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Shell Canada

Stormwater Management Report

5 Orchard Drive, Stittsville, City of Ottawa

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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:

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

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*

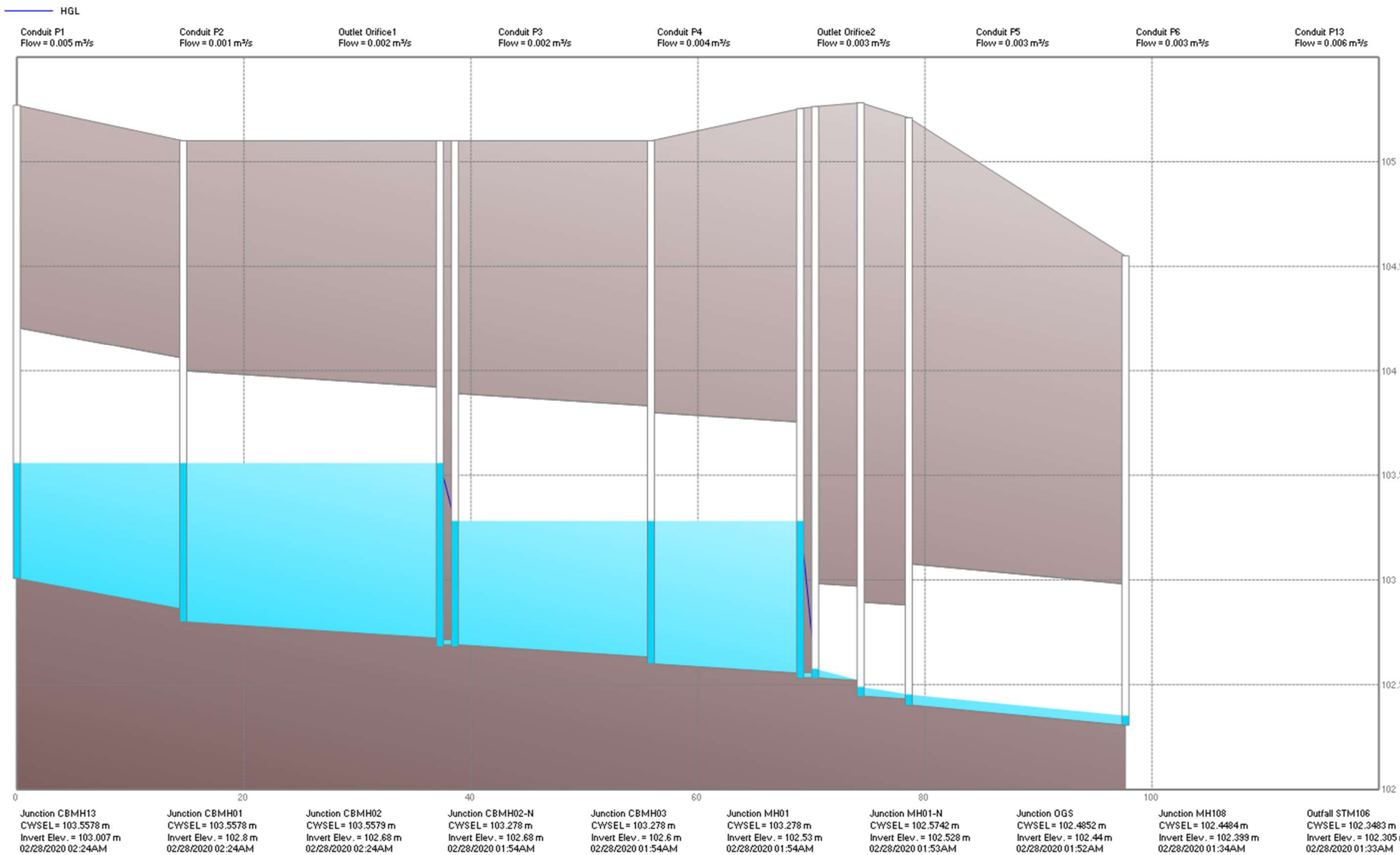

Figure 2: Proposed 5 - Year Storm HGL (From CBMH13 to Ex.STM106) – With SWM Control

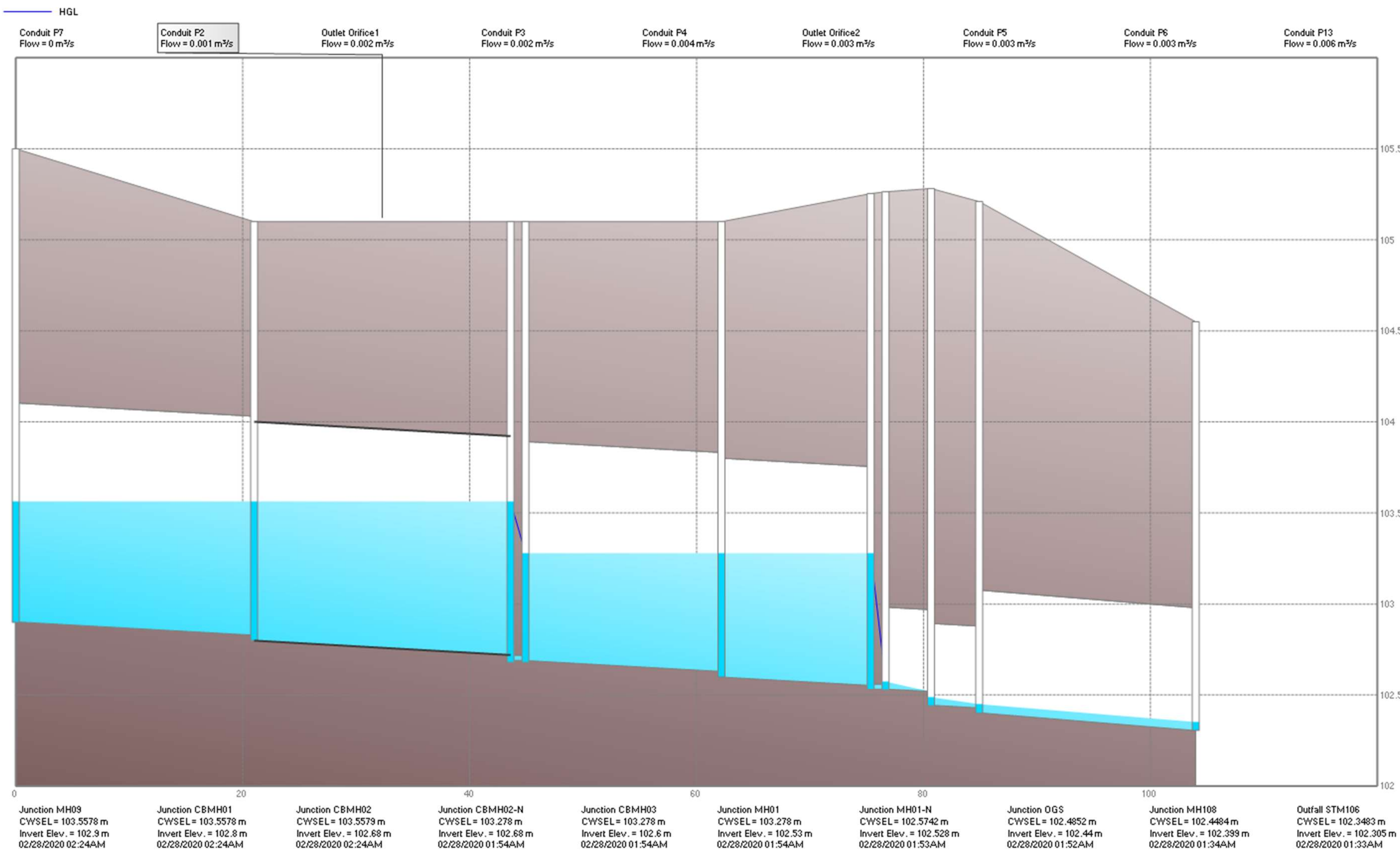

Figure 3: Proposed 5 - Year Storm HGL (From MH09 to Ex.STM106) – With SWM Control

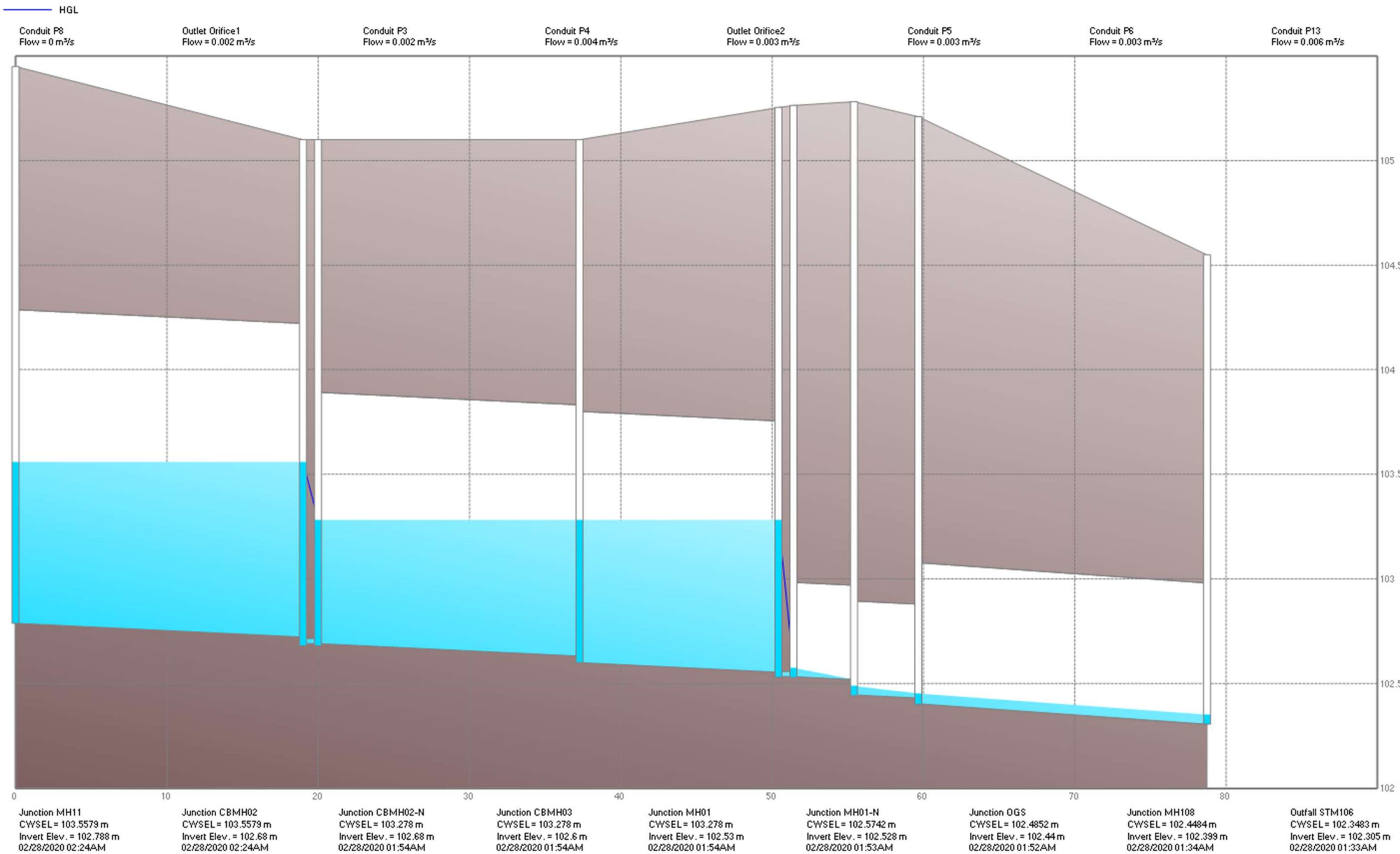

Figure 4: Proposed 5 - Year Storm HGL (From MH11 to Ex.STM106) – With SWM Control

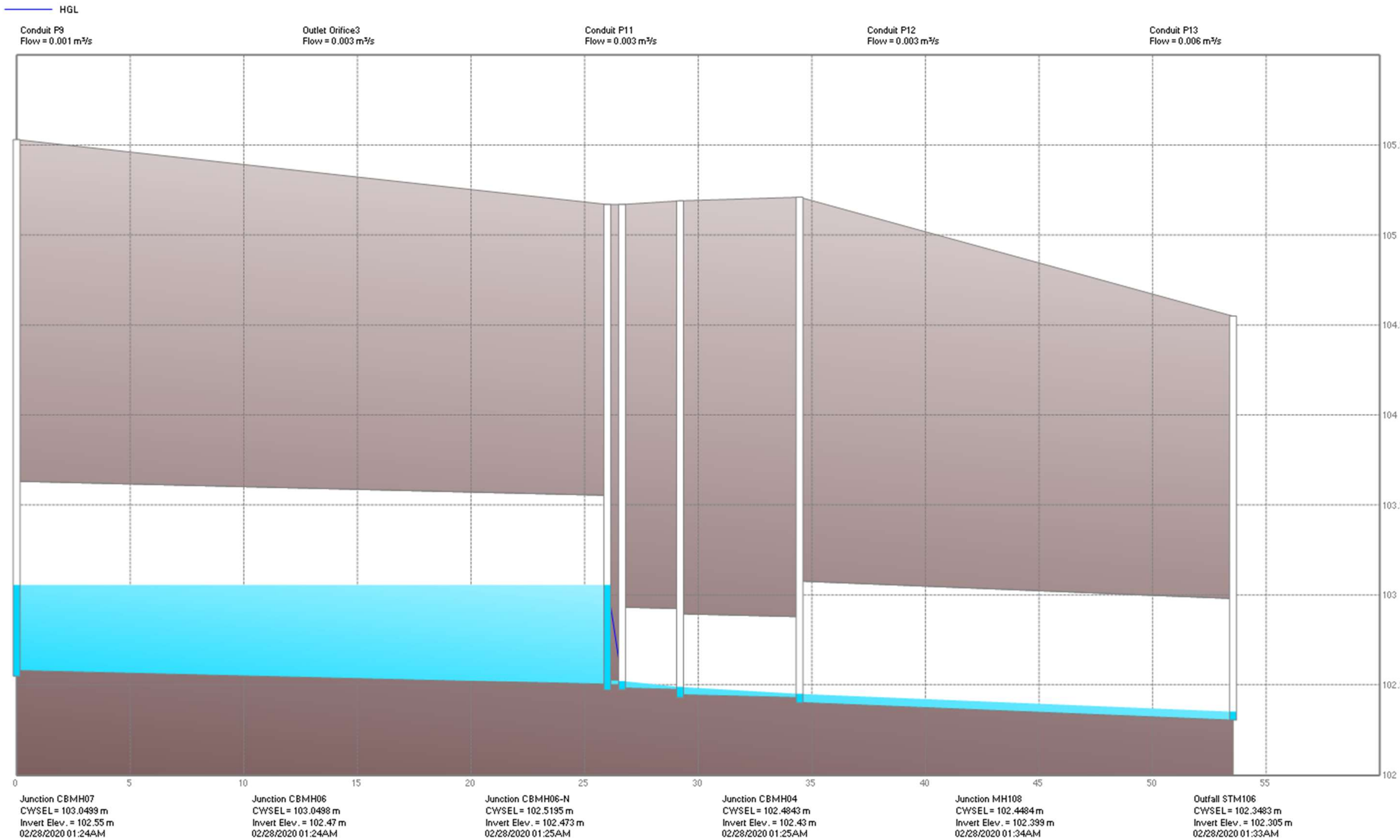

Figure 5: Proposed 5 - Year Storm HGL (From CBMH7 to Ex.STM106) – With SWM Control

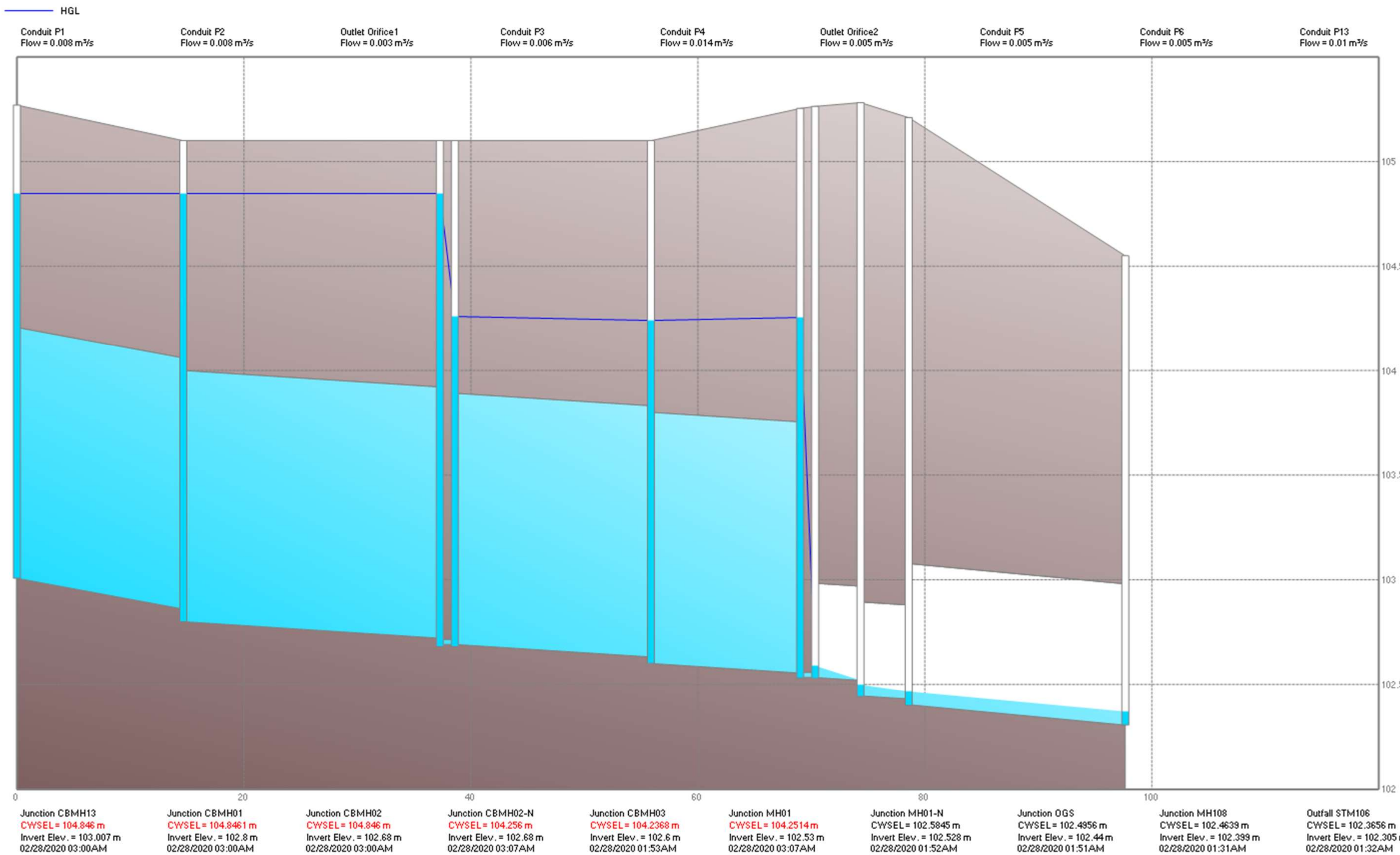

Figure 6: Proposed 100 - Year Storm HGL (From CBMH13 to Ex.STM106) – With SWM Control

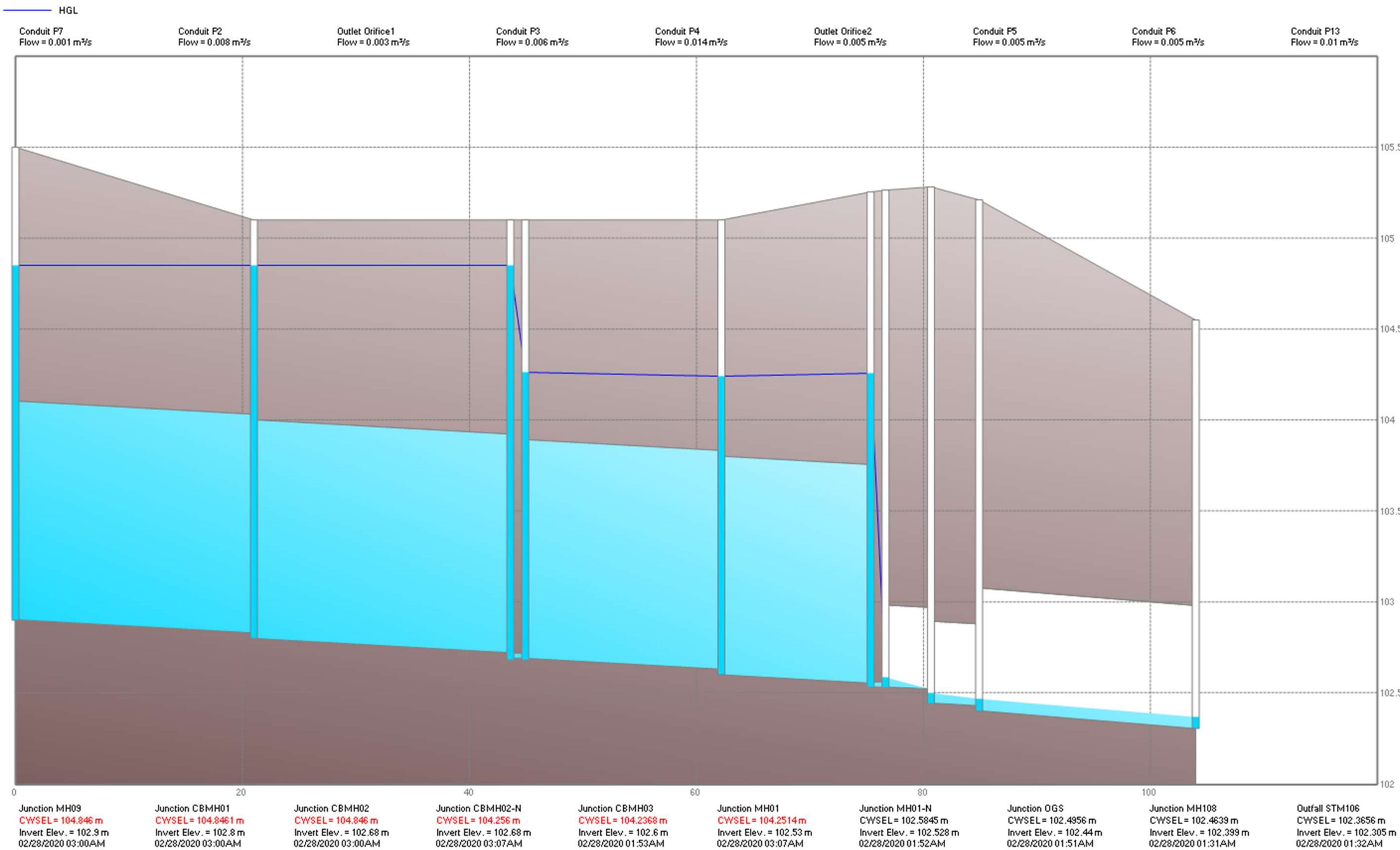

Figure 7: Proposed 100 - Year Storm HGL (From MH09 to Ex.STM106) – With SWM Control

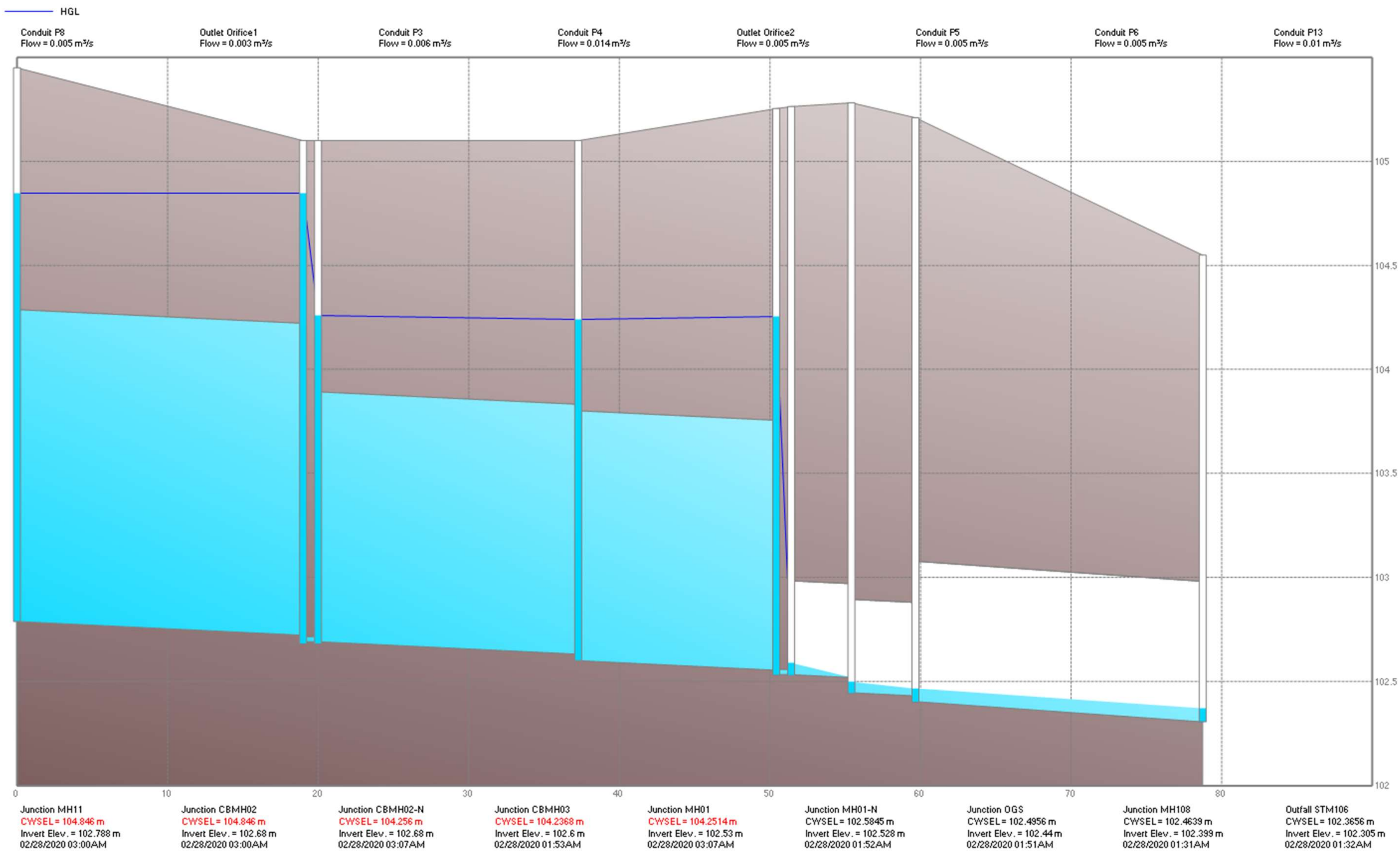

Figure 8: Proposed 100 - Year Storm HGL (From MH11 to Ex.STM106) – With SWM Control

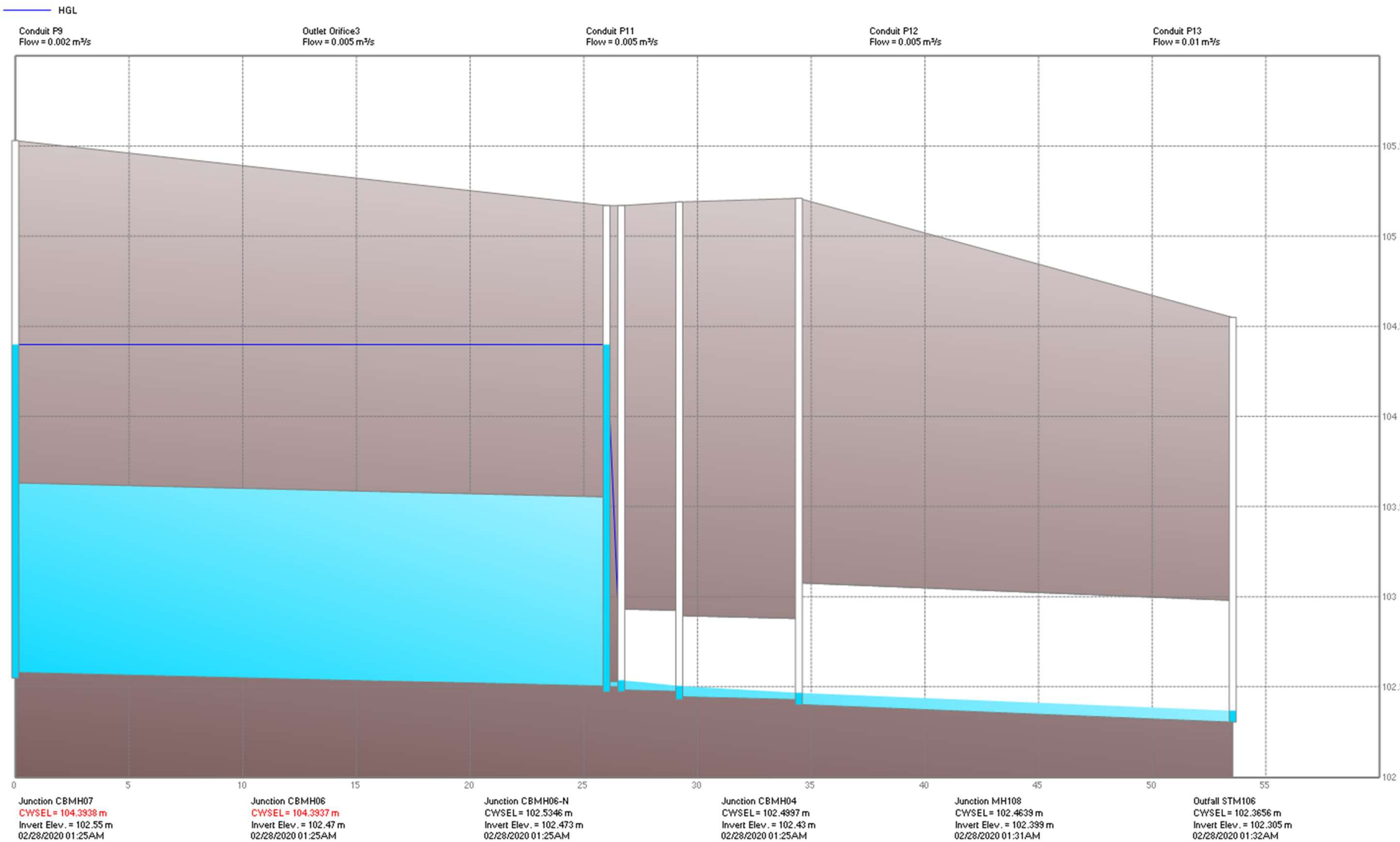

Figure 9: Proposed 100 - Year Storm HGL (From CBMH07 to Ex.STM106) – With SWM Control

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

GEMTEC
www.gemtec.ca

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

experience • knowledge • integrity   expérience • connaissance • intégrité 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

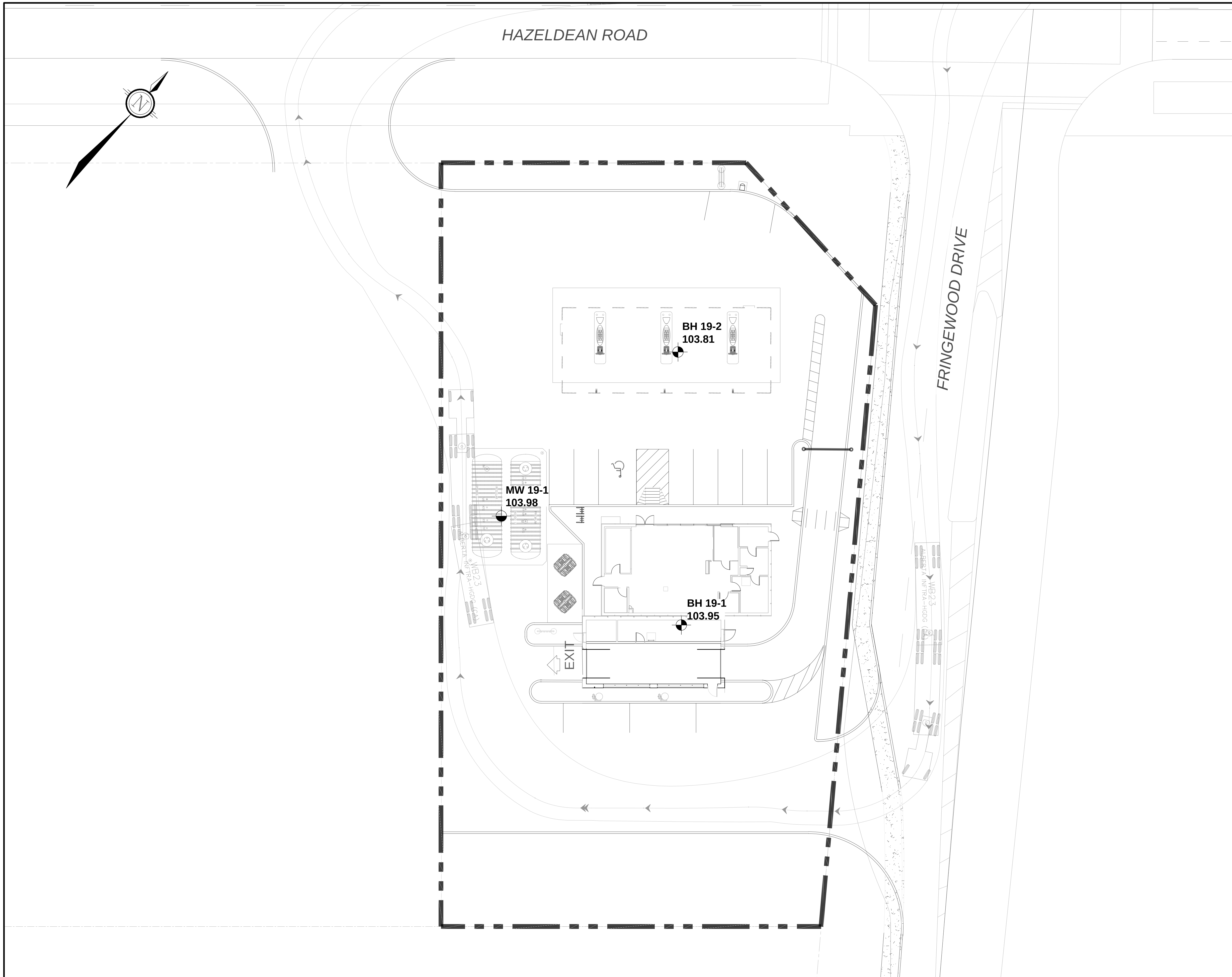
The details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the final design drawings be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

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.



KEY PLAN
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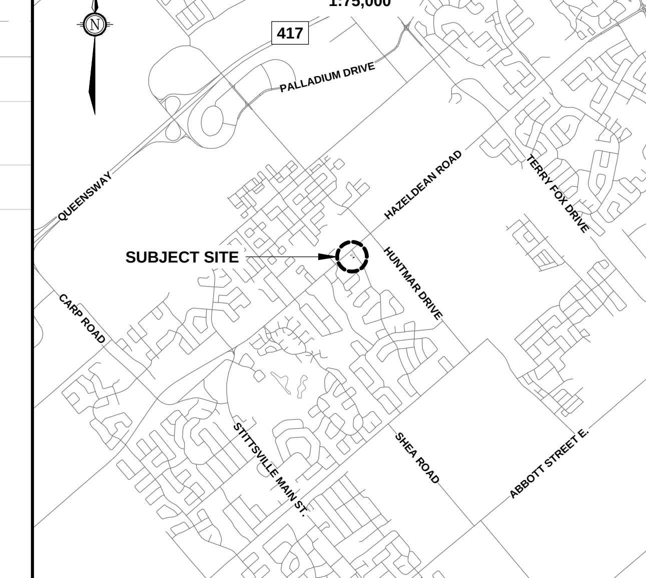

LEGEND

BOREHOLE LOCATION IN PLAN
(current investigation by GEMTEC)

MONITORING WELL LOCATION IN PLAN
(current investigation by GEMTEC)

BH/MW # — BOREHOLE/MONITORING WELL ID
XX.XX — 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

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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 1**	

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

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

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
ADDITIONAL LAB. TESTING

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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	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
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	103.08 / 0.90	1	SS	350	5		
1				2	SS	350	16		
		Compact, brown gravelly silty sand, trace clay, cobbles and boulders (GLACIAL TILL)		3	SS	450	23	MH	Bentonite
2	Power Auger / Hollow Stem Auger (210mm OD)		101.08 / 2.90	~~5~~	~~SS~~	~~25~~	~~50 for 25mm~~		
3	Diamond Rotary Core / HQ (89mm OD)	Moderately fractured, slightly weathered LIMESTONE BEDROCK, banded with shale		5	RC	1370	TCR= 89%, SCR= 59%, RQD= 44%	UC= 146 MPa	Filter sand
4									50mm diameter well screen, 1.5m long
5			98.57 / 5.41	6	RC	1140	TCR= 84%, SCR= 80%, RQD= 80%		
		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

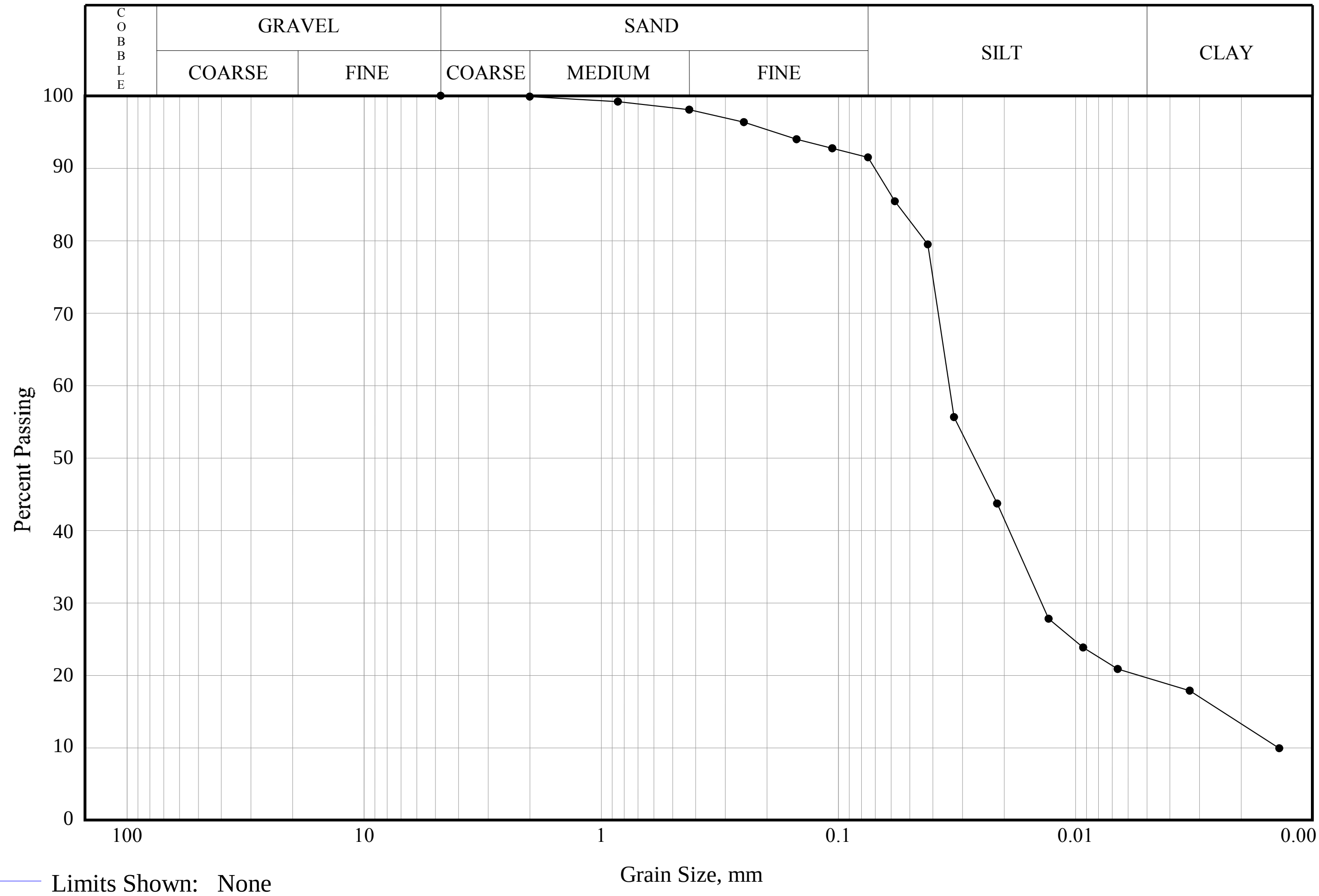
GROUNDWATER OBSERVATIONS

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

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

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	

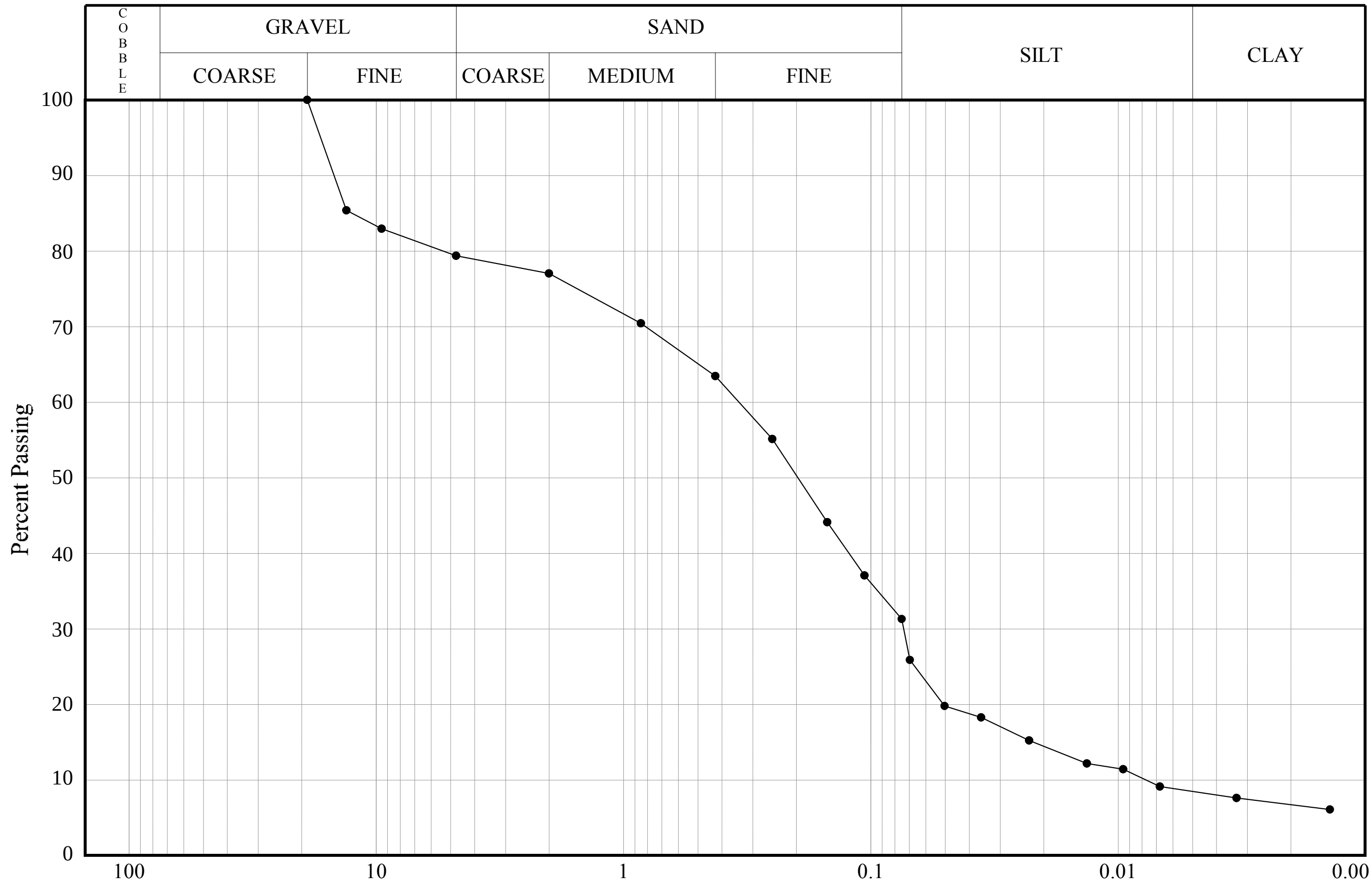


Limits Shown: None

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

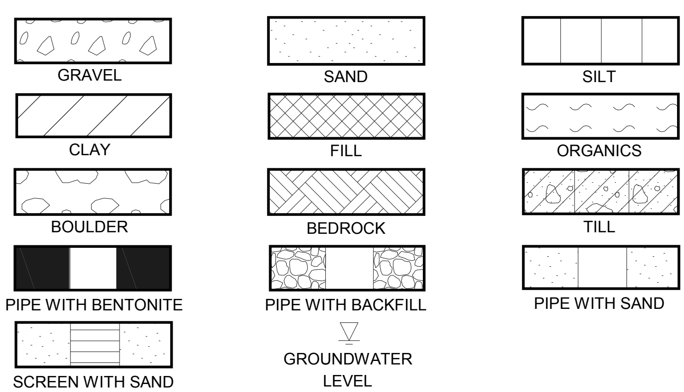
PENETRATION RESISTANCE

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

Scale (mm): 0.01 | 0.1 | 1.0 | 10 | 100 | 1000mm

SILT CLAY	SAND: Fine	SAND: Medium	SAND: Coarse	GRAVEL	COBBLE	BOULDER
< 0.08	0.08 – 0.4	0.4 – 2	2 – 5	5 – 80	80 – 200	> 200

DESCRIPTIVE TERMINOLOGY
(Based on the CANFEM 4th Edition)

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

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descriptive terms.pub
Modified May 2018

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

MW19-1
BORING DATE: June 4, 2019
DEPTH: 2.90 to 5.41 mbgs

BH19-2
BORING DATE: June 4, 2019
DEPTH: 3.71 to 5.34 mbgs

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T: (613) 836-1422 | www.gemtec.ca | ottawa@gemtec.ca

Project		
GEOTECHNICAL INVESTIGATION PROPOSED SHELL SERVICE STATION 5 ORCHARD DRIVE, OTTAWA, ON	**FIGURE B1** File No. 63993.69	ROCK CORE PHOTOGRAPHS

P:\10. Drafting\1. Titleblock Templates\Drawing2.dwg, B1, 19/06/11 1:58:54 PM

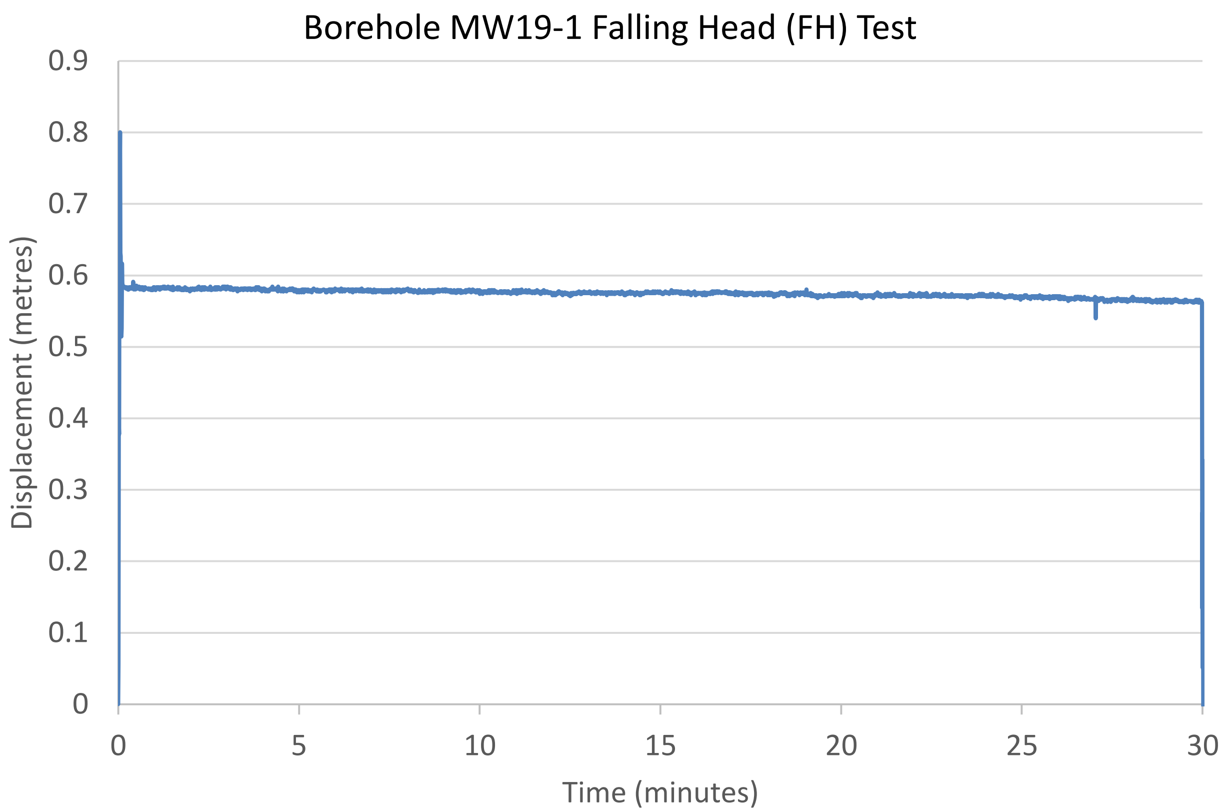
APPENDIX C

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

Certificate of Analysis

Report Date: 17-Jun-2019

Client: GEMTEC Consulting Engineers and Scientists Limited

Order Date: 11-Jun-2019

Client PO:

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	-	-	-

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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
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toktam.nikfarjam@aecom.com

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

Ottawa

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

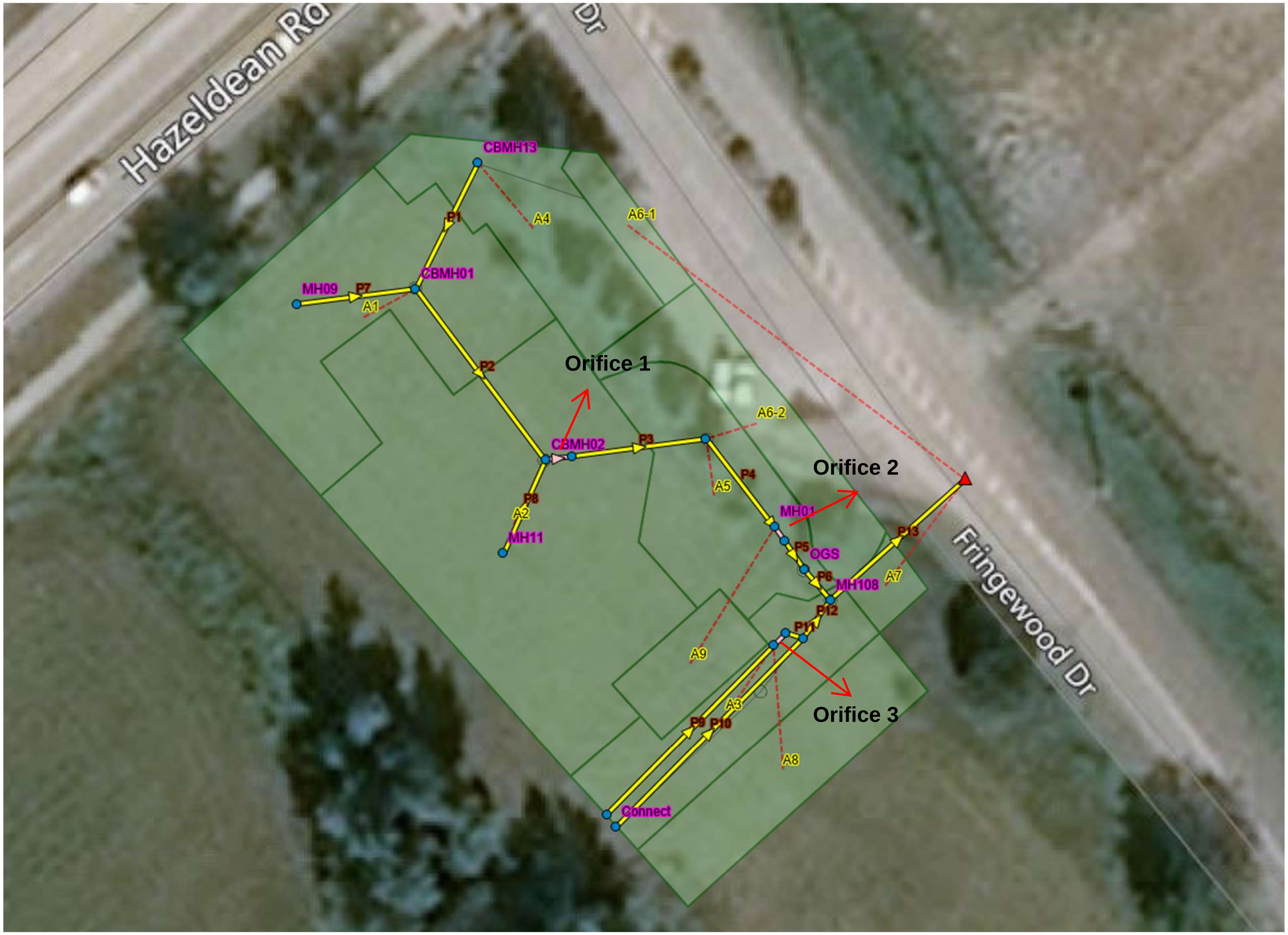
It is important to note that the need for additional studies and plans may result during application review. If following the submission of your application, it is determined that material that is not identified in this checklist is required to achieve complete application status, in accordance with 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

N

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	Time of Max Occurrence 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

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	Hours Full Upstream	Hours Full 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

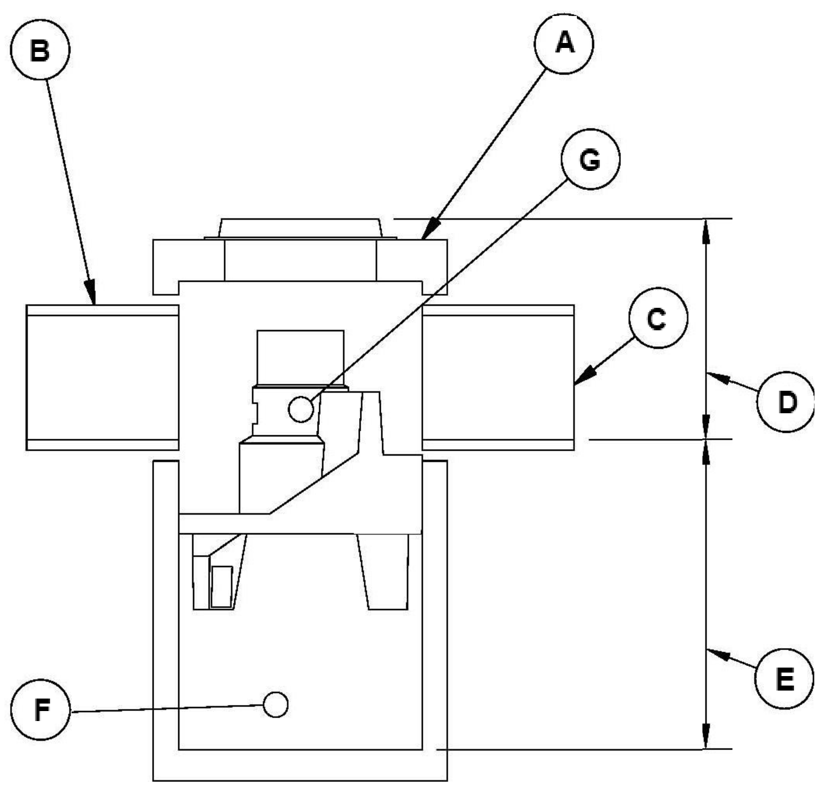
Treatment Requirements		
Treatment Goal:	Enhanced (MOE)	
Selected Parameters:	80% TSS	90% Volume
Selected Unit:	**ADS FD-5HC**	

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

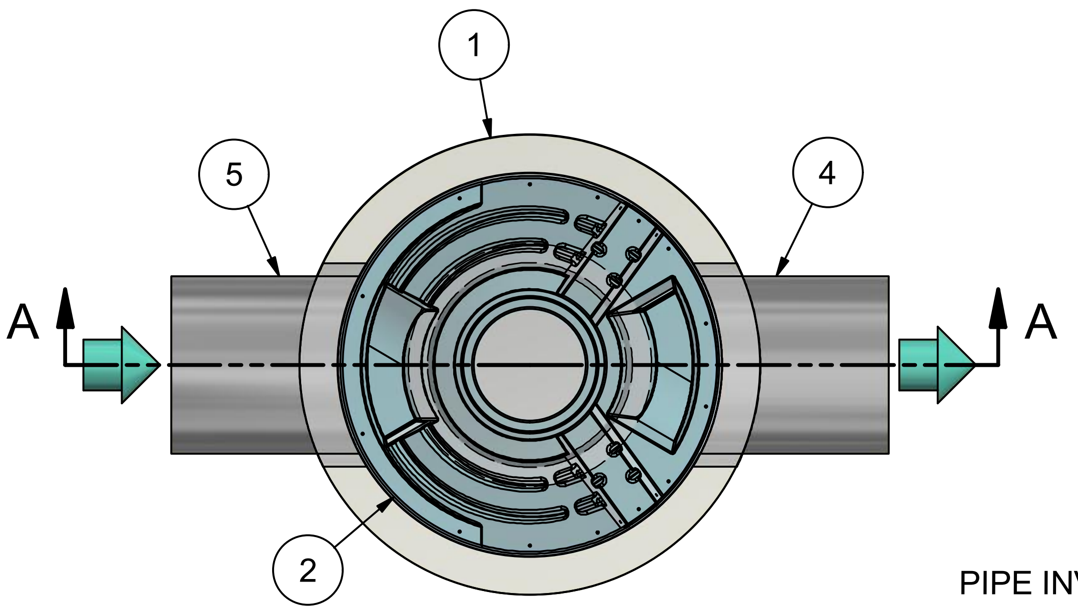
Rainfall Intensity(1)	Fraction of Rainfall(1)	ADS FD-5HC Removal Efficiency(2)	Weighted Net-Annual Removal Efficiency
mm/hr	%	%	%
0.50	0.1%	100.0%	0.1%
1.00	14.1%	100.0%	14.1%
1.50	14.2%	100.0%	14.2%
2.00	14.1%	100.0%	14.1%
2.50	4.2%	100.0%	4.2%
3.00	1.5%	100.0%	1.5%
3.50	8.5%	100.0%	8.5%
4.00	5.4%	99.2%	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:

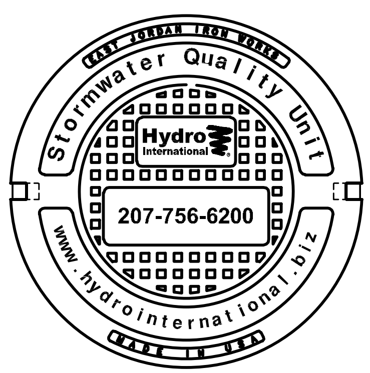
(1) Rainfall Data: 1960:2007, HLY03, Ottawa, ONT, 6105976 & 6105978.

(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.

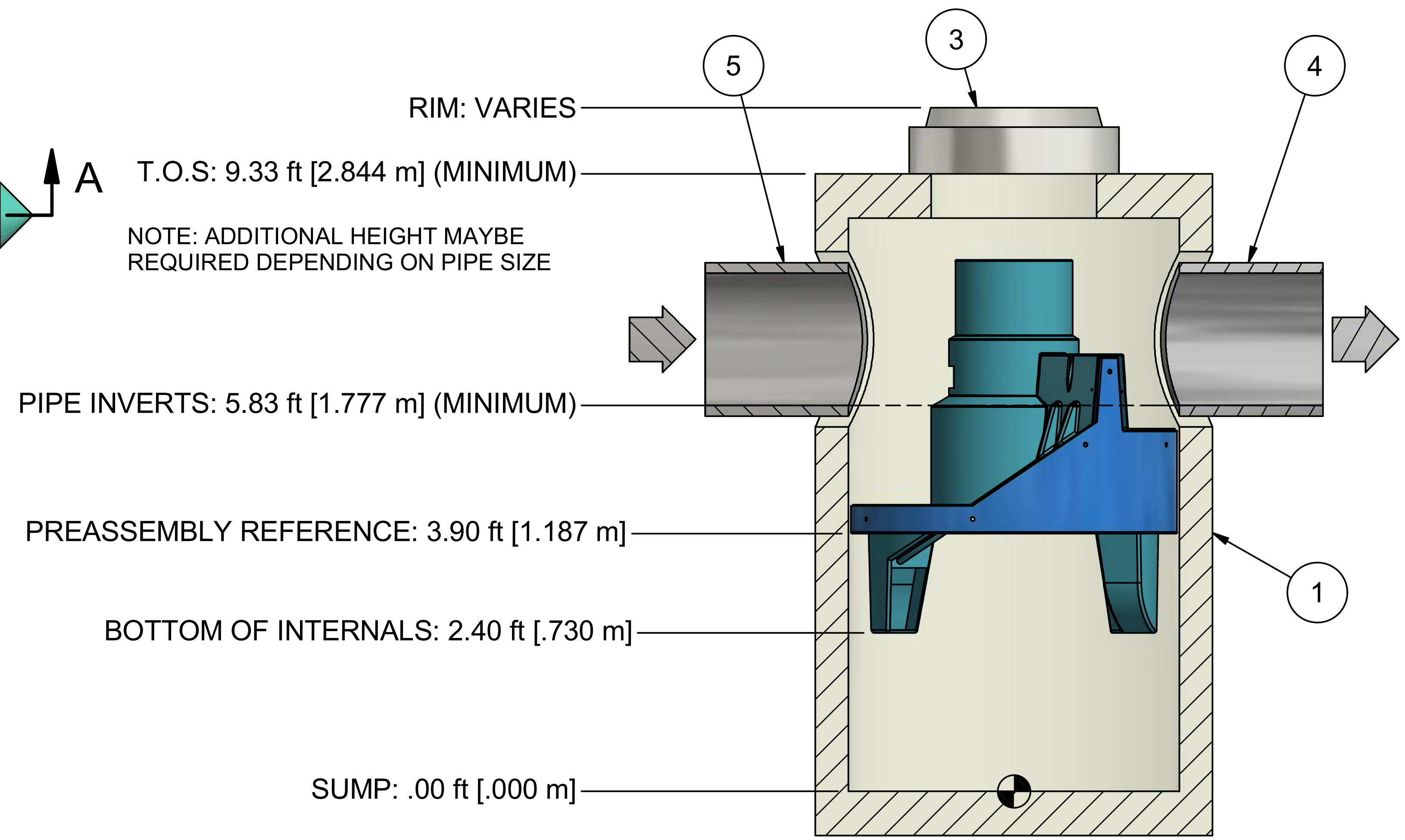


PLAN VIEW



HYDRO FRAME AND COVER (INCLUDED)

GRADE RINGS BY OTHERS
AS REQUIRED



SECTION A-A

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.

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°

DATE: 6/11/2019	SCALE: 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: N/A	MATERIAL:	
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.

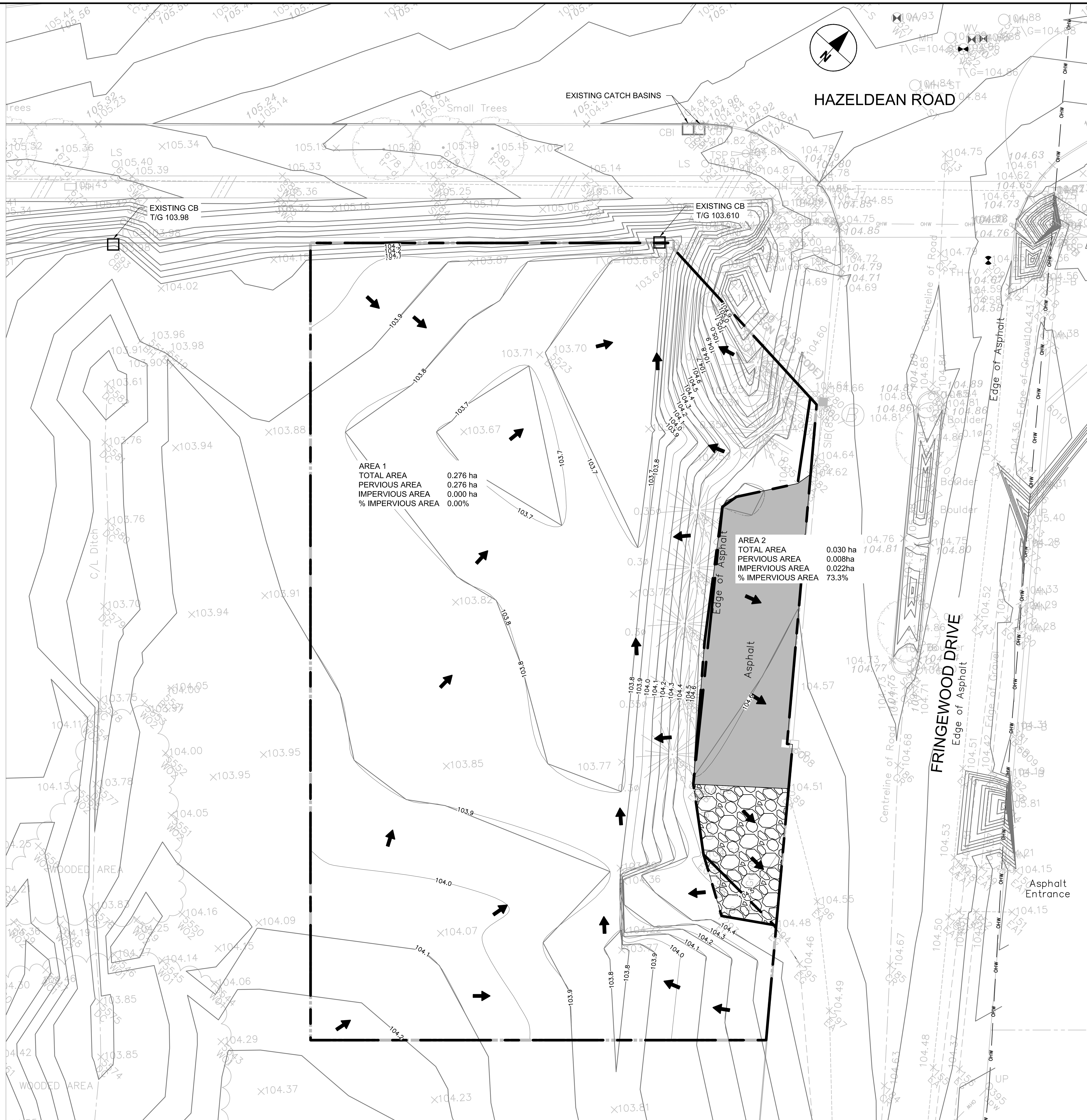
Particle Size (micron)	Distribution (%)
20	20
60	20
150	20
400	20
2000	20

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
604.444.6400 tel 604.294.8597 fax
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

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

Last saved by: REIDJ2(2020-03-27) Last Plotted: 2020-03-27
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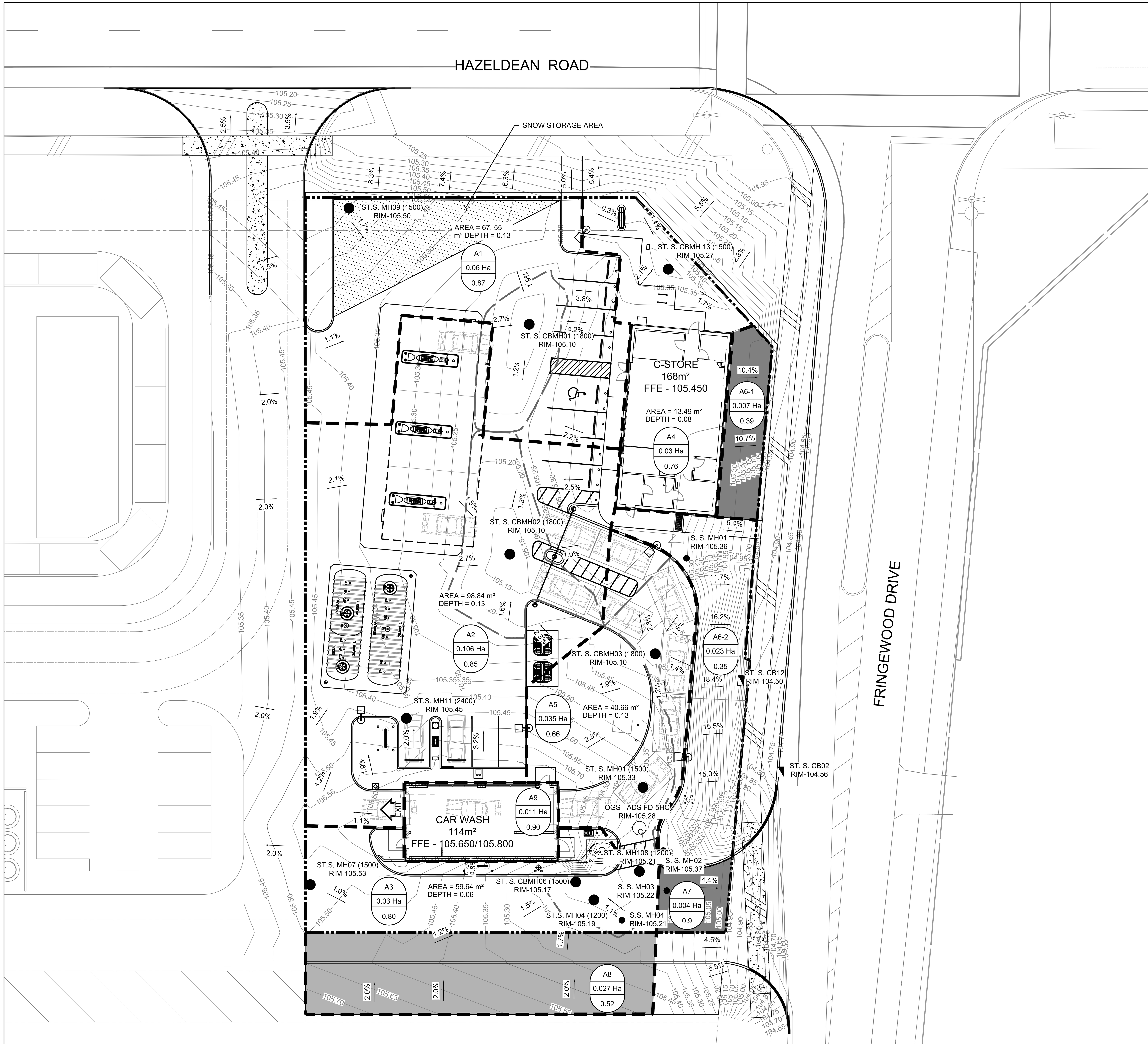
Appendix G

Stormwater Management Plan



STORM WATER MANAGEMENT PLAN

Scale 1:200

0 5 10 m
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

AECOM

PROJECT

Shell Canada Products
Hazeldean Road and
Fringewood Drive NTI

5 Orchard Drive
Stittsville, Ontario

CLIENT

Shell Canada
400-4th Avenue SW
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403.252.4554 tel
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CONSULTANT

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Burnaby, BC V5A 4R4
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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

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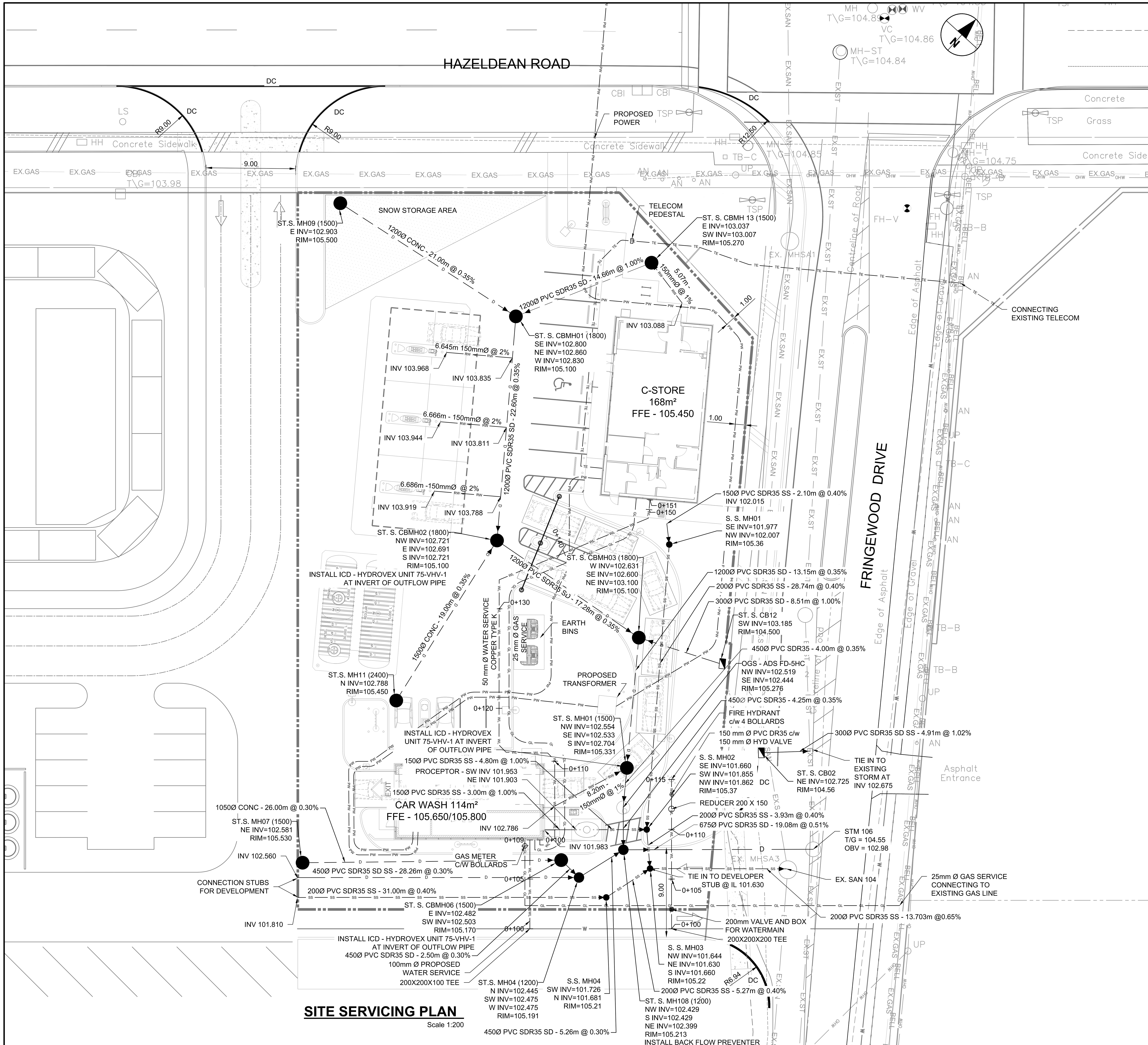
Appendix H

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
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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Shell Canada Products
Hazeldean Road and Fringewood Drive NTI
5 Orchard Drive
Stittsville, Ontario

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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
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: ____

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ANSI D 864mm x 559mm

Approved: ____

Checked: ____

Designer: ____

Project Management Initials: ____

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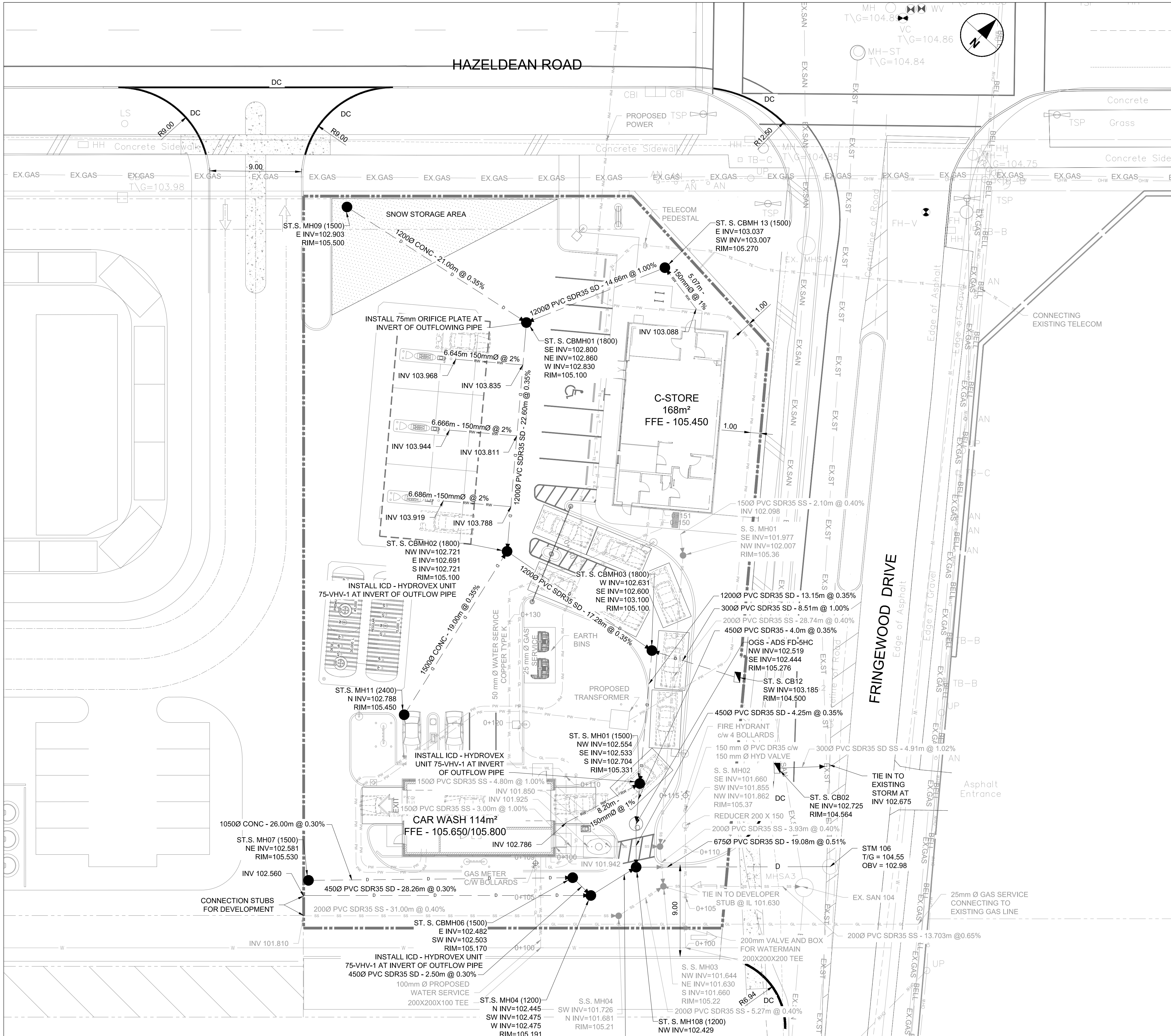

STORM SERVICING PLAN

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 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
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0+103.150	105.218	102.000	SANITARY LINE CROSSING
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0+109.00	105.800	102.000	CONNECTION TO CAR WASH

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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
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0+116.310	105.465	102.000	45° HORIZONTAL BEND
0+121.210	105.343	102.000	POWER LINE CROSSING
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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

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

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REGISTRATION

LEGAL DESCRIPTION

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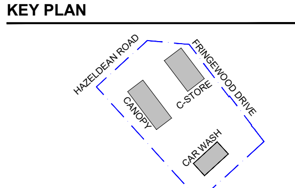
ISSUE/REVISION

I/R	DATE	DESCRIPTION
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

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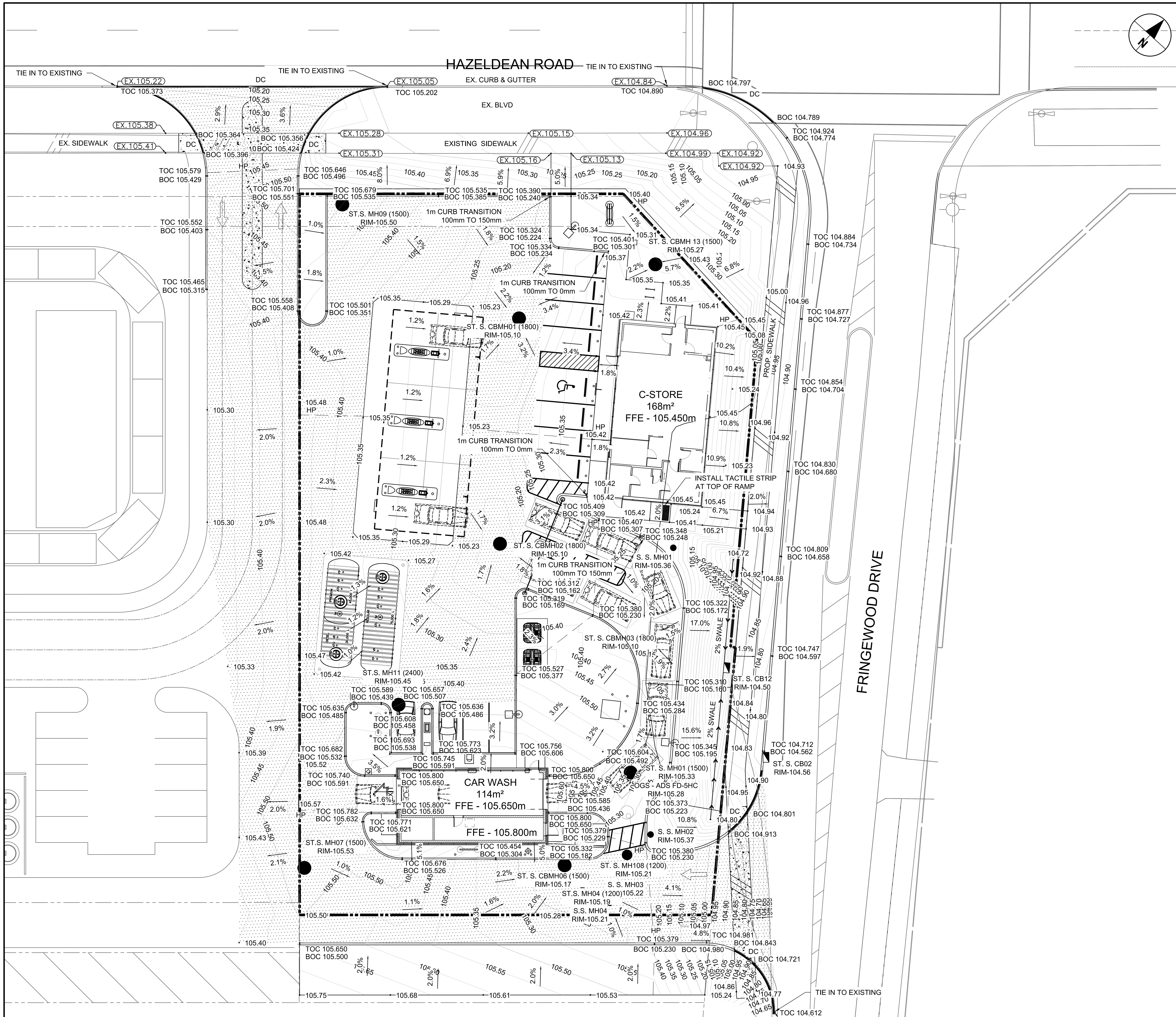
Appendix I

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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Shell Canada Products
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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

SITE GRADING PLAN

AECOM FILE NAME

C104.0-SGP-HZLX

SHEET NUMBER

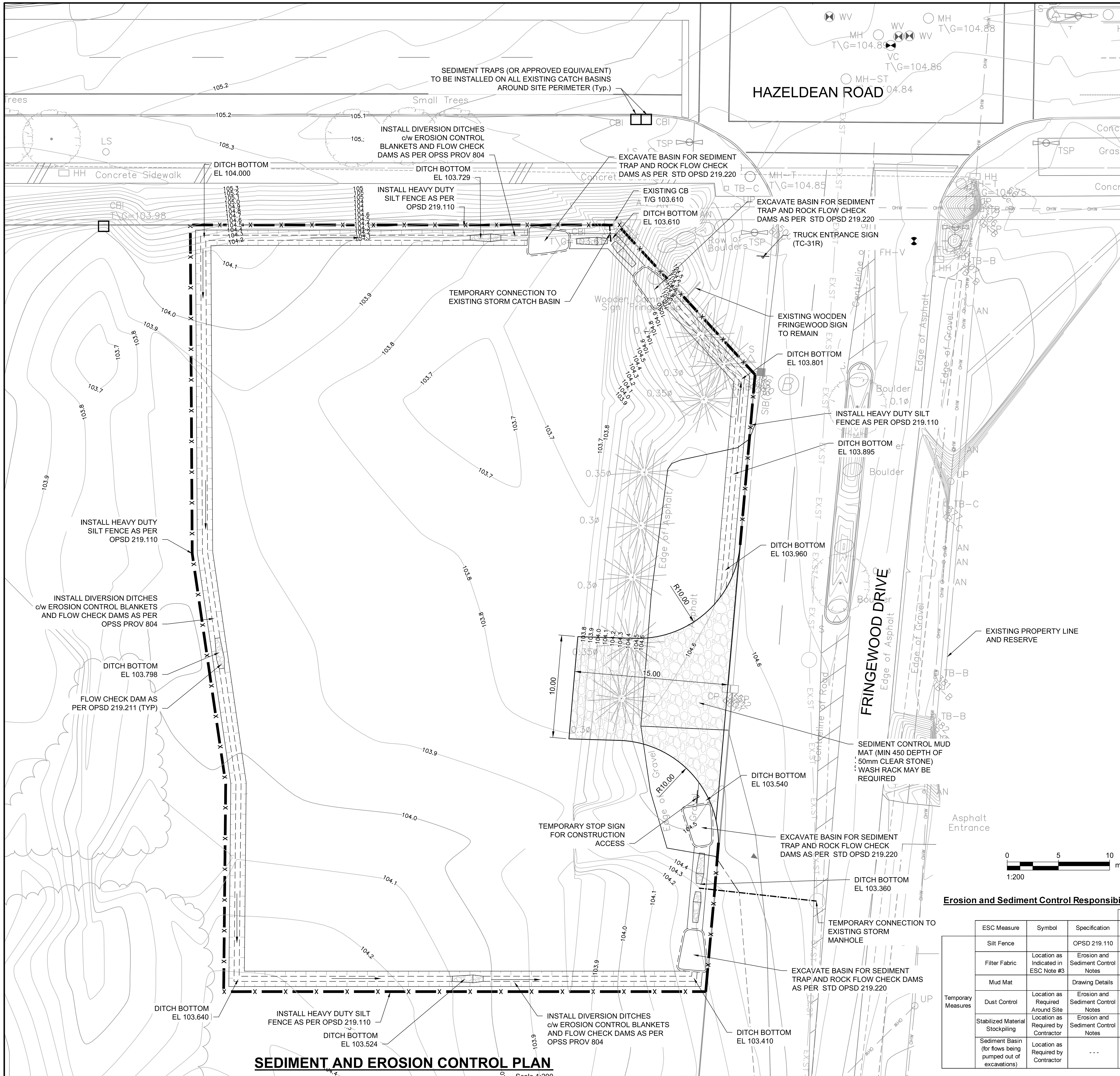
C104.0

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

Last saved by: REIDJ2(2020-03-31) Last Plotted: 2020-03-31
Filename: C:\USERS\REIDJ2\DESKTOP\HAZELDEAN\20200326_CVL_SUBM_HZLX NTI\20200326_CVL_SUBM_HZLX NTI\03.CAD_(01)\HAZELDEAN_DP_2603\DP\C104.0-SGP-HZLX.DWG

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

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

PROJECT

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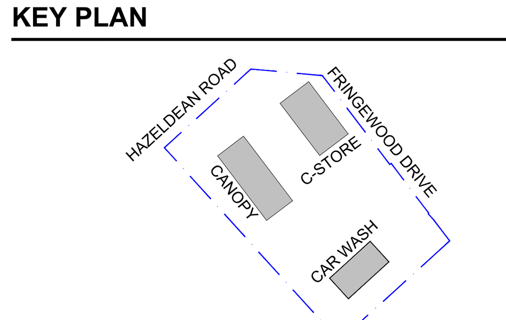
SG

KEY PLAN



GLOBAL PROJECT ID NUMBER

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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: ___

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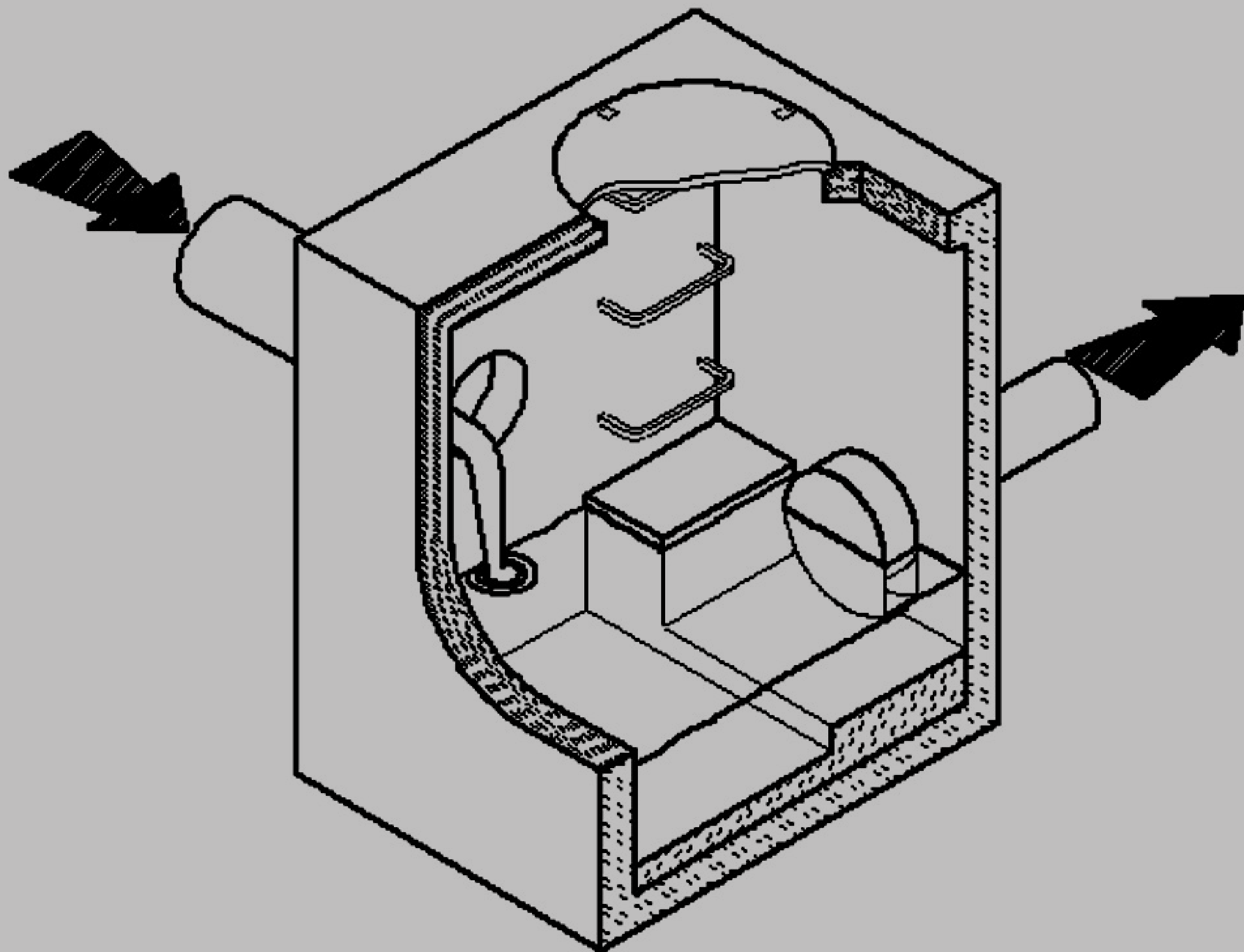
Appendix K

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.

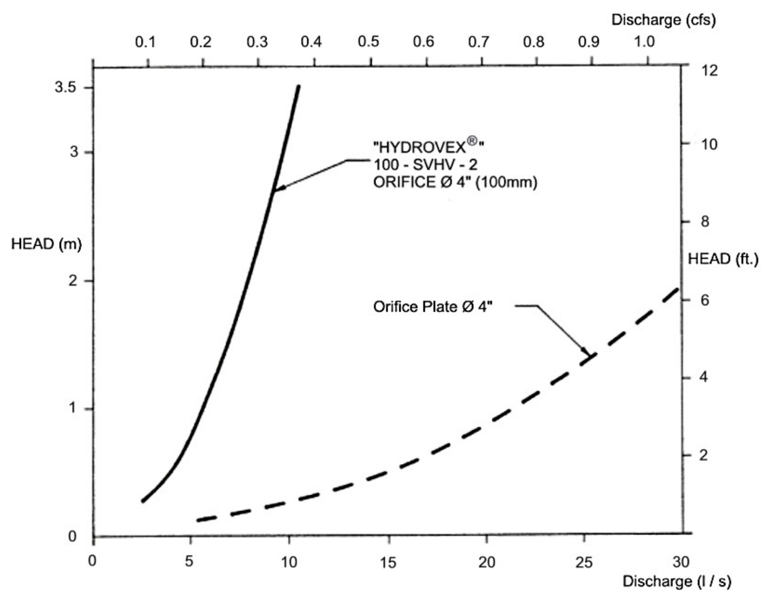


Figure 1:HYDROVEX® VHV/SVHV compared to an orifice plate

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

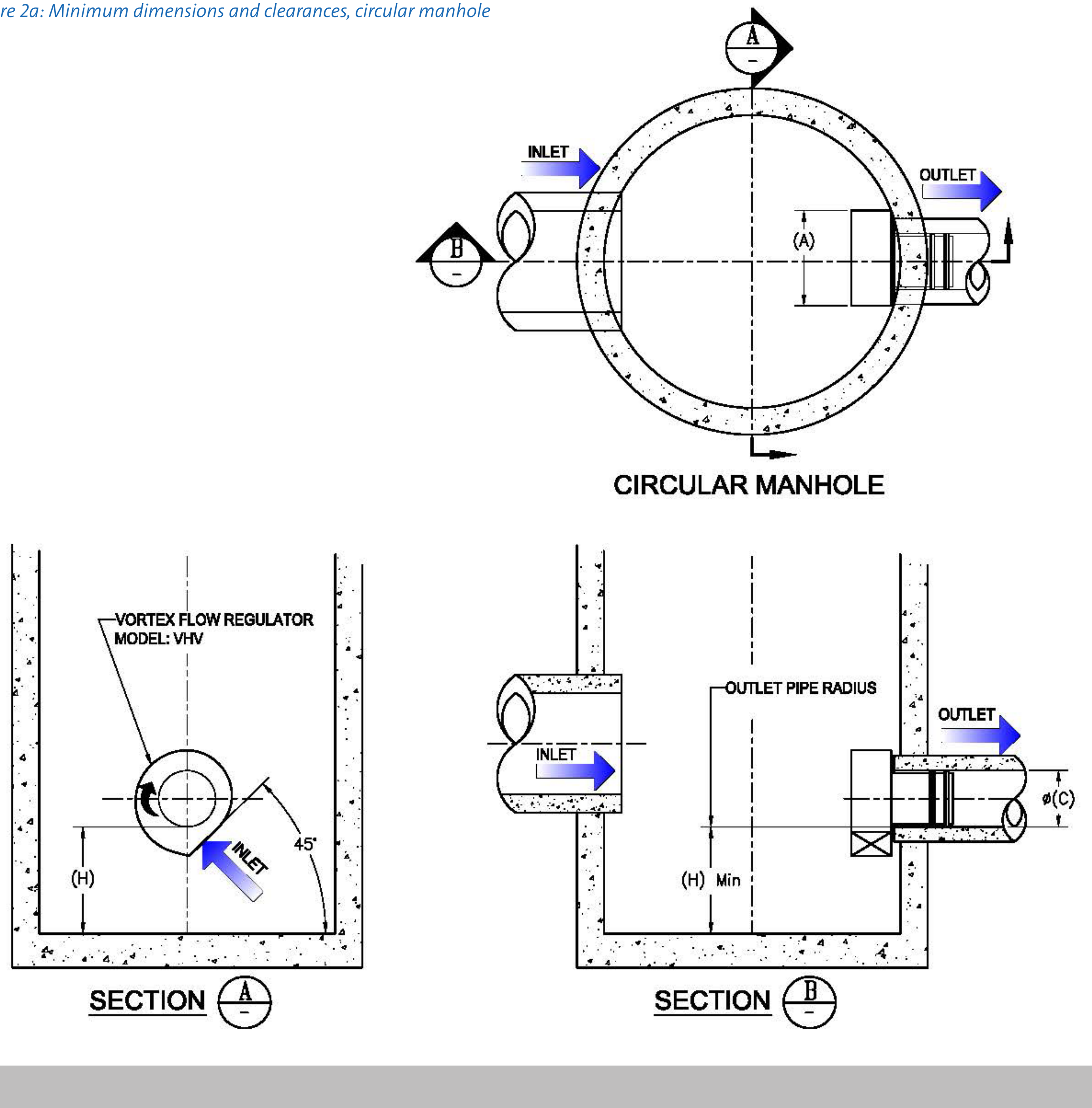
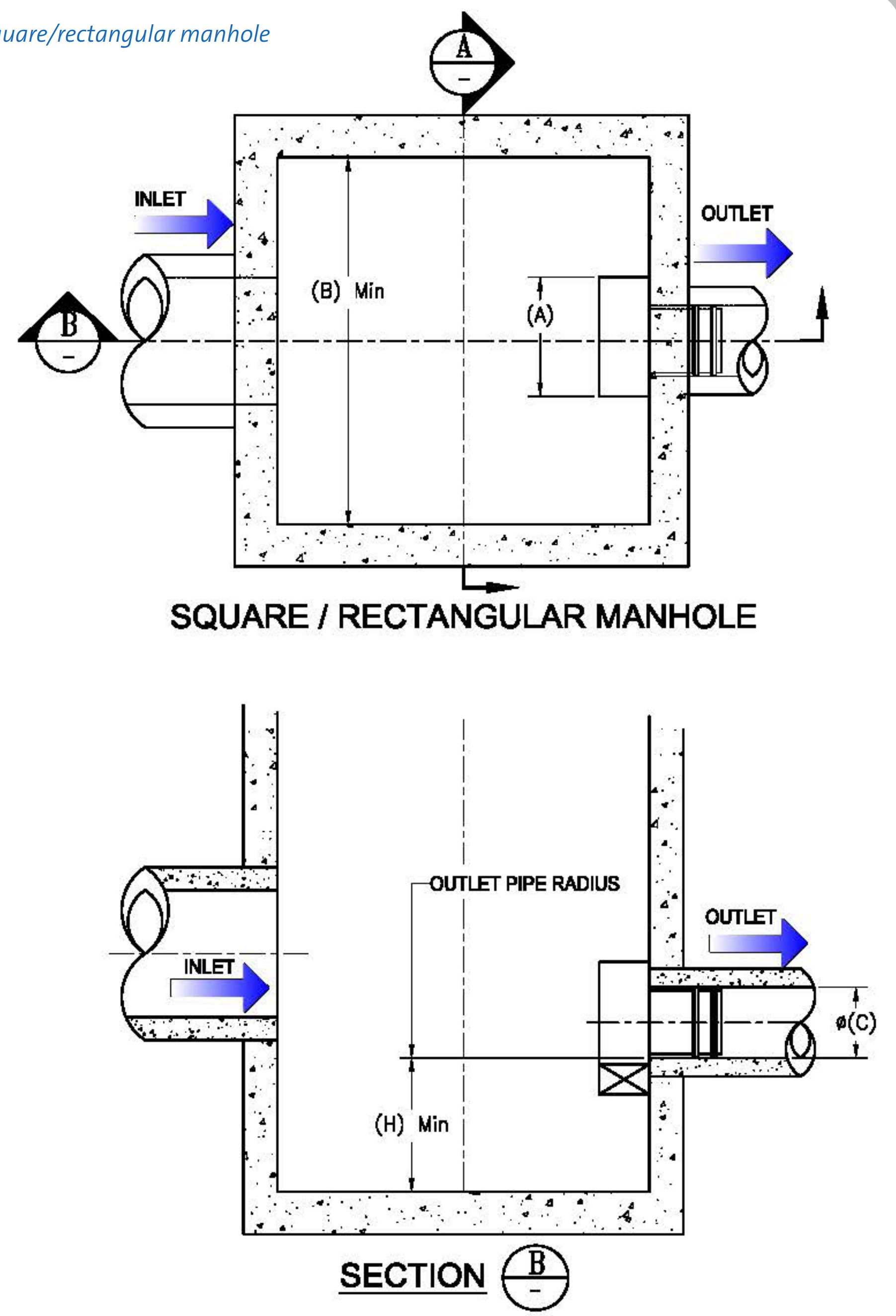


Figure 2b: Minimum dimensions and clearances, square/rectangular manhole



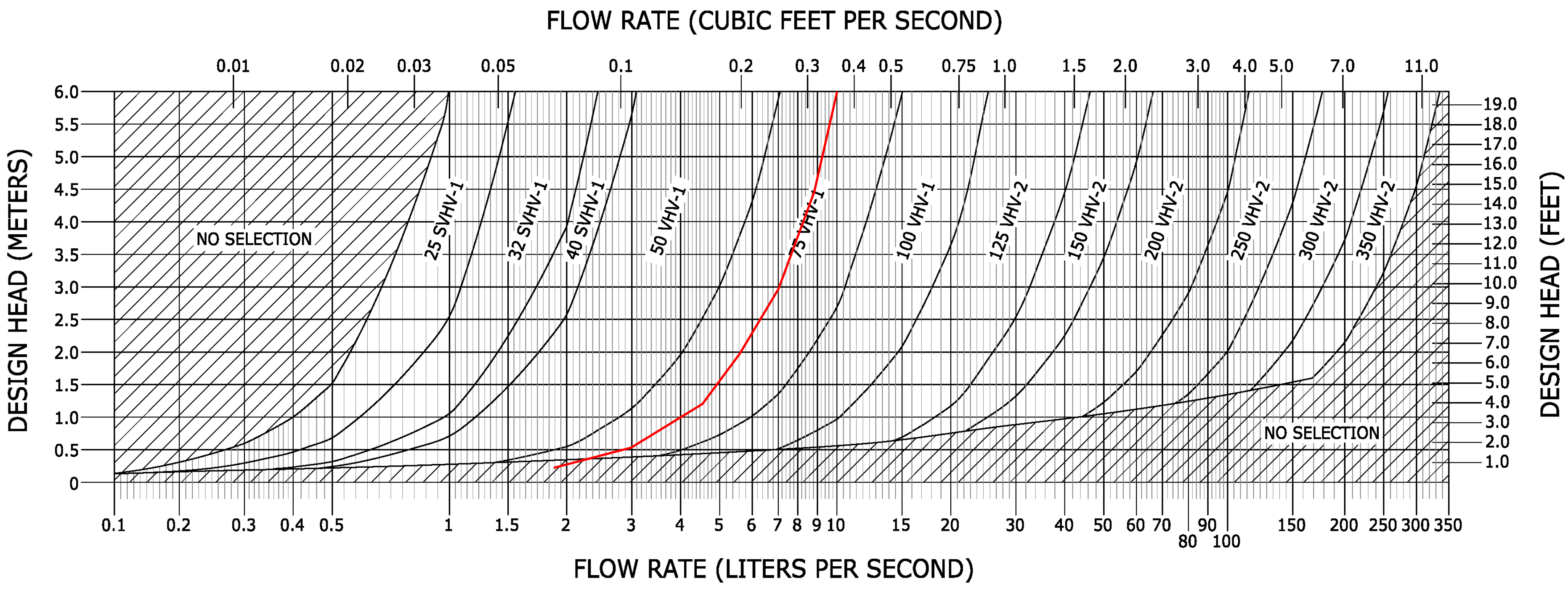


Figure 3 : HYDROVEX® VHV/SVHV Selection Chart

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

Veolia Water Technologies

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