000	SANITARY LINE CROSSING
0+110.10	105.620	102.000	45° HORIZONTAL BEND
0+111.11	105.609	102.000	45° HORIZONTAL BEND
0+115.30	105.629	102.000	45° HORIZONTAL BEND
0+116.310	105.465	102.000	45° HORIZONTAL BEND
0+121.210	105.343	102.000	POWER LINE CROSSING
0+132.91	105.158	102.000	45° HORIZONTAL BEND
0+136.750	105.197	102.000	STORM LINE CROSSING
0+140.650	105.252	102.000	POWER LINE CROSSING
0+147.620	105.382	102.000	GAS LINE CROSSING
0+149.100	105.414	102.000	45° HORIZONTAL BEND
0+151.00	105.450	102.000	CONNECTION TO C-STORE

0 5 10 m
1:200

AECOM

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DRAWN BY
SG

KEY PLAN

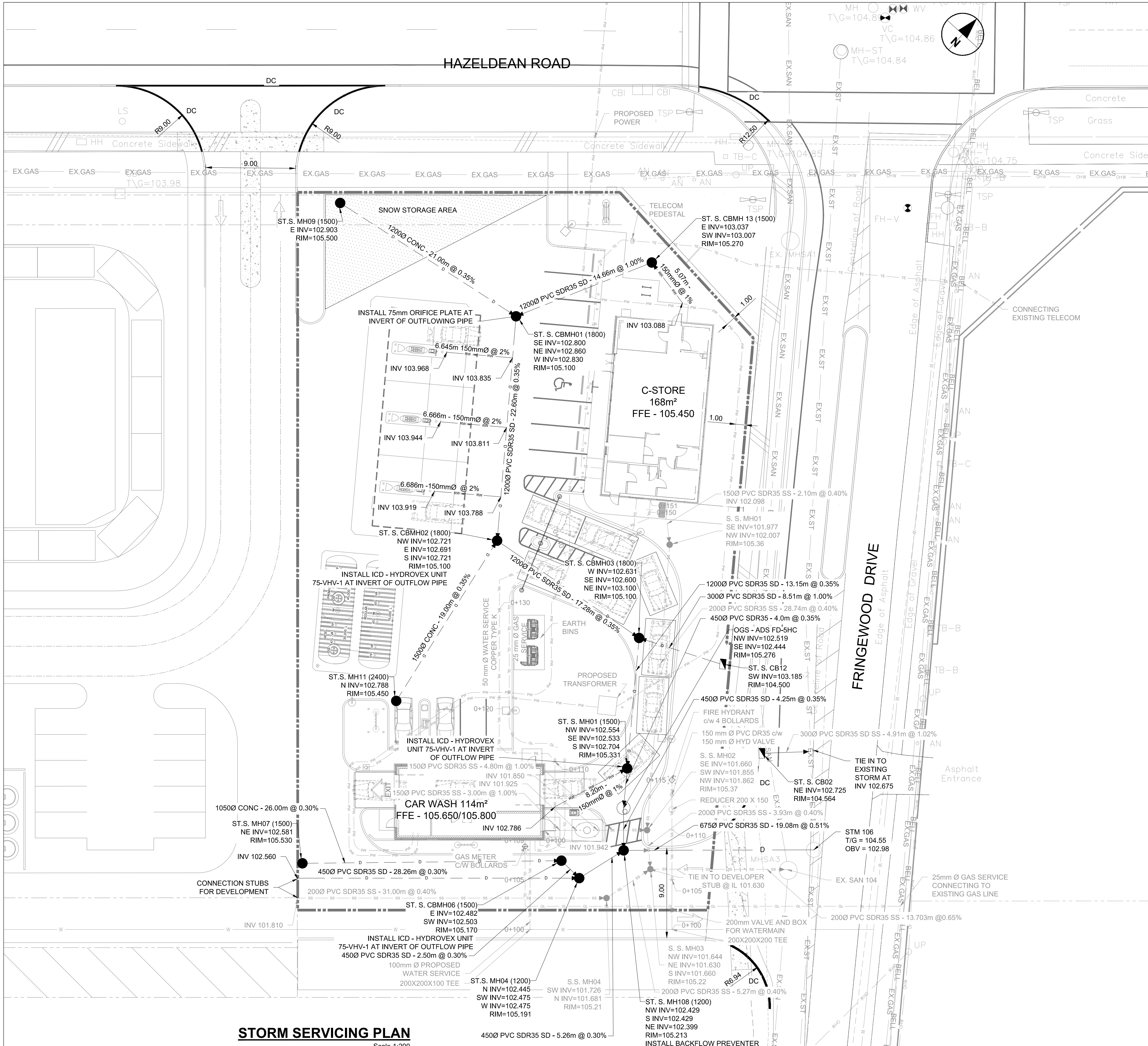
GLOBAL PROJECT ID NUMBER
CAN01444

SHEET TITLE
SITE SERVICING PLAN

AECOM FILE NAME
C103.0-SSP-HZLX

SHEET NUMBER
C103.0

ANSI D 864mm x 559mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____
Last saved by: REIDJ2(2020-03-31) Last Plotted: 2020-03-31
Filename: C:\USERS\REIDJ2\DESKTOP\HAZELDEAN\20200326_CVL_SUBM_HZLX NTI\20200326_CVL_SUBM_HZLX NTI\03.CAD_(01)\HAZELDEAN_DP_2603\DP\C103.0-SSP-HZLX.DWG



STORM SERVICING PLAN

Scale 1:200

NOTES

1. FOR GENERAL NOTES SEE DRAWING C001.0

LEGEND

- PROPOSED LEASE AND PROPERTY LINE
- PROPOSED STORMWATER / CATCH BASIN MANHOLE / MANHOLE — ST.S. CBMH / ST.S MH
- PROPOSED SANITARY MANHOLE — S.S. MH
- PROPOSED STORMWATER CATCH BASIN — ST.S. CB
- PROPOSED OGS ADS FD-5HC
- PROPOSED HYDRANT AND VALVE — HYD, VB
- PROPOSED LIGHT STANDARD
- SANITARY PROCEPTOR
- PROPOSED GAS LINE — GL
- PROPOSED WATER LINE — WL
- PROPOSED RAIN WATER COLLECTOR — RW
- PROPOSED TELECOM — TE
- PROPOSED SANITARY SEWER — SS
- PROPOSED STORM SEWER — D
- PROPOSED POWER LINE — PW
- EXISTING STORM LINE — EX.ST
- EXISTING SANITARY LINE — EX.SAN
- EXISTING GAS LINE — EX.GAS
- EXISTING TELECOMMUNICATION — BELL
- EXISTING WATER LINE — W
- GAS METER
- WATER VALVE

PROPOSED 200mmØ WATERMAIN TABLE (TO FIRE HYDRANT)			
STATION	SURFACE ELEVATION	T/WM ELEVATION	COMMENTS
0+100	105.156	102.000	CONNECTION 200 X 200 X 200 TEE
0+101.96	105.153	102.000	WATER VALVE WITH BOX
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0+111.11	105.609	102.000	45° HORIZONTAL BEND
0+115.30	105.629	102.000	45° HORIZONTAL BEND
0+116.310	105.465	102.000	45° HORIZONTAL BEND
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0+149.100	105.414	102.000	45° HORIZONTAL BEND
0+151.00	105.450	102.000	CONNECTION TO C-STORE

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AECOM

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A	2020-03-31	ISSUED FOR SERVICING REPORT

DRAWN BY
SG

KEY PLAN

GLOBAL PROJECT ID NUMBER
CAN01444

SHEET TITLE
STORM SERVICING PLAN

AECOM FILE NAME
C803.0-SSP-HZLX

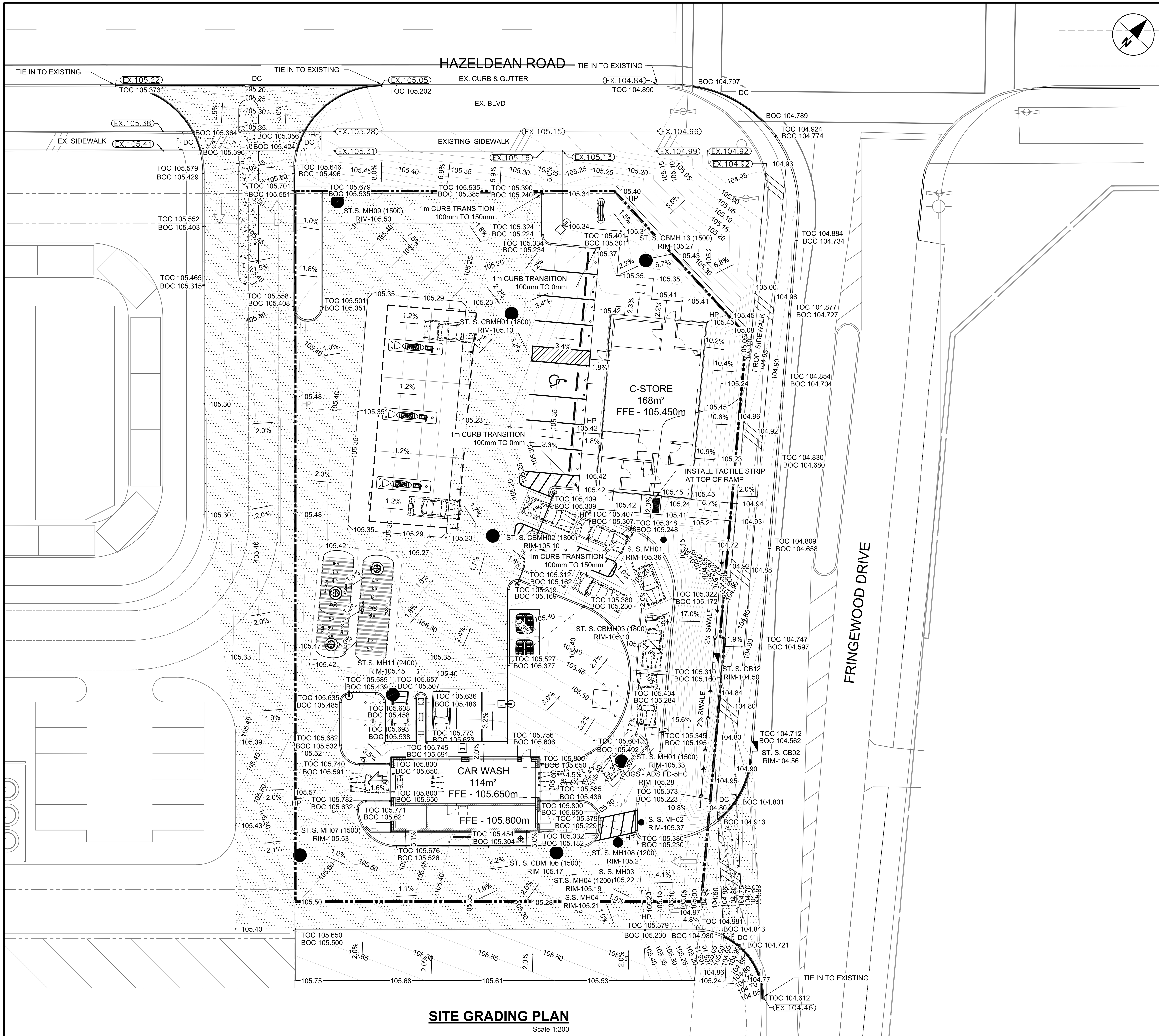
SHEET NUMBER
C803.0

ANSI D 864mm x 559mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____

Last saved by: REIDJ2(2020-03-31) Last Plotted: 2020-03-31
Filename: C:\USERS\REIDJ2\DESKTOP\HAZELDEAN\20200326_CVL_SUBM_HZLX NTI\20200326_CVL_SUBM_HZLX NTI\03 CAD_(01)HAZELDEAN_DP_2603\DP\C803.0-SSP-HZLX.DWG

Appendix I

Site Grading Plan



SITE GRADING PLAN

Scale 1:200

0 5 10 m
1:200

NOTES

1. FOR GENERAL NOTES SEE DRAWING C001.0
2. CONTRACTOR IS RESPONSIBLE TO CONFIRM EXISTING GRADES IN FIELD.

LEGEND

PROPOSED GRADE	657.000
TOP OF CURB PROPOSED GRADE	TOC 657.150
BOTTOM OF CURB PROPOSED GRADE	BOC 657.000
EXISTING SURFACE GRADE	EX.105.15

PAVEMENT STRUCTURES:

LIGHT DUTY (NEW PAVEMENT)
50mm HL3 or SUPERPAVE 19.0 ASPHALTIC CONCRETE
150mm GRANULAR "A" BASE CRUSHED STONE
300mm GRANULAR "B" TYPE II SUBBASE
ASPHALT GRADE PG-58-34
*INSTALLED PER GEOTECHNICAL REPORT

HEAVY DUTY (NEW PAVEMENT)
40mm HL3 or SUPERPAVE 12.5 ASPHALTIC CONCRETE
50mm HL8 or SUPERPAVE 19.0 ASPHALTIC CONCRETE
150mm GRANULAR "A" BASE CRUSHED STONE
450mm GRANULAR "B" TYPE II SUBBASE
ASPHALT GRADE PG 58-34
* INSTALLED PER GEOTECHNICAL REPORT

PROPOSED LEASE AND PROPERTY LINE

ST.S. CBMH / ST.S MH — PROPOSED STORMWATER / CATCH BASIN MANHOLE / MANHOLE

S.S. MH — PROPOSED SANITARY MANHOLE

ST.S. CB — PROPOSED STORMWATER CATCH BASIN

PROPOSED OGS - ADS FD-5HC

HYD — PROPOSED FIRE HYDRANT

PROPOSED LIGHT STANDARD

EXISTING HYDRANT

DC — PROPOSED DEPRESSED CURB (AS PER SC7.1)

AECOM

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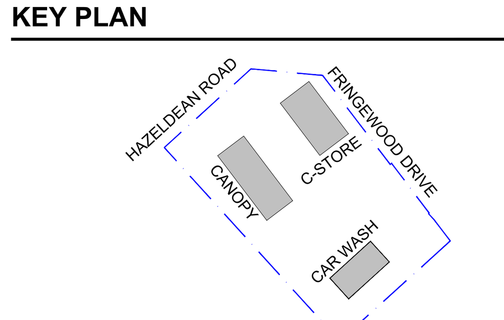

GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

SITE GRADING PLAN

AECOM FILE NAME

C104.0-SGP-HZLX

SHEET NUMBER

C104.0

ANSI D 864mm x 559mm

Approved: Checked: Designer: Project Management Initials:

Last saved by: REIDJ2(2020-03-31) Last Plotted: 2020-03-31
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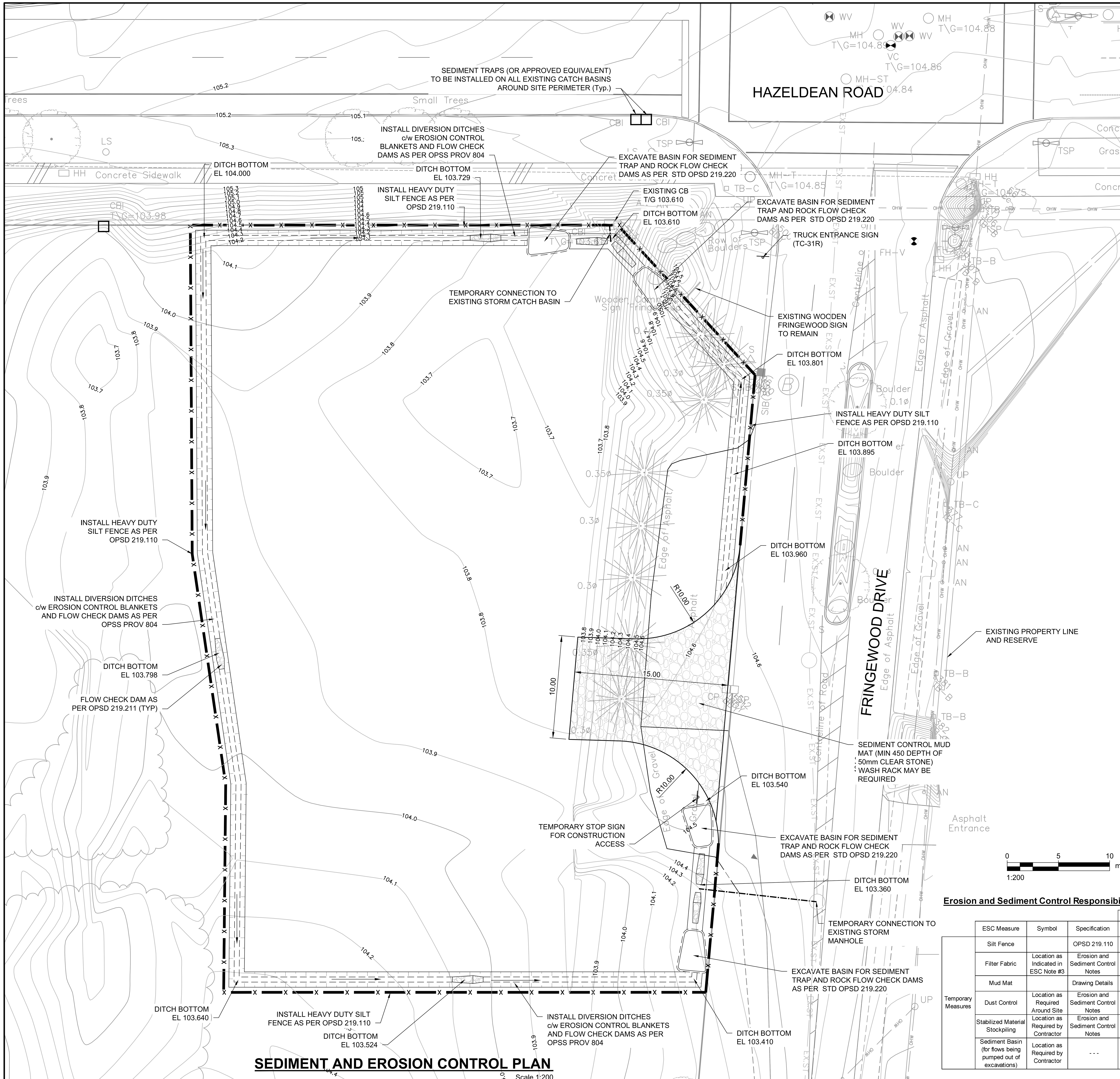
Appendix J

Sediment and Erosion Control Plan



SEDIMENT AND EROSION CONTROL PLAN

Scale 1:200

NOTES:

1. FOR GENERAL NOTES SEE DRAWING C001.0.
2. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE PROPER INSTALLATION, MAINTENANCE, AND REMOVAL OF ALL TEMPORARY EROSION AND SEDIMENT CONTROL MEASURES DURING CONSTRUCTION AND AS DIRECTED BY THE ENGINEER.
3. ADDITIONAL EROSION AND SEDIMENT CONTROL (ESC) MEASURES MAY BE REQUIRED AND SHALL BE DETERMINED BY THE ENGINEER.
4. SILT CONTROL FENCING SHALL BE INSTALLED ACCORDING TO THIS DRAWING AND MAINTAINED UNTIL COMPLETION OF THE LANDSCAPING AND SITE STABILIZATION.
5. NO CONSTRUCTION ACTIVITY OR MACHINERY SHALL INTRUDE BEYOND THE SILT/SNOW FENCE OR LIMIT OF DEVELOPMENT. ALL CONSTRUCTION VEHICLES SHALL LEAVE THE SITE AT DESIGNATED LOCATIONS AS SHOWN ON THE PLANS. ALL MATERIALS AND EQUIPMENT SHALL BE STORED ON SITE IN A DESIGNATED AREA. NO MATERIAL OR EQUIPMENT SHALL BE STORED ON THE MUNICIPAL RIGHT OF WAY. NO CONSTRUCTION VEHICLES WILL PARK ON THE MUNICIPAL ROADS.
6. STOCKPILES SHALL BE SET BACK FROM ANY WATERCOURSE AND STABILIZED AGAINST EROSION AS SOON AS POSSIBLE. A SETBACK OF AT LEAST 15m FROM ANY TOP OF BANK OR WATERCOURSE IS REQUIRED. ALL EXPOSED SOILS SHALL BE IMMEDIATELY STABILIZED WITH A SEED AND MULCH APPLICATION AS DIRECTED BY THE ENGINEER.
7. SERVICING OF CONSTRUCTION EQUIPMENT ON-SITE IS PROHIBITED.
8. CLEANING OF EXISTING ROAD(S) AT SITE ACCESS POINTS SHALL BE DONE DAILY DURING CONSTRUCTION OR AS NECESSARY THROUGH REGULAR INSPECTION OR AS DIRECTED BY THE ENGINEER.
9. DUST CONTROL TO BE REVIEWED DAILY. WATER TRUCK TO BE PROVIDED ON-SITE AND ALL HAUL ROAD / WORKING AREAS TO BE SPRAYED WITH WATER AS REQUIRED TO ENSURE DUST IS CONTROLLED ON-SITE.
10. ALL RE-GRADED AREAS WITHIN THE SITE WHICH ARE NOT OCCUPIED BY BUILDINGS, ROADWAYS, SIDEWALKS OR DRIVEWAYS SHALL BE TOP-SOILED AND SODDED / SEEDED IMMEDIATELY AFTER COMPLETION OF FINAL GRADING OPERATIONS OR AS DIRECTED BY THE ENGINEER.
11. SEDIMENT TRAPS (OR APPROVED EQUIVALENT) ARE TO BE INSTALLED AT ALL CATCHBASINS AND CATCHBASIN MANHOLE LOCATIONS UPON COMPLETION OF SERVICING.
12. THE ESC STRATEGIES ON THESE PLANS ARE NOT STATIC AND MAY NEED TO BE UPGRADED / AMENDED AS SITE CONDITION CHANGES TO PREVENT SEDIMENT RELEASE TO THE NATURAL ENVIRONMENT. FAILED ESC MEASURE MUST BE REPAIRED IMMEDIATELY.
13. MATERIALS TO REPAIR DAMAGED EROSION AND SEDIMENT CONTROL MEASURES MUST BE KEPT ON-SITE AT ALL TIMES.
14. INSPECTION OF THE PROPOSED EROSION AND SEDIMENT CONTROL MEASURES WILL OCCUR ON A WEEKLY BASIS, AFTER SIGNIFICANT RAINFALL OR SNOW MELT EVENTS AND DAILY DURING EXTENDED RAIN OR SNOW MELT PERIODS.
15. SEDIMENT / SILT SHALL BE REMOVED FROM THE SEDIMENT CONTROL DEVICE AND THE CATCHBASIN BUFFERS AFTER STORM EVENTS AND DISPOSED OF IN AREAS AS APPROVED BY THE ENGINEER.
16. ALL LITTER AND DEBRIS SHALL BE MONITORED AND DISPOSED OF DAILY OR AS NECESSARY THROUGH REGULAR INSPECTION.
17. ROCK CHECK DAMS ARE TO BE CLEANED OF ALL ACCUMULATED SEDIMENT AS SOON AS SEDIMENT HAS ACCUMULATED TO DEPTH GREATER THAN 50% OF THE UPSTREAM CHECK DAM.
18. THE SILT FENCE MUST BE INSPECTED WEEKLY AND IMMEDIATELY AFTER RAINFALL OR SIGNIFICANT SNOW MELT EVENTS FOR RIPS AND TEARS, BROKEN STAKES, BLOW OUTS (STRUCTURAL FAILURE) AND ACCUMULATION OF SEDIMENT. THE SILT FENCE MUST BE FIXED AND / OR REPLACED IMMEDIATELY WHEN DAMAGED. ACCUMULATED SEDIMENT MUST BE REMOVED FROM THE SILT FENCE WHEN ACCUMULATION REACHES 50% OF THE HEIGHT OF THE FENCE.

MUD MAT

STONE SIZE - USE CLEAR CRUSHED 50mm STONE.

THICKNESS - NOT LESS THAN 300mm

LENGTH - AS REQUIRED

WIDTH - 10m MINIMUM, BUT NOT LESS THAN THE WIDTH AT POINTS WHERE INGRESS OR EGRESS OCCURS.

FILTER CLOTH - NON-WOVEN GEOTEXTILE WILL BE PLACED OVER THE ENTIRE AREA PRIOR TO PLACING STONE.

MAINTENANCE - THE ENTRANCE SHALL BE MAINTAINED IN A CONDITION WHICH WILL PREVENT TRACKING OR FLOWING OF SEDIMENT ONTO PUBLIC RIGHT-OF-WAY. THIS MAY REQUIRE PERIODIC TOP DRESSING WITH ADDITIONAL STONE AS CONDITIONS DEMAND AND REPAIR AND/ OR CLEANOUT OF ANY MEASURES USED TO TRAP SEDIMENTS. ALL SEDIMENTS SPILLED, DROPPED, WASHED OR TRACKED ONTO PUBLIC RIGHT-OF-WAY MUST BE REMOVED IMMEDIATELY.

TIRE WASH STATION - WHEELS SHALL BE CLEANED TO REMOVE SEDIMENT PRIOR TO ENTRANCE ONTO PUBLIC RIGHT-OF-WAY. WHEN WASHING IS REQUIRED IT SHALL BE DONE ON A DESIGNATED AREA STABILIZED WITH STONE AND WHICH DRAINS INTO AN APPROVED SEDIMENT TRAPPING DEVICE.

INSPECTION AND REQUIRED - INSPECTION AND MAINTENANCE MAINTENANCE SHALL BE PROVIDED PERIODCALLY AND AFTER AFTER SIGNIFICANT RAINFALL AND SNOWMELT.

SEDIMENT CONTROL CONSTRUCTION SCHEDULE:

1. INSTALL PERIMETER ENVIRONMENTAL FENCE AND CONSTRUCTION VEHICLE ACCESS.
2. EXCAVATE PERIMETER SWALES AND INSTALL CHECK DAMS.
3. STRIP SITE OF TOPSOIL AND REMOVE OFF SITE.
4. INSTALL MINOR STORM SEWER SYSTEM ALONG WITH OTHER SERVICES.
5. INSTALL CATCH BASIN FILTRATION ON ALL CATCH BASINS AND CATCH BASIN MANHOLES.
6. SEDIMENT CONTROL MEASURES ARE TO BE MAINTAINED UNTIL ALL AREAS OF THE SITE HAVE BEEN STABILIZED WITH SOD OR ASPHALT.

LEGEND:

HEAVY DUTY SILT FENCE

MUD MAT CLEAR STONE

MUD MAT LIMESTONE

SEDIMENT TRAP

0 5 10 m
1:200

Erosion and Sediment Control Responsibilities:

				During Construction			After Construction Prior to Final Acceptance		After Final Acceptance
	ESC Measure	Symbol	Specification	Installation Responsibility	Inspection/Maintenance Responsibility	Inspection Frequency	Approval to Remove	Removal Responsibility	Inspection/Maintenance Responsibility
Temporary Measures	Silt Fence		OPSD 219.110	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Consultant	Developer's Contractor	N/A
	Filter Fabric	Location as Indicated in ESC Note #3	Erosion and Sediment Control Notes	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Consultant	Developer's Contractor	N/A
	Mud Mat		Drawing Details	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Developer's Contractor	Developer's Contractor	N/A
	Dust Control	Location as Required Around Site	Erosion and Sediment Control Notes	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Consultant	Developer's Contractor	N/A
	Stabilized Material Stockpiling	Location as Required by Contractor	Erosion and Sediment Control Notes	Developer's Contractor	Developer's Contractor	Weekly (as a minimum)	Developer's Contractor	Developer's Contractor	N/A
	Sediment Basin (for flows being pumped out of excavations)	Location as Required by Contractor	- - -	Developer's Contractor	Developer's Contractor	After Every Rainstorm	Developer's Contractor	Developer's Contractor	N/A

AECOM

PROJECT

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CONSULTANT

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REGISTRATION

LEGAL DESCRIPTION
PART OF BLOCK 21 OF DRAFT PLAN OF SUBDIVISION OF PARTS OF LOTS 26 AND 27
CONCESSION 11
GEOGRAPHIC TOWNSHIP OF GOULBOURN
(CITY OF OTTAWA)

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A	2020-03-31	ISSUED FOR SPA

DRAWN BY

SG

KEY PLAN

GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

SEDIMENT AND EROSION CONTROL PLAN

AECOM FILE NAME

C101.0-SDC-HZLX

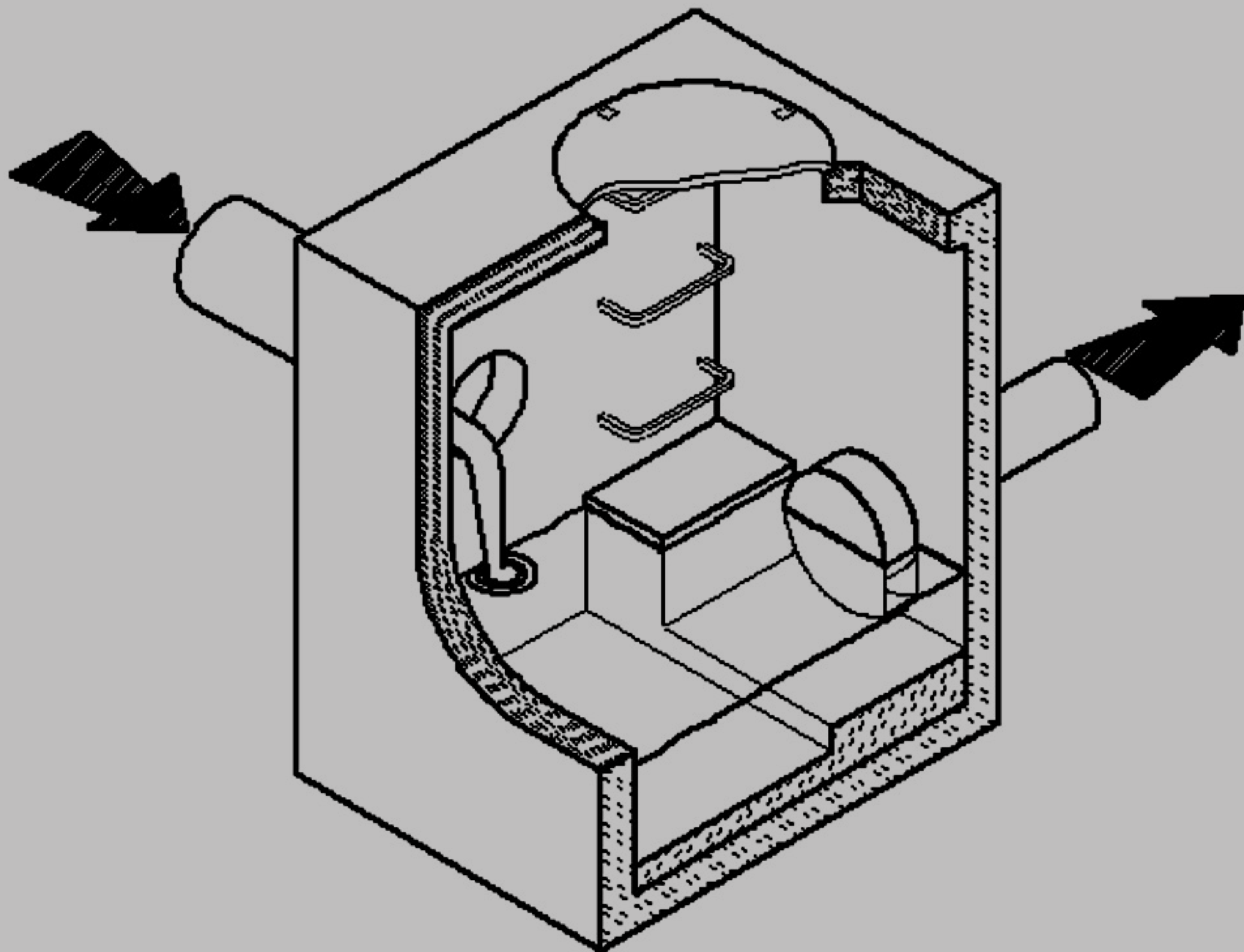
SHEET NUMBER

C101.0

ANSI D 864mm x 559mm
Approved: ____ Checked: ____ Designer: ____ Project Management Initials: ____

Last saved by: REIDJ2(2020-03-26) Last Plotted: 2020-03-31
Filename: C:\USERS\REIDJ2\DESKTOP\HAZELDEAN\20200326_CVL_SUBM_HZLX NTI\20200326_CVL_SUBM_HZLX NTI\03.CAD_(01)HAZELDEAN_DP_2603\DP\C101.0-SDC-HZLX.DWG

Appendix K
HYDROVEX ICD



VEOLIA

HYDROVEX® VHV/SVHV
Vertical Vortex Flow Regulator
CSO, SSO, Stormwater Management

WATER TECHNOLOGIES

HYDROVEX® VHV / SVHV Vertical Vortex Flow Regulator

Application

One of the major problems of urban wet weather flow management is the runoff generated by heavy rainfall. During a storm event, uncontrolled flows may overload the drainage system and cause flooding. Wear and deterioration on the network are increased dramatically as a result of increased flow velocities. In a combined sewer system, the wastewater treatment plant will experience a significant increase in flows during storms, thereby losing its treatment efficiency. A simple means of managing excessive storm water runoff is to control the flows at their point of origin, the manhole.

The HYDROVEX® VHV / SVHV line of vortex flow regulators is ideal for point source control of low to medium stormwater flows in manholes, catch basins and other retention structures. The HYDROVEX® VHV / SVHV design is based on the fluid mechanics principle of the forced vortex. The discharge is controlled by an air-filled vortex which reduces the effective water passage area without physically reducing orifice size. This effect grants precise flow regulation without the use of moving parts or electricity, and allows for larger inlet and outlet openings compared to the basic orifice. Although the concept is quite simple, many years of research and testing have been invested to optimize the performance of our vortex technology.

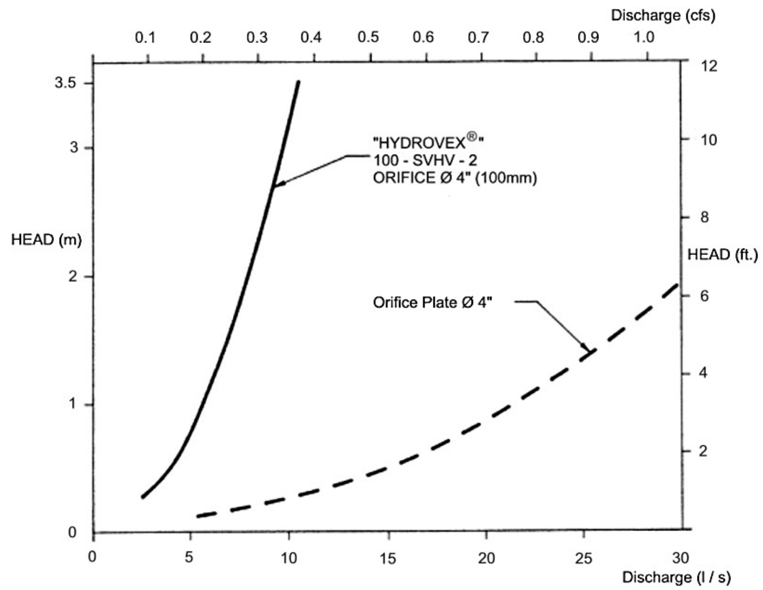


Figure 1:HYDROVEX® VHV/SVHV compared to an orifice plate

Vortex valves have openings typically 4 to 6 times larger than an orifice plate for the same design. Larger opening sizes decrease the chance of blockage caused by sediments and debris found in storm water flows. Figure 1 shows the discharge curve of a vortex regulator compared to an equally sized orifice plate. For an identical opening size, the flow is approximately four times smaller than the orifice plate for the same upstream water pressure.

Advantages

- Large inlet/outlet openings reduce the chance of clogging
- Openings typically 4-6 times larger than the basic orifice (Figure 1)
- Outlet orifice always equal or larger than inlet
- Ideal for precise control of low to medium stormwater flow applications
- Submerged inlet for floatables control
- No moving parts or electricity required
- Durable and robust stainless steel construction
- Minimal maintenance
- Easy to install

Selection

Selecting a VHV/SVHV regulator is easily achieved using Figure 3. Each selection is made using the maximum allowable flow rate and the maximum allowable upstream water pressure (head). The area in which the design point falls will designate the required model. The maximum design head is defined as the difference between the maximum upstream water level and the invert of the outlet pipe. All selections should be verified by a John Meunier Inc. representative prior to fabrication.

Design example:

- Maximum discharge: 6 L/s (0.2 cfs)*
- Maximum design head 2m (6.56 ft.)**
- Using Figure 3, model 75 VHV-1 is selected

**The selection chart provided assumes free flowing downstream conditions. Should the outlet pipe be >80% full at design flow, a larger pipe diameter should be used. In the above example, the minimum outlet pipe diameter and slope would be 150mm (6in), 0.3%.*
***The design head is defined as the difference between the maximum upstream water level and the outlet pipe invert.*

The HYDROVEX® VHV / SVHV vortex flow regulators can be installed in circular or square manholes. The table below lists the minimum dimensions and clearances required for each regulator model. It is imperative to respect the minimum clearances shown to ensure ease of installation and proper functioning of the regulator.

Model	Regulator Diameter A (mm) [in]	CIRCULAR Minimum Manhole Diameter B (mm) [in]	SQUARE Minimum Chamber Width B (mm) [in]	Minimum Outlet Pipe Diameter C (mm) [in]	Minimum Clearance H (mm) [in]
25 SVHV-1	125 [5]	600 [24]	600 [24]	150 [6]	150 [6]
32 SVHV-1	150 [6]	600 [24]	600 [24]	150 [6]	150 [6]
40 SVHV-1	200 [8]	600 [24]	600 [24]	150 [6]	150 [6]
50 VHV-1	150 [6]	600 [24]	600 [24]	150 [6]	150 [6]
75 VHV-1	250 [10]	600 [24]	600 [24]	150 [6]	150 [6]
100 VHV-1	325 [13]	900 [36]	600 [24]	150 [6]	200 [8]
125 VHV-2	275 [11]	900 [36]	600 [24]	150 [6]	200 [8]
150 VHV-2	350 [14]	900 [36]	600 [24]	150 [6]	225 [9]
200 VHV-2	450 [18]	1200 [48]	900 [36]	200 [8]	300 [12]
250 VHV-2	575 [23]	1200 [48]	900 [36]	250 [10]	350 [14]
300 VHV-2	675 [27]	1600 [64]	1200 [48]	250 [10]	400 [16]
350 VHV-2	800 [32]	1800 [72]	1200 [48]	300 [12]	500 [20]

Figure 2a: Minimum dimensions and clearances, circular manhole

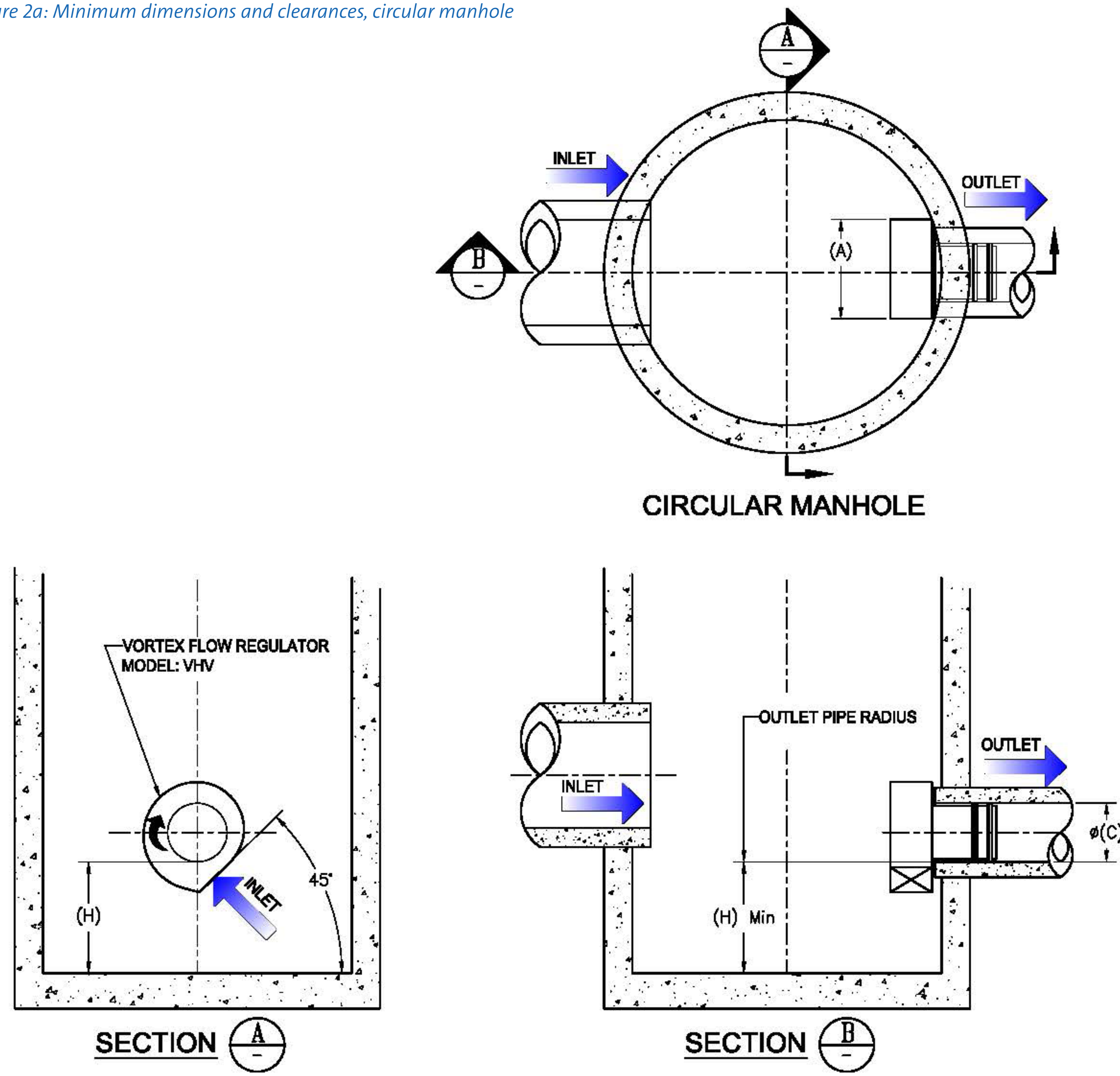
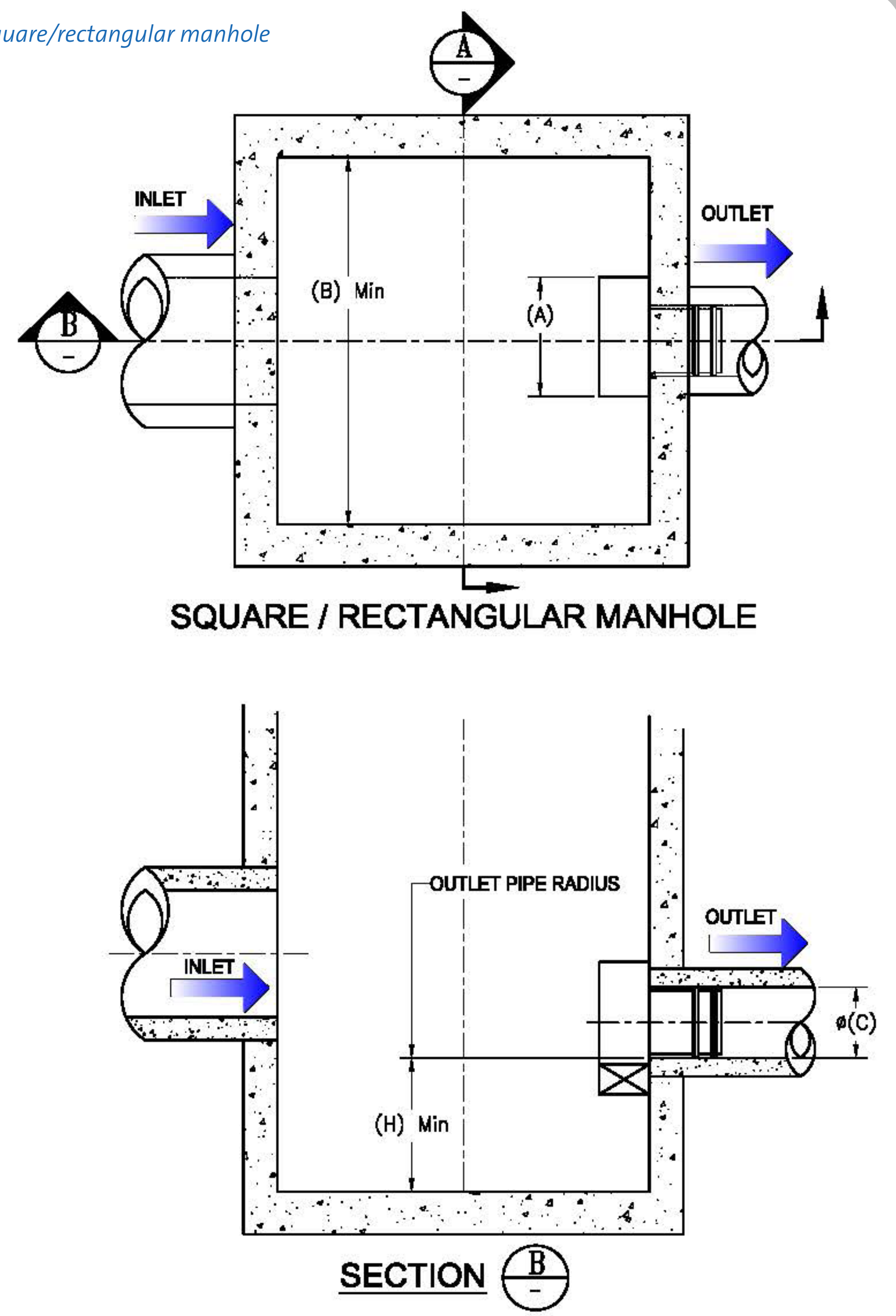


Figure 2b: Minimum dimensions and clearances, square/rectangular manhole



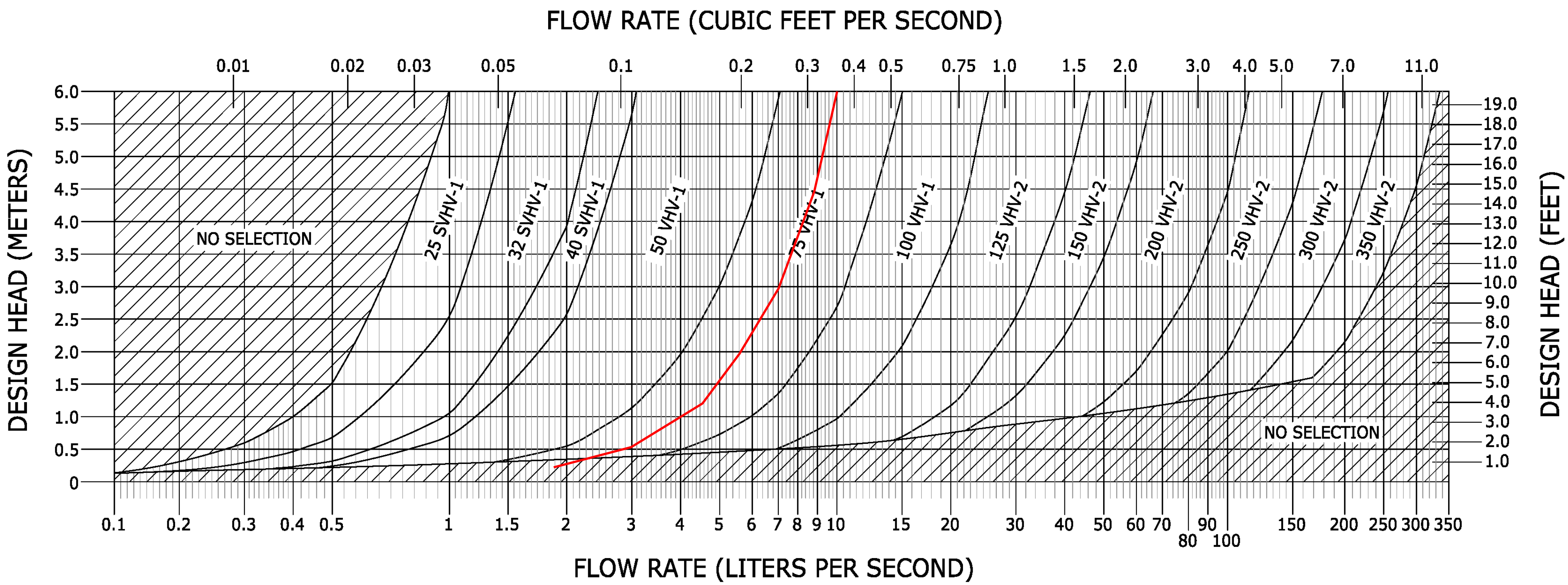


Figure 3 : HYDROVEX® VHV/SVHV Selection Chart

Options

A variety of options are available for the HYDROVEX® VHV / SVHV vortex flow regulators, including:

- Type O: extended inlet for odor control
- FV-VHV: sliding plate mounted
- Gooseneck: for shallow or no sump installations
- Vent: for low slope applications

DT: roof drainage applications

Specifications

In order to specify a HYDROVEX® VHV/SVHV flow regulator, the following parameters must be clearly indicated:

- Model number, ex: 75-VHV-1
- Outlet pipe diameter and type, ex: ø 150mm [6”], SDR 35
- Design discharge rate, ex: 6.0 L/s [0.21 CFS]
- Design head, ex: 2.0 m [6.56 ft] *
- Manhole diameter, ex: ø 900 mm [ø 36”]
- Minimum clearance “H”, ex: 150 mm [6 in]
- Construction material type (304 stainless steel standard)

**The design head is defined as the difference between the maximum upstream water level and the outlet pipe invert.*

Installation

The installation of a HYDROVEX® VHV/SVHV flow regulator can be accomplished quickly and does not require any special tools. The sleeve of the vortex flow regulator is simply inserted into the outlet pipe of the manhole and the unit is then secured to the concrete wall using the supplied anchor.

Maintenance

HYDROVEX® regulators are designed to minimize maintenance requirements. We recommend a periodic visual inspection in order to ensure that the unit is free of debris. The manhole sump beneath the unit should be inspected and cleaned with a vacuum truck periodically to remove accumulated sediments.

Guaranty

The HYDROVEX® line of VHV / SVHV regulators are guaranteed against both design and manufacturing defects for a period of 5 years after sale. The unit will be modified or replaced should it be found to be defective within the guarantee period.

Resourcing the world

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