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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 a Stormwater Management design 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 a 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 is a total area of 0.306 ha, part of the 1.82 ha commercial block with 0.027 ha external area.

This stormwater management report (Report) addresses the storm sewer design and stormwater management evaluation and design for the Shell Site (refer to **Figure 1** for its location). The report summarized the storm sewer design and stormwater management (SWM) design requirements and proposed works to address stormwater runoff flow from the site under post-development conditions and identify any stormwater servicing concerns for the proposed Shell Site development located at 5 Orchard Drive, Ottawa.

Figure 1: Site Location

1.1 Background

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

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

As part of the assessment of the entire development area referred to as 5 Orchard Drive and reported in the 2019 Functional Servicing Report, a hydraulic evaluation was undertaken of the downstream recipient storm sewer within Hazeldean Road. The purpose of the assessment was to establish the available capacity within the downstream system for the development site. Based on the results of the analysis, it was determined that the downstream system has only 251.9 l/s for the entire development site (5 Orchard Drive – total commercial and residential areas) during the 100-year storm event. From the 2019 Functional Servicing Report, 200 l/s was assigned for residential design and the remainder 51.9 l/s was assigned to the commercial area assuming a site runoff coefficient of C=0.9. The total commercial area (including the Shell Site) is 1.82 ha and total outflow from the site is 30.26 l/s for the 5 year event and 51.85 l/s for the 100 year event. This results in a level of service rate of 16.62 l/s/ha for the 5 year and 28.5 l/s/ha for the 100 year. The level of service was confirmed by the developer and their engineer and the communication is provided in **Appendix E**.

Governing design criteria applied for the storm sewer and stormwater management was obtained from the following documents:

- Ottawa Sewer Design Guidelines, Second Edition (SDG002), October 2012;
- Technical Bulletin PIEDTB – 2016-01, Revisions to Ottawa Design Guidelines – Sewer;
- Mississippi Valley Conservation Authority (MVCA); and,
- Ministry of Environment and Conservation and Parks (MECP), *Stormwater Management Planning and Design Manual* (March 2003).

The documents provide guidance on the control the storm discharge quantity and quality from the site to meet the allowable flow rate. This can include, but is not limited to, a combination of absorbent landscaping, Oil and Grit Separator and on-site oversized pipe as well as depression surface areas for storage.

The Shell 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 (**Appendix A**);
- *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 (excerpts in **Appendix B**);
- *Stormwater Management Report for the 5 Orchard Drive, City of Ottawa*, prepared by David Schaeffer Engineering Ltd (DSEL), March 2020 (excerpts in **Appendix C**);
- Sewer Design Guidelines, Second Edition (SDG002), City of Ottawa, October 2012;
- Technical Bulletin PIEDTB – 2016-01, Revisions to Ottawa Design Guidelines – Sewer, September 2016;
- Technical Bulletin ISDTB-2014-01, Revisions to Ottawa Design Guidelines – Sewer, City of Ottawa, February 2014;
- City of Ottawa Technical Bulletin ISTB-2018-04, City of Ottawa, June 2018;
- 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. Rational Method Parameters

The Rational Method is used to as the basis for both the storm sewer design and the stormwater evaluation and design of the site. The runoff flow rates for the storm sewer design are based on the 5 year storm intensity whereas the stormwater evaluation was based on the flow rates generated from the 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year storm intensity. The flows are determined based on the below Rational Method formula:

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

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 subject site includes the following three main sub-areas: buildings, green landscape areas (grass/vegetation), and asphalt surfacing. Upon review of the Geotechnical Report prepared by GEMTEC Consulting Engineers and Scientists (see **Appendix A**), the surface grade at the borehole locations consists of dark brown clay silt topsoil and a deposit of brown silt with some clay and trace sand was encountered below the topsoil (refer to **Appendix A**). This soil type was taken into consideration when selecting the Runoff Coefficient for the green landscape area.

Based on the above, the following Runoff Coefficients were selected as per **Table 1** for use under existing and proposed site conditions:

Table 1: Summary of Runoff Coefficients Values from Ottawa Sewer Design Guidelines (SDG002)

Description	Runoff Coefficent,C	Runoff Coefficient for 100 Year Storm Event
Building	0.9	1.0*
Pavement – Asphalt	0.9	1.0*
Grass – Vegetation*	0.3	0.38

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

It should be noted that based on the type of the soil (silt and some clay), the Runoff Coefficient of 0.3 was selected for the grass/vegetation areas based on the below **Table 2** which is a reproduction of Table 5.7 from the Ottawa Sewer Design Guidelines (page 5-26).

Table 2: Runoff Coefficients for Various Soil Conditions (Table 5.7 SDG002)

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.

The Ottawa Sewer Design Guidelines (SDG002) stipulated that minimum initial Time of Concentration is to be 10 minutes. This is used for the sizing of the storm sewer system. For the stormwater management evaluation, the 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 F** 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 subject 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. Storm Sewer Design

3.1 Storm Servicing Strategy

The storm servicing strategy for the subject site includes:

- Site grading to maintain sufficient site lines and tie-in with surrounding right-of-way and areas;
- Minimize cut and fill earth operations;
- Reduce or eliminate retaining walls, where feasible;
- Minimize impact to abutting properties;
- Site grading to contain the runoff up to the 100-year storm event within the property boundaries;
- Enable gravity servicing outlets;
- Conveyance of runoff to catchbasins strategically located throughout the site;
- Grading of low points for ponding to a maximum of 0.3 m at catchbasin locations, where feasible;
- Storm sewers through site sized for the 5-year Rational Method flow (see **Section 2**);
- Due to the higher water quality risk due to the site use as a gas station, an oil grit separator with filtration is required to treat the site to an Enhanced Level of Protection (80% TSS removal) prior to discharge. This will require a direct submission to the Ontario Ministry of Environment, Conservation and Park (MECP) for Environmental Compliance Approval (ECA);
- All site storm sewers will be conveyed to the oil grit separator for capture of any potential on-site spills;
- All roof drains directed to a storm sewer via an overland flow route to a catchbasin;
- On-site storage provided for all storm events up to, and including, the 100-year storm event;
- Emergency overflow route provided for the subject site out to Fringewood Drive;
- Storm sewer from subject site connected into future storm sewer in Fringewood Drive.

A storm sewer in Fringewood Drive, and the ultimate outlet for the site, has been designed, approved by the City of Ottawa and a Ontario Ministry of Environment, Conservation and Parks (MECP) Environmental Compliance Approval (ECA) has been issued. The storm sewer is anticipated to be constructed concurrently with the future subdivision to the south of the subject site. Following discussions with the adjacent developer, it is anticipated that the subdivision and installation of the Fringewood Drive storm sewer will be in advance of the construction of the subject site. Therefore, for the purposes of this report and design, it is assumed that the storm sewer within Fringewood Drive exists.

3.2 Proposed Storm Sewer

3.2.1 Design Criteria

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

- Intensity Duration Frequency curves as per OSDG Section 5.4.2 and outlined in **Section 2**);
- Runoff Coefficients as per OSDG Section 5.4.5.2.1 and Table 5.7 (discussed in **Section 2**);
- Time of concentration 10 minutes for most upstream drainage area of each sewer run;
- Storm sewer minor flow should be controlled to meeting the existing recipient sewer level of service or runoff from the design storm (2 or 5 year), whichever is less.
- On-site storage provided for all storm events up to, and including, the 100-year event;
- Particle size distribution for oil grit separator sizing (discussed in **Section 6.6**);
- Minimum inlet control device size is 83 mm (round) or a minimum flowrate of 6 l/s for a Vortex-type unit (as per the City Approved Sewer and Miscellaneous Products Listing);
- Storm sewer pipe size gravity capacity determined using Manning's equation;
- Manning's n value = 0.013;
- Minimum storm sewer pipe used is 250 mm diameter;
- Minimum storm sewer slope = 0.1%;
- Minimum storm sewer velocity = 0.8 m/s;
- Maximum storm sewer velocity = 3.0 m/s;
- Storm sewer diameter and minimum slope as per OSDG Table 6.1;
- Depth of cover (pipe obvert to finished grade) is 2.0 m, but no less than 1.0 m;
- Storm sewers match obvert to obvert where practical, lowered in some areas to increase cover; and,
- On-site ponding depth not to exceed 350 mm under static or dynamic conditions.

3.2.2 Storm Sewer Sizing

The storm sewer system was sized using the 5 year Rational Method and the criteria noted in **Sections 2, 3.1 and 3.2.1)**. The supporting calculations are provided in **Appendix E**. The storm sewer system was designed with the dual purpose of conveying subject runoff to the recipient future storm sewer in Fringewood Drive and providing underground or in-line storage for the subject site. Therefore, the some of the pipes proposed for the site are oversized. The size of storm pipes to service the site range from a 450 mm diameter PVC SDR35 to 1050 mm diameter concrete pipes. The storm sewer system is presented in **Sheet C803.0** (presented in **Appendix G**) and the drainage area plan is presented in **Sheet C105.0** (presented in **Appendix G**). Generally, the storm system captures flow via CBs and CBMH structures from the north portion of the subject site to the southwest corner. With the exception to two areas released uncontrolled to Fringewood Drive by sheet flow (Areas A6-2 and A7, see **Sheet C105.0, Appendix G**), the majority of the site is conveyed via the storm system to an oil grit separator prior to discharging to an east-west storm pipe in the southern portion of the property (675 mm diameter) that will connect to the future storm system in Fringewood Drive. There is a branch of the storm sewer system servicing the subject site that provides a service connection for the future commercial site (**Sheet C803.0, Appendix G**, Stub to MH201).

The cover provided for the storm pipes throughout the site are between 0.61 m to 2.15 m. There are three instances where the cover is less than 1.0 m and it occurs along the following storm sewer runs (see **Sheet C803.0, Appendix G**):

- CB12 to CBMH03 – 0.61 m of cover;
- CBMH06 to MH07 – 0.84 m of cover; and,
- CBMH01 to CBMH02 – 0.98 m of cover.

For those storm sewers with less than 2 m of cover are proposed to be insulated and frost protection details are provided on **Sheet C103.0, Appendix G**.

The dual purpose of the storm system to provide flow conveyance during frequent storm events and storage during less frequent storm events (100 year storm event), the storm sewer system was modeled hydraulically to confirm on-site storage (pipes and surface), site outflow targets, ICD function and the resultant hydraulic grade line. The details of the hydraulic assessment are presented in **Section 6.3**.

To achieve the restricted outflow rate established for the site (28 l/s/ha), one inlet control devices (ICDs) is proposed for the site. The location of the ICD is indicated in **Sheet C803.0** (**Appendix G**). The ICDs proposed is the Hydrovex® VHV Vertical Vortex Flow Regulator model 75 VHV-1. The units are listed on the City of Ottawa *Approved Sewer and Miscellaneous Products Listing* (MS-22.15 March 2019). Further discussion regarding the evaluation of the stormwater system is discussed in **Section 6.3**.

3.3 Storm Sewer Summary and Conclusions

From **Sections 3.1 to 3.2**, calculations supporting the storm pipe design and **Sheets C105.0 and C803.0** presented in **Appendix G**, the storm sewer design for the subject site meets the City of Ottawa requirements.

4. Stormwater Design Standards and Criteria Review

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

4.1 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 Guidelines and Technical Bulletin PIEDTB are the governing documents 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.

4.1.1 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 for return periods of 2-year to 100-year, 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 without ponding during event and surcharging.
- Sewer system should be protected against critical surcharging during the 100-year storm event without overtopping the manhole.
- 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 as per *City of Ottawa Approved Sewer and Miscellaneous Products Listing* (MS-22.15 March 2019).
- The maximum depth of flow on local and collector streets can be used up to 0.35 m during the 100-year event.
- The maximum HGL should be remained at 0.3 m below the underside of footing.
- When using the Rational Method for stormwater design 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 on the Runoff Coefficient 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.

4.1.2 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. Details related to the sediment and erosion control plan proposed for the Shell Site are provided in **Section 8**.

4.1.3 Water Quality

The water quality target for the site is the long-term average removal of 80% Total Suspended Solids (TSS) on an annual loading basis as per correspondence with the MVCA . This target is applied to the Shell Site due to its primary use as a gas station.

5. Existing Conditions

5.1 General

Under existing conditions, the total site area is 0.306 ha of rural undeveloped land. There is no on-site water retention and/or detention observed. **Sheet C131.0** (refer to **Appendix G**) provides lot drainage configuration for the existing conditions. As shown in **Sheet C131.0**, most of the runoff generated from site sheet flows to the adjacent properties and City road right-of-way. The surface runoff is generally split in two directions consisting of the following:

- 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.

Following summarizes the proportions of existing pervious and impervious areas:

- Pervious Areas: 92.81%
- Impervious Areas:7.19%

Under existing conditions, runoff from the Shell Site is collected via a ditch inlet catchbasin near the intersection of Hazeldean Road and Fringewood Drive and conveyed by storm sewers within Hazeldean Road to the existing interim SWM pond located on Hazeldean Road on the northeast corner of Huntmar Drive. The interim pond ultimately discharges into the Carp River via Hazeldean Creek.

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

Table 5: Existing Peak Flow from the Subject Site

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

5.2 Site Soil Conditions

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 silt 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 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**).

6. Proposed Conditions

6.1 Site Developments and Site Grading

The proposed Shell Site 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 hard surface walkways and landscaped areas, and two (2) underground fuel storage tanks. The proposed site is shown on **Sheets C104.0 and C105.0** (refer to **Appendix G**).

Following are the proportions of pervious and impervious areas under proposed development conditions for the Shell Site:

- Pervious Areas: 26.70%
- Impervious Areas:58.89%
- Roof Impervious Areas:14.41%

All grading has been undertaken to satisfy the following:

- 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 Shell Site grading is such that low points or ponding is available throughout the site at some of the catchbasins or catchbasin manholes. The intent of the design is to convey runoff directly to the oversized storm sewer system for underground storage. The roof of the 'C-Store' and Carwash have been designed with rainwater leaders that discharge directly onto a grassed/landscape area that then sheet flows to CBMH13 and CBMH6, respectively.

During all storm events (2 to 100-year), there is no ponding of water on the surface at each catchbasin. However, there is surface ponding available at depths between 0.06 to 0.3 m at the low points on-site (refer to **Sheet C105.0, Appendix G**). With the exception of two drainage areas (A6-1, (0.007 ha), grass area located on the back of Convenience Store, and A7 (0.004 ha), the access portion of the driveway), the remainder of the Shell Site is graded to and captured by the site stormwater system. These two catchment areas (A6-1 and A7, total area of 0.011 ha) release uncontrolled into the Fringewood Drive right-of-way. Refer to **Sheets C103.0** and **C803.0** (see **Appendix G)** as well as **Section 6.2.2** for an overview of the proposed development.

As shown in the **Sheet C131.0** (see **Appendix G**), the existing site footprint is 0.306 ha. In the proposed condition, Catchment Area A8 (0.027 ha) is located outside of the lease line and site footprint, but drains into the Shell Site. As a result, the total catchment area evaluated is 0.333 ha in proposed conditions (0.306 + 0.027 = 0.333 ha).

6.2 Water Quantity

6.2.1 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. Table 9 from the *Functional Servicing and Stormwater Management Report for Campanale Homes Development* (excerpts in **Appendix B**), the release rate for the 1.82 ha commercial block was calculated as 30.26 L/s and 51.85 L/s for the 5-year and 100-year storm events, respectively. As a result, the release rate for the Shell Site is 5.54 L/s and 9.5 L/s for 5-year and 100-year storm events, respectively. The calculation is provided in **Table 6** below.

- Release Rate for 5-year storm event (Future Commercial Area plus Shell Site): 30.26 L/s/1.82 ha = 16.62 L/s/ha
- Release Rate for 100-year storm event (Future Commercial Area plus Shell Site): 51.85 L/s/1.82 ha = 28.50 L/s/ha
- Target Release Rate for 5-year storm event (Shell Site): 16.62 L/s/ha x 0.333 ha = 5.54 L/s
- Target Release Rate for 100-year storm event (Shell Site): 51.85 L/s/ha x 0.333 ha = 9.5 L/s

Table 6: Allowable Release Flow Rate

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

Note: **The value is extracted from the Functional Servicing and Stormwater Management Report (DESL, 201, see* ***Appendix B****).*

6.2.2 Control and Uncontrolled Areas

The proposed Shell Site 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 stormwater management evaluation and design, the site has been divided into uncontrolled and controlled areas as outlined on **Sheet C105.0** (refer to **Appendix G**). Runoff from Catchment Area A6-1 (0.007 ha of grassed area) and Catchment Area A7 (0.004 ha of access driveway to Fringewood Drive) are uncontrolled and sheet flow into the Fringewood Drive right-of-way.

Runoff from Catchment Areas A1 through A5, A8 (External Area) and A9 will be captured by the proposed catchbasin (CB) /catchbasin manhole (CBMH) throughout the site. Stormwater will then be conveyed to the new sewer system servicing the Shell Site and eventually be directed to the proposed 675 mm diameter concrete storm sewer located at Fringewood Drive. In order to capture the flow generated from Catchment Area A6-2, a CB and swale system are proposed that will collect the runoff and convey the flow via a 675 mm diameter storm sewer pipe connected to CBMH-03.

Therefore, the Controlled Area (0.322 ha) is represented by Catchment Areas A1 through A5 and A9 as well as External Area A8 and Uncontrolled Area (0.011 ha) is represented by Catchment Areas A6-1 and A7.

6.2.3 Proposed Quantity Control and Post-Development Restricted Flow

The stormwater management quantity control system will consist of detention storage within the oversized storm sewers proposed throughout the site. A restrictor (orifice) is proposed to be installed inside the downstream manhole to control the release rate with storage provided in the proposed oversize pipe systems as well as within the CB or CBMH structures, as needed. For the Shell Site, excess runoff from the 5-year design storm event can be stored underground to meet the target release rate. In addition to the underground storage provided, there is surface storage available at those CB and CBMH throughout the site where grading allows, if needed.

The calculated peak flow rates for the proposed site condition during 2 to 100-year storm events are summarized in **Table 7**. Detailed calculations are provided in **Appendix F**. It should be noted that, the peak flow was calculated using the Rational Method. For those storm events greater than 5 year, the adjustment factors applied to the Runoff Coefficients, as indicated in **Sections 2** and **5.2**, and were applied to account for the increase in runoff due to saturation of the catchment soils during less frequent storm events. As indicated in **Section 6.2.2**, the flow from Catchment Areas A6-1 and A7 will be uncontrolled and discharge via sheet flow to the Fringewood Drive right-of-way.

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	0.059	10.07	13.66	16.01	20.87	25.40	29.29
Catchment Area 2	0.106	19.34	26.24	30.76	40.09	48.80	52.62
Catchment Area 3	0.030	5.12	6.95	8.15	10.62	12.93	14.89
Catchment Area 4	0.031	5.06	6.86	8.05	10.49	12.77	14.71
Catchment Area 5	0.035	4.93	6.69	7.84	10.22	12.44	14.33
Catchment Area 6-1	0.007	0.45	0.61	0.71	0.93	1.13	1.30
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	0.011	2.11	2.87	3.36	4.38	5.33	5.46
Total	0.333	52.60	71.36	83.65	109.01	132.71	148.36

A comparison of existing and post-development peak flow rates at the Shell Site are 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	52.60	----
5-Year	30.41	71.36	5.54
10-Year	35.65	83.65	----
25-Year	42.24	109.01	----
50-Year	47.13	132.71	----
100-Year	52.12	148.36	9.5

*Note: *The calculation summary has been included in **Appendix F**.*

As indicated in **Table 8** above, the uncontrolled post-development peak flow rates are higher than existing peak flow, therefore it is required that the 2 to 100-year post-development peak flows be controlled to the target release flow rate as indicated in the approved *Functional Servicing and Stormwater Management Report* (DSEL, 2019). **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 events.

As indicated in **Section 6.2.2**, under the proposed development conditions there are uncontrolled areas (Catchment Areas 6-1 and 7) will sheet flow to Fringewood Drive. However, External Catchment Area A8 (0.027 ha), directed into the Shell Site by sheet flow, will be controlled with the proposed stormwater system.

Various options were evaluated to best control site outflow to the restricted rates noted in **Section 6.2.1**. It was determined that the best option is to control the flow from the site by using one (1) Vortech-type inlet control device (ICD) in combination with oversize storm pipe storage. The ICD is proposed to be located at the outlet pipe manhole MH01 to control the release of stormwater from subject site. The restriction of flow will allow storage of stormwater within the oversized storm sewers. The ICD proposed is a Hydrovex "VHV Vertical Vortex Flow Regulator" Vortech-type orifices, Unit 75VHV-1 (see **Appendix F** for details). The location of the ICD is indicated on **Sheet C105.0** in **Appendix G**).

The required storage volume is 132 m^3 to control the 100-year post-development flow rate to the target release rate of 9.17 L/s. This was calculated utilizing the Modified Rational Method and the calculations are presented in **Appendix F**. To determine the required storage volume within the Shell Site, the release rate of 9.17 L/s for the Controlled Areas (0.322 ha) was assigned to the Modified Rational Method for 100-year storm event:

- Release rate from Controlled Areas: 9.17 L/s (28.50 L/s/ha x 0.322 ha = 9.17 L/s)

Oversized pipes are proposed for the Shell Site to provide storage to meet the controlled release rate. In addition the site has been graded to provide some surface storage that is available, if needed (see **Section 6.1**). **Table 9** provides a summary of the available storage volume based on the proposed grading and storm sewer design and Vortech ICD controls (refer to **Appendix F** for detailed calculation). The total surface storage volume for the site was calculated based on the lowest catchbasin rim and the overland spill elevations (Refer to **Appendix F** for detailed calculation).

Table 9: Potential Available Storage Within the Site

Potential Storage	Storage Volume (m^3)
Oversized Pipe Storage	116.53
CBs/CBMHs Storage	73.39
Surface Depressions	20.54
Total	210.45

A PCSWMM model was used to evaluate the system and assess the hydraulic grade line and release rate at the site and is discussed in more detail in **Section 6.3**. The target release rate from the Shell Site is presented in **Table 8** and is 5.54 l/s and 9.5 l/s for the 5 and 100 year storm events, respectively. The results of the PCSWMM model (refer to **Table 10** and **Appendix H**) indicates that the the total release flow rate from the Shell Site (total controlled and uncontrolled) is 5.65 L/s and 8.29 L/s for 5-year and 100-year storm events, respectively. It should be noted that the release rate during the 5 year storm event is slightly higher than the targeted (5.65 l/s versus 5.54 l/s. respectively). The difference is 0.11 l/s during the 5 year storm event. During the 100 year storm event, the release rate is less than the target (8.29 l/s versus 9.5 l/s). For the entire site (Future Commercial Block plus Shell Site), the PCSWMM results indicate that the total release flow at the future Fringewood Drive storm sewer is less than targeted in the *Functional Servicing and Stormwater Management Report* (DESL, 2019) and the results are presented in **Table 11.** Taking into consideration that the target release rates are met for the total site (Future Commercial Block plus Shell Site) for both 5 and 100 year storm events and from the Shell Site during the 100 year storm event, the slight increase in release rate is considered negligible in the entire on-site and off-site systems.

Table 10: Comparison of Peak Flow for the Target Release Rate and Controlled Peak Flow under Proposed Conditions (Shell Site)

Return Period	Controlled Peak Flow from Shell Site (Outlet, MH-05 at Fringewood Drive) Proposed (L/s)	Target Release Flow Rate (Outlet MH-05 at Fringewood Drive) *** (L/s)
5-Year	5.65*	5.54
100-Year	8.29**	9.5

*Notes: *The value extracted from PCWMM model for 5-year storm events at the location of orifice plus flow generated from two uncontrolled areas of A6-1 and A7 calculated by Rational Method (4 L/s+0.61L/s+1.04L/s=5.65 L/s)*

*** The value extracted from PCWMM model for 100-year storm events at the location of orifice plus flow generated from two uncontrolled areas of A6-1 and A7 calculated by Rational Method (5 L/s+1.30 L/s+1.99 L/s=8.29 L/s)*

****Refer to Table 8 above.*

Table 11: Comparison of Peak Flow for the Target Release Rate and Controlled Peak Flow under Proposed Conditions (Future Commercial Block and Shell Site)

Return Period	Controlled Peak Flow from Commercial Block (Outlet, MH-05 at Fringewood Drive) Proposed * (L/s)	Target Release Flow Rate (Outlet MH-05 at Fringewood Drive) ** (L/s)
5-Year	29	30.26
100-Year	47	51.58

*Notes: *The value extracted from PCWMM model. The Future Commercial Block flow is included in this result.*

***The value extracted from Functional Servicing and Stormwater Management Report (DESL, 2019, see **Appendix B**).*

6.3 Proposed Stormwater System

A hydrodynamic model (PCSWMM) was developed for the hydraulic simulation 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 fully 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 oversized pipe storage.

6.3.1 Modelling Approach and Parameters

The approach involved the following major milestones:

- The following data utilized in the development of the model: Detailed survey information, Aerial photo, proposed 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 the 3 hour Chicago storm event using City's IDF information, for 2-year, 5-year and 100-year storm events.
- The pipe roughness coefficient of 0.013 was used in the Manning Formula.
- A tailwater time-depth/elevation curve was developed for Outfall (MH-05 located on Fringewood Drive with respect to the maximum HGL of 102.845 m at MH-05 assessed by DSEL (see to **Appendix C**, Table 3A and 3B of the *Stormwater Management Report for 5 Orchard Drive*, March 2020) to create an estimated tailwater condition from the storm sewer on Fringewood Drive.
- PCSWMM model parameters had defined on model based on Table 12 below as per 2012 City of Ottawa Sewer Design Guidelines:

Table 12: PCSWMM Model Parameters

Parameter	Value
Depression Storage for impervious areas	1.57 mm
Depression Storage for pervious areas	4.67 mm
Horton Infiltration Constant	Max Infiltration Rate (fc): 76.2 mm Min. Infiltration Rate (f0): 13.2 mm Decay Constant (k): 4.14 1/hr (equal to 0.00115 1/s)

- Average slope was taken from **Sheet C104.0** Site Grading Plan (**Appendix G**) for each catchment areas.
- Future Commercial Block was modelled assuming a maximum flows of 24.77 and 42.45 L/s for 5 and 100-year storm events, respectively, through the Shell Site and ultimately to MH-05 on Fringewood Drive (refer to **Table 13**). It should be noted that to accommodate the difference in flow from the external area during the 5 and 100 year storm events, two separate models were developed.

Table 13: Peak Flow from Future Commercial Block

Description	Results
Total Commercial Area	1.82 ha
Shell Site	0.333 ha
Future Area	1.82 ha - 0.333 ha = 1.49 ha
Commercial Flow 5-Year	16.626l/s/hax1.49=24.77 L/s
Commercial Flow 100-Year	28.489l/s/hax1.49=42.45 L/s

- Stage storage area for subcatchment areas where surface ponding is available (Catchment Areas A1, A2, A3, A4, A5 and A6-2) were included in the model (refer to **Appendix F**).
- Available storage within CBs and CBMHs were defined in model.
- The average flow length of each subcatchments was calculated for different potential paths that runoff could be directed from delineated areas to the catchbasin/catchbasin manhole/manhole. It should be noted that the subcatchment width was calculated automatically by PCSWMM based on the flow length information.

- A stress test evaluation of the site was undertaken using a 20% increase in the 100-year 3-hour Chicago storm.
- The rating curve for inlet control device proposed (Vertical Vortex Flow Regulators Unit 75VHV-1) was created and added to the model as a head-discharge curve (refer to **Appendix F**).
- Minor system losses accounted in the model used City of Ottawa Guidelines (OSDG) Appendix 7-A.9 and assumed no deflection or benching (refer to **Appendix F**).

6.3.2 Modelling Results:

Figure 2 to **Figure 13** presents the 2-year, 5-year and 100-year HGL profiles, respectively, that represent the underground pipe storage as well as surface storage, as simulated in PCSWMM. As shown, a combination of 1050 mm, 750 mm, 675 mm and 450 mm diameter pipes provide sufficient storage volume and the HGL remains below proposed surface grade. In addition, no ponding occurs during the 2. 5 and 100-year storm events with the proposed design and with the exception of the two small uncontrolled areas, the Shell Site is self-contained from the stormwater storage perspective.

Table 14: Proposed Storm Sewer Size

Pipe ID	Upstream Manhole ID	Downstream Manhole ID	Pipe Diameter (mm)	Roughness	Pipe Length (m)	Entry Loss Coeff.**	Exit Loss Coeff.**	Velocity* (m/s)	Flow* (L/s)
P1	CBMH13	CBMH01	1050	0.013	14.12	0.02	0.39	0.47	9
P2	MH09	CBMH01	1050	0.013	21.00	0.02	0.39	0.14	17
P3	CBMH01	CBMH02	1050	0.013	22.60	0.39	0.39	0.46	16
P4	CBMH02	CBMH03	1050	0.013	17.28	1.33	0.39	0.46	19
P5	DICB12	CBMH03	675	0.013	9.488	0.02	1.72	0.34	8
P6	CBMH06	MH07	750	0.013	26	0.02	1.33	0.74	35
P7	MH07	MH11	750	0.013	20.203	1.72	0.41	0.69	32
P8	MH11	CBMH02	1050	0.013	19.01	0.21	1.33	0.42	23
P9	CBMH03	MH01	1050	0.013	13.15	0.39	0.02	0.69	14
P10	MH01	OGS	450	0.013	3.29	0.02	0.02	0.52	5
P11	OGS	CMH201	675	0.013	3.84	0.02	1.33	0.41	5

*Note: *The velocity and flow simulated with PCSWMM for 100 Year Chicago using City of Ottawa IDF*
***Used City of Ottawa Guidelines (OSDG) Appendix 7-A.9 (refer to **Appendix E**)*

Figure 2: Proposed 2-Year Storm HGL (From CBMH13 to MH-05) – With SWM Control

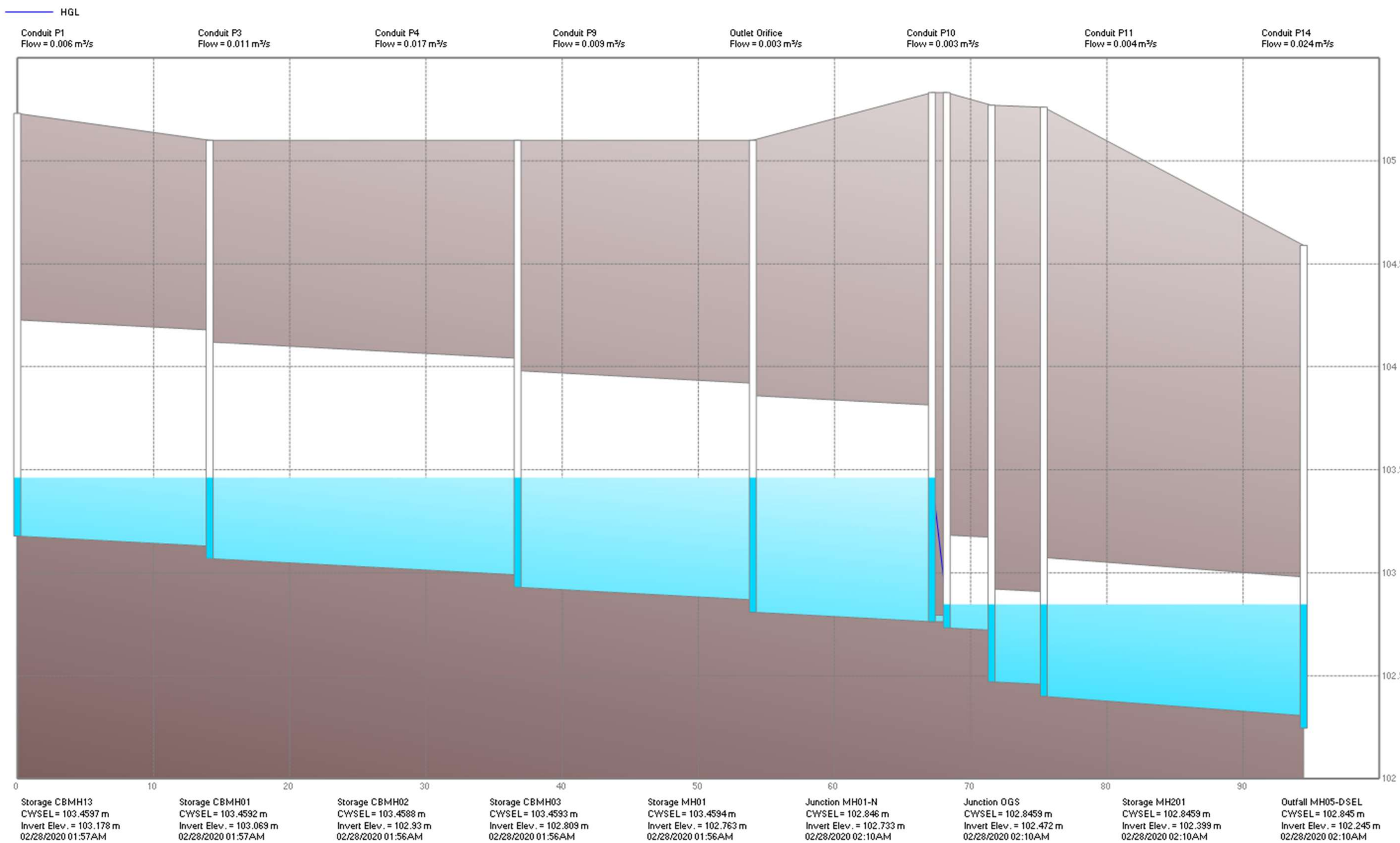


Figure 3: Proposed 2 - Year Storm HGL (From CBMH6 to MH-05) – With SWM Control

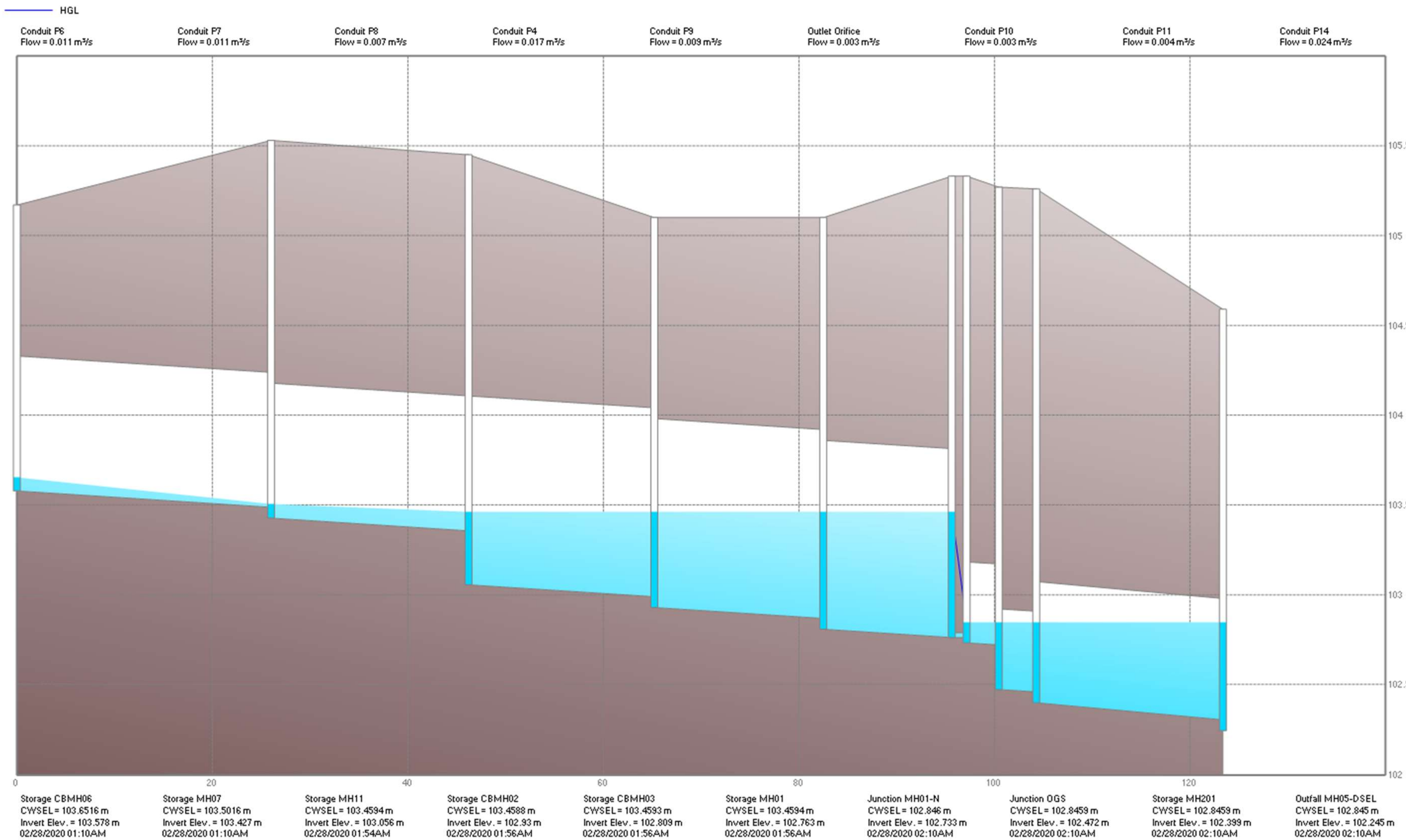


Figure 4: Proposed 2 - Year Storm HGL (From MH9 to MH-05) – With SWM Control

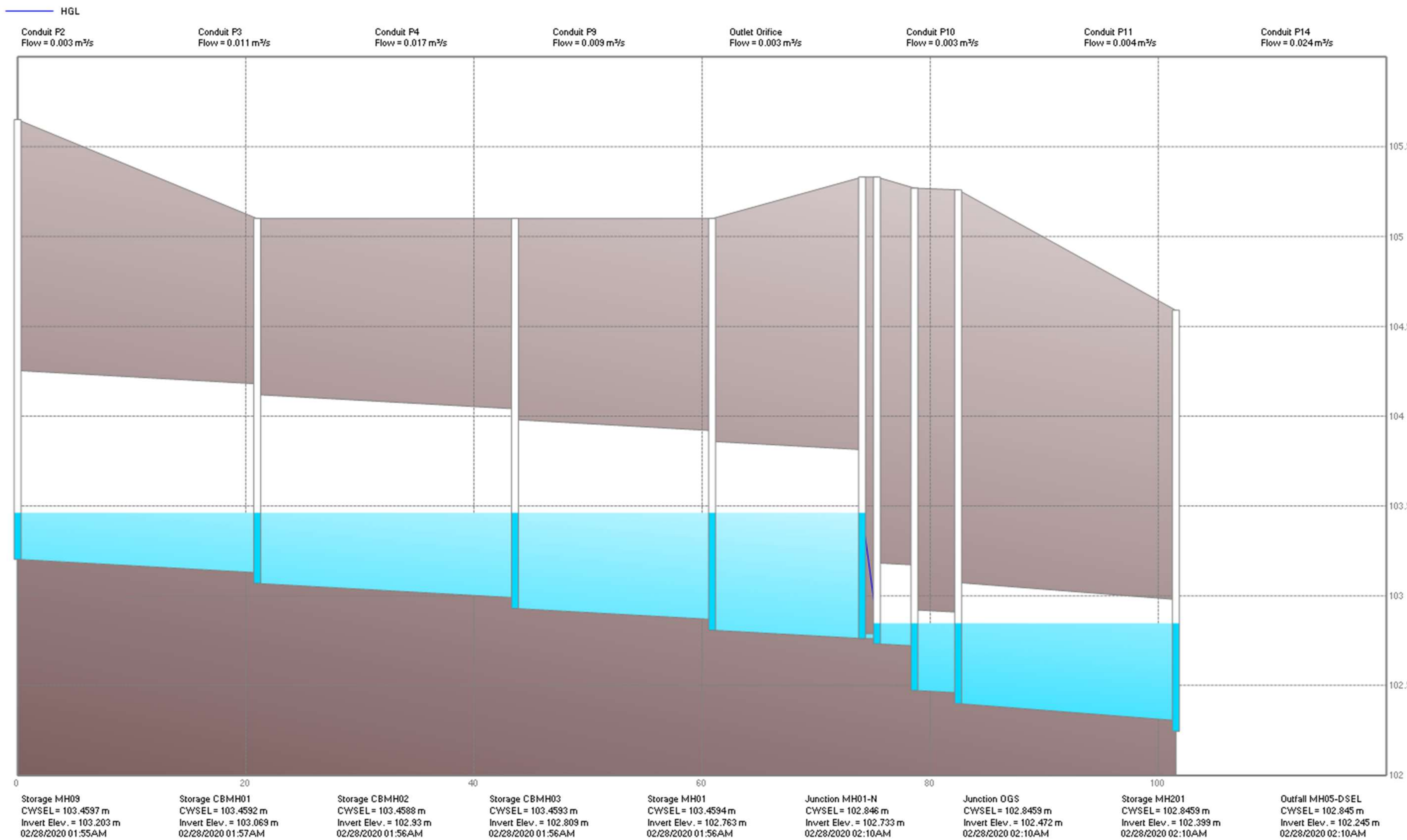


Figure 5: Proposed 2 - Year Storm HGL (From DICB12 to MH-05) – With SWM Control

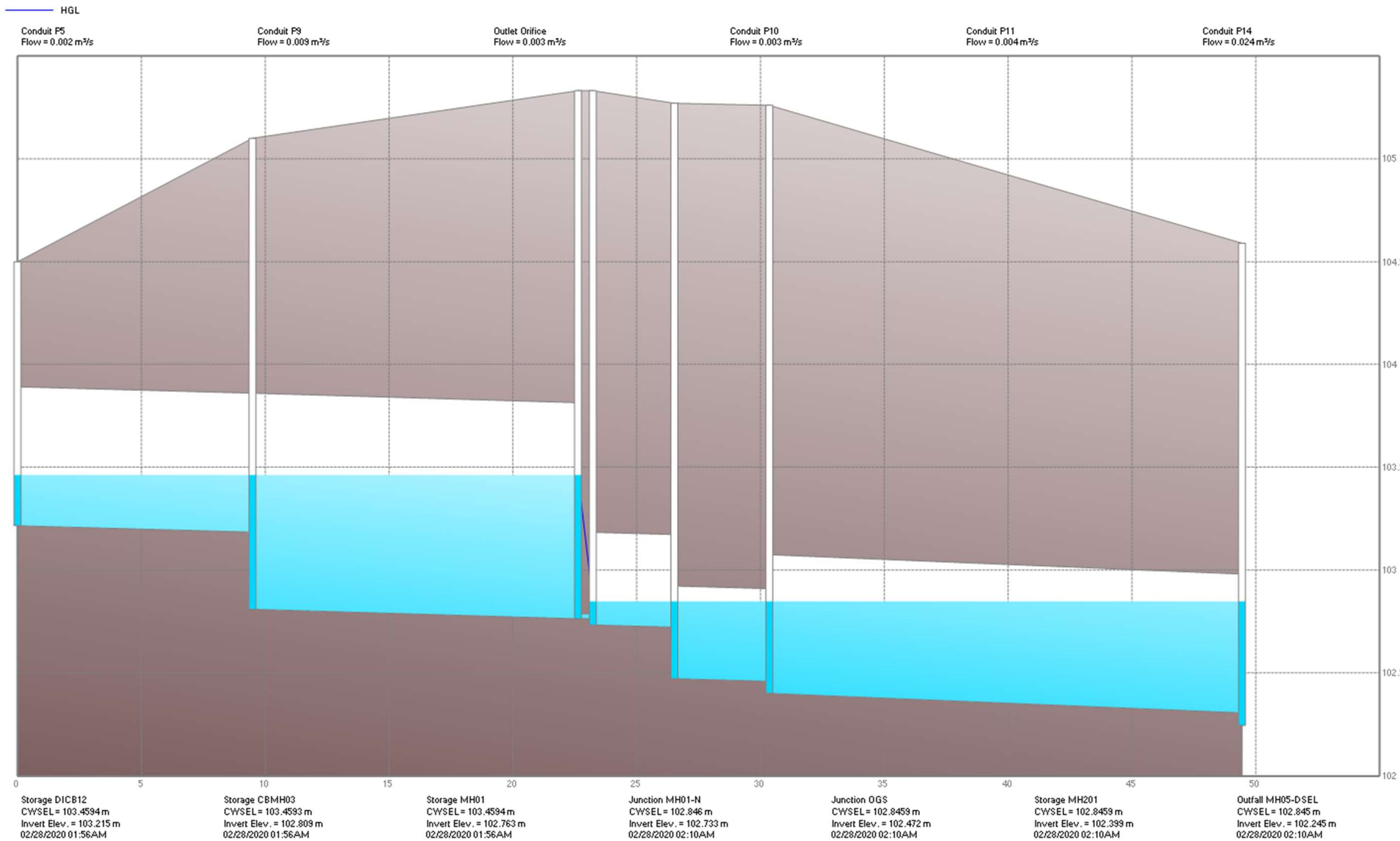


Figure 6: Proposed 5 - Year Storm HGL (From CBMH13 to MH-05) – With SWM Control

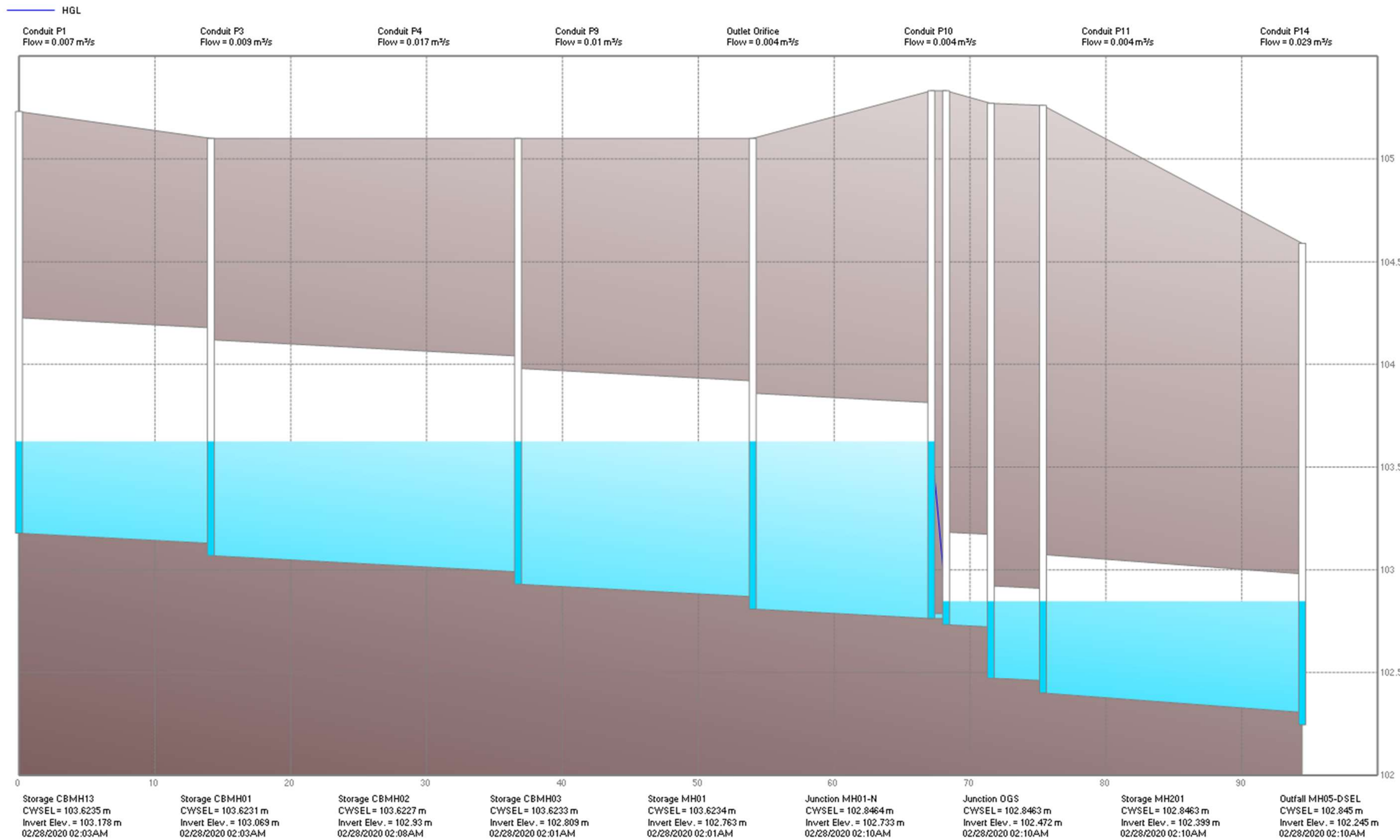


Figure 7: Proposed 5 - Year Storm HGL (From MH09 to MH-05) – With SWM Control

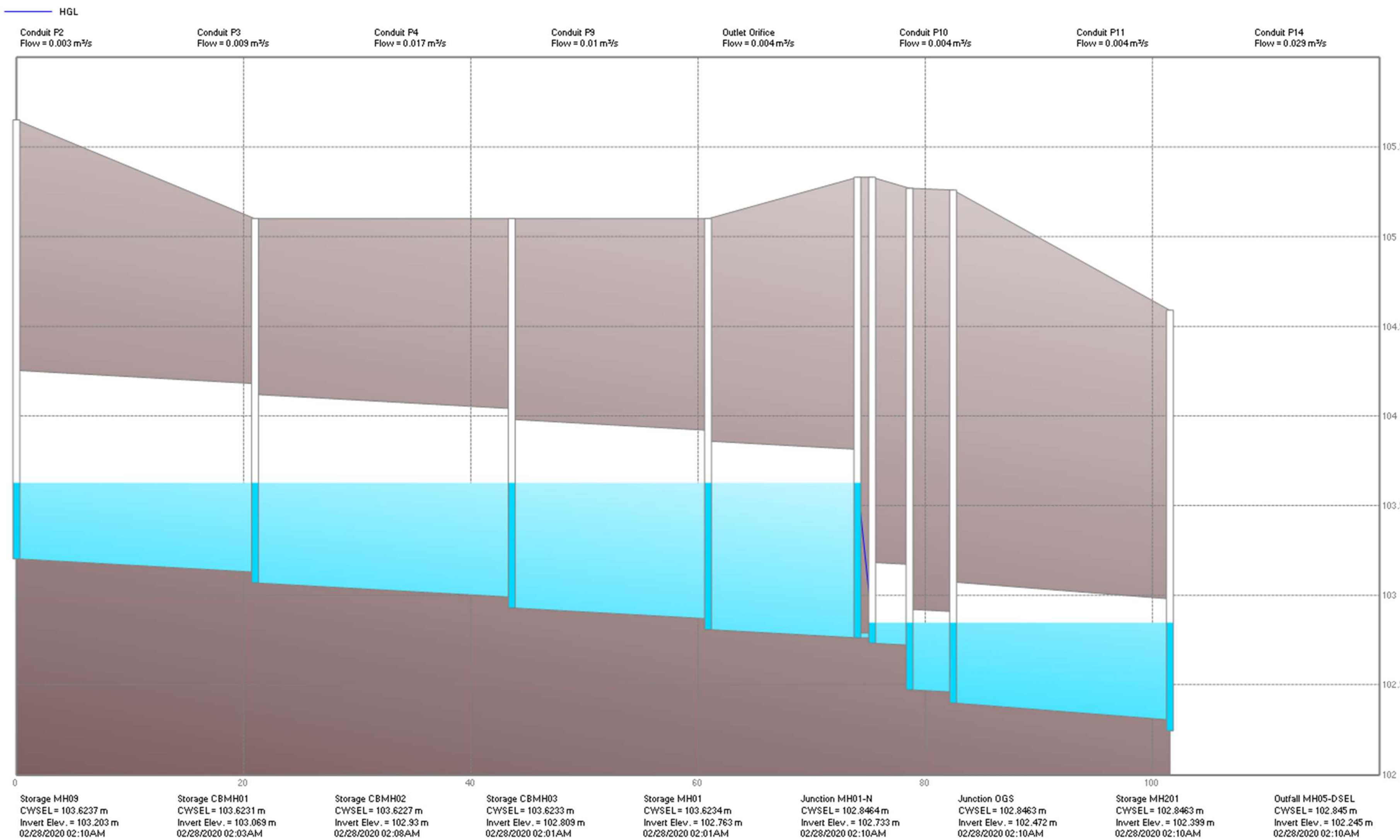


Figure 8: Proposed 5 - Year Storm HGL (From CBMH6 to MH-05) – With SWM Control

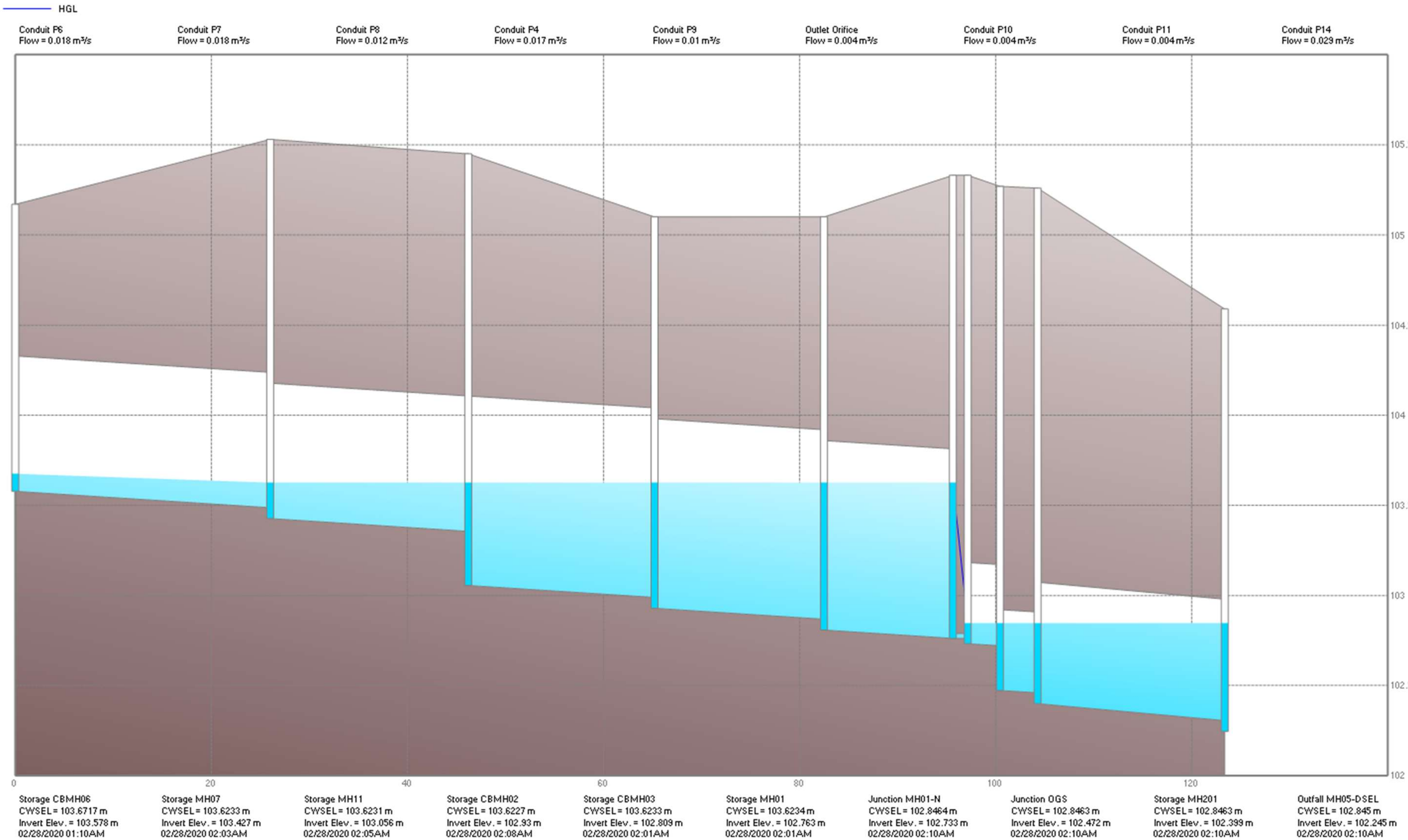


Figure 9: Proposed 5 - Year Storm HGL (From DICB12 to MH-05) – With SWM Control

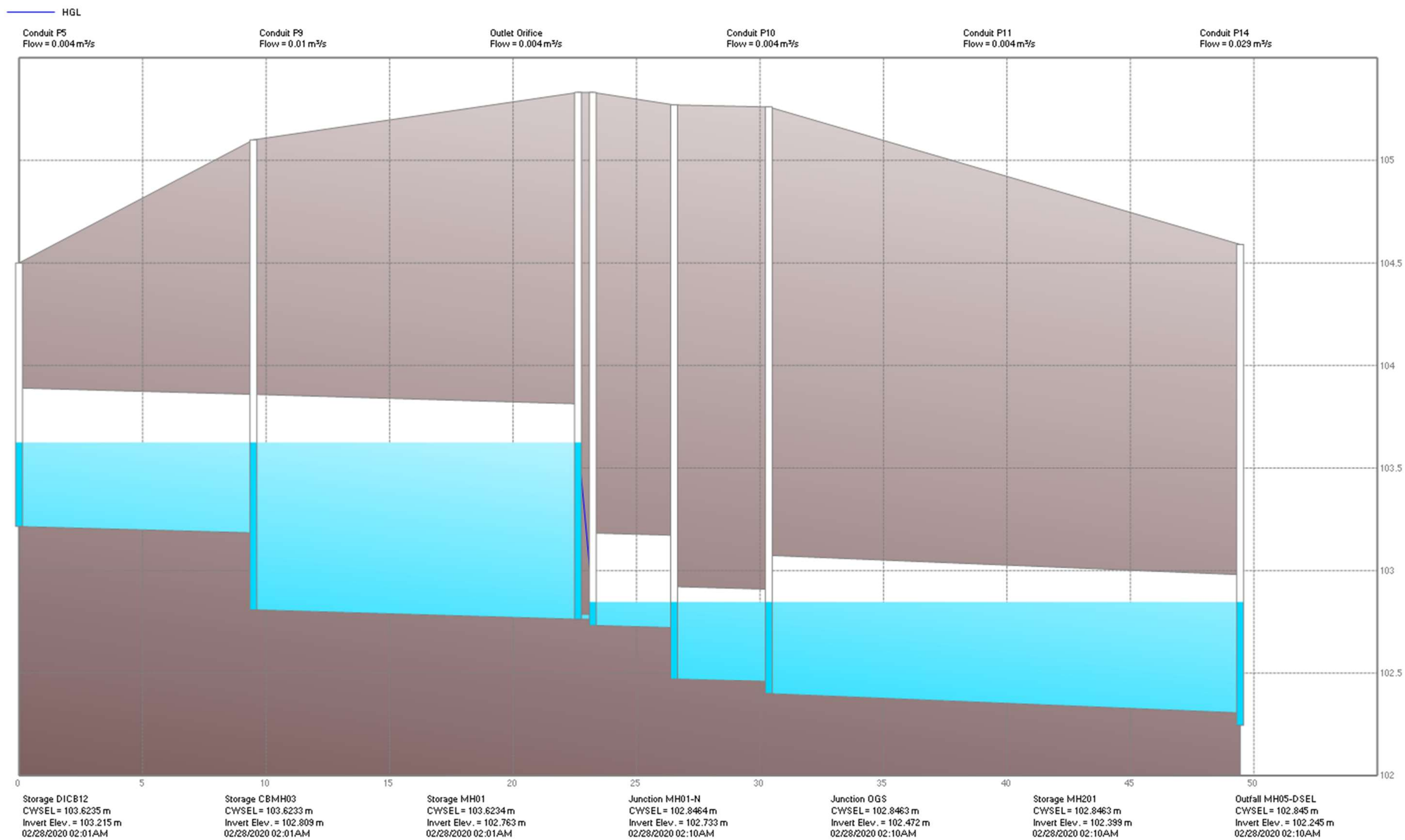


Figure 10: Proposed 100 - Year Storm HGL (From CBMH13 to MH-05) – With SWM Control

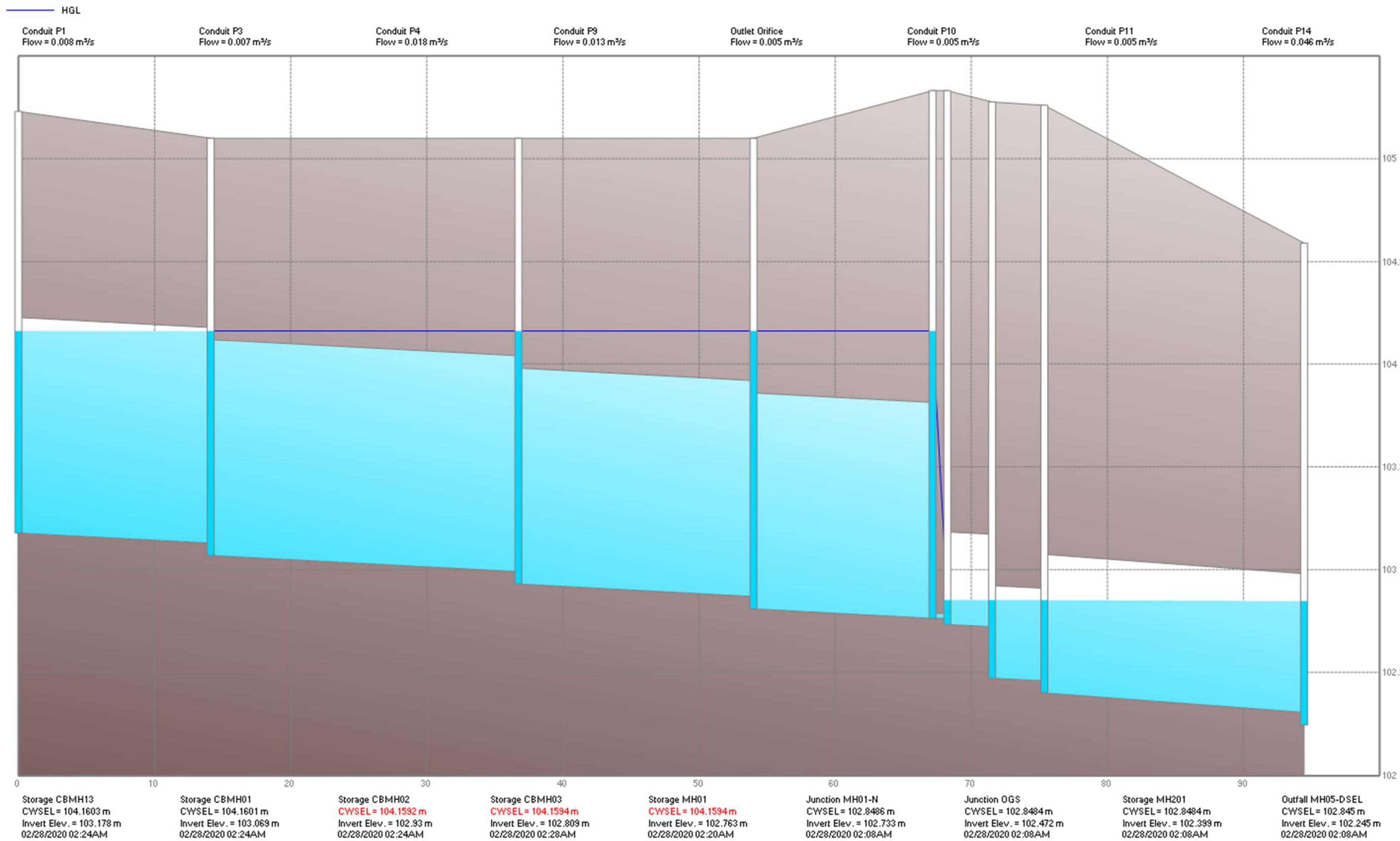


Figure 11: Proposed 100 - Year Storm HGL (From MH09 to MH-05) – With SWM Control

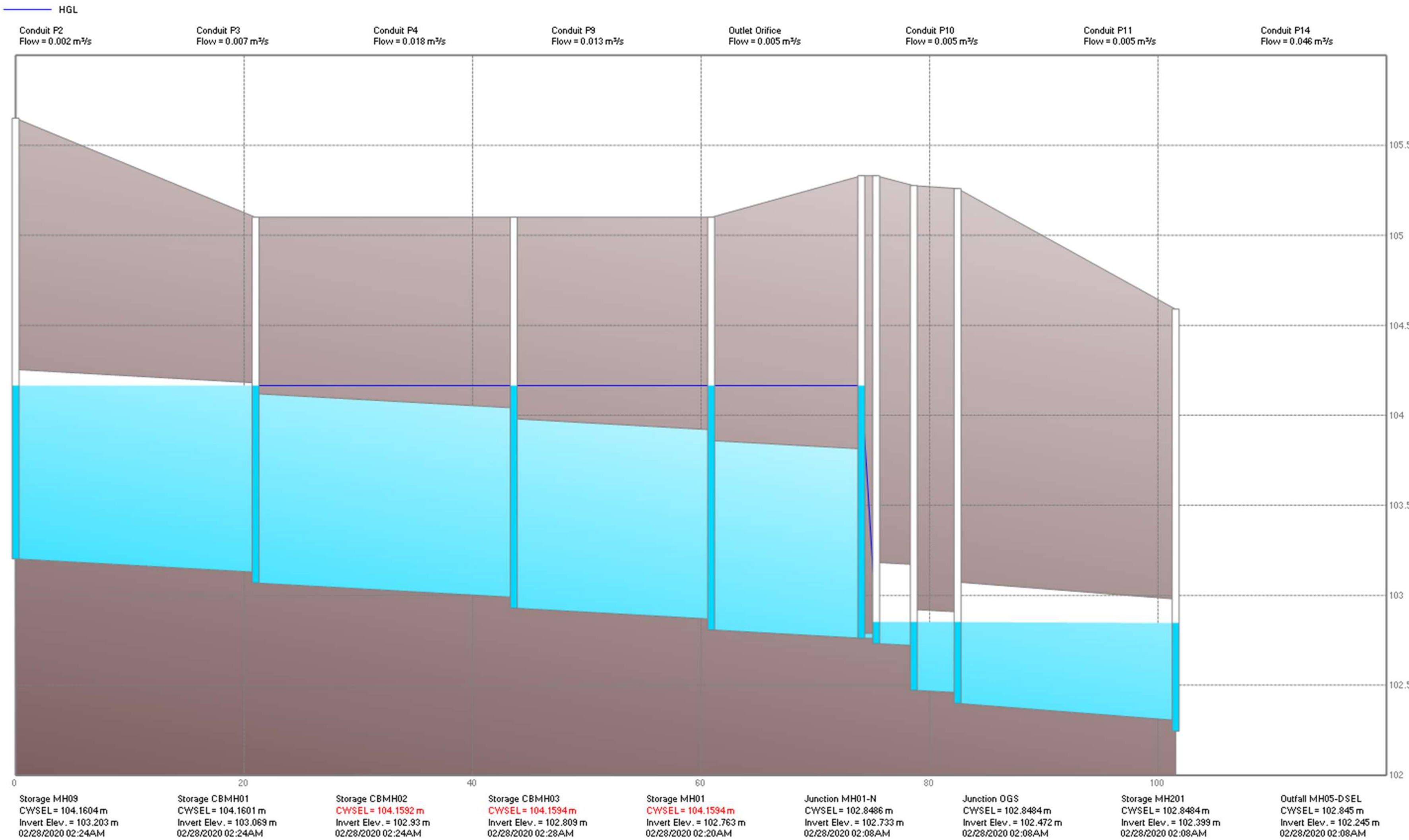


Figure 12: Proposed 100 - Year Storm HGL (From CBMH6 to MH-05) – With SWM Control

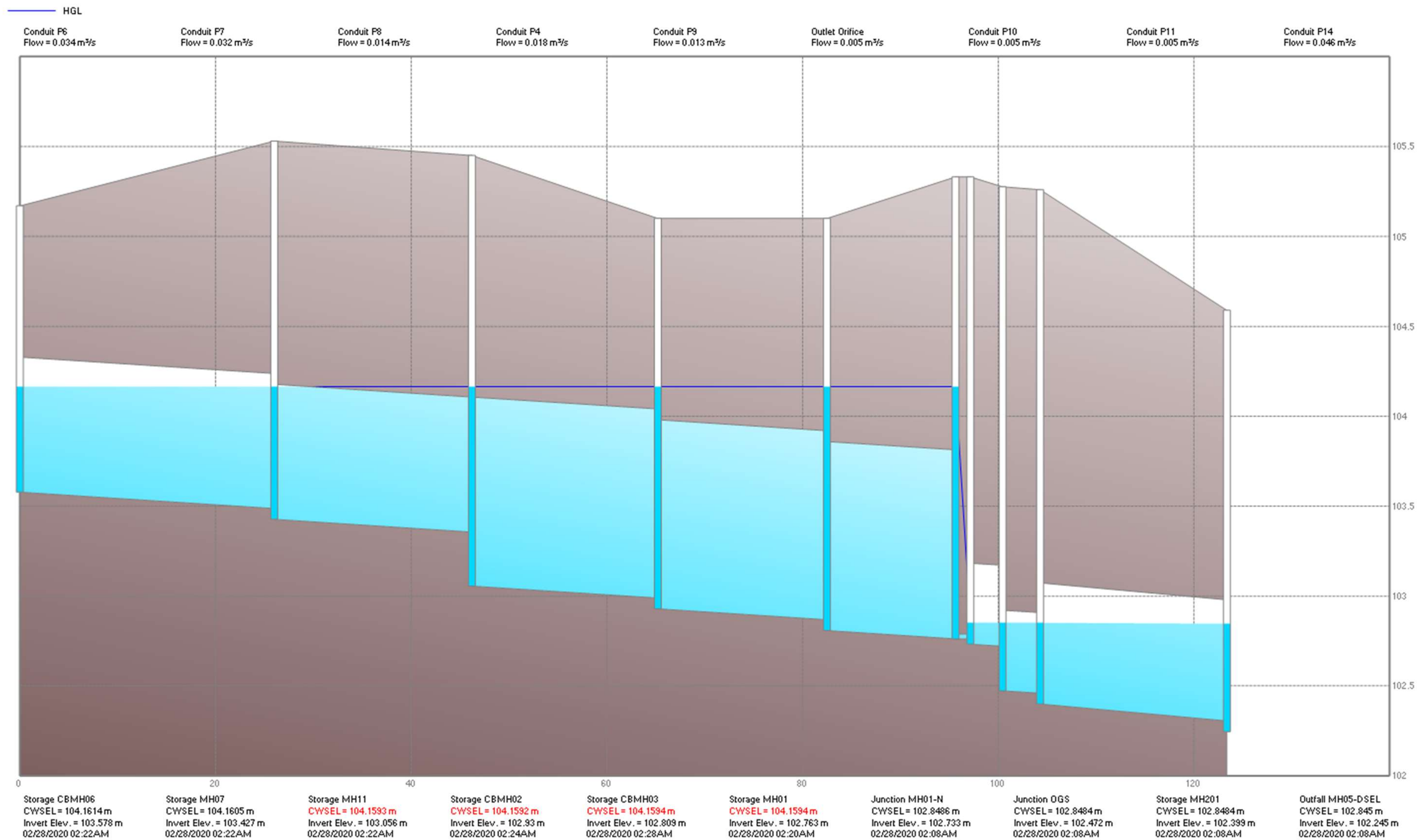
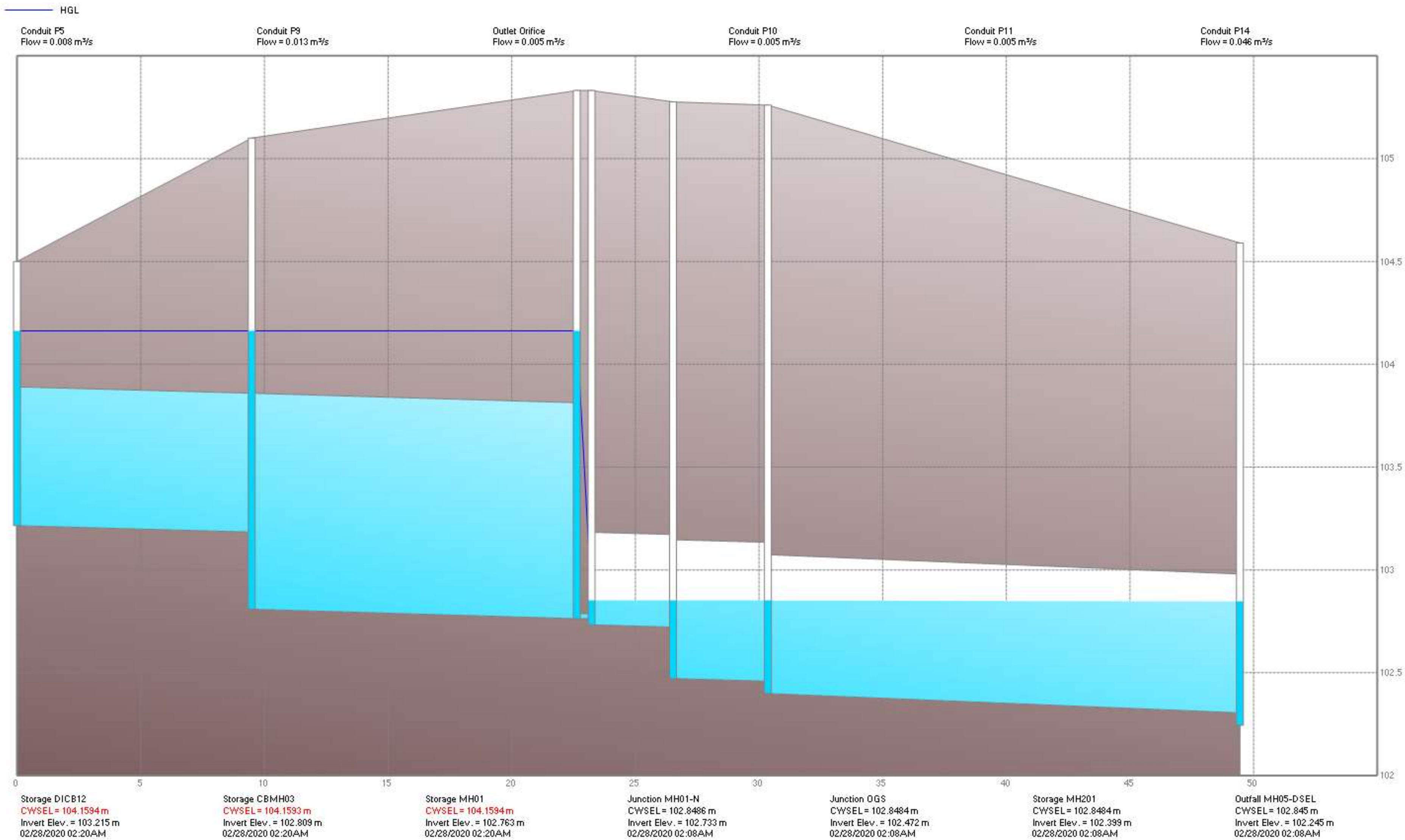


Figure 13: Proposed 100 - Year Storm HGL (From DICB12 to MH-05) – With SWM Control



In addition to the model results, **Table 15** demonstrates that the catchbasin grate capacity is sufficient for the 100 year catchment area runoff directed to it to be conveyed into the subsurface storm system.

Table 15: Ponding at Major Low Points for the 100-Year Storm

Name	Rim Elevation (m)	Ponding Depth* (m)	Q (m3/s)**	Max Convey*** Capacity (m3/s)	Spill
CBMH13	105.23	0.11	0.015	0.070	NO
CBMH01	105.1	0.13	0.029	0.100	NO
CBMH02	105.1	0.13	0.053	0.100	NO
CBMH03	105.1	0.13	0.014	0.100	NO
CB12	104.50	0.3	0.005	0.200	NO
MH201	105.26	0.06	0.015	0.020	NO

*Notes: *Refer to **Sheet C105.0, Appendix G**.*
***Refer to **Appendix F** for Calculation.*
**** Used City of Ottawa Guidelines (OSDG) Appendix 7-A.9, refer to **Appendix F**.*

As per the Ottawa Sewer Design Guidelines (OSDG), the maximum HGL will remain at 0.3 m below the underside of footing. With respect to the **Figure 10** the HGL at CBMH13 for 100-year storm event is 104.16 m, while the C-Store Finish Floor Elevation (FFE) is 105.45 m. Similarly, the HGL at CBMH6 for 100-year storm event is 103.58 m (refer to Figure 12), while the C-Store FFE is 105.65 m

6.4 Stress Test Results

In order to assess the climate change condition, the 100-year 3-hour Chicago storm was increased by 20% (stress test storm) was used to asses the impact on the proposed development site with its simulation in PCSWMM as per the October 2012 Ottawa Sewer Design Guidelines. The results indicate that at one location on the site where water will pond on the surface at CB12 (Catchment Area A6-2). The available surface storage is approximately 1 m^3 and the PCSWMM results indicate that 8 m^3 is required during the stress test storm. The balance of volume will sheet flow to the Fringewood Drive right-of-way. It should be noted that the off-site flow during the stress test storm does not include runoff from asphalt areas.

6.5 Water Balance

Both the City of Ottawa and the MVCA do not have any requirements for water balance for this area or this specific site. The purpose of this section is to assess the water balance calculation for both existing and proposed conditions on a best-efforts basis. With respect to the MECP *Stormwater Management Planning and Design Manual* (2003) Section 4.1.1, infiltration controls are not appropriate for applications with the potential for highly contaminated stormwater (e.g., industrial land uses) since there is a high potential for groundwater contamination and/or dry weather spills.

Considering the end-use of the site, several infiltration alternatives such as Permeable Pavements, Bioretention, Enhanced Grass Swale, and so on have not been considered to be appropriate for the site to meet the water balance. Given the subject site constraints, the only feasible LID measures include the absorbent landscape features. With respect to the MECP *Stormwater Management Planning and Design Manual* (2003) Section 4.1.1, infiltration controls are not appropriate for applications with the potential for highly contaminated stormwater (e.g., industrial land uses) since there is a high potential for groundwater contamination and/or dry weather spills.

Considering the end-use of the site, several infiltration alternatives such as Permeable Pavements, Bioretention, Enhanced Grass Swale, and so on have not been considered to be appropriate for the site to meet the water balance. Directing 'clean' rooftop drainage to pervious landscape surfaces was applied within the site to improve the water balance to the extent possible. Following this approach, the roof of the 'C-Store' and Carwash (with the exception of canopies) have been designed with rainwater leaders that splash onto the grassed/landscaped area. These areas then sheet flow to CBMH13 and CBMH6, respectively, and enter the site storm sewer system.

Water balance can be expressed in terms of inputs (precipitation (P)) and outputs (evapotranspiration (ET), runoff (R), and infiltration (I)).

A monthly average water balance approach was developed, utilizing monthly average climate information for Ottawa, ON (1981-2010), from the Federal Ministry of the Environment and Climate Change. Parameters such as average monthly temperature and total monthly precipitation were utilized to estimate the monthly heat index, potential evapotranspiration, daylight correction value and the surplus and deficit potential based on the Thornthwaite and Mather method (1957). If precipitation exceeds evapotranspiration and the excess is not used by plants, there is a surplus of water in soil moisture conditions. When evapotranspiration exceeds precipitation, there is a deficit of moisture, and recharge occurs until this deficit is recovered. There is typically a water surplus in the winter months which results in runoff and infiltration when melting and thaw occurs. From this information, it was determined that the annual water budget (based on average monthly data) results in 943.6 mm of precipitation, 482 mm of evapotranspiration (adjusted), including a surplus and deficit of 449 mm and 158 mm, respectively (refer to **Appendix F**). This equates to a total water surplus of 462 mm annually (includes both total annual infiltration + runoff). Furthermore, both the existing and proposed subdivision catchments were analyzed using on the MECP *Stormwater Management Planning and Design Manual* (2003) Table 3.1: Hydrological Cycle Component Values. The relative infiltration and runoff split is consistent with the values in this table. It should be noted that for the proposed condition, the External Catchment Area A8 was not considered part of the assessment.

Using the climate information and derived parameters summarized above, both existing and proposed conditions water balance was evaluated, compared and presented in **Table 16.**

Table 16: Comparison of Water Balance Calculation for Existing, Uncontrolled Proposed and Controlled Proposed Conditions

Parameter	Existing Conditions		Uncontrolled Proposed Conditions		Controlled Proposed Conditions*** (Roof Downspout Disconnection)	
	(mm)	(m^3)	(mm)	(m^3)	(mm)	(m^3)
Rainfall	944	2,900	944	2,890	944	2,890
Evapotranspiration *	482	1,500	113	350	113	350
Infiltration **	231	700	54	170	125	380
Runoff	231	700	776	2,370	705	2,160
Total	944	2,900	944	2,890	944	2,890

*Notes: * Evapotranspiration assumed to be across the subject site.*

***External Catchment Area A8 was not considered for the assessment of water balance in proposed conditions.*

****Assumes 'clean' rooftop runoff (15 mm of daily retention depth, accounts for approximately 82% (refer to **Figure 15**) of the total average annual rainfall depth from the roof areas of the C-Store and Carwash) to be infiltrated.*

Figure 14: Comparison of Water Balance Calculation for Existing, Proposed (Uncontrolled) and Proposed (Roof Downspout Disconnection) Conditions

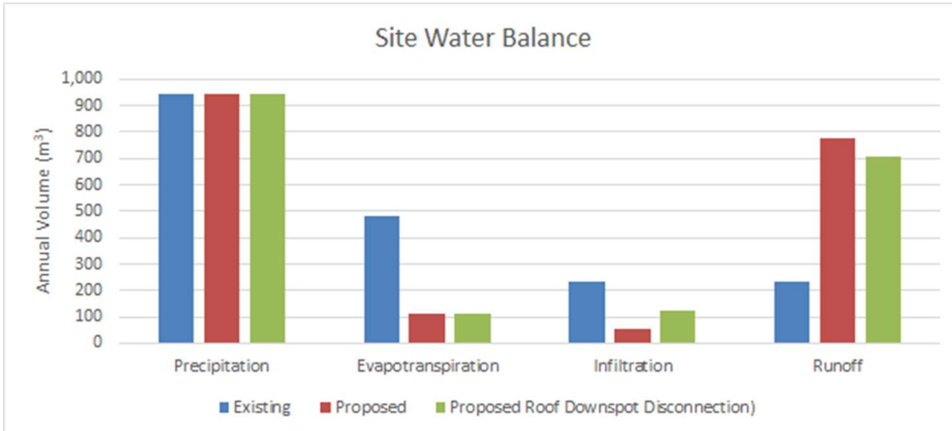
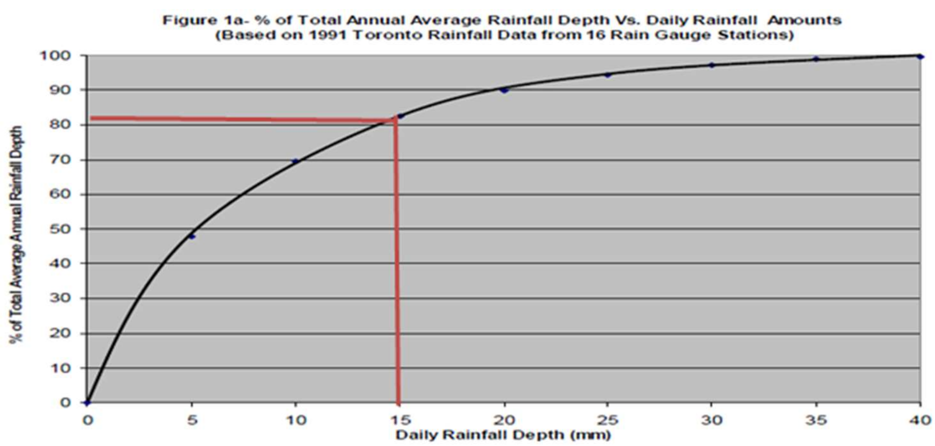


Figure 15: Figure 1a from City of Toronto Wet Weather Flow Guidelines (2006)



As shown in **Table 16** and **Figure 14** roof downspout disconnection will improve infiltration within the Shell Site . This is the best appropriate management practice that can be applied to the Shell Site considering its size and use.

6.6 Water Quality

The water quality target for the project is the long-term average removal of 80% Total Suspended Solids (TSS) on an annual loading basis from runoff leaving the site as per MVCA . As shown in **Sheet C 103.0** (refer to **Appendix G**), a filter type Oil Grit Separator model ADS UP FLOW FILTER (UFF-5) with five filters (or approved equivalent) is proposed. This unit will provide a volume capacity of 1,000 L for oil storage and it has been sized to provide Enhanced Level of Treatment for the 0.322 ha of the Shell Site. As per the manufacturer, 1000 L capacity to be met via additional baffle design.
The particle size distribution used and provided by the manufacturer meets the requirement of Procedure for Laboratory Testing of Oil-Grit Separators prepared by Toronto and Region Conservation Authority as shown in **Table 17**.

Table 17: Particle Size Distribution of Test Sediment (as per Table 1 of Procedure for Laboratory Testing of Oil-Grit Separators)

Particle Size (μm)	Percent Less Than	Particle Size Fraction (μm)	Percent
1000	100	500-1000	5
500	95	250-500	5
250	90	150-250	15
150	75	100-150	15
100	60	75-100	10
75	50	50-75	5
50	45	20-50	10
20	35	8-20	15
8	20	5-8	10
5	10	2-5	5
2	5	<2	5

As indicated in previous sections, all upstream areas where there is a potential for contamination will be directed to this OGS unit for treatment (refer to **Appendix F**). This proposed water quality treatment unit will be located downstream of MH01 (see **Sheet C103.0, Appendix G**). The only exceptions are Catchment Areas A6-1 and A7 that cannot be directed to the OGS unit. All other areas, including External Catchment Area A8 (mostly green area) will be directed to the filter type OGS unit (0.333 ha – 0.004 ha – 0.007 ha = 0.322 ha).
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For those uncontrolled areas from the Shell Site, both areas are isolated from the remainder of the site (no surface flow from potentially contaminated areas is conveyed to or through the area via surface flow). As noted in the *Functional Servicing and Stormwater Management Report for Campanale Homes Development* (DSEL, 2019 in **Appendix B**), runoff from these areas into the Fringewood Drive right-of-way and captured in the storm sewer will eventually be conveyed to downstream stormwater quality treatment facilities. On an interim basis, the Fringewood Drive storm sewer flow is conveyed to the downstream interim Hazeldean Stormwater Facility. Ultimately, the interim pond will be replaced with two oil grit separators. Therefore, water quality treatment is provided for these uncontrolled areas from the Shell Site in the downstream recipient system. **Table 18** presents a summary of the proposed filter type oil grit separator for water quality control for the Shell Site. As indicated below, the average annual TSS removal efficiency achieved for the proposed site is 80%.

Table 18: Summary of Proposed Oil and Grit Separator

Item	Specification
Model	ADS UP FLOW FILTER (UFF- 5)
Net Annual TSS Removal Efficiency	82.6
Sediment Capacity (L)	1,580
Oil Capacity (L)	1,000
Total Holding Capacity (L)	2,580
Diameter of Outlet Pipe (mm)	450
Number of Filter Modules	5
Rated Treatment Flow Rate (L/s)	8

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7. Temporary Storm Works Required

As noted in **Section 1.1**, the entire commercial site is serviced by two sets of ditch inlet catchbasins (DICBs). Their locations are indicated on **Sheet C103.0** presented in **Appendix G**. Briefly, one DICB is located on the subject property and the other is located to the west outside the subject site. As part of the proposed works for the subject site, the DICB is proposed to be abandoned by being filled with sand and concrete and left in place (see Sheet C103.0, **Appendix G**) once it is no longer required as a temporary outlet during construction. The western DICB is proposed to remain in place to continue to service the remaining undeveloped portion of the commercial site.

The western lease boundary of the subject site is the high point where all flow west of this high point discharges by sheet flow into the future commercial area site. The runoff from the undeveloped portion of the commercial site (west of the subject site) will continue to flow northeast toward and adjacent to the subject site and its future Hazeldean Road entrance. The subject site and entrance will be raised above existing ground and the interface tie-in on the west of side will be sloped at 3:1 into the future commercial site (see **Sheet C104.0** presented in **Appendix G**). Since the existing topography of the future commercial site is also to the north, the existing topography will convey any rural runoff toward the west existing DICB (see **Sheet C104.0, Appendix G**).

As noted in **Section 1.1**, the entire commercial site (Future Commercial Block and Shell Site) are limited to 28 l/s/ha outflow during the 100 year storm event. For the remaining undeveloped portion of the site (1.82 – 0.33 ha = 1.49 ha), the restricted flowrate is 42 l/s during the 100 year event. Using the modified rational method, it was determined that 202 m^3 of storage volume is required during the 100 year event to limit outflow from the site to 42 l/s. Using the information related to the existing westerly DICB from the Functional Serviceability and Stormwater Report (DSEL, 2019, referred to as DICB1 in Table 8 of that report, see **Appendix B**) and the stage-area curve based on existing and adjusted grading for external future commercial site under interim conditions, it was determined an 139 mm diameter orifice is required in the DICB to limit outflow. Supporting calculations are provided in **Appendix E**.

It is anticipated that the interim runoff conveyance to the west DICB will be in place until the future commercial site is constructed. The timing of this is expected to occur after the subject site has been constructed and is operational.

8. Sediment and Erosion Control

The sediment and erosion control proposed for the construction of the subject site is discussed and documented in a separate report entitled *Site Servicing Report 5 Orchard Drive, Ottawa Hazeldean Road and Fringewood Drive* (AECOM, 2020). For any sediment and erosion control discussion and plans the above noted report should be referred. A copy of the report accompanies the application package.

9. Conclusion

This report has demonstrated that the proposed storm sewer sizing, site drainage and stormwater management are consistent with the requirements of the City of Ottawa. The findings of this study are summarized as follows:

- The storm sewer is designed to convey the 5-year post development flows from the Shell Site. The proposed internal sewer system will connect the future storm sewer on Fringewood Drive. The timing of the installation of that storm sewer is expected prior to the development of the Shell Site.
- Dedicated stormwater systems will collect all runoff from all the catchment areas for 2 to 100-year storm events with the exception of Catchment Areas 6-1 and 7 that discharge uncontrolled via sheet flow into the Fringewood Drive right-of-way.
- Provide one (1) – Vortech-type ICD (Unit 75VHV-1) to control 5 and 100-year post-development flows to meet the target release rate 30.26 L/s and 51.85 L/s with considering Future Commercial Block (1.8 ha) as identified in **Table 10** and **Table 11**. The required storage will be provided in the oversized underground storm sewer system.
- The 5 to 100-year post-development flows from the Shell Site will be controlled with on-site water quantity controls. It should be noted that the release rate during the 5 year storm event is slightly higher than the targeted (5.65 l/s versus 5.54 l/s. respectively). The difference is 0.11 l/s during the 5 year storm event. During the 100 year storm event, the release rate is less than the target (8.29 l/s versus 9.5 l/s). Taking into consideration that the target release rates are met for the total site (Future Commercial Block plus Shell Site) for both 5 and 100 year storm events and from the Shell Site during the 100 year storm event, the slight increase in release rate is considered negligible in the entire on-site and off-site systems.
- The result from the PCSWMM evaluation indicate that the proposed oversize pipes and storage available within CBMHs/MHs provide sufficient storage for the site with the outflow restrictions and the HGL remains below the MH rims for the 2, 5 and 100 year storm events.
- There is available surface storage throughout the site to depths between 0.06 m to 0.3 m above the catchbasin grates. However based on the PCSWMM results, there is no ponding occurring on the surface for the 2 to 100-year storm events at the site.
- An OGS model ADS UP FLOW FILTER (UFF- 5) with 5 filters (or approved equivalent) will provide Enhance Level of Protection for water quality for the Shell Site. It will provide 80% TSS removal (refer to **Appendix F**). In general, the Enhanced Level of Protection will be provided for the total Shell Site area with the exception of Catchment Areas A6-1 (grass area located on the back of Convenience Store) and A7 (access portion of the entrance from Fringewood Drive). The *Functional Servicing and Stormwater Management Report* (DSEL, 2019 in **Appendix B**) noted that storm sewer flow from Fringewood Drive is eventually conveyed to a downstream interim stormwater pond that will eventually be replaced by two oil grit separators. Therefore, water quality treatment is provided for these uncontrolled areas from the Shell Site in the downstream recipient system.
- During the stress test storm event (100-year + 20%), there is flooding at one location on the site (CB12) that will store on the surface before conveyed by sheet flow to the Fringewood Drive right-to-way.
- The proposed storage (underground and surface) for the Shell Site will be able to capture the 100-year storm event for all the controlled areas with no runoff to the adjacent properties and municipal right-of-way.

- During the 100 year storm event, the maximum HGL is a minimum of 0.3 m below the underside of footing.
- There is 0.22 m of vertical clearance between the spill elevation on street and the ground elevation at the C-Store building envelope which is greater than the minimum of 15 cm required.

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

GEMTEC
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

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1.0 INTRODUCTION

1.1 General

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

1.2 Project and Site Description

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

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

2.0 SUBSURFACE INVESTIGATION

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

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

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

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

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

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

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

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

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

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

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

3.0 SUBSURFACE CONDITIONS

3.1 General

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

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

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

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

3.2 Topsoil

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

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

3.3 Silt

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

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

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

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

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

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

3.4 Glacial Till

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

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

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

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

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

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

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

3.5 Bedrock

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

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

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

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

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

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

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

3.6 Groundwater Levels

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

Table 3.4 – Groundwater Level – June 10, 2019

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

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

3.7 Soil Chemistry Relating to Corrosion

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

Table 3.5 – Summary of Corrosion Testing - Soil

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

4.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

4.1 General

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

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

4.2 Overburden Excavation

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

4.3 Bedrock Excavation

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

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

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

4.4 Groundwater Pumping

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

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

4.5 Site Grade Raise Restrictions

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

4.6 Foundation Design

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

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

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

Table 4.1 – Foundation Bearing Pressures

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

Notes:

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

4.7 Frost Protection of the Foundations

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

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

4.8 Foundation Backfill and Drainage

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

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

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

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

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

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

4.9 Slab on Grade Support (Heated Areas Only)

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

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

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

4.10 Seismic Design of Proposed Structures

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

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

5.0 PROPOSED UNDERGROUND FUEL STORAGE TANKS

5.1 Excavation and Groundwater Pumping

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

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

5.2 Bedding

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

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

5.3 Backfill

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

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

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

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

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

Table 5.1 – Backfill Earth Pressure Parameters

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

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

5.4 Buoyant Uplift of Tanks

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

6.0 SITE SERVICES

6.1 Overburden Excavation

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

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

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

Additional comments on overburden excavation are provided in Sections 4.2.

6.2 Bedrock Excavation

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

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

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

6.3 Groundwater Pumping and Management

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

6.4 Pipe Bedding

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

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

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

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

6.5 Trench Backfill

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

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

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

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

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

6.6 Seepage Barriers

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

7.0 ACCESS ROADWAY AND PARKING AREAS

7.1 Subgrade Preparation

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

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

7.2 Flexible Pavement Structures for the Parking Areas and Access Roadway

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

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

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

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

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

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

7.3 Compaction Requirements

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

8.0 ADDITIONAL CONSIDERATIONS

8.1 Corrosion of Buried Concrete and Steel

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

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

8.2 Effects of Construction Induced Vibration

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

8.3 Winter Construction

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

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

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

8.4 Excess Soil Management Plan

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

8.5 Design Review and Construction Observation

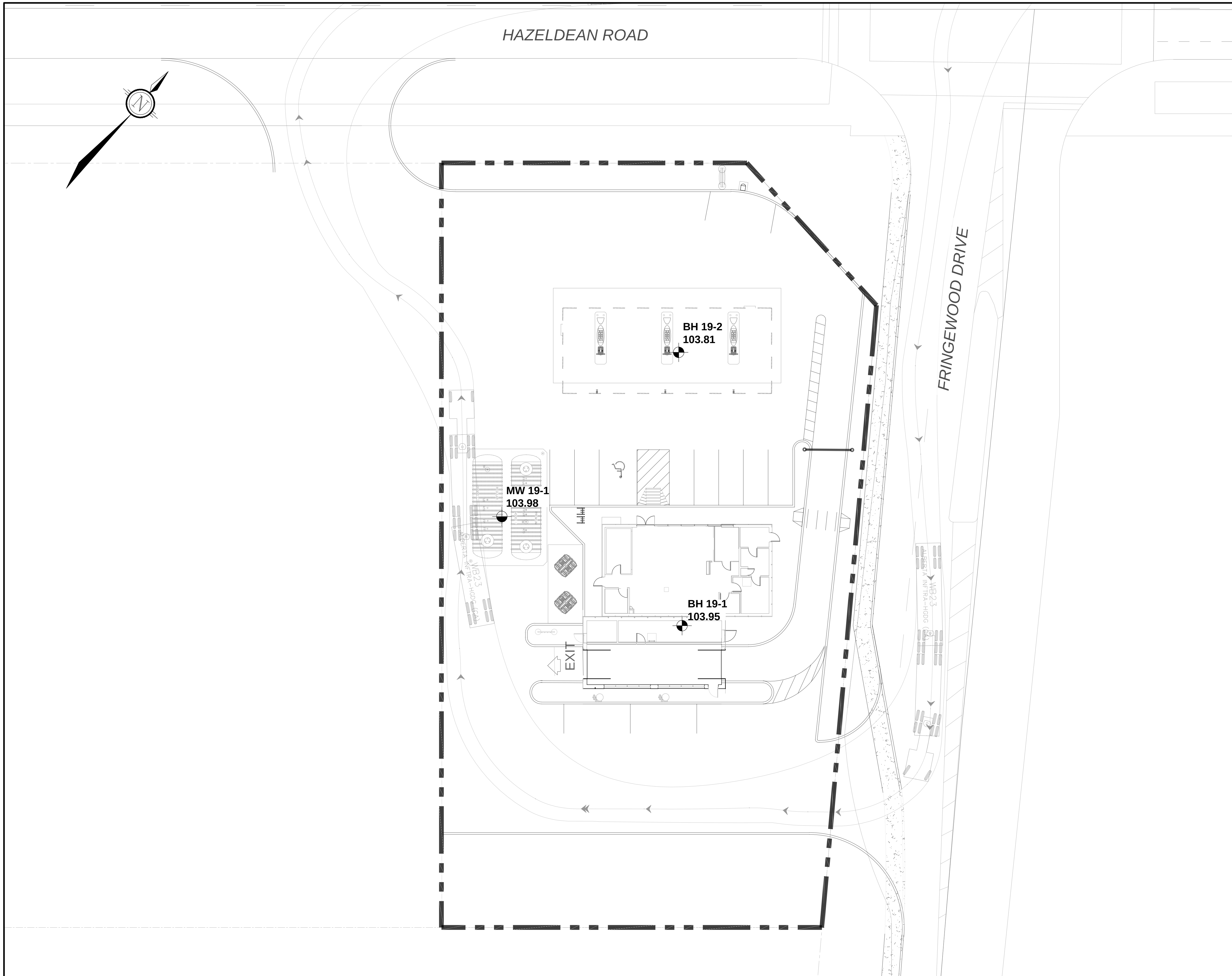
The details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the final design drawings be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

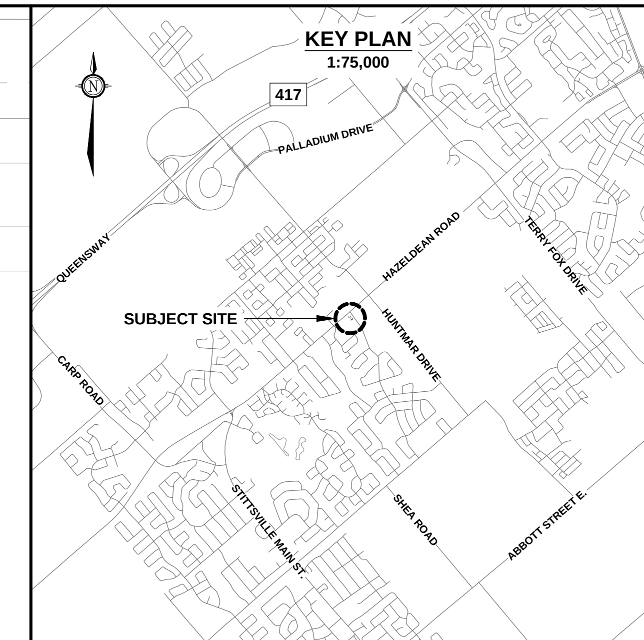
The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent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

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

Luc Bouchard, P.Eng., ing.

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





LEGEND

BOREHOLE LOCATION IN PLAN (current investigation by GEMTEC)

MONITORING WELL LOCATION IN PLAN (current investigation by GEMTEC)

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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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

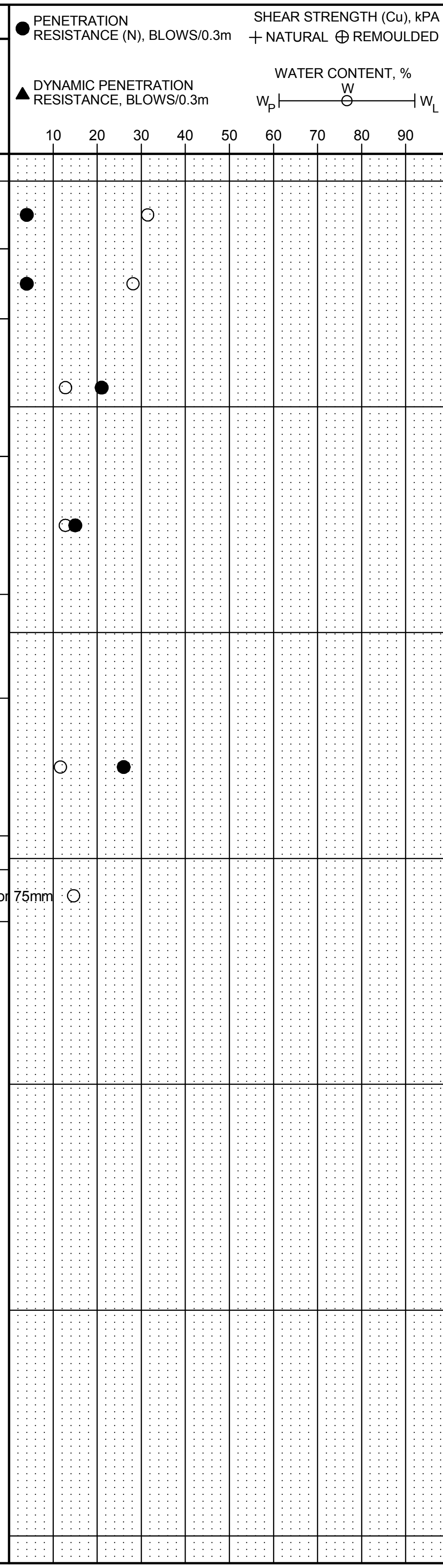
Record of Borehole Sheets
Results of Laboratory Classification Testing
List of Abbreviations and Terminology
Lithological and Geotechnical Rock Description Terminology

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	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Hydrovacuum	Ground Surface		103.95						
		Dark brown clayey silt (TOPSOIL)								Backfilled with auger cuttings
		Very loose, brown SILT, some clay, trace sand		103.80 0.15	1a	SS	350	4		
					1b	SS	350	4	MH	
1		Compact, gravelly silty sand, trace clay, cobbles and boulders (GLACIAL TILL)		103.05 0.90	2	SS	450	21		
	Power Auger, Hollow Stem Auger (210mm OD)				3	SS	450	15		
2										
					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										



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LOGGED: ML
CHECKED: LB

RECORD OF BOREHOLE 19-2

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

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

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

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

ADDITIONAL LAB. TESTING

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CHECKED: LB

RECORD OF BOREHOLE MW19-1

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

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

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	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		Compact, brown gravelly silty sand, trace clay, cobbles and boulders (GLACIAL TILL)		2	SS	350	16		
				3	SS	450	23	MH	Bentonite
2	Power Auger, Hollow Stem Auger (210mm OD)			5	SS	25	50 for 25mm		
			101.08 / 2.90						
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

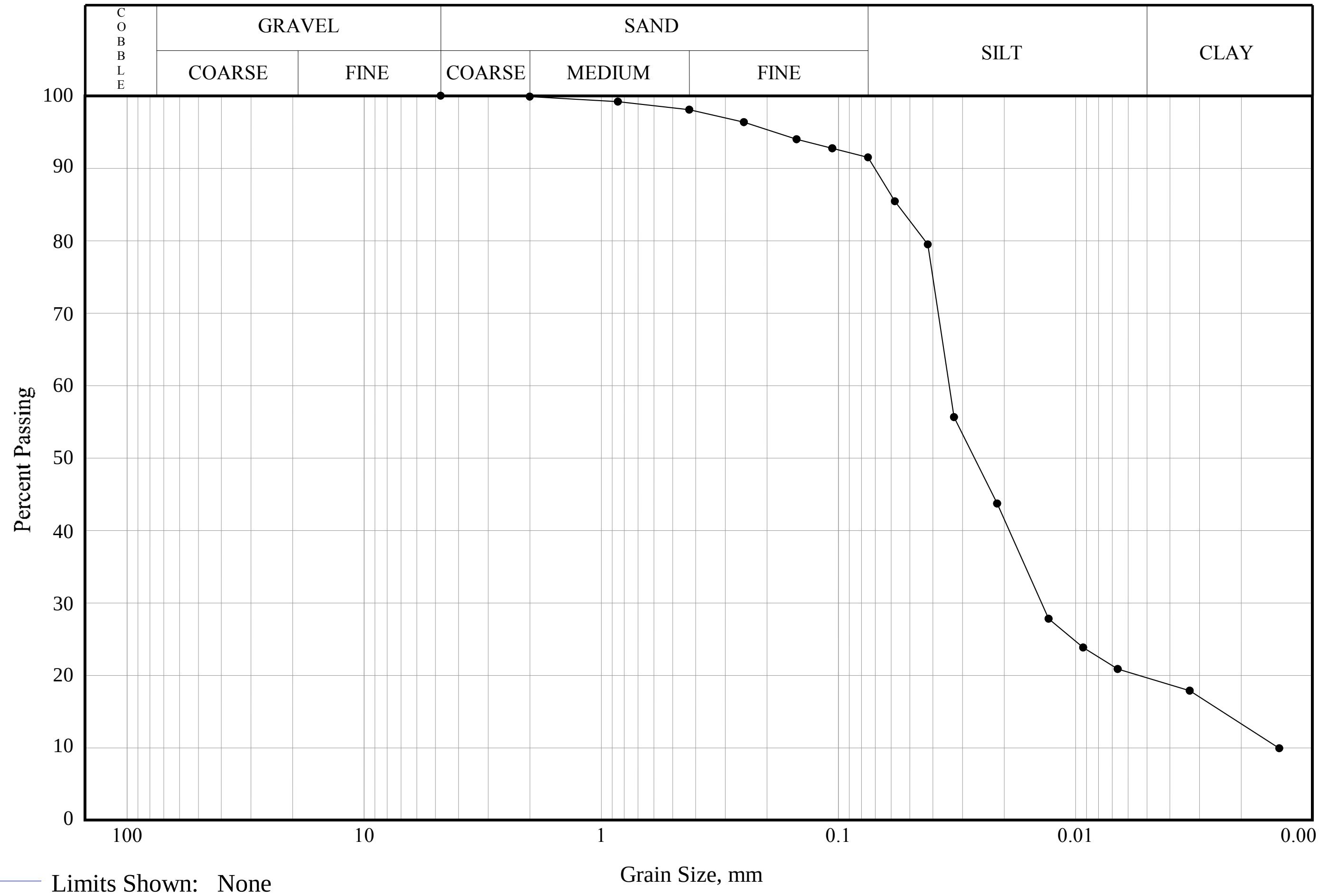
GROUNDWATER OBSERVATIONS

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			Soils Grading Chart
	Client:	AECOM Canada Ltd	
	Project:	Geotechnical Investigation - 5 Orchard Drive, Ottawa	
	Project #:	6399369	

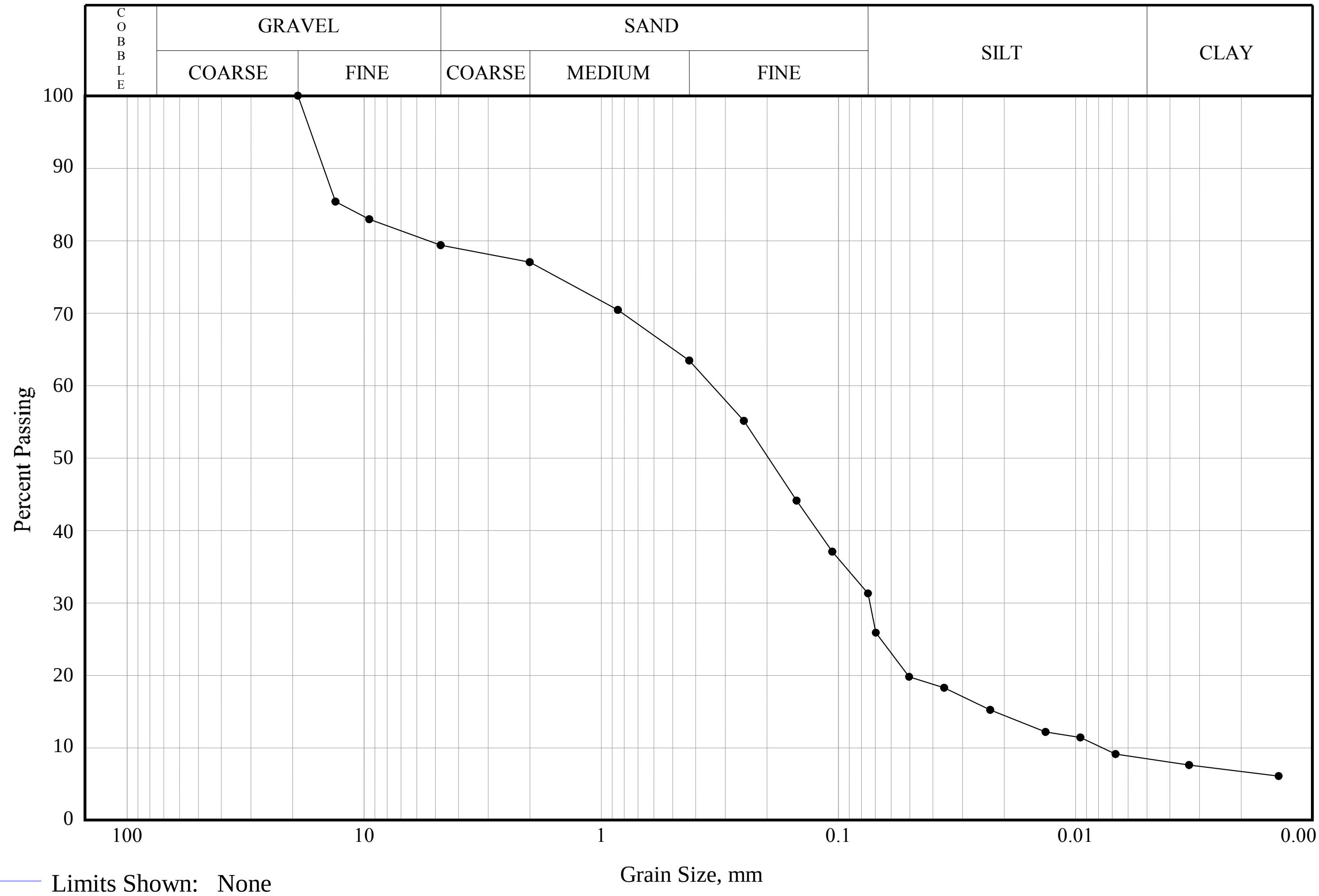


Limits Shown: None

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

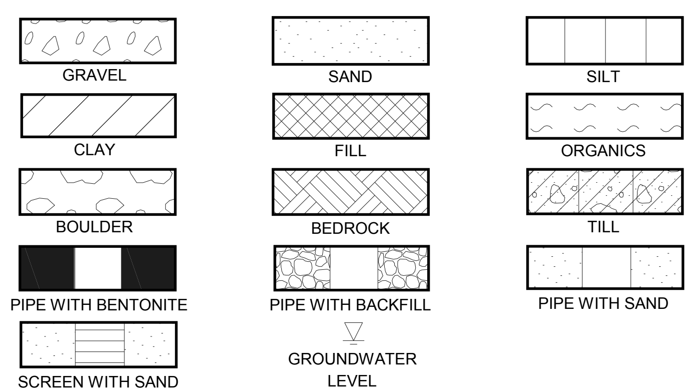
PENETRATION RESISTANCE

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			GRAVEL	COBBLE	BOULDER
	Fine	Medium	Coarse			
0.08	0.4	2	5	80	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.

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

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

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

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

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

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

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

APPENDIX B

Rock Core Photo – Figure B1

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

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

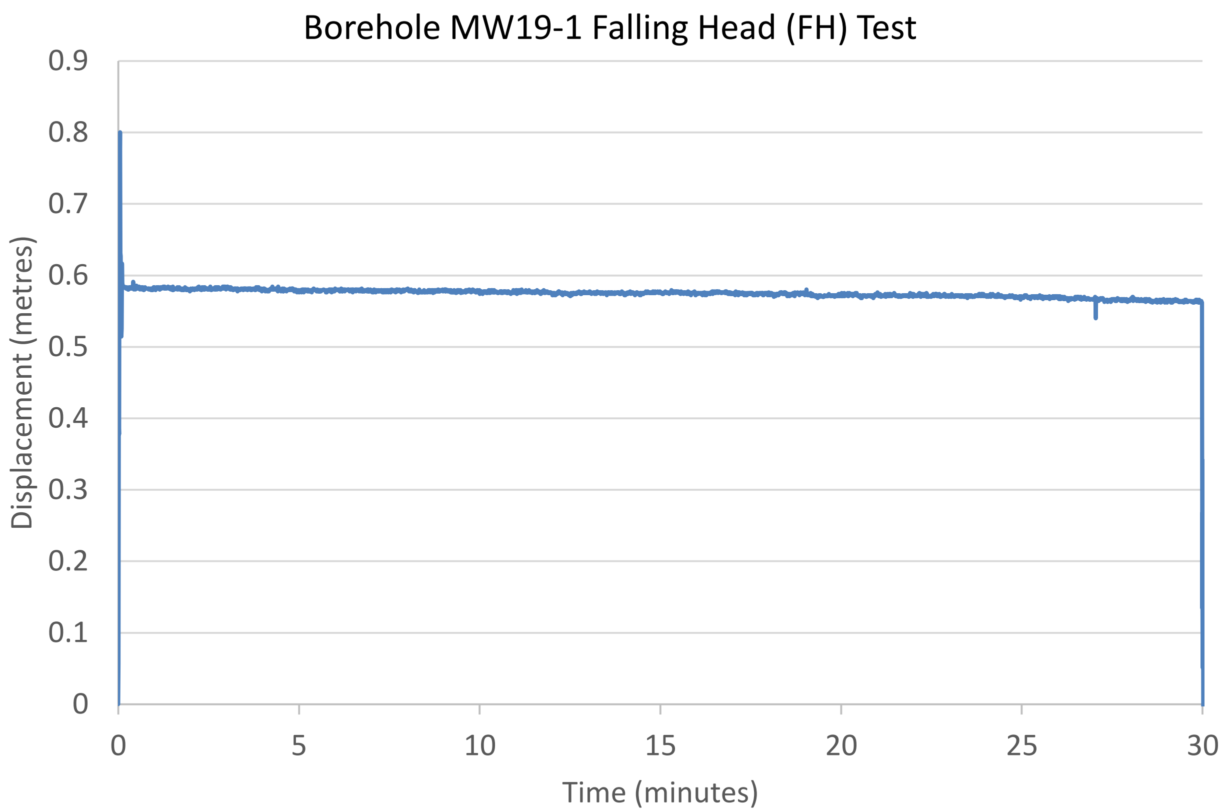
APPENDIX C

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

Client: GEMTEC Consulting Engineers and Scientists Limited

Client PO:

Report Date: 17-Jun-2019

Order Date: 11-Jun-2019

Project Description: 63993.69

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

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Appendix B

Excerpts from Functional Servicing and Stormwater Management Report for Campanale Homes 5 Orchard Drive City of Ottawa (DSEL, March 2019 – Rev 3)

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FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

FOR

CAMPANALE HOMES
5 ORCHARD DRIVE

CITY OF OTTAWA

PROJECT NO.: 18-1006

MARCH 2019 – REV. 3

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

MARCH 2019 – REV. 3

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CITY OF OTTAWA

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

1.0 INTRODUCTION

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

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

Figure 1: Site Location

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

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

1.1 Existing Conditions

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

Hazeldean Road:

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

Fringewood Drive:

- 200 mm watermain.

1.2 Required Permits / Approvals

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

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

1.3 Pre-consultation

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

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

2.0 GUIDELINES, PREVIOUS STUDIES, AND REPORTS

2.1 Existing Studies, Guidelines, and Reports

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

3.0 WATER SUPPLY SERVICING

3.1 Existing Water Supply Services

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

3.2 Water Supply Servicing Design

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

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

Table 1
Water Supply Design Criteria

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

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

Table 2
Proposed Water Demand

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

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

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

3.3 Watermain Modelling

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

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

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

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

Table 3
Model Simulation Output Summary

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

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

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

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

3.4 Water Supply Conclusion

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

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

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

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

4.0 WASTEWATER SERVICING

4.1 Existing Wastewater Services

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

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

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

4.2 Wastewater Design

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

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

Table 4
Wastewater Design Criteria

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

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

Table 5
Summary of Proposed Wastewater Flows

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

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

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

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

4.3 Wastewater Servicing Conclusions

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

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

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

5.0 STORMWATER MANAGEMENT

5.1 Existing Stormwater Services

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Table 8
Summary of Existing DICB Capture Rate

Parameter	DICB 1	DICB 2
DICB Grate Invert Elevation (m)	103.98	103.65
DICB Lead Invert (m)	102.94	102.71
Ponding Level[1] (m)	104.49	104.49
Assumed Downstream HGL[2] (m)	103.08	102.77
Total Head[3] (m)	1.41	1.72
DICB Grate Capture Rate[4] (L/s)	660	660
375mm DICB Lead Capture[5] (L/s)	354	391

1) Spill Elevation across Fringewood Drive per topographic survey
2) Downstream HGL assumed equal to obvert of Ex. 675mm Storm within Hazeldean Road
3) Total Head equal to Ponding Level less the downstream HGL
4) DICB capture rate determined from Design Chart 4.20 from the MTO Drainage Management Manual, 1997 using 0.51m of ponding, capture rate multiplied by1.2 to account for 1200mm x 600mm grate and then by 0.5 to account for blockages. DICB2 has a higher ponding than DICB1 so the capture rate for DICB1 was used for both DICBs conservatively.
5) Orifice equation used per the ***City Standards***, refer to ***Appendix D*** for orifice equation

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

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

5.2 Post-development Stormwater Management Target

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

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

5.3 Proposed Stormwater Management System

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

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

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

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

5.4 Proposed Quantity Controls

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

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

Table 9
Stormwater Flowrate and Storage Summary

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

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

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

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

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

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

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

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

5.5 Proposed Quality Control

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

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

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

Table 10
Interim Wetland Quality Control

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

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

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

5.6 Stormwater Management Conclusions

Post development stormwater runoff will be required to be restricted to the allowable target release rate for storm events up to and including the 100-year storm, in accordance with City of Ottawa, ***City Standards***. The post-development allowable release rate to the sewer within Hazeldean Road was calculated to be ***251.9 L/s***; with an estimated ***416.9 m³*** of storage required for the residential development and ***843.1 m³*** of storage required in the future commercial development in order to meet this release rate.

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

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

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

6.0 UTILITIES

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

7.0 EROSION AND SEDIMENT CONTROL

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

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

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

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

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

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

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

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

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

8.0 CONCLUSION AND RECOMMENDATIONS

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

- Based on boundary conditions provided by the City the existing municipal water infrastructure is capable of providing the proposed development with water within the City's required pressure range. Pressure check will need to be completed during construction to determine if pressure reducing valves will be required;
- The proposed development is anticipated to have a peak wet weather flow of ***4.51 L/s*** directed to the Stittsville Trunk Sewer, the property has been contemplated in the sizing of the future sewer to be installed within Fringewood Drive;
- The proposed development will be required to attenuate post development flows to an equivalent release rate of ***251.9 L/s*** to the sewer within Hazeldean Road, for all storms up to and including the 100-year storm event;
- It is anticipated that ***416.9 m^3*** of storage will be required for the residential development and ***843.1 m^3*** of storage will be needed for the future commercial development to attenuate stormwater to the allowable release rate to the storm sewer within Hazeldean Road; and
- Utility services would need to be coordinated with utility companies prior to development.

Prepared by,
David Schaeffer Engineering Ltd.

JBB# 18-1006

Per: Steven L. Merrick, P.Eng

Reviewed by,
David Schaeffer Engineering Ltd.

Per: Stephen Pichette, P.Eng.

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LEGEND:

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

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EXISTING CONDITIONS
5 ORCHARD DRIVE

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

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LEGEND:

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

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EXISTING STORMWATER DRAINAGE PLAN 5 ORCHARD DRIVE

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

Appendix C

Excerpts from Stormwater Management Report for the 5 Orchard Drive City of Ottawa (DSEL, March 2020)

Stormwater Management Report for the 5 Orchard Drive

City of Ottawa

March 2020

Prepared for: David Schaeffer Engineering Ltd.

JFSA Ref. No.: 1733

Prepared by :

JFSA Water Resources and Environmental Consultants

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Stormwater Management Report for the 5 Orchard Drive

in the City of Ottawa

March 2020

Prepared for :

David Schaeffer Engineering Ltd.

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Stormwater Management Report for the 5 Orchard Drive Development
in the City of Ottawa

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in the City of Ottawa
March 2020

1 INTRODUCTION AND OBJECTIVES

J.F. Sabourin and Associates Inc. (JFSA) were retained by David Schaeffer Engineering Ltd. (DSEL) to prepare a Stormwater Management (SWM) Plan for the proposed residential development at 5 Orchard Drive, located in Stittsville Ontario. As shown in Figure 1, the proposed development is located south of Hazeldean Road, off Fringewood Drive. The proposed development will back onto the existing residential development on Cloverloft Court to the south and a 1.82 ha proposed commercial development to the north. The 5 Orchard Drive Development has a total drainage area of approximately 2.13 ha and will comprise of a mix of single-detached houses and townhouses. The proposed development will have 3 onsite storage units in place along the 8 m city block between the commercial and residential lots. The onsite storage will control runoff from the proposed development and will connect to the existing trunk sewer that runs along Hazeldean Road, which connects to an interim SWM pond before discharging to Hazeldean Creek. To make full use of the proposed onsite storage units, ICD's will be implemented downstream of the units to control the runoff from this development. Under ultimate conditions, the Interim SWM pond will be decommissioned and replaced with 2 Oil and Grit Separators (OGS) units. For this analysis, it has been assumed that the runoff from the proposed commercial development site to the north will also be controlled through the use of onsite storage.

The purpose of this report is to evaluate the major and minor system flows for the proposed development concerning the latest stormwater management guidelines and to check the adequacy of the proposed pipe sizes to convey the 2-year (5-year on collector and 10-year on arterial roads) and the 100-year storm flows from within the development and from external areas. Background documents that were reviewed in preparing this report include the following:

- *Stormwater Management Planning and Design Manual, Ministry of the Environment, March 2003.*
- *Hazeldean Road Widening Poole Creek to Terry Fox Drive Stormwater Management, IBI November 2009.*
- *Erosion and Sediment Control Guidelines for Urban Construction, Conservation Halton et al., December 2006.*
- *Draft City of Ottawa Stormwater Management Facility Design Guidelines, IBI Group, April 2012.*
- *City of Ottawa Sewer Design Guidelines, City of Ottawa, October 2012.*
- *Technical Bulletin ISDTB-2014-01, Revisions to Ottawa Design Guidelines – Sewer, City of Ottawa, February 2014.*
- *City of Ottawa Technical Bulletin PIEDTB-2016-01, City of Ottawa, September 2016.*
- *City of Ottawa Technical Bulletin ISTB-2018-04, City of Ottawa, June 2018.*
- *Kanata West Master Servicing Study - Stantec Consulting Ltd., June 2006*

- *Functional Servicing Report for Campanale Homes 5 Orchard Drive, David Schaeffer Engineering Limited, August 2019.*
- *5 Orchard Drive – Stormwater Functional Servicing Analysis, JFSA December 2018*

As per the new approach formalized in the September 2016 City of Ottawa Technical Bulletin PIEDTB-2016-01, the proposed subdivision has been designed with a 2-year minor system level of service on local roads and 5-year level of service on collector roads. Where possible with grading and minor system capture limitations, road ponding areas up to 35 cm deep were used to contain the 100-year major system flows.

The PCSWMM program was used to model the major and minor systems to ensure that all of the City of Ottawa's stormwater management requirements are assessed and satisfied. The general SWM design criteria and guidelines that are to be met are described in Section 5.

Figure 1: General Location of Subject Site

2 EXISTING CONDITIONS

Under existing conditions, runoff from the subject site is collected and conveyed by storm sewers within Hazeldean Road to the existing interim SWM pond located on Hazeldean Road, east of the intersection of Hazeldean Road and Huntmar Drive. The interim pond then discharges to Hazeldean ditch before ultimately discharging to the Carp River. Within the subject property, there are two parallel ditches which lead to two existing DICBs. The majority of the flow from the subject site is picked up by the ditch draining to DICB 1, with the flow from the east portion of the site directed to DICB 2. The runoff from the rear yards of the Cloverloft Court properties that bound the south edge of the subject property, flow into a rear yard ditch that runs along the southern boundary. Drainage from the existing subdivision to the south of the subject property drains east towards the intersection of Fringewood Drive and Cloverloft Court. Note that a culvert crossing Fringewood Drive at Cloverloft Court is perched and would not accept flow from these lands, and it is assumed that all runoff by-passes this culvert and is directed north to DICB 2. Both DICB 1 and DICB 2 discharge to the existing 675 mm diameter storm sewer within Hazeldean Road.

As both the existing and proposed development site will discharge to the interim SWM pond on Hazeldean Road, this pond has been included in this analysis. The pond storage volume and outlet controls have been included in the PCSWMM model as specified in IBI's November 2009 "Hazeldean Road Widening Poole Creek to Terry Fox Drive Stormwater Management" Report. Existing catchment areas on Hazeldean Road that drain to this facility have been delineated by DSEL and have been assumed to have a 10-year capture to the minor system.

3 INTERIM CONDITIONS

Under interim conditions, a new storm sewer will be implemented along Fringewood Drive, which will service both the proposed commercial and residential developments. The release rate for these subject properties will be limited to the capacity of the existing storm sewers within Hazeldean Road. As per the Functional Servicing Report for 5 Orchard Drive, DSEL August 2019, a hydraulic grade line analysis was completed for the existing sewers to determine the maximum available capacity of the sewers. To ensure that the hydraulic grade line in the proposed condition does not impact the proposed development or have a negative impact on the downstream system, the allowable release rate for the total future developments has been determined to be 251.9 L/s. For this analysis, the total residential lands will have an allowable release rate of 200 L/s and the commercial site will have an allowable release rate of 51.9 L/s. To obtain these release rates onsite storage units will be implemented within the respective developments. Note that 0.525 ha of existing rear yards along the northern extent of the Frindgewood residential development will drain through the proposed 5 Orchard Drive residential property. Although runoff from these rear yards will discharge through the proposed development, this runoff will not be controlled to the rates specified above, as these lands already drain to the trunk sewer under existing conditions. The remaining runoff from the existing Fringewood Drive development will be redirected to the Hazeldean tributary on 2 Iber Road located on the east side of Fringewood Drive.

4 ULTIMATE CONDITIONS

Under ultimate conditions, the proposed development will remain as set out under interim conditions, but the Interim SWM pond that these developments discharge to on Hazeldean Road will be decommissioned and replaced with 2 Oil and Grit Separators (OGS) units.

5 DESIGN CRITERIA AND GUIDELINES

The design criteria and guidelines used for the stormwater management of the subject site are those that were developed in the background documents, as well as those provided in the October 2012 *City of Ottawa Sewer Design Guidelines* and subsequent technical memorandums, and generally accepted stormwater management design guidelines.

During the detailed design of the proposed developments, it was assumed that the 2.13 ha residential development will have an average imperviousness of 53%, and the 1.82 ha commercial site will have an average imperviousness of 86%. A detailed analysis of the proposed dual drainage system was required to confirm that the following general design criteria and guidelines for the minor and major systems would be met.

5.1 Minor System

a) Storm sewers are to be designed to provide a minimum 2-year level of service, plus 5-year inflows on collector roads and 10-year inflows on arterial roads.

b) The 100-year hydraulic grade line (HGL) within the development minor systems must be maintained at least 0.3 m below the underside of footing elevation where gravity house connections are installed.

c) For less frequent storms (i.e. larger than 1:2 year or 1:5 year on collector / 1:10 year on arterial roads), the minor system shall, if required, be limited with the use of inlet control devices to prevent excessive hydraulic surcharges and to maximize the use of surface storage on the road where desired.

d) Catchbasins on the road are to be equipped with City standard type S19 (fish) grates or City standard type S22 side inlets, and grates for catchbasins in rear yards, park and open spaces with pedestrian traffic are to be City standard type S19, S30 and S31.

e) Single catchbasins are to be equipped with 200 mm minimum lead pipes, and double catchbasins are to be equipped with 250 mm minimum lead pipes, or two 200 mm minimum lead pipes where two inlet control devices are specified.

f) Rearyard catchbasins are to be equipped with 250 mm minimum lead pipes. Catchbasins installed on the street, where rearyard catchbasins connect to the main storm sewer through the catchbasin, are to be equipped with 250 mm minimum lead pipes for single or double catch basins, or two 200 mm minimum lead pipes for double catchbasins unless otherwise noted.

g) Under full flow conditions, the allowable velocity in storm sewers is to be no less than 0.80 m/s and no greater than 3.0 m/s. Where velocities in excess of 3.0 m/s are proposed, provisions shall be made to protect against displacement of sewers by sudden jarring or movement. Velocities greater than 6 m/s are not permitted.

5.2 Major System

a) The major system shall be designed with sufficient road surface storage to allow the excess runoff of a 100-year storm to be retained within road ponding areas where desired.

b) Inlet control devices should be sized such that they do not create surface ponding on the road during the 2-year design storm on local roads (5-year design storm on collector and 10-year design storm on arterial roads); it should be noted that surface ponding over grates is present during rainfall under any design, as an appropriate depth of water is required for runoff to enter the grate.

c) Roof leaders shall be installed to direct the runoff to splash pads and on to grassed areas.

d) For the 100-year storm, the maximum total depth of water (static + dynamic) for all roads shall not exceed 35 cm at the gutter.

e) During the 100-year + 20% stress test, the maximum extent of surface water on streets, rearyards, public space and parking areas shall not touch the building envelope.

f) When catchbasins are installed in rear yards, safe overland flow routes are to be provided to allow the release of excess flows from such areas.

g) The product of the maximum flow depths on streets and maximum flow velocity must be less than 0.60 m^2/s on all roads.

h) The excess major system flows up to the 100-year return period are to be retained on-site in development blocks such as parks, schools, commercial, etc.

i) There must be at least 15 cm of vertical clearance between the spill elevation on the street and the ground elevation at the nearest building envelope that is in the proximity of the flow route or ponding area.

j) There must be at least 30 cm of vertical clearance between the rearyard spill elevation and the ground elevation at the adjacent building envelope.

6 ASSUMPTIONS AND SOURCE OF DATA USED IN THIS STUDY

Sources of information and assumptions made in this study are listed below:

- Stormwater management model:	*PCSWMM (version 7.2)*
- Minor system design:	*1:2 year, plus 1:5 year inflows on collector roads and 1:10 year on arterial roads. See the Rational Method Calculations in Appendix A.*
- Major system design:	*1:100 year*
- Max. 100-yr water depth on roads:	*35 cm above the gutter*
- Extent of the major system:	*Shall not touch the building envelope during the 100-year + 20% stress test*
- PCSWMM model parameters:	*CN calculated based on land use and soil type present, D.Stor.Imp. = 1.57 mm, D.Stor.Per. = 4.67 mm (as per 2012 City of Ottawa Sewer Design Guidelines)* *Detailed Area Imperviousness: based on development layout.*
- Design storms:	*2-, 5-, 10- and 100-year 3-hour Chicago and 100-year 24-hour SCS Type II storms as per 2012 City of Ottawa Sewer Design Guidelines; peak averaged over 10 minutes.*
- Historical Events:	*July 1st, 1979; August 4th, 1988; and August 8th, 1996 events as per 2012 City of Ottawa Sewer Design Guidelines.*
- Stress Test:	*20% increase in the 100-year 3-hour Chicago storm.*
- Street catchbasin covers:	*City Standard Type S19 (fish) or City Standard Type S22 (side inlet). Type S19 approach flow-capture curves as per MTO design charts (equivalent to OPSD 400.010). Type S22 approach flow-capture curves as per the 2004 City of Ottawa Guidelines.*
- Rearyard catchbasin covers:	*City Standard Type S19, S30 and S31*
- Curb and gutter:	*City Standard SC1.3 (mountable) and SC1.1 (barrier). In the absence of flow capture curves for these curbs and gutters, OPSD 600.010 curb and gutters are assumed.*
- Manning's' roughness coeff.:	*0.013 for concrete and PVC pipes (free flow).*
- Minor system losses:	*Refer to Appendix C for manhole loss coefficients.*
- Underside of footing elevations:	*As provided by DSEL.*
- Freeboard in HGL analysis:	*0.3 m between the underside of footing elevation and 100-year hydraulic gradeline.*
- Inlet Control Devices:	*Refer to Appendix B for Plas-Tech ICD details.*
- Depth of backyard swales:	*As per DSEL's Grading Plan*
- Street and pipe dimensions:	*As per DSEL's Plan and Profiles*
- Right-of-way characteristics:	*As per DSEL's Details of Roads*
- Downstream HGL:	*Free outfall condition on Hazeldean Creek.*

7 PROPOSED MINOR AND MAJOR SYSTEM DRAINAGE

The proposed minor and major system drainage routes are shown in plan view in Figures 2 and 3. The proposed development has been modelled in PCSWMM, as this program is well suited for modelling small urban drainage areas.

In accordance with the new proposed standards, the minor system has been designed to accommodate a minimum of the 2-year post-development flows from within the site and from external areas, plus 5-year inflows on collector roads and 10-year inflows on arterial roads (such as those on Hazeldean Road). A Rational Method design was conducted by DSEL (refer to Appendix A) in order to estimate minor system flows based on the City of Ottawa IDF relationship and selected runoff coefficients.

As noted earlier in this report, where possible with grading limitations, road ponding areas up to 35 cm deep were used to contain the 100-year major system flows in the development. Note that rearyard catchbasins were connected to catch basins on the road where possible, in order to allow rearyard runoff access to the storage in road ponding areas at regular intervals. In a design of this type where lots are serviced by gravity house connections, inlet control devices (ICDs) can be used to limit minor system capture at each catchbasin to the appropriate level of service.

Within the development, circular orifice plate type Inlet Control Devices (ICDs) of City standard diameters 83 mm, 94 mm, 102 mm, 108 mm, 127 mm, 152 mm and 178 mm will be used to limit minor system capture to a minimum of the 2-year flow (refer to Appendix B for Plas-Tech ICD details), allowing for sub-surface storage of 0.5 m^3 in single catchbasins, 1.0 m^3 in double catch basins, and 1.9 m^3 in catchbasin manholes.

The street segments within the proposed development have been designed using a 'saw tooth' or 'sagged' road profile. The runoff from within these segments will be conveyed to catchbasins located at the lowest point within the street segment. Flows in excess of the catchbasin capture rate will be temporarily stored within the 'sagged' street segments and released slowly to the storm sewers, up to the 100-year design storm. When the storage on a specific street segment is surpassed due to blockage or an event greater than the 100-year storm, the excess water will flow towards the next downstream street sag, and eventually to the pond. It should be noted that the major system would outlet during the 100-year + 20% stress test without flooding any of the properties within the subdivision.

In the event that the drainage system’s capacity to capture surface flows is exceeded, Figure 4 presents the maximum extent of static surface ponding and volume on the streets based on grading. Note that no surface storage volumes were accounted for in the PCSWMM model in the rear yard swales.

The PCSWMM analyses have demonstrated that the proposed drainage system for the subdivision will have sufficient capacity to control the excess flow during a 100-year storm and safely capture and convey the 2-year (plus 5-year on collector roads and 10-year on arterial roads) flow to the pond/OGS unit.

7.1 Major System and SWM Analysis

The PCSWMM models were developed based on the information provided in Figures 2 and 3. Nine (18) simulations were conducted, one for each of the following rainfall events:

1) The 25 mm, 4-hour Chicago storm;
2) the 2-year, 3-hour Chicago storm;
3) the 5-year, 3-hour Chicago storm;
4) the 10-year, 3-hour Chicago storm;
5) the 25-year, 3-hour Chicago storm;
6) the 50-year, 3-hour Chicago storm;
7) the 100-year, 3-hour Chicago storm;
8) the 100-year, 3-hour Chicago storm + 20%;
9) the 2-year, 24-hour SCS Type II storm;
10) the 5-year, 24-hour SCS Type II storm;
11) the 10-year, 24-hour SCS Type II storm;
12) the 25-year, 24-hour SCS Type II storm;
13) the 50-year, 24-hour SCS Type II storm;
14) the 100-year, 24-hour SCS Type II storm;
15) the 100-year, 24-hour SCS Type II storm +20%;
16) the July 1st, 1979 historical event;
17) the August 4th, 1988 historical event;
18) the August 8th, 1996 historical event.

Note that the purpose of simulating the 100-year, 3-hour Chicago storm with a 20% increase is to stress test the drainage system for potential flooding, as per the October 2012 *City of Ottawa Sewer Design Guidelines*.

The depression storage parameters in the PCSWMM model are as per the October 2012 *City of Ottawa Sewer Design Guidelines*. The percent imperviousness of the detailed drainage areas was measured based on the development layout. CN infiltration parameters for each subcatchment is based on underlying land use and soil types present in that location (see Attachment C for full details).

In the PCSWMM model where required inflows are limited by circular orifice plate type Inlet Control Devices (ICDs) of City standard diameters 83 mm, 94 mm, 102 mm, 108 mm, 127 mm, 152 mm and 178 mm. Note that 200 mm diameter lead pipes were assumed and are required between single catchbasins and the storm sewers, and 250 mm diameter lead pipes were assumed and are required between rearyard catchbasins or single catchbasin manholes and the storm sewers. Double catchbasins and double catchbasin manholes are to be equipped with 250 mm diameter lead pipes, or two 200 mm diameter lead pipes where two ICDs are specified. Refer to Table D-6 of Appendix D for a summary of inlet controls implemented within the development.

Within the proposed subdivision, the dynamic flow depth on the road (at the gutter) will be minimal during the 100-year Chicago storm, as the 100-year flows are mostly retained within the road ponding areas and do not accumulate as in a typical subdivision design (refer to Calculation Sheet 1A of Appendix D). Furthermore, it was determined that, for the 100-year storm and for all major system segments, the product of the depth of water (m) at the gutter multiplied by the velocity of flow (m/s) will not exceed the maximum allowable 0.6 m^2/s (refer to Calculation Sheet 1A of Appendix D, where the calculated maximum was 0.093m^2/s).

Calculation Sheet 1B of Appendix D presents the stress test results for dynamic flow depth on the road based on a 20% increase in the 100-year storm, as per the October 2012 *City of Ottawa Sewer Design Guidelines*. As shown in Calculation Sheet 1B, the product of the depth of water at the gutter multiplied by the velocity of flow is 0.110 m^2/s.

Details of 100-year street storage results (i.e. storage available and depth of water at ponding areas) are provided in Table D-6 of Appendix D. This information, calculated by the PCSWMM model, demonstrates that total 100-year depth of water (static and dynamic) on the street at these ponding areas will not exceed the maximum depth of 35 cm.

Table D-6 of Appendix D also presents the street storage stress test results based on a 20% increase in the 100-year storm, as per the October 2012 *City of Ottawa Sewer Design Guidelines*. As shown in Table D-2, the maximum depth of water (static + dynamic overflow) at any ponding area under these conditions is calculated as 35 cm. The maximum extent of surface water during the 100-year + 20% stress test will not touch the building envelopes.

Table 1 presents a summary of the major system results simulated in PCSWMM during the 100-year Chicago storm.

Table 1: Summary of Major System Results for the 100-Year 3-Hour Chicago Storm

Catch Basin ID	Approach Flow (m3/s)	Captured Flow (m3/s)	Flow Depth (cm)
CB_1	0.211	0.065	25
CB_2	0.091	0.072	25
CB_3	0.080	0.044	20
CB_4	0.090	0.044	20
CB_5	0.296	0.088	24
CB_6	0.070	0.088	24

7.2 Minor System and Hydraulic Gradeline Analysis

The minor system analysis was completed using the PCSWMM program based on the peak flows captured during the rainfall events. Note that the storm sewer design is as provided by DSEL, and a Manning's roughness coefficient of 0.013 was used for concrete and PVC storm sewer pipes.

Refer to Appendix C for manhole loss coefficients used in the PCSWMM model. Note for ultimate conditions that a loss coefficient of 1.3 (as specified by the manufacture) has been incorporated into the models at the inlet pipe to the proposed oil-and-grit separators to account for losses through the unit. The minor system performance was analyzed under free downstream conditions on Hazeldean Creek. Table 2A presents the peak minor system outflows obtained with the above-mentioned simulations.

Table 2A: Comparison of Minor System Flows to the Interim Pond

Location	DSEL Rational Method Flow (m^3/s)	2-Year PCSWMM Flow (m^3/s)	5-Year PCSWMM Flow (m^3/s)	10-Year PCSWMM Flow (m^3/s)	100-Year PCSWMM Flow (m^3/s)
MH 27 to Interim Pond	0.908	0.545	0.746	0.869	1.077

Table 2A shows that the 2-year flows simulated with the PCSWMM models are lower than the Rational Method flow. This is in part explained by the fact that the Rational Method calculations include a combination of 2-, 5-, 10-, and 100-year flows based on minimum capture requirements through the various locations draining to the SWM Pond. The PCSWMM simulations have determined that for the selected 2-, 5-, 10- and 100-year storms, the total minor system flows to the Interim SWM Pond / OGS unit would be 0.545 m^3/s, 0.746 m^3/s, 0.869 m^3/s and 1.077 m^3/s, respectively. Tables 2B and 2C summarize the peak flows and total runoff volumes to the SWM Pond / OGS unit and downstream of this feature on Hazeldean Creek under Existing, Interim and Ultimate Conditions.

Table 2B: Comparison of Minor System Flows to the Interim Pond/OGS

Storm	Existing Conditions		Interim Conditions		Ultimate Conditions	
	Peak Flow (m^3/s)	Total Volume (m^3)	Peak Flow (m^3/s)	Total Volume (m^3)	Peak Flow (m^3/s)	Total Volume (m^3)
25mmChicago4Hr	0.350	1,268	0.403	1,623	0.401	1,627
2yrChicago3hr	0.565	1,808	0.545	2,166	0.544	2,170
5yrChicago3hr	0.758	2,786	0.746	3,042	0.743	3,045
10yrChicago3hr	0.917	3,471	0.869	3,629	0.867	3,632
25yrChicago3hr	0.960	4,354	0.977	4,361	0.983	4,363
50yrChicago3hr	0.995	5,029	1.031	4,907	1.036	4,908
100yrChicago3hr	1.080	5,725	1.077	5,474	1.084	5,474
100yrChicago3hr+20%	1.174	7,090	1.141	6,635	1.148	6,672
2YrSCS24	0.452	3,632	0.563	3,597	0.564	3,598
5YrSCS24	0.674	5,262	0.753	4,919	0.754	4,920
10YrSCS24	0.826	6,362	0.872	5,796	0.874	5,794
25YrSCS24	1.006	7,735	1.000	6,869	1.002	6,866
50YrSCS24	1.105	8,778	1.064	7,694	1.066	7,692
100YrSCS24	1.173	9,873	1.121	8,580	1.126	8,577
100YrSCS24+20%	1.200	12,180	1.146	10,330	1.154	10,300

Table 2C: Total Flows to Hazeldean Creek

Storm	Existing Conditions		Interim Conditions		Ultimate Conditions	
	Peak Flow (m^3/s)	Total Volume (m^3)	Peak Flow (m^3/s)	Total Volume (m^3)	Peak Flow (m^3/s)	Total Volume (m^3)
25mmChicago4Hr	0.063	1,244	0.149	1,627	0.392	1,631
2yrChicago3hr	0.127	1,784	0.244	2,172	0.534	2,176
5yrChicago3hr	0.236	2,762	0.365	3,050	0.734	3,054
10yrChicago3hr	0.318	3,446	0.439	3,638	0.846	3,641
25yrChicago3hr	0.433	4,333	0.532	4,375	0.980	4,376
50yrChicago3hr	0.527	5,019	0.603	4,934	1.042	4,932
100yrChicago3hr	0.630	5,745	0.667	5,526	1.104	5,523
100yrChicago3hr+20%	0.779	7,262	0.881	6,815	1.319	6,846
2YrSCS24	0.204	3,559	0.310	3,597	0.557	3,601
5YrSCS24	0.358	5,185	0.424	4,919	0.745	4,921
10YrSCS24	0.466	6,282	0.498	5,795	0.855	5,794
25YrSCS24	0.607	7,652	0.587	6,869	0.986	6,867
50YrSCS24	0.673	8,695	0.650	7,697	1.053	7,693
100YrSCS24	0.800	9,800	0.746	8,593	1.114	8,589
100YrSCS24+20%	1.060	12,190	1.068	10,420	1.250	10,390

Through the diversion of the existing residential development on Cloverloft Court to the Hazeldean tributary on 2 Iber Road located on the east side of Fringewood Drive, in conjunction with the proposed onsite storage measures within the proposed development the peak flows into the SWM pond/ OGS are generally less than those under existing conditions. The total runoff volumes at this location under the proposed conditions are also generally less than that under existing conditions, especially for the larger events.

The proposed onsite storage units and corresponding downstream ICD's have been sized to ensure that peak flows from the development in the minor system do not exceed 200 L/s for the 100-year event. Note that 0.525 ha of existing rear yards along the northern extent of the Frindgewood residential development will drain through the proposed 5 Orchard Drive residential property via the onsite storage units, although the runoff from these catchments will not be controlled to the rates specified above, as these lands already drain to the trunk sewer under existing conditions. For the residential development, storage will be provided through one 1.35m, and two 1.8m diameter culverts, buried along the northern edge of the development. The details of the proposed onsite storage for the commercial site are still preliminary, irrespective of these details flows been restricted to ensure that they do not exceed 52 L/s for the 100-year event. Through an iterative sizing process, it was found that the following storage volume and ICD will be required for the respective locations.

Table 2D: Onsite Storage Requirements

Location	Required Storage Volume (m^3)	Peak Flow (L/s)	Downstream ICD
Residential West	189	109	178mm
Residential Middle	90	75	152mm
Residential East	149	132	220 mm Orifice Plate
Commercial Lot	900	49	127mm

Although the 100-year flow will surcharge most parts of the minor system, a freeboard of 0.3 m between the 100-year hydraulic grade line and the underside of footings has been provided throughout the proposed development, with the exception for the resideintal blocks that will have sump pump units in place. Tables C-1A through to C-2A-F of Appendix C summarizes the pipe data and hydraulic simulation results for the 100-year 3-hour Chicago storm, 100-year 24-hour SCS Type II storm and the three historical events for the interim and Future scenarios. From this analysis, a minimum freeboard of 0.3 m between the hydraulic grade line and the underside of footings has been provided throughout the proposed developments for the 100-year storms, and a minimum freeboard of 0 m has been provided throughout the proposed development for the historical events. Note that four blocks of townhomes (Blocks 12, 13, 14 & 15) will be required to be sump pumped due to the shallow connection to the existing storm sewer within Hazeldean Road, and do not need to meet these freeboard requirements.

Additionally, note that the majority of the flowing full pipe velocities are no less than 0.80 m/s and no greater than 3.0 m/s for all proposed pipes with one exception. Where velocities in excess of 3.0 m/s are proposed, provisions shall be made to protect against displacement of sewers by sudden jarring or movement. Velocities greater than 6 m/s are not permitted.

Table C-1C and C-2C of Appendix C presents the interim and ultimate climate change stress test results for the hydraulic gradeline analysis based on a 20% increase in the 100-year storm, as per the October 2012 *City of Ottawa Sewer Design Guidelines*. Under these conditions, no locations within the proposed developments have freeboards of less than 0 m, with the exception of the lots that will have sump pumps.

Table 3A and 3B present the composite hydraulic gradeline results for the 100-year 3-hour Chicago and 100-year 24-hour SCS Type II design storms under Interim and Ultimate Conditions

Table 3A: Interim Composite Hydraulic Gradeline Results for 100-Year Design Storms

U/S MH	D/S MH	Max. U/S HGL (m)	Max. D/S HGL (m)	Pipe Length (m)	Lot Number	USF (m)	Freeboard [(2)] (m)	Interpolated HGL Length HGL (m)	Interpolated HGL Dist. From D/S MH (m)	Interpolated HGL HGL (m)
MH-28	MH-1	104.749	104.371	25.717						
					19-4	105.34	0.910	25.7	4	104.430
					19-5	105.34	0.844	25.7	8.5	104.496
					19-6	105.34	0.768	25.7	13.7	104.572
					2	105.87	1.252	25.7	16.8	104.618
					3	105.82	1.183	25.7	18.1	104.637
					7	105.49	0.768	25.7	23.9	104.722
					4	105.63	0.881	25.7	25.7	104.749
					5	105.63	0.881	25.7	25.7	104.749
					6	105.63	0.881	25.7	25.7	104.749
MH-1	MH-30_US	104.041	103.875	148.15						
					15-4	104.73	0.854	148.2	1.2	103.876
					12-2	104.94	1.061	148.2	3.6	103.879
					15-5	104.98	1.098	148.2	6.3	103.882
					12-1	104.94	1.053	148.2	10.5	103.887
					15-6	104.98	1.090	148.2	13.1	103.890
					11-6	105.03	1.135	148.2	18.2	103.895
					16-1	104.98	1.082	148.2	20.9	103.898
					11-5	105.03	1.127	148.2	25	103.903
					16-2	104.98	1.074	148.2	27.7	103.906
					11-4	105.03	1.121	148.2	30.1	103.909
					16-3	104.98	1.068	148.2	32.8	103.912
					11-3	105.03	1.114	148.2	36.9	103.916
					16-4	104.98	1.061	148.2	39.6	103.919
					11-2	105.03	1.108	148.2	42.1	103.922
					16-5	104.98	1.055	148.2	44.8	103.925
					11-1	105.03	1.101	148.2	48.4	103.929
					16-6	104.98	1.047	148.2	51.6	103.933
					10-6	105.11	1.172	148.2	56.6	103.938
					17-1	105.05	1.109	148.2	59.3	103.941
					10-5	105.11	1.164	148.2	63.4	103.946
					17-2	105.05	1.100	148.2	66.6	103.950
					10-4	105.11	1.158	148.2	68.6	103.952
					17-3	105.05	1.095	148.2	71.3	103.955
					10-3	105.11	1.151	148.2	75.4	103.959
					17-4	105.05	1.087	148.2	78.1	103.963
					10-2	105.11	1.145	148.2	80.5	103.965
					17-5	105.05	1.082	148.2	83.2	103.968
					10-1	105.11	1.137	148.2	87.3	103.973
					17-6	105.05	1.074	148.2	90	103.976
					9-6	105.11	1.128	148.2	95.1	103.982
					18-1	105.05	1.066	148.2	97.7	103.984
					9-5	105.11	1.121	148.2	101.9	103.989
					18-2	105.05	1.058	148.2	104.6	103.992
					9-4	105.11	1.115	148.2	107	103.995
					18-3	105.05	1.052	148.2	109.7	103.998
					9-3	105.11	1.107	148.2	113.8	104.003
					18-4	105.05	1.044	148.2	116.5	104.006
					9-2	105.11	1.102	148.2	119	104.008
					18-5	105.05	1.039	148.2	121.6	104.011
					9-1	105.11	1.094	148.2	125.8	104.016
					18-6	105.05	1.031	148.2	128.5	104.019
					8-2	105.13	1.104	148.2	134.6	104.026
					19-1	105.34	1.312	148.2	136.2	104.028
					8-1	105.13	1.097	148.2	141.4	104.033
					19-2	105.34	1.305	148.2	143	104.035
MH-30_DS	MH-3	103.039	102.914	75						
					13-6	102.65	-0.291	75.0	16.4	102.941
					14-1	102.8	-0.146	75.0	19	102.946

Table 3A: Interim Composite Hydraulic Gradeline Results for 100-Year Design Storms

U/S MH	D/S MH	Max. U/S HGL (m)	Max. D/S HGL (m)	Pipe Length (m)	Lot Number	USF (m)	Freeboard [2] (m)	Interpolated HGL: Length HGL (m)	Interpolated HGL: Dist. From D/S MH (m)	Interpolated HGL: HGL (m)
					13-5	102.65	-0.303	75.0	23.2	102.953
					14-2	102.8	-0.157	75.0	25.9	102.957
					13-4	103.13	0.169	75.0	28.3	102.961
					14-3	103.28	0.314	75.0	31	102.966
					13-3	103.13	0.157	75.0	35.1	102.973
					14-4	103.28	0.303	75.0	37.8	102.977
					13-2	103.65	0.669	75.0	40.3	102.981
					14-5	103.78	0.794	75.0	42.9	102.986
					13-1	103.65	0.657	75.0	47.1	102.993
					14-6	103.78	0.783	75.0	49.8	102.997
					12-6	104.19	1.185	75.0	54.8	103.005
					15-1	104.34	1.330	75.0	57.5	103.010
					12-5	104.19	1.173	75.0	61.6	103.017
					15-2	104.34	1.319	75.0	64.3	103.021
					12-4	104.64	1.615	75.0	66.8	103.025
					15-3	104.73	1.700	75.0	69.4	103.030
					12-3	104.64	1.603	75.0	73.6	103.037
MH-7	MH-8	103.878	103.491	79						
MH-8	MH-9	103.491	102.765	100						
MH-9	MH-10	102.765	102.632	95						
MH-10	MH-11	102.582	102.25	102						
MH-11	MH-12	102.14	102.02	55.5						
MH-12	MH-13	101.94	101.53	93.5						
MH-13	MH-14	101.45	101.48	10						
MH-14	MH-15	101.4	100.98	76.5						
MH-15	MH-16	100.94	100.34	95						
MH-16	MH-17	100.32	99.98	94.85						
MH-17	MH-25	99.94	99.94	26.15						
MH-25	MH-26	99.9	99.78	23.9						
MH-26	MH-27	99.73	99.63	85.21						
MH-203	MH-202	102.8	102.847	20.834						
MH-202	MH-201	102.797	102.849	11.946						
MH-201	MH-5	102.799	102.845	18.995						

Note: [1] A negative surcharge implies that the pipe is not flowing full

[2] Conservative estimate of freeboard based on U/S HGL and lowest USF connected to pipe. Actual HGL / freeboard at all connecting lots

[3] Interim USF elevations estimated as 1.8 m below the upstream top of manhole elevations.

(green)	Interpolated HGL elevation
(yellow)	Freebaord Less than 0.3m from USF
(red)	Freeboard above USF

Table 3B: Future Composite Hydraulic Gradeline Results for 100-Year Design Storms

U/S MH	D/S MH	Max. U/S HGL (m)	Max. D/S HGL (m)	Pipe Length (m)	Lot Number	USF (m)	Freeboard [2] (m)	Interpolated HGL Length HGL (m)	Interpolated HGL Dist. From D/S MH (m)	Interpolated HGL HGL (m)
MH-28	MH-1	104.749	104.371	25.717						
					19-4	105.34	0.910	25.7	4	104.430
					19-5	105.34	0.844	25.7	8.5	104.496
					19-6	105.34	0.768	25.7	13.7	104.572
					2	105.87	1.252	25.7	16.8	104.618
					3	105.82	1.183	25.7	18.1	104.637
					7	105.49	0.768	25.7	23.9	104.722
					4	105.63	0.881	25.7	25.7	104.749
					5	105.63	0.881	25.7	25.7	104.749
					6	105.63	0.881	25.7	25.7	104.749
MH-1	MH-30_US	104.041	103.875	148.15						
					15-4	104.73	0.854	148.2	1.2	103.876
					12-2	104.94	1.061	148.2	3.6	103.879
					15-5	104.98	1.098	148.2	6.3	103.882
					12-1	104.94	1.053	148.2	10.5	103.887
					15-6	104.98	1.090	148.2	13.1	103.890
					11-6	105.03	1.135	148.2	18.2	103.895
					16-1	104.98	1.082	148.2	20.9	103.898
					11-5	105.03	1.127	148.2	25	103.903
					16-2	104.98	1.074	148.2	27.7	103.906
					11-4	105.03	1.121	148.2	30.1	103.909
					16-3	104.98	1.068	148.2	32.8	103.912
					11-3	105.03	1.114	148.2	36.9	103.916
					16-4	104.98	1.061	148.2	39.6	103.919
					11-2	105.03	1.108	148.2	42.1	103.922
					16-5	104.98	1.055	148.2	44.8	103.925
					11-1	105.03	1.101	148.2	48.4	103.929
					16-6	104.98	1.047	148.2	51.6	103.933
					10-6	105.11	1.172	148.2	56.6	103.938
					17-1	105.05	1.109	148.2	59.3	103.941
					10-5	105.11	1.164	148.2	63.4	103.946
					17-2	105.05	1.100	148.2	66.6	103.950
					10-4	105.11	1.158	148.2	68.6	103.952
					17-3	105.05	1.095	148.2	71.3	103.955
					10-3	105.11	1.151	148.2	75.4	103.959
					17-4	105.05	1.087	148.2	78.1	103.963
					10-2	105.11	1.145	148.2	80.5	103.965
					17-5	105.05	1.082	148.2	83.2	103.968
					10-1	105.11	1.137	148.2	87.3	103.973
					17-6	105.05	1.074	148.2	90	103.976
					9-6	105.11	1.128	148.2	95.1	103.982
					18-1	105.05	1.066	148.2	97.7	103.984
					9-5	105.11	1.121	148.2	101.9	103.989
					18-2	105.05	1.058	148.2	104.6	103.992
					9-4	105.11	1.115	148.2	107	103.995
					18-3	105.05	1.052	148.2	109.7	103.998
					9-3	105.11	1.107	148.2	113.8	104.003
					18-4	105.05	1.044	148.2	116.5	104.006
					9-2	105.11	1.102	148.2	119	104.008
					18-5	105.05	1.039	148.2	121.6	104.011
					9-1	105.11	1.094	148.2	125.8	104.016
					18-6	105.05	1.031	148.2	128.5	104.019
					8-2	105.13	1.104	148.2	134.6	104.026
					19-1	105.34	1.312	148.2	136.2	104.028
					8-1	105.13	1.097	148.2	141.4	104.033
					19-2	105.34	1.305	148.2	143	104.035
MH-30_DS	MH-3	103.039	102.914	75						
					13-6	102.65	-0.291	75.0	16.4	102.941
					14-1	102.8	-0.146	75.0	19	102.946

Table 3B: Future Composite Hydraulic Gradeline Results for 100-Year Design Storms

U/S MH	D/S MH	Max. U/S HGL (m)	Max. D/S HGL (m)	Pipe Length (m)	Lot Number	USF (m)	Freeboard [2] (m)	Interpolated HGL Length HGL (m)	Interpolated HGL Dist. From D/S MH (m)	Interpolated HGL HGL (m)
					13-5	102.65	-0.303	75.0	23.2	102.953
					14-2	102.8	-0.157	75.0	25.9	102.957
					13-4	103.13	0.169	75.0	28.3	102.961
					14-3	103.28	0.314	75.0	31	102.966
					13-3	103.13	0.157	75.0	35.1	102.973
					14-4	103.28	0.303	75.0	37.8	102.977
					13-2	103.65	0.669	75.0	40.3	102.981
					14-5	103.78	0.794	75.0	42.9	102.986
					13-1	103.65	0.657	75.0	47.1	102.993
					14-6	103.78	0.783	75.0	49.8	102.997
					12-6	104.19	1.185	75.0	54.8	103.005
					15-1	104.34	1.330	75.0	57.5	103.010
					12-5	104.19	1.173	75.0	61.6	103.017
					15-2	104.34	1.319	75.0	64.3	103.021
					12-4	104.64	1.615	75.0	66.8	103.025
					15-3	104.73	1.700	75.0	69.4	103.030
					12-3	104.64	1.603	75.0	73.6	103.037
MH-7	MH-8	103.878	103.491	79						
MH-8	MH-9	103.491	102.765	100						
MH-9	MH-10	102.765	102.632	95						
MH-10	MH-11	102.582	102.25	102						
MH-11	MH-12	102.14	102.02	55.5						
MH-12	MH-13	101.94	101.51	93.5						
MH-13	MH-14	101.45	101.45	10						
MH-14	MH-15	101.4	100.97	76.5						
MH-15	MH-16	100.94	100.33	95						
MH-16	MH-17	100.32	99.96	94.85						
MH-17	MH-25	99.94	99.92	26.15						
MH-25	MH-26	99.89	99.74	23.9						
MH-26	MH-27	99.72	99.536	85.21						
MH-203	MH-202	102.8	102.847	20.834						
MH-202	MH-201	102.797	102.839	11.946						
MH-201	MH-5	102.799	102.845	18.995						

Note: [1] A negative surcharge implies that the pipe is not flowing full

[2] Conservative estimate of freeboard based on U/S HGL and lowest USF connected to pipe. Actual HGL / freeboard at all connecting lots

[3] Interim USF elevations estimated as 1.8 m below the upstream top of manhole elevations.

Interpolated HGL elevation (green)

Freebaord Less than 0.3m from USF (yellow)

Freeboard above USF (red)

8 EROSION AND SEDIMENT CONTROL DURING AND AFTER CONSTRUCTION

Silt and erosion control strategies shall be implemented during construction activities in order to minimize the transfer of silt off-site. The following measures should be implemented:

i) Silt control fences shall be installed as required in order to prevent the movement of silt off-site during rainfall events.

ii) Construction of a mud mat shall be installed at the site entrance in order to promote self-cleaning of truck tires when leaving the site.

iii) All catch basins shall be equipped with a crushed stone filter in order to prevent the capture of silt in the storm sewer system.

iv) Regular cleaning of the adjacent roads shall be undertaken during the construction activities.

v) Regular inspection and maintenance of the silt control measures shall be undertaken until the site has been stabilized.

vi) The erosion and sediment control devices shall be removed after the site has been stabilized.

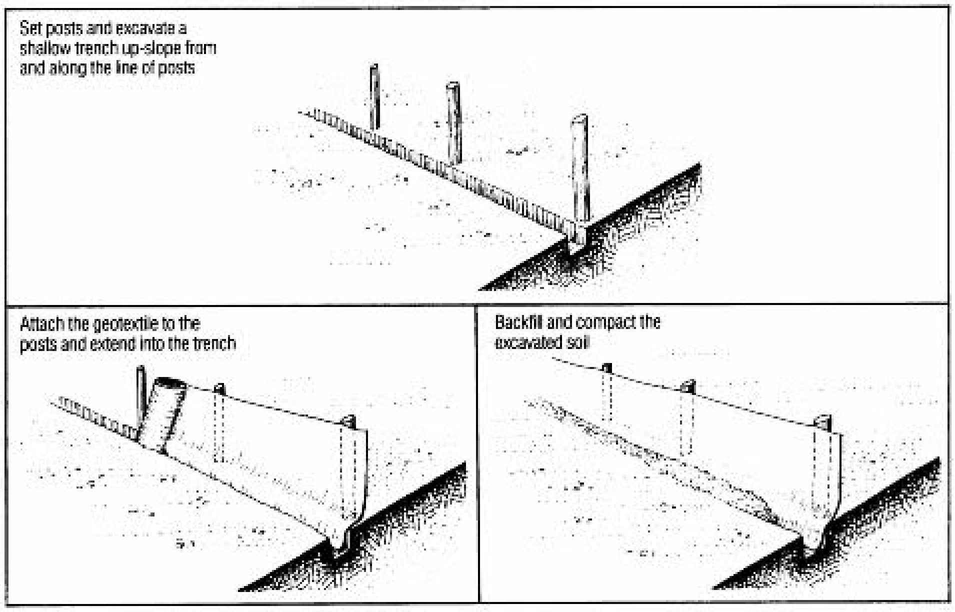


Figure 5: Typical installation of silt fences

Figure 6: Catchbasin with geotextile to protect storm sewer pipes from sediment contamination

9 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

JFSA has prepared a Stormwater Management (SWM) Plan for the proposed residential development at 5 Orchard Drive, located in Stittsville Ontario, south of Hazeldean Road, off Fringewood Drive. The 5 Orchard Drive Development has a total drainage area of approximately 2.13 ha and will comprise of a mix of single-detached houses and townhouses. The proposed development will have 3 onsite storage units in place along an 8 m City block between the commercial and residential on the northern extent of the residential development to control runoff from the proposed development and will connect to the existing trunk sewer that runs along Hazeldean Road, that connects to an interim SWM pond before discharging to Hazeldean Creek

In accordance with the City of Ottawa design guidelines, the minor system has been designed to accommodate a minimum of the 2-year post-development flows from within the site and from external areas (plus 5-year flows on collector and 10-year flows on arterial roads). PCSWMM model analyses have determined that the minor system will surcharge in most parts of the system. However, with the use of Inlet Control Devices, a minimum freeboard of 0.3 m is provided between the 100-year hydraulic gradeline and the underside of footings throughout the subdivision.

The PCSWMM simulations have determined that for the selected 2-, 5-, 10- and 100-year storms, the total minor system flows to the Interim SWM Pond / OGS unit would be 0.545 m^3/s, 0.746 m^3/s, 0.869 m^3/s and 1.077 m^3/s, respectively.

Within the subdivision, the peak water depths do not exceed the maximum allowable 35 cm depth at the gutter for the simulated 100-year storm (refer to Calculation Sheet 1A and Table D-2 of Appendix D). Furthermore, it was determined that for the 100-year event, the product of the depth of water (m) at the gutter multiplied by the velocity of flow (m/s) will not exceed the maximum allowable 0.6 m^2/s (refer to Calculation Sheet 1A of Appendix D, where the calculated maximum was 0.093 m^2/s). Refer below for an assessment of static ponding depth on the road.

Table C-2A and C-3A of Appendix C summarize the hydraulic grade line analysis. Note that the full pipe velocities are generally no less than 0.80 m/s and no greater than 3.0 m/s for the proposed pipes. Where velocities in excess of 3.0 m/s are proposed, provisions shall be made to protect against displacement of sewers by sudden jarring or movement.

Stress test results for the major and minor drainage systems based on a 20% increase in the 100-year storm, as per the October 2012 *City of Ottawa Sewer Design Guidelines*, are summarized in Section 5. Recommendations for silt and erosion control strategies to be implemented during construction are presented in Section 8.

In conclusion, the proposed design satisfies all selected design guidelines and requirements.

Appendix D

Pre - Consultation

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

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 E

Supporting Storm Sewer Information and Calculations

- Email Communication from Campanale Group regarding Restricted Outflow Rate (November 2019)
- Rational Method Spreadsheet Calculations for Site Storm Sewer System
- Stormwater Calculations – Interim Conditions External Future Commercial Site

Brown, Rikke

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

Hi Qasim,
Any issues with the Second item below?

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

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

Hi Joel,

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

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

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

Let me know if you have any questions.

Thanks,
Cody

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

Direct Line: 613-247-3089

STORM SEWER DESIGN - COMPUTATION FORM

Project Name: Shell Hazeldean and Fringewood Ottawa
Job No: 2020072
Date Created: 8/28/2020

1:5 year intensity (mm/hr) **104.19**
ST Ci **2084**

storm event	constant	tc	constant	intensity
1:5	2084	10.00	10.00	104.19

Storm Event 5
a 998.017
b 0.814
c 6.053
I=A/(t + C)^B

Use PVC Pipe for sizes less than:	Use Concrete Pipe for sizes greater than:	Friction Coefficients: (n)	
		0.011	0.013
450	450	UR	Concrete
		PVC	

0.90<v<3.0

Computed By: Yvonne
Date: 9/15/2020

Catchment Area Design														Pipe Design									Capacity Check	
Drainage Area Label	Manhole U/S	Manhole D/S	Drainage Area (ha	Sum Area (ha)	Runoff Factor C	Equivalent Area (ha) AxC	Cumulative Area (ha) AxC	U/S Tc (min)	D/S Tc (min)	Intensity (mm/hr)	Incremental Design Flow, (L/s)	Design Flow (L/s)	Restricted Flow, (L/s)	Pipe Slope (%)	Nominal Pipe Diameter (mm)	Pipe Material	Friction Coeff. (n)	Pipe Length (m)	Pipe Capacity (L/s)	Velocity (m/s)	Time of Flow (min)	Actual Pipe Diameter (mm)	Design / Cap. Ratio	Pipe OK or SURCHARGED
A6-2	CB12	CBMH03	0.0230	0.023	0.35	0.008	0.008	10.00	10.12	104.19	2.3	2.3	2.3	0.325%	675	CON	0.013	9.488	500.3	1.35	0.12	686.0	0.00	OK
A3, A9, A8	CBMH06	MH07	0.0680	0.068	0.71	0.048	0.048	10.00	10.29	104.19	13.9	13.9	13.9	0.350%	750	CON	0.013	26.000	687.1	1.51	0.29	762.0	0.02	OK
	MH07	MH11	0.0000	0.068	1.00	0.000	0.048	10.29	10.51	102.71	0.0	13.9	13.9	0.350%	750	CON	0.013	20.203	687.1	1.51	0.22	762.0	0.02	OK
	MH11	CBMH02	0.0000	0.068	1.00	0.000	0.048	10.51	10.68	101.59	0.0	13.9	13.9	0.350%	1050	CON	0.013	19.006	1686.2	1.89	0.17	1067.0	0.01	OK
	MH09	CBMH01	0.0000	0.000	1.00	0.000	0.000	10.00	10.19	104.19	0.0	0.0	0.0	0.350%	1050	CON	0.013	21.000	1686.2	1.89	0.19	1067.0	0.00	OK
A4	CBMH13	CBMH01	0.0310	0.031	0.76	0.024	0.024	10.00	10.12	104.19	6.8	6.8	6.8	0.350%	1050	CON	0.013	14.123	1686.2	1.89	0.12	1067.0	0.00	OK
A1	CBMH01	CBMH02	0.0590	0.090	0.80	0.047	0.071	10.12	10.32	103.54	13.6	20.4	20.4	0.350%	1050	CON	0.013	22.603	1686.2	1.89	0.20	1067.0	0.01	OK
A2	CBMH02	CBMH03	0.1060	0.264	0.85	0.090	0.209	10.32	10.48	102.53	25.7	60.0	60.0	0.350%	1050	CON	0.013	17.276	1686.2	1.89	0.15	1067.0	0.04	OK
A5	CBMH03	MH01	0.0350	0.322	0.66	0.023	0.240	10.48	10.59	101.76	6.5	68.8	68.8	0.350%	1050	CON	0.013	13.145	1686.2	1.89	0.12	1067.0	0.04	OK
	MH01	OGS	0.0000	0.322	1.00	0.000	0.240	10.59	10.64	101.19	0.0	68.8	3.71	0.350%	450	PVC	0.011	3.290	196.8	1.25	0.04	447.9	0.02	OK
	OGS	MH201	0.0000	0.322	1.00	0.000	0.240	10.64	10.68	100.97	0.0	68.8	3.71	0.350%	675	CON	0.013	3.840	519.2	1.40	0.05	686.0	0.01	OK
	MH201	STM MH 5	0.0000	0.322	1.00	0.000	0.240	10.68	10.87	100.75	0.0	68.8	28.6	0.490%	675	CON	0.013	19.085	614.3	1.66	0.19	686.0	0.05	OK
C1*	PLUG	MH202	1.4900	1.490	0.80	1.192	1.192	10.00	10.50	104.19	345.3	345.3	24.9	0.300%	675	CON	0.013	39.066	480.7	1.30	0.50	686.0	0.05	OK
	MH202	MH201	0.0000	1.490	1.00	0.000	1.192	10.50	10.60	101.65	0.0	345.3	24.9	0.150%	675	CON	0.013	5.460	339.9	0.92	0.10	686.0	0.07	OK

*Note the Restricted Flow from the connection to the adjacent commercial development (C1) is based on a 5-year release rate of 30.3 L/s per 1.82ha (source: Functional Servicing and Stormwater Management Report for Campanale Homes 5 Orchard Drive" by DSEL, March 2019.
Calculation used: 30.3L/s x (1.82 ha - 0.333ha) / 1.82ha = 24.9 L/s

Project Number:	ctm 2020072		Revsied by:	Yvonne Faas
Name	Hazeldean & Fringewood		Date Started:	Aug 1 2020
client	AECOM/Shell		Date revised:	Aug 12 2020
Location:	Ottawa ON			

AREA STATEMENT (IN HECTARES)					
Sub Catchment	PAVED AREA (ha)	LANDSCAPE AREA (ha)	ROOF TOP AREA (ha)	TOTAL AREA (ha)	Weighted Runoff Coefficient, c
A1	0.0491	0.0099		0.0590	0.80
A2	0.0780	0.0080	0.0200	0.1060	0.85
A3	0.0250	0.0050		0.0300	0.80
A4	0.0070	0.0070	0.0170	0.0310	0.76
A5	0.0210	0.0140		0.0350	0.66
A6-1*	0.0000	0.0070		0.0070	0.30
A6-2	0.0020	0.0210		0.0230	0.35
A7*	0.0040			0.0040	0.90
A8 (External Area - Runon)	0.0100	0.0170		0.0270	0.52
A9			0.011	0.0110	0.90
B1 (External Area, Interim)	0.0583	1.4317		1.4900	0.32
C1 (External Area, Ultimate)	1.2417	0.2483		1.4900	0.80

Appendix F

Supporting Stormwater Management Information and Calculations

- Airport Calculation
- Peak Flow Calculation, Existing Conditions
- Peak Flow Calculation, Proposed Conditions
- Required Storage, Modified Rational Method
- Available Storage within Shell Site
- Stage-Storage Tables for Ponding Areas
- Time-Depth/Elevation Curve for Outfall
- Detailed OGS Sizing Report
- Hydrovex ICD
- Hydraulic Losses at Bends Chart, as per Ottawa Sewer Design Guidelines
- Surface Inlet Capacity at Road Sages Chart, as per Ottawa Sewer Design Guidelines
- Water Balance 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

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)

2 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	0.059	0.0491	0.010		0.90	0.3	0.80	76.805	10.07
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	0.031	0.0070	0.007	0.017	0.90	0.3	0.76	76.805	5.06
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	76.805	4.93
Catchment A6-1	0.007	0.0000	0.007		0.90	0.3	0.30	76.805	0.45
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	0.1961	0.089	0.048	0.90	0.3	0.74	76.805	52.60

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

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	0.059	0.0491	0.010		0.90	0.3	0.80	104.193	13.66
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	0.031	0.0070	0.007	0.017	0.90	0.3	0.76	104.193	6.86
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	104.193	6.69
Catchment A6-1	0.007	0.0000	0.007		0.90	0.3	0.30	104.193	0.61
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	0.1961	0.089	0.048	0.90	0.3	0.74	104.193	71.36

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

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	0.059	0.0491	0.010		0.90	0.3	0.80	122.142	16.01
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	0.031	0.0070	0.007	0.017	0.90	0.3	0.76	122.142	8.05
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	122.142	7.84
Catchment A6-1	0.007	0.0000	0.007		0.90	0.3	0.30	122.142	0.71
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	0.1961	0.089	0.048	0.90	0.3	0.74	122.142	83.65

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

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	0.059	0.0491	0.010		0.90	0.3	0.80	0.88	144.693	20.87
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	0.031	0.0070	0.007	0.017	0.90	0.3	0.76	0.84	144.693	10.49
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	0.0000	0.007		0.90	0.3	0.30	0.33	144.693	0.93
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	0.1961	0.089	0.048	0.90	0.3	0.74	0.81	144.693	109.01

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)

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	C Weighted (Including Add 20% Value)	New C Weighted (Including Add 20% Value)	I (mm/hr)	Q (l/s)
Catchment A1	0.059	0.0491	0.010		0.90	0.3	0.80	0.96	1.00	161.471	25.40
Catchment A2	0.106	0.0780	0.008	0.020	0.90	0.3	0.85	1.03	1.00	161.471	48.80
Catchment A3	0.030	0.0250	0.005		0.90	0.3	0.80	0.96	0.96	161.471	12.93
Catchment A4	0.031	0.0070	0.007	0.017	0.90	0.3	0.76	0.92	0.92	161.471	12.77
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	0.79	0.79	161.471	12.44
Catchment A6-1	0.007	0.0000	0.007		0.90	0.3	0.30	0.36	0.36	161.471	1.13
Catchment A6-2	0.023	0.0020	0.021		0.90	0.3	0.35	0.42	0.42	161.471	4.36
Catchment A7	0.004	0.0040			0.90	0.3	0.90	1.08	1.00	161.471	1.94
Catchment A8	0.027	0.0100	0.017		0.90	0.3	0.52	0.63	0.63	161.471	7.60
Catchment A9	0.011			0.011	0.90	0.3	0.90	1.08	1.00	161.471	5.33
Total Area	0.333	0.1961	0.089	0.048	0.90	0.3	0.74	0.89	0.88	161.471	132.71

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)

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	C Weighted (Including Add 25% Value)	New C Weighted (Including Add 25% Value)	I (mm/hr)	Q (l/s)
Catchment A1	0.059	0.0491	0.010		0.90	0.3	0.80	1.00	1.00	178.559	29.29
Catchment A2	0.106	0.0780	0.008	0.020	0.90	0.3	0.85	1.07	1.00	178.559	52.62
Catchment A3	0.030	0.0250	0.005		0.90	0.3	0.80	1.00	1.00	178.559	14.89
Catchment A4	0.031	0.0070	0.007	0.017	0.90	0.3	0.76	0.96	0.96	178.559	14.71
Catchment A5	0.035	0.0210	0.014		0.90	0.3	0.66	0.83	0.83	178.559	14.33
Catchment A6-1	0.007	0.0000	0.007		0.90	0.3	0.30	0.38	0.38	178.559	1.30
Catchment A6-2	0.023	0.0020	0.021		0.90	0.3	0.35	0.44	0.44	178.559	5.03
Catchment A7	0.004	0.0040			0.90	0.3	0.90	1.13	1.00	178.559	1.99
Catchment A8	0.027	0.0100	0.017		0.90	0.3	0.52	0.65	0.65	178.559	8.75
Catchment A9	0.011			0.011	0.90	0.3	0.90	1.13	1.00	178.559	5.46
Total Area	0.333	0.1961	0.089	0.048	0.90	0.3	0.74	0.92	0.90	178.559	148.36

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

Modified Rational Method

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

Area =	**0.322**	ha
"C" =	**0.91**	
AC=	**0.29302**	
Tc =	**10.0**	min
Time Increment =	**5.0**	min
Release Rate =	**9.17**	l/s
Max.Storage =	**132**	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	145.45	87.3	5.5	81.8	
15.0	142.9	116.40	104.8	8.3	96.5	
20.0	120.0	97.71	117.3	11.0	106.2	
25.0	103.8	84.59	126.9	13.8	113.1	
30.0	91.9	74.84	134.7	16.5	118.2	
35.0	82.6	67.27	141.3	19.3	122.0	
40.0	75.1	61.21	146.9	22.0	124.9	
45.0	69.1	56.25	151.9	24.8	127.1	
50.0	64.0	52.10	156.3	27.5	128.8	
55.0	59.6	48.57	160.3	30.3	130.0	
60.0	55.9	45.53	163.9	33.0	130.9	
65.0	52.6	42.89	167.3	35.8	131.5	
70.0	49.8	40.56	170.3	38.5	131.8	
75.0	47.3	38.49	173.2	41.3	132.0	<<<<
80.0	45.0	36.65	175.9	44.0	131.9	
85.0	43.0	34.99	178.4	46.8	131.7	
90.0	41.1	33.49	180.8	49.5	131.3	
95.0	39.4	32.12	183.1	52.3	130.8	
100.0	37.9	30.88	185.3	55.0	130.2	
105.0	36.5	29.73	187.3	57.8	129.5	
110.0	35.2	28.68	189.3	60.5	128.7	
115.0	34.0	27.70	191.1	63.3	127.9	
120.0	32.9	26.80	192.9	66.0	126.9	
125.0	31.9	25.95	194.7	68.8	125.9	

Available Storage Within the Shell Site:

Surface Storage			
Name	**Ponding Depth (m)**	**Area(m2)**	**Storage (m3)**
Catchment A1	0.13	177	7.67
Catchment A2	0.13	156	6.76
Catchment A3	0.06	45	0.90
Catchment A4	0.11	38	1.39
Catchment A5	0.13	88	3.81
Catchment A6-2	0.3	25	2.50
Total Volume (m3)			**20.54**

CBMHs/MHs Storage					
Name	**Dia (mm)**	**Dia (m)**	**Depth (m)**	**Area(m2)**	**Volume(m3)**
CBMH13	1800	1.8	2.052	2.54	5.22
MH09	1800	1.8	2.447	2.54	6.22
CBMH01	2400	2.4	2.031	4.52	9.18
CBMH02	2400	2.4	2.17	4.52	9.81
CBMH03	2400	2.4	2.231	4.52	10.09
DICB12	1500	1.5	1.285	1.77	2.27
MH01	1800	1.8	2.568	2.54	6.53
MH201	1500	1.5	2.821	1.77	4.98
CBMH06	1500	1.5	1.592	1.77	2.81
MH07	1800	1.8	2.103	2.54	5.35
MH11	2400	2.4	2.414	4.52	10.92
Total Volume (m3)					**73.39**

Oversized Pipe Storage		
Pipe Dia (mm)	**Length (m)**	**Storage (m3)**
1050	107.153	92.74
750	46.203	20.40
675	9.488	3.39
Total Volume (m3)		**116.53**

Stage Storage Tables for Ponding Areas

Area A1				
Depth (m)	Elevation (m)	Area (m^2)	Incremental Volume (m^3)	Volume (m^3)
0.000	105.100	0		0
0.100	105.200	80	4.00	4.00
0.130	105.230	177	3.86	7.86

Area A2				
Depth (m)	Elevation (m)	Area (m^2)	Incremental Volume (m^3)	Volume (m^3)
0.000	105.100	0		0
0.100	105.200	70	3.50	3.50
0.130	105.230	156	3.39	6.89

Area A3				
Depth (m)	Elevation (m)	Area (m^2)	Incremental Volume (m^3)	Volume (m^3)
0.000	105.170	0		0
0.030	105.200	5	0.08	0.08
0.060	105.230	45	0.75	0.83

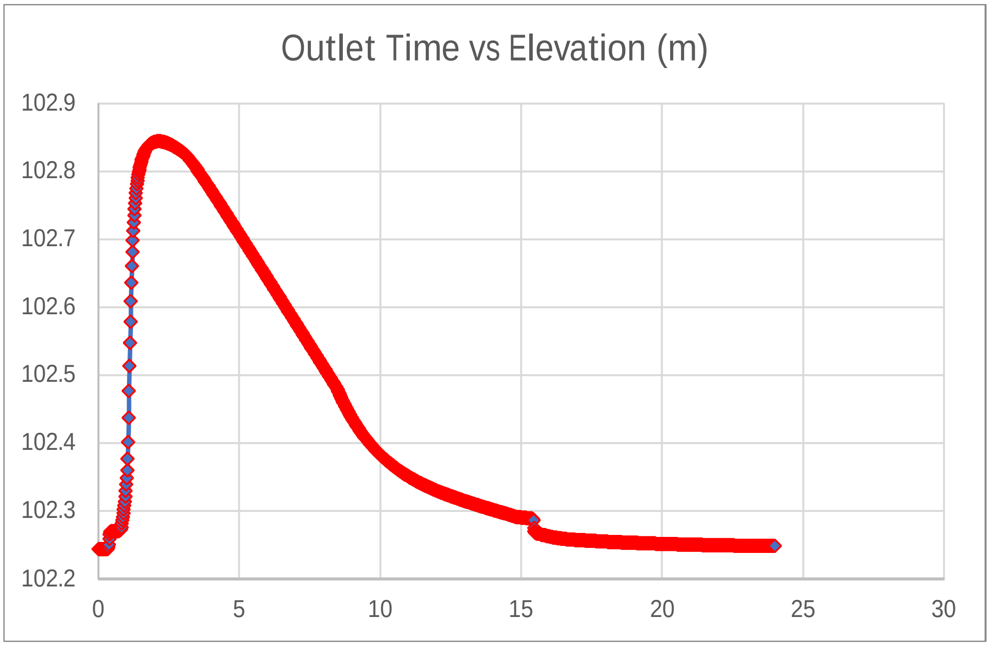
Area A4				
Depth (m)	Elevation (m)	Area (m^2)	Incremental Volume (m^3)	Volume (m^3)
0.000	105.230	0		0
0.100	105.330	22	1.10	1.10
0.110	105.340	38	0.30	1.40

Area A5				
Depth (m)	Elevation (m)	Area (m^2)	Incremental Volume (m^3)	Volume (m^3)
0.000	105.100	0		0
0.100	105.200	35	1.75	1.75
0.130	105.230	88	1.85	3.60

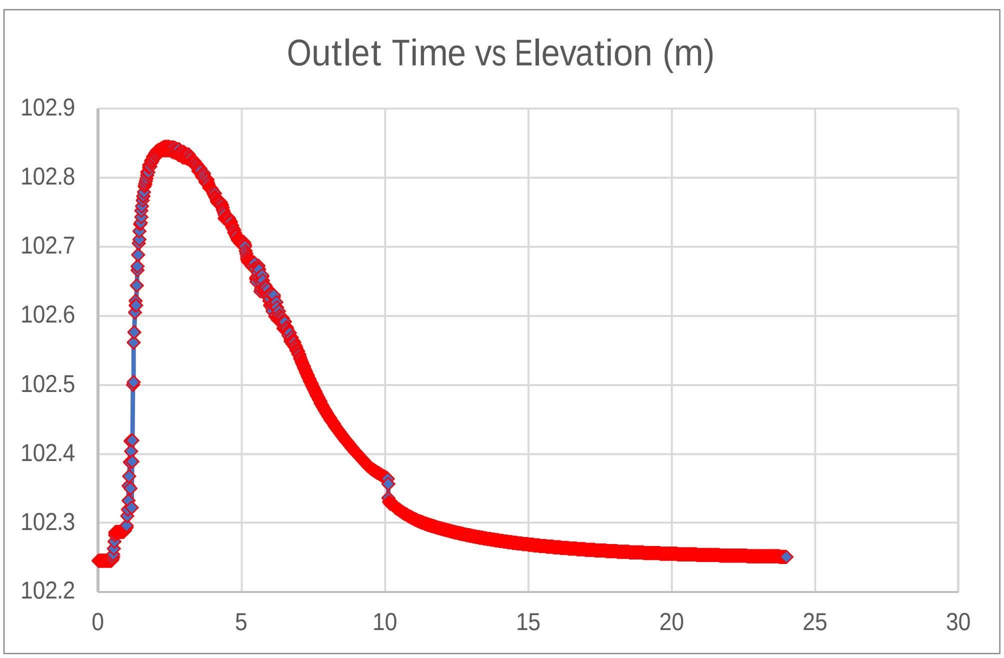
Area A6-2				
Depth (m)	Elevation (m)	Area (m^2)	Incremental Volume (m^3)	Volume (m^3)
0.000	104.500	0		0
0.150	104.650	8	0.60	0.60
0.300	104.800	25	2.47	3.07

Outlet Invert:	102.245
HGL (DSEL Report)	102.845

Time-depth/elevation curve developed for Outfall, 100-Year Storm Event



Time-depth/elevation curve developed for Outfall, 5-Year Storm Event



September 5, 2020

ADS UFF Sizing Summary

Project Name: Shell, 5 Orchard Drive
Consulting Engineer: AECOM
Location: Ottawa, ON
Sizing Completed By: Haider Nasrullah **Email:** haider.nasrullah@ads-pipe.com

Recommended Unit	
Recommended Model:	UFF-5
TSS Removal Percentage:	82.6%
Total Site Volume Treated:	91.4%

Site Details	
Site Area:	0.322 ha
% Impervious:	100%
Rational C:	0.91
Rainfall Station:	Ottawa, ONT
Particle Size Distribution:	ETV / NJDEP

Unit Specifications:	
Number of Filter Modules:	5
Maximum Treatment Flowrate:	8 L/s
Inlet - Outlet Drop:	240 mm*
Max. Pipe Diameter:	600 mm
Operating Head:	760 mm

* Drop across unit can be reduced when required.

Site Elevations:	
Rim Elevation:	0.00
Inlet Pipe Elevation:	0.00
Outlet Pipe Elevation:	0.00

Consult approved shop drawings for final elevations. Riser sections (and/or grade rings) may be required to reach final grade on site.

Rainfall Intensity[1]	Fraction of Rainfall[1]	Removal Efficiency[2]	Weighted Net-Annual Removal Efficiency
mm/hr	%	%	%
0.50	0.1%	92.4%	0.1%
1.00	14.1%	91.6%	12.9%
1.50	14.2%	90.8%	12.9%
2.00	14.1%	90.1%	12.7%
2.50	4.2%	89.3%	3.7%
3.00	1.5%	88.5%	1.3%
3.50	8.5%	87.7%	7.5%
4.00	5.4%	86.9%	4.7%
4.50	1.2%	86.1%	1.0%
5.00	5.5%	85.3%	4.7%
6.00	4.3%	83.8%	3.6%
7.00	4.5%	82.2%	3.7%
8.00	3.1%	80.6%	2.5%
9.00	2.3%	79.1%	1.8%
10.00	2.6%	77.5%	2.0%
20.00	9.2%	61.8%	5.7%
30.00	2.6%	46.1%	1.2%
40.00	1.2%	30.4%	0.4%
50.00	0.5%	14.7%	0.1%
100.00	0.7%	0.0%	0.0%
150.00	0.1%	0.0%	0.0%
Net Annual Treatment			**82.6%**
Total Runoff Volume Treated:			**91.4%**

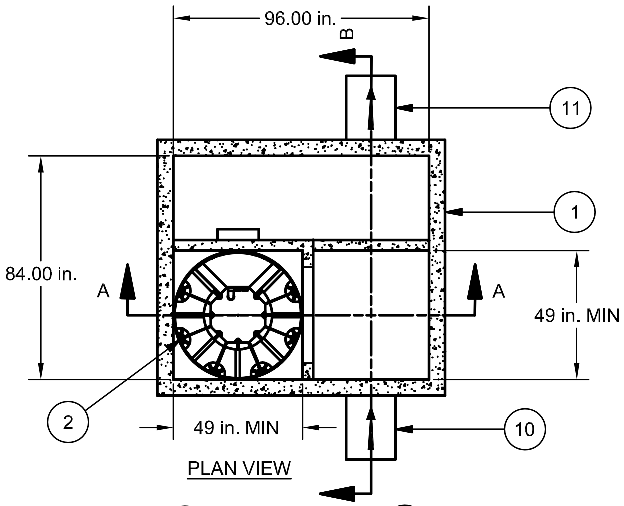
Rainfall Data: 1960:2007, HLY03, Ottawa, ONT, 6105976 & 6105978.

Notes:

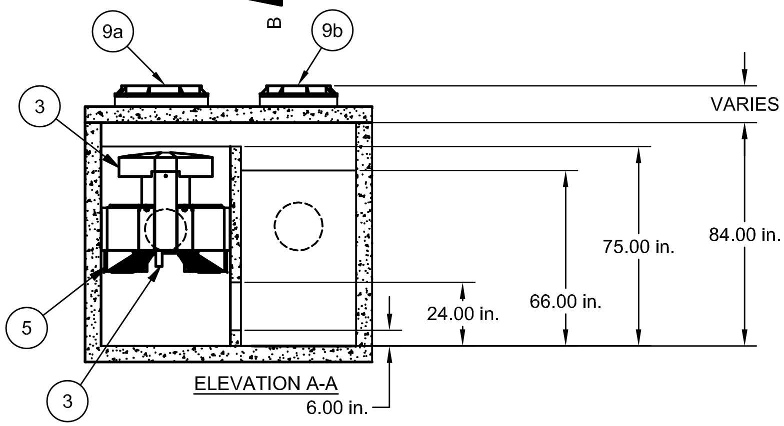
Removal efficiencies are based on NJDEP Test Protocols and independently verified.

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.

NOT FOR CONSTRUCTION - CONTACT HYDRO INTERNATIONAL FOR SITE SPECIFIC DRAWINGS

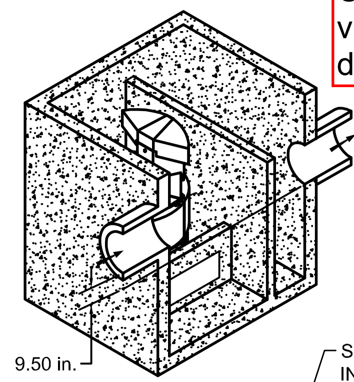


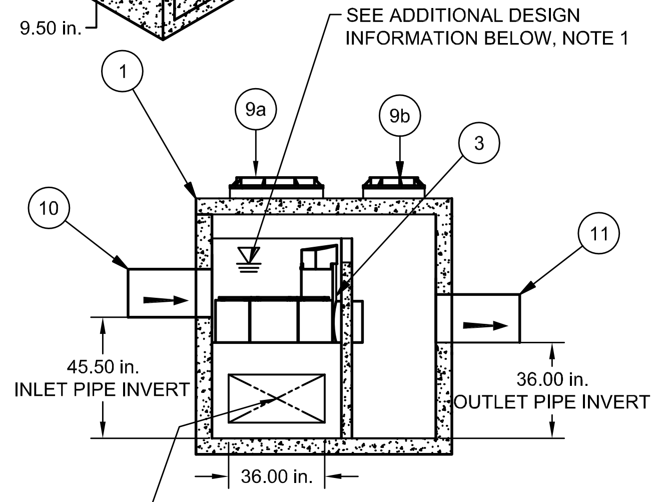
PLAN VIEW



ELEVATION A-A

1,000L Oil Storage Capacity to be met via additional baffle design.





ELEVATION B-B

Notes

1. MANHOLE WALL AND SLAB THICKNESSES ARE NOT TO SCALE.
2. CONTACT HYDRO INTERNATIONAL FOR A BOTTOM OF STRUCTURE ELEVATION PRIOR TO SETTING DOWNSTREAM DEFENDER MANHOLE.

REVISION HISTORY			
REV	BY	DATE	DESCRIPTION
B	JLL	2/02/15	NOTES

Date	Scale
12/28/2011	NTS

Drawn	Checked	Approved
EMH	MRJ	MRJ

Title

7-FT x 8-FT VAULT
UP-FLO FILTER
1-7 MODULES

GENERAL ARRANGEMENTS

Parts List			
ITEM	QTY	DESCRIPTION	SIZE
1	1	PRECAST VAULT (BY HYDRO VIA PRECASTER)	84 in x 96 in
2	1-7	FILTER MODULE (7 SHOWN)	
	1-7	FILTER BAG SET	
3	1	OUTLET MODULE W/ BRACKET, BYPASS HOOD, DRAINDOWN	
4	0	DUAL SUPPORT BRACKET W/ ANGLED SCREEN	
5	1-7	SINGLE SUPPORT BRACKET W/ ANGLED SCREEN	
6	1-7	BACKER PLATE FOR SINGLE SUPPORT	
7	1	MOUNTING SPOOL	
8	0	ADDITIONAL DRAINDOWN	
9a, 9b	1,2	FRAME AND COVERS	30 in, 24 in
10	1	INLET PIPE (BY OTHERS)	12 in
11	1	OUTLET PIPE (BY OTHERS)	12 in

CAPACITIES:

1. Minimum performance: 80% removal of Sil-Co-Sil 106 (d50 = 22 microns) at the peak treatment flow.
2. NJDEP peak treatment flow: 0.056 cfs/module * OR 0.66 acres of imperviousness/module, whichever results in the greater number of modules.
3. Maximum number of modules: 7 **

ADDITIONAL DESIGN INFORMATION:

1. * Normal operating W.S.E. is 2.46' above the outlet invert at the peak treatment flow of 0.056 cfs/module. For a given flow the head requirement can be reduced by adding additional filters.
2. ** Treatment flows that require more modules will require a larger vault design.
3. Media Type: CPZ

Hydro International®

Stormwater Solutions
94 Hutchins Drive
Portland, Maine 04102
Tel: (207) 756-6200
Fax: (207) 756-6212
stormwaterinquiry@hydro-int.com

CAD Ref: **UFGA - 1RV**

Project No.

Drawing No. Rev. B

Nikfarjam, Toktam

From:	Haider Nasrullah <Haider.Nasrullah@ads-pipe.com>
Sent:	Tuesday, September 8, 2020 12:18 PM
To:	Nikfarjam, Toktam
Cc:	Cody Neath; Michael Reid
Subject:	[EXTERNAL] RE: OGS Sizing, City of Ottawa
Attachments:	ETV-OGS-Procedure_final_revised-June_2014.pdf; Up-Flo 7 Module Standard Detail.pdf

Hi Toktam,

My weekend was good. Hope yours was as well! The oil capacity of 1000L will be met by implementing an additional baffle. Please see below the Particle Size Distribution used to size the unit. This data is derived from the above attached report that indicates the procedure for ETV testing.

3.1 Test Sediment

The test sediment used for sediment removal performance testing shall be comprised of inorganic ground silica with a specific gravity of 2.65, uniformly mixed to meet the particle size distribution shown in Table 1. The PSD includes a broad range of particles from clay to coarse sand.

Table 1: Particle Size Distribution of Test Sediment

Particle Size (µm)	Percent Less Than	Particle Size Fraction (µm)	Percent
1000	100	500-1000	5
500	95	250-500	5
250	90	150-250	15
150	75	100-150	15
100	60	75-100	10
75	50	50-75	5
50	45	20-50	10
20	35	8-20	15
8	20	5-8	10
5	10	2-5	5
2	5	<2	5

Regards,

Haider Nasrullah, P.Eng.
Engineered Products Manager

ADS Canada

Cell: 647-850-9417
Email: haider.nasrullah@ads-pipe.com
Website: www.ads-pipecanada.com

From: Nikfarjam, Toktam <toktam.nikfarjam@aecom.com>
Sent: September 8, 2020 10:45 AM
To: Haider Nasrullah <Haider.Nasrullah@ads-pipe.com>
Cc: Cody Neath <Cody.Neath@ads-pipe.com>; Michael Reid <Michael.Reid@ads-pipe.com>
Subject: RE: OGS Sizing, City of Ottawa

CAUTION: This email originated outside of ADS. Be cautious when opening any links or documents. If you have questions, contact ITSecurity@ads-pipe.com.

Hi Haider,

Hope you had a great weekend. I just have a question, I need the oil capacity of OGS as well. As I mentioned before, OGS on Shell site will require 1000 L oil holding capacity. Does this OGS provide the require 1000 L oil capacity? Also, can you please provide the particle size distribution of OGS?

Thanks,
Toktam

From: Haider Nasrullah <Haider.Nasrullah@ads-pipe.com>
Sent: Saturday, September 5, 2020 1:19 PM
To: Nikfarjam, Toktam <toktam.nikfarjam@aecom.com>
Cc: Cody Neath <Cody.Neath@ads-pipe.com>; Michael Reid <Michael.Reid@ads-pipe.com>
Subject: [EXTERNAL] RE: OGS Sizing, City of Ottawa

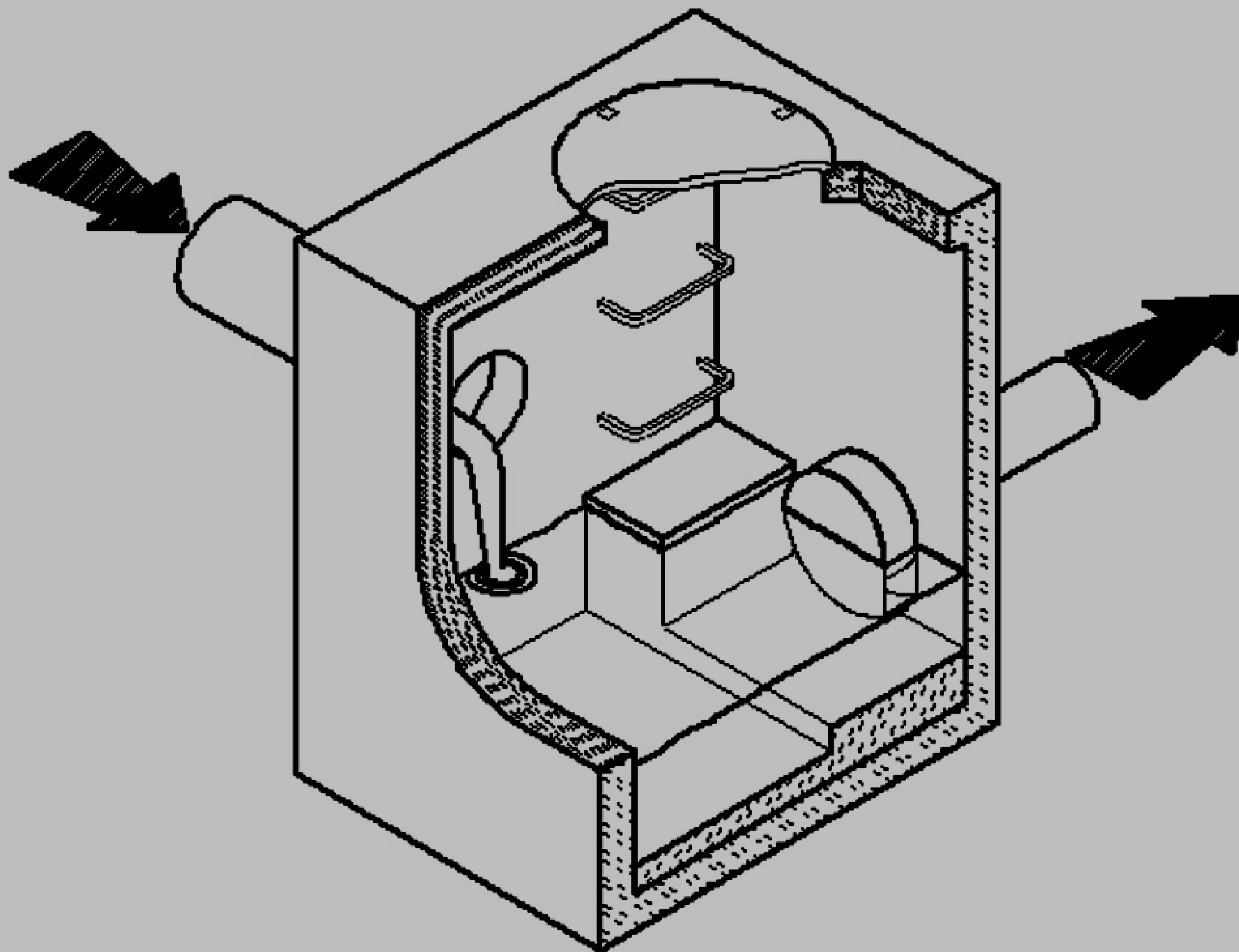
Hi Toktam,

Please see the revised sizing report with the updated runoff coefficient of 0.91. We are still looking at similar size vault (2.1x2.4m) with 5 filters now instead of 4. The change in pipe should not be a problem since the incoming pipe to the Upflo filter remains the same at 450mm. Please ensure that a minimum drop of 0.240m is provided from the inlet to the outlet of the vault. The sizing for the Filter is based on the Canadian ETV Particle Size Distribution. Feel free to give me a call if you have any questions. Thanks!

Regards,

Haider Nasrullah, P.Eng.
Engineered Products Manager
ADS Canada
Cell: 647-850-9417
Email: haider.nasrullah@ads-pipe.com
Website: www.ads-pipecanada.com

From: Nikfarjam, Toktam <toktam.nikfarjam@aecom.com>
Sent: September 5, 2020 12:48 PM
To: Haider Nasrullah <Haider.Nasrullah@ads-pipe.com>
Cc: Cody Neath <Cody.Neath@ads-pipe.com>; Michael Reid <Michael.Reid@ads-pipe.com>
Subject: RE: OGS Sizing, City of Ottawa



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.

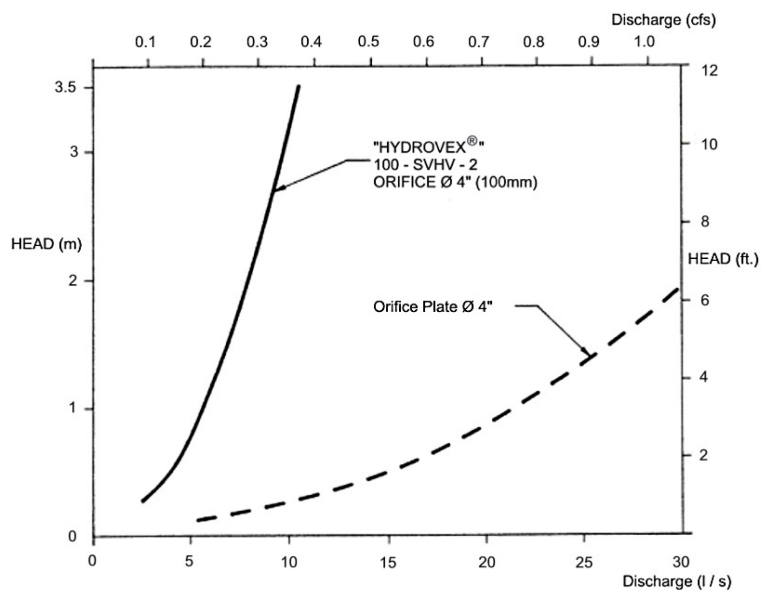


Figure 1:HYDROVEX® VHV/SVHV compared to an orifice plate

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

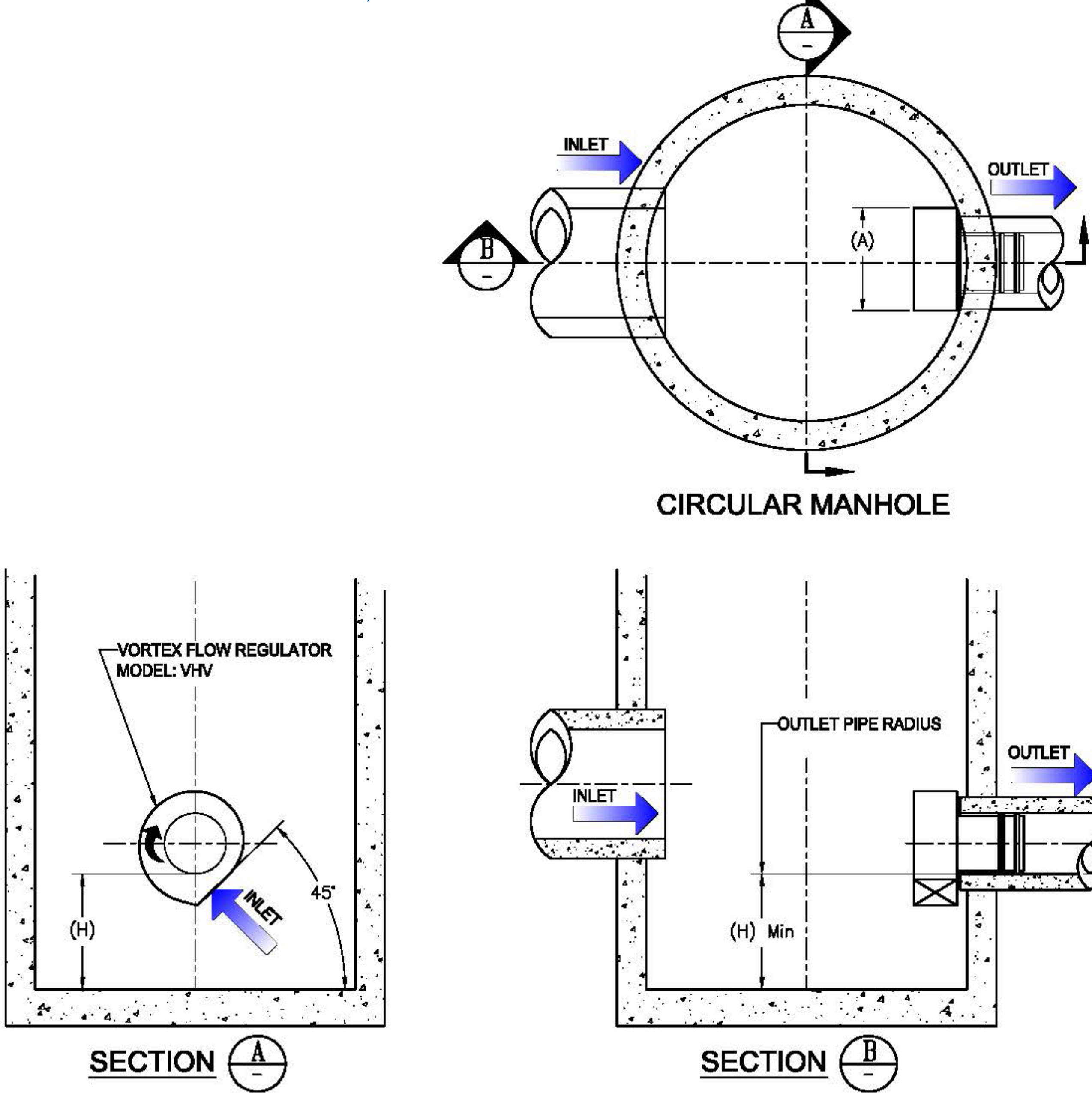
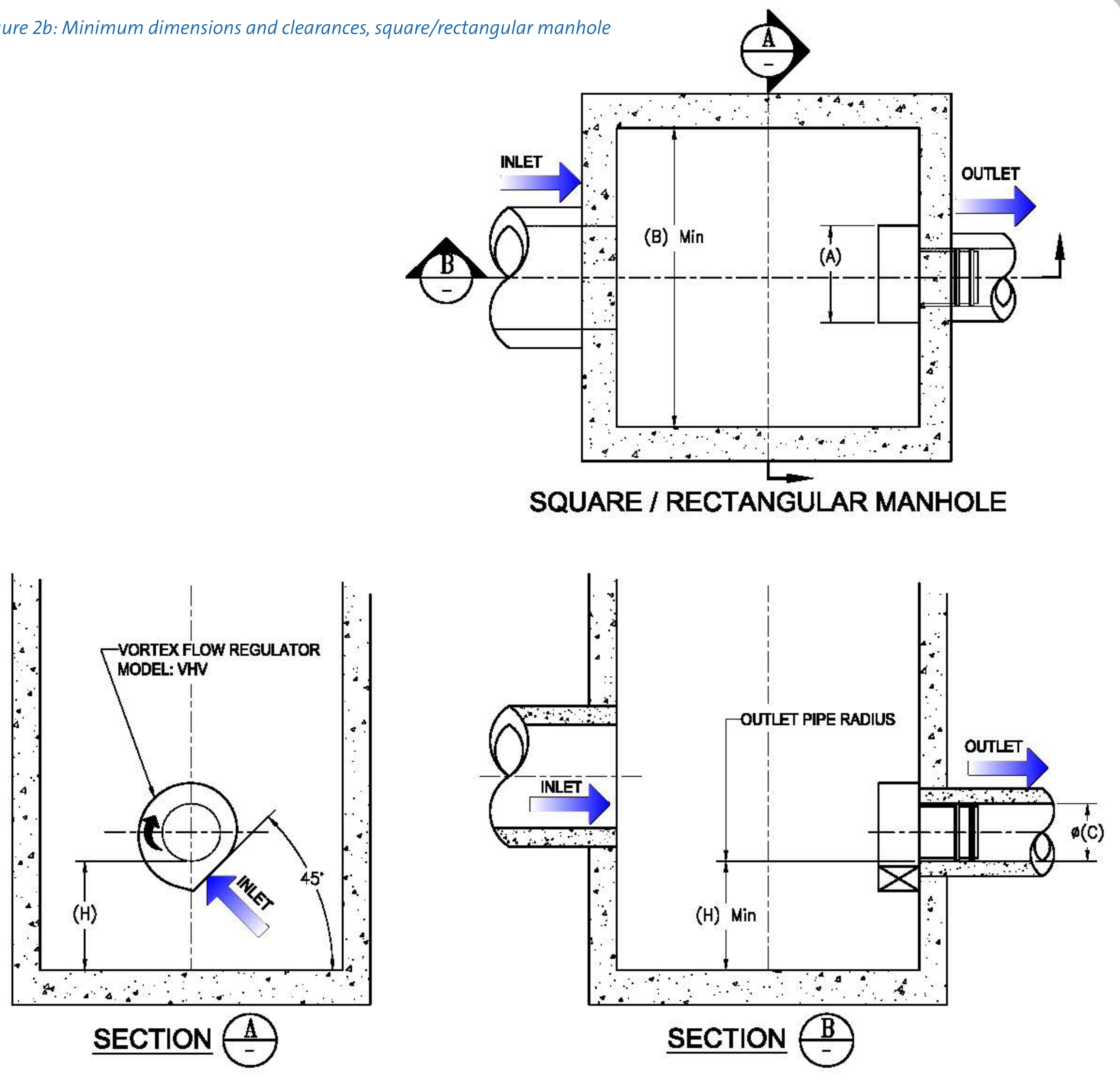


Figure 2b: Minimum dimensions and clearances, square/rectangular manhole



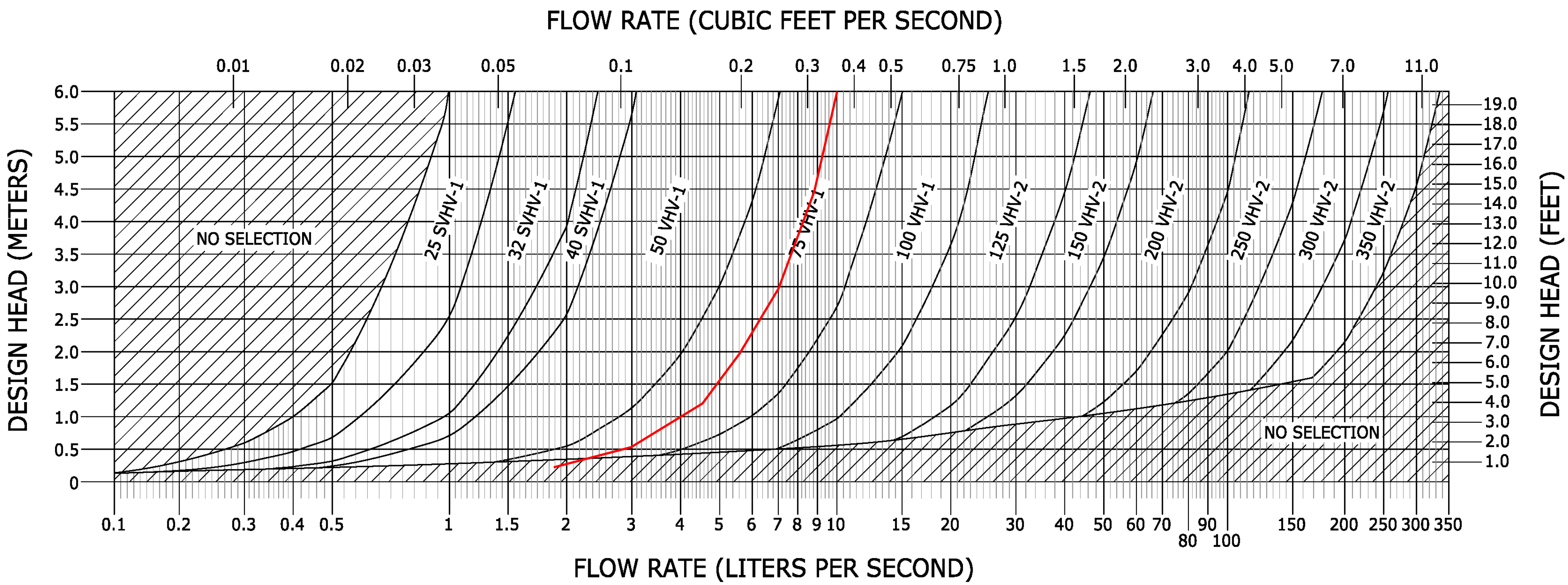


Figure 3 : HYDROVEX® VHV/SVHV Selection Chart

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

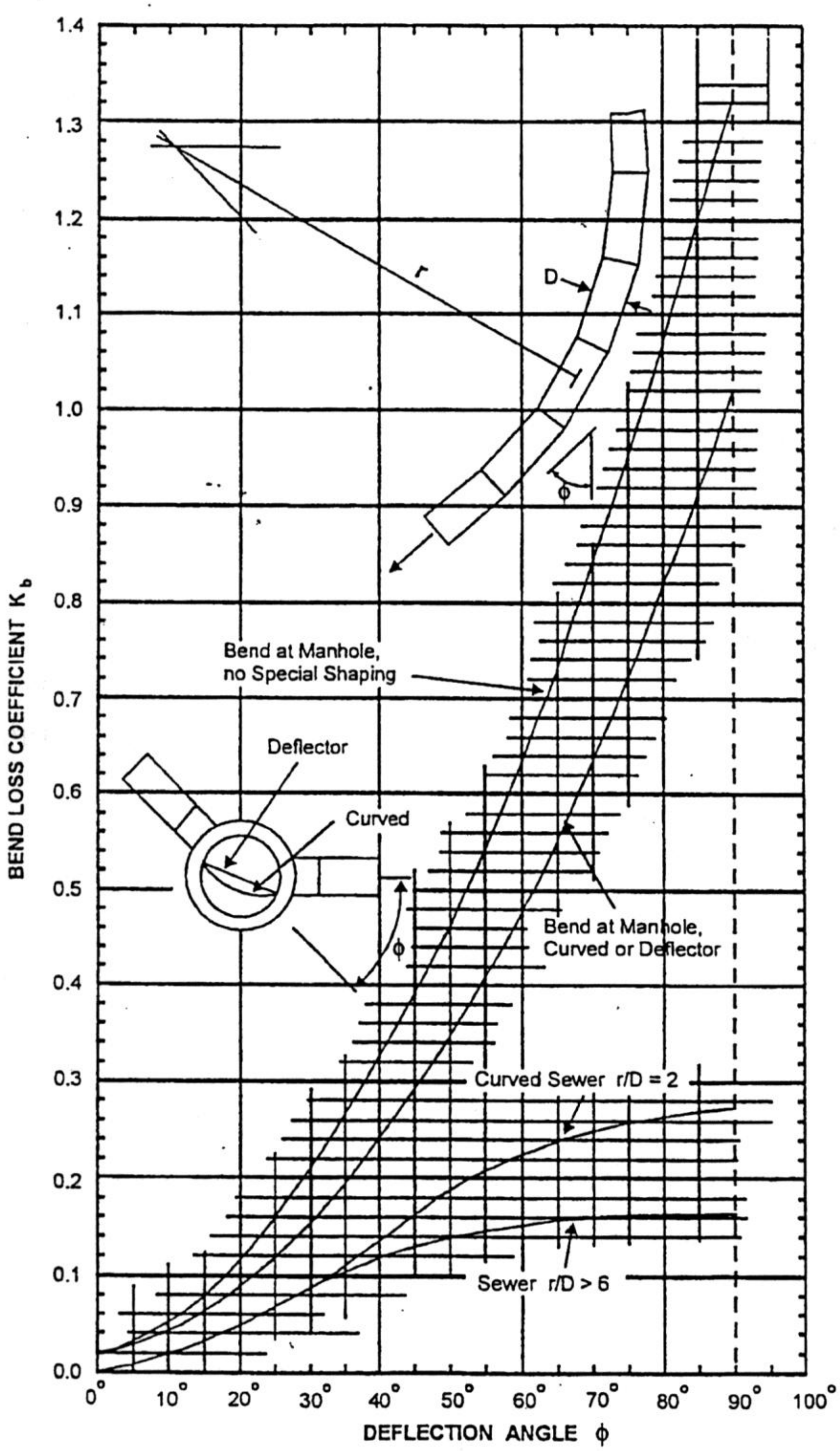
The HYDROVEX® line of VHV / SVHV regulators are guaranteed against both design and manufacturing defects for a period of 5 years after sale. The unit will be modified or replaced should it be found to be defective within the guarantee period.

Resourcing the world

Veolia Water Technologies

4105 Sartelon • Saint-Laurent, Quebec • H4S 2B3 Canada
T.: 514-334-7230 • F.: 514-334-5070 • Sales Direct Line: 1-855-564-3747
cso@veolia.com • www.veoliawatertechnologies.ca

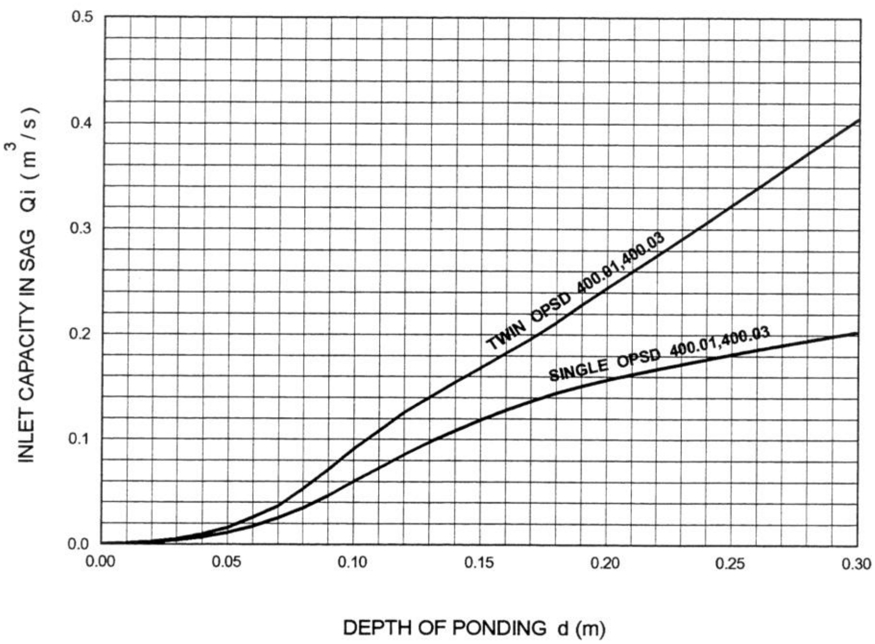
Design Chart : Sewer Bend Loss Coefficients



Source: American Iron and Steel Institute (1980)

Surface Inlet Capacity At Road Sags[8]

Design Chart 4.19: Inlet Capacity at Road Sag



[8] From the *MTO Drainage Management Manual*

Month	Average Monthy Temp	Heat Index	Potential ET	Daylight Correction Value	Adjusted ET	Total Precipitation	Surplus	Deficit
(-)	(°C)	(-)	(mm)	(-)	(mm)	(mm)	(mm)	(mm)
Jan	-10.3	0	0	0.78	0	65.4	65.4	
Feb	-8.1	0	0	0.88	0	54.3	54.3	
Mar	-2.3	0	0	0.99	0.0	64.4	64.4	
Apr	6.3	1.4	28.3	1.12	31.6	74.5	42.9	
May	13.3	4.4	63.8	1.22	77.9	80.3	2.4	-2.4
Jun	18.5	7.2	91.4	1.28	91.7	92.8	0.0	-1.1
Jul	21.0	8.8	105.0	1.25	82.7	91.9	0.0	-9.2
Aug	19.8	8.0	98.5	1.16	82.9	85.5	0.0	-2.6
Sep	15.0	5.3	72.8	1.05	76.4	90.1	13.7	-13.7
Oct	8.0	2.0	36.7	0.92	33.8	86.1	52.3	-52.3
Nov	1.5	0.2	5.9	0.81	4.8	81.9	77.1	-77.1
Dec	-6.2	0.0	-	0.75	0	76.4	76.4	
Totals		37.4			**482**	**943.6**	**449**	**-158**
	a	1.09						
					Total Water Surplus	**462**		

1. Average Ottawa International Airport, ON monthly temperature and precipitation from Canadian Climate Normals (Government of Canada) from 1981-2010.
2. Daylight correction values from Thornthwaite's Equation for estimating potential evaportranspiration (Hydrology: An Environmental Approach, I. Watson & A.D. Burnett)
3. Water balance based on Ontario Ministry of the Environment and Conservation and Parks Stormwater Management Planning and Design Manual (2003), Table 3.1.

$PET\ (in) = 0.63(10T/I)^a$

Where $a = 6.751 \times 10^{-7}(I^3) - 7.71 \times 10^{-5}(I^2) + 1.792 \times 10^{-2}(I) + 0.49239$

And I is the sum of the i values for the year,

Where $i = (T/5)^{1.514}$

Latitude correction for daylight hours

$(24\cos^{-1}(-\tan(L\Pi/180)\tan(0.4093\sin(2\Pi(int(30.4m-15))(365-1.39)))/12\Pi$

Where L is latitude, and m is month number (1 to 12).

Parameter	Description	Factor
Topography	Flat Land < 0.6 m/km (0.06%)	0.3
	Rolling Land 2.8 m to 3.8 m/km (0.28%-0.38%)	0.2
	Hilly Land 28 m to 47 m/km (2.8%-4.7%)	0.1
Soils	Tight Impervious Clay	0.1
	Medium Combinations of Clay and Loam	0.2
	Open Sandy Loam	0.4
Cover	Cultivated Land	0.10
	Woodland	0.20

Infiltration Factors	Pervious	Impervious
Topography Factor	0.30	0.30
Soil Infiltration Factor	0.10	0.10
Land Cover Infiltration Factor	0.10	0.10
MECP Infiltration Factor	**0.50**	**0.00**
Runoff Coefficient	0.50	1.00

Outputs		
recipitation Surplus	462	mm/year
Net Surplus	462	mm/year
Evapotranspiration	482	mm/year
Infiltration	231	mm/year
Runoff	231	mm/year
Total Outputs	944	mm/year

Existing Conditions

	Catchment	Area	Surface Type	Land-Use	Area	Precipitation	Evapotranspiration	Infiltration	Runoff
						Annual Water Balance			
	(ID)	(ha)	(Desc)	(Desc)	(ha)	(mm)	(mm)	(mm)	(mm)
Subject Property	A1	0.28	Impervious	Grassed	0	944	0	0	944
			Pervious		0.28	944	482	231	231
	A2	0.03	Impervious	Grassed	0	944	0	0	944
			Pervious		0.03	944	482	231	231
Total Subject Property		0.31			0.31	944	482	231	231
Total (m^3)						2,900	1,500	700	700

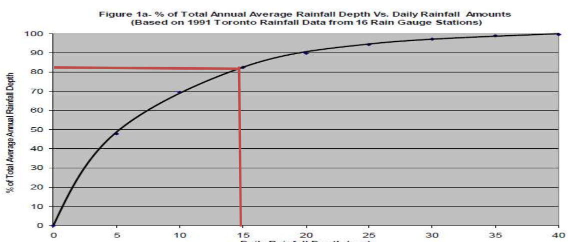
Proposed Conditions (Un-Controlled)

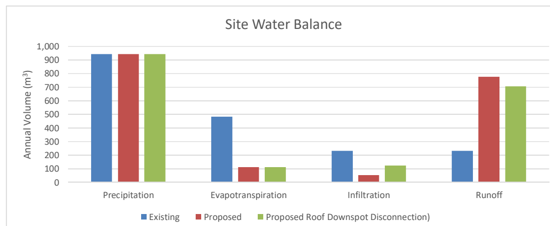
	Catchment	Area	Surface Type	Land-Use	Area	Precipitation	Evapotranspiration	Infiltration	Runoff
							Annual Water Balance		
	(ID)	(ha)	(Desc)	(Desc)	(ha)	(mm)	(mm)	(mm)	(mm)
Subject Property	A1	0.0590	Impervious		0.049	944	0	0	944
			Pervious		0.010	944	482	231	231
	A2	0.1060	Impervious		0.078	944	0	0	944
			Impervious (Roof)		0.020	944	0	0	944
			Pervious		0.008	944	482	231	231
	A3	0.0300	Impervious		0.025	944	0	0	944
			Pervious		0.005	944	482	231	231
	A4	0.0310	Impervious		0.007	944	0	0	944
			Impervious (Roof)		0.017	944	0	0	944
			Pervious		0.007	944	482	231	231
	A5	0.0350	Impervious		0.021	944	0	0	944
			Pervious		0.014	944	482	231	231
	A6-1	0.0070	Impervious		0.000	944	0	0	944
			Pervious		0.007	944	482	231	231
	A6-2	0.0230	Impervious		0.002	944	0	0	944
			Pervious		0.021	944	482	231	231
	A7	0.0040	Impervious		0.004	944	0	0	944
			Pervious		0.000	944	482	231	231
	A9	0.0110	Impervious (Roof)		0.011	944	0	0	944
			Pervious		0.000	944	482	231	231
Total Subject Property		0.306			0.306	944	113	54	776
Total (m^3)						2,890	350	170	2,370

Proposed Conditions (Controlled - Roof Downspout Disconnection)

	Catchment	Area	Surface Type	Area	Annual Water Balance: Precipitation	Evapotranspiration	Infiltration	Runoff
	(ID)	(ha)	(Desc)	(ha)	(mm)	(mm)	(mm)	(mm)
Subject Property	A1	0.0590	Impervious	0.049	944	0	0	944
			Pervious	0.010	944	482	231	231
	A2	0.1060	Impervious	0.078	944	0	0	944
			Impervious (Roof)	0.020	944	0	0	944
			Pervious	0.008	944	482	231	231
	A3	0.0300	Impervious	0.025	944	0	0	944
			Pervious	0.005	944	482	231	231
	A4	0.0310	Impervious	0.007	944	0	0	944
			Impervious (Roof)	0.017	944	0	774	170
			Pervious	0.007	944	482	231	231
	A5	0.0350	Impervious	0.021	944	0	0	944
			Pervious	0.014	944	482	231	231
	A6-1	0.0070	Impervious	0.000	944	0	0	944
			Pervious	0.007	944	482	231	231
	A6-2	0.0230	Impervious	0.002	944	0	0	944
			Pervious	0.021	944	482	231	231
	A7	0.0040	Impervious	0.004	944	0	0	944
			Pervious	0.000	944	482	231	231
	A9	0.0110	Impervious (Roof)	0.011	944	0	774	170
			Pervious	0.000	944	482	231	231
Total Subject Property		0.306		0.306	944	113	125	705
Total (m^3)					2,890	350	380	2,160

Parameter	Existing Conditions		Proposed Conditions (Uncontrolled)		Proposed Conditions (Roof Downspout Disconnection)	
	(mm)	(m^3)	(mm)	(m^3)	(mm)	(m^3)
Rainfall	944	2,900	944	2,890	944	2,890
Evapotranspiration (1)	482	1,500	113	350	113	350
Infiltration (2)	231	700	54	170	125	380
Runoff	231	700	776	2,370	705	2,160
Total	944	2,900	944	2,890	944	2,890





Notes:

1. Evapotranspiration assumed to be across the subject site.
2. Assumes 'clean' rooftop runoff to be infiltrated, 15 mm of daily rainfall to be infiltrated (82% of average annual rainfall depth).
3. Table 1a from City of Toronto Wet Weather Flow Guidelines (2006).

Monthly average climate information for Ottawa, ON (1981-2010), from the Federal Ministry of the Environment and Climate Change

Precipitation

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year	Code
Rainfall (mm)	25.0	18.7	31.1	63.0	80.1	92.8	91.9	85.5	90.1	82.2	64.5	33.5	758.2	A
Snowfall (cm)	53.9	43.3	38.3	11.3	0.2	0.0	0.0	0.0	0.0	3.7	20.2	52.5	223.5	A
Precipitation (mm)	65.4	54.3	64.4	74.5	80.3	92.8	91.9	85.5	90.1	86.1	81.9	76.4	943.4	A
Average Snow Depth (cm)	24	28	20	1	0	0	0	0	0	0	2	13	7	A
Median Snow Depth (cm)	24	27	20	0	0	0	0	0	0	0	1	13	7	A
Snow Depth at Month-end (cm)	27	25	5	0	0	0	0	0	0	0	4	19	7	A
Extreme Daily Rainfall (mm)	43.6	40.4	44.2	46.2	47.8	66.6	69.6	67.0	**135.4**	76.0	46.2	36.3		
Date (yyyy/dd)	2010/25	1997/21	1980/21	2005/02	2008/31	1995/03	1967/09	2004/10	**2004/09**	1995/06	2000/26	1941/24		
Extreme Daily Snowfall (cm)	38.6	39.6	**40.6**	29.8	15.0	0.0	0.0	0.0	1.5	29.2	28.2	35.6		
Date (yyyy/dd)	1966/30	1954/16	**1947/02**	1993/01	1963/10	1939/01	1939/01	1939/01	1946/30	1988/22	1987/25	2007/16		
Extreme Daily Precipitation (mm)	43.6	40.4	44.2	46.2	47.8	66.6	69.6	67.0	**135.4**	76.0	46.2	43.7		
Date (yyyy/dd)	2010/25	1997/21	1980/21	2005/02	2008/31	1995/03	1967/09	2004/10	**2004/09**	1995/06	2000/26	1973/09		
Extreme Snow Depth (cm)	77	119	**135**	58	10	0	0	0	0	20	42	75		
Date (yyyy/dd)	1979/26	1971/24	**1993/14**	1971/01	1963/11	1955/01	1955/01	1955/01	1955/01	1997/27	1995/29	2007/17		

Table 3.1: Hydrologic Cycle Component Values

	Water Holding Capacity mm	Hydrologic Soil Group	Precipitation mm	Evapo-transpiration mm	Runoff mm	Infiltration* mm
Urban Lawns/Shallow Rooted Crops (spinach, beans, beets, carrots)						
Fine Sand	50	A	940	515	149	276
Fine Sandy Loam	75	B	940	525	187	228
Silt Loam	125	C	940	536	222	182
Clay Loam	100	CD	940	531	245	164
Clay	75	D	940	525	270	145
Moderately Rooted Crops (corn and cereal grains)						
Fine Sand	75	A	940	525	125	291
Fine Sandy Loam	150	B	940	539	160	241
Silt Loam	200	C	940	543	199	199
Clay Loam	200	CD	940	543	218	179
Clay	150	D	940	539	241	160
Pasture and Shrubs						
Fine Sand	100	A	940	531	102	307
Fine Sandy Loam	150	B	940	539	140	261
Silt Loam	250	C	940	546	177	217
Clay Loam	250	CD	940	546	197	197
Clay	200	D	940	543	218	179
Mature Forests						
Fine Sand	250	A	940	546	79	315
Fine Sandy Loam	300	B	940	548	118	274
Silt Loam	400	C	940	550	156	234
Clay Loam	400	CD	940	550	176	215
Clay	350	D	940	549	196	196

Notes: Hydrologic Soil Group A represents soils with low runoff potential and Soil Group D represents soils with high runoff potential. The evapotranspiration values are for mature vegetation. Streamflow is composed of baseflow and runoff.

* *This is the total infiltration of which some discharges back to the stream as base flow. The infiltration factor is determined by summing a factor for topography, soils and cover.*

Topography	Flat Land, average slope < 0.6 m/km	0.3
	Rolling Land, average slope 2.8 m to 3.8 m/km	0.2
	Hilly Land, average slope 28 m to 47 m/km	0.1
Soils	Tight impervious clay	0.1
	Medium combinations of clay and loam	0.2
	Open Sandy loam	0.4
Cover	Cultivated Land	0.1
	Woodland	0.2

Appendix G

Drawings

- Sheet C131, Existing Stormwater Drainage
- Sheet C101, Sediment and Erosion Control Plan
- Sheet C103, Site Servicing Plan
- Sheet C104, Site Grading Plan
- Sheet C105, Stormwater Management Plan
- Sheet C803, Storm Servicing Plan

SEDIMENT AND EROSION CONTROL PLAN

Scale 1:200

0 5 10 m
1:200

LEGEND

OVERLAND FLOW	→
EXISTING ASPHALT SURFACE	
EXISTING GRAVEL SURFACE	

AECOM

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REGISTRATION

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

ISSUE/REVISION

I/R	DATE	DESCRIPTION
A	2020-02-07	ISSUED FOR REVIEW

DRAWN BY

AS

KEY PLAN

HAZELDEAN ROAD
FRINGEWOOD AVENUE
CANOPY
C-STORE
CAR WASH
N

GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

EXISTING STORMWATER DRAINAGE

AECOM FILE NAME

C131.0-SWM-HZLX

SHEET NUMBER

C131.0

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

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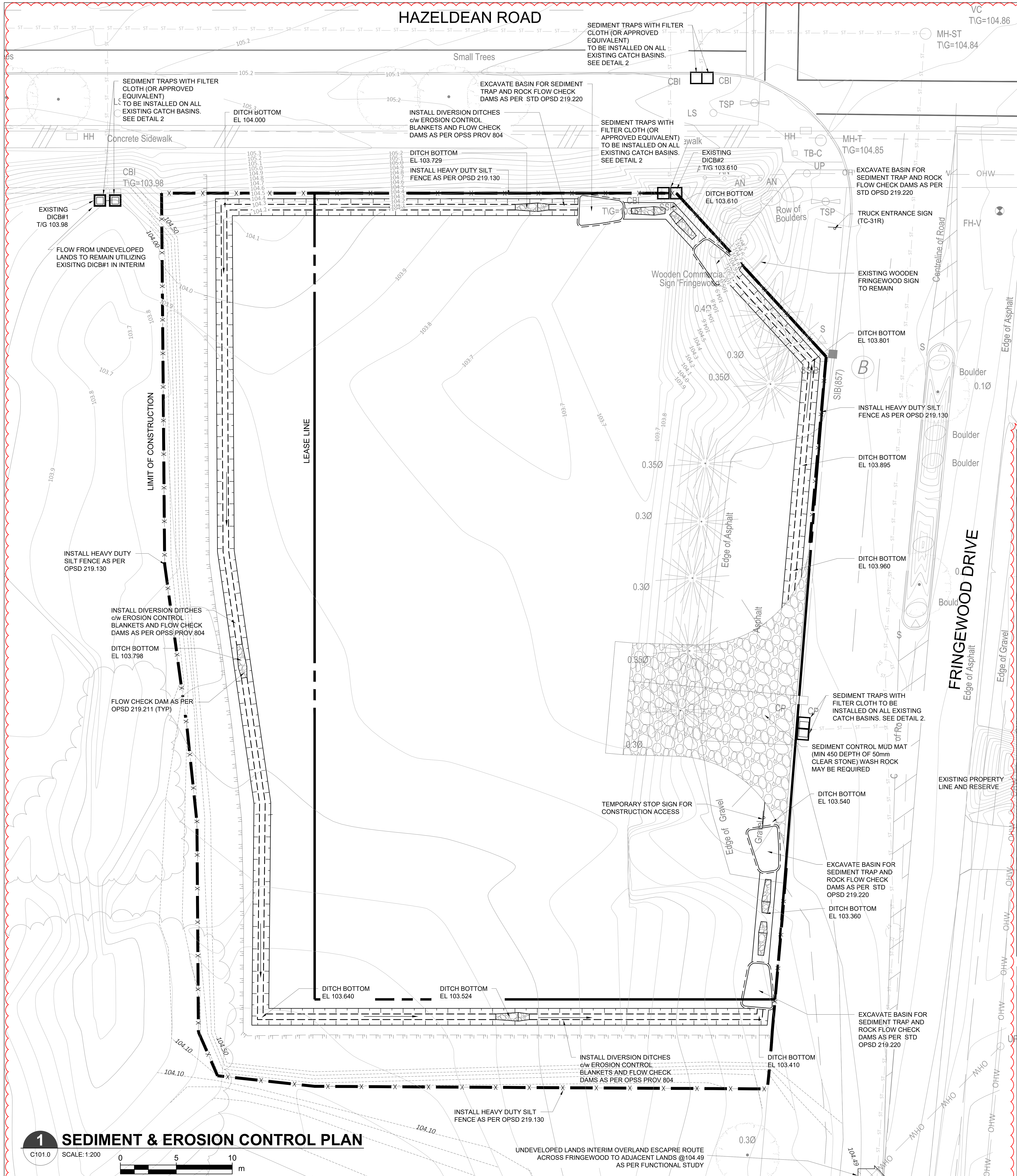
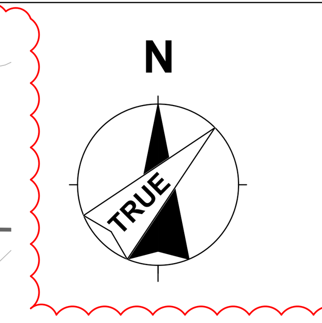
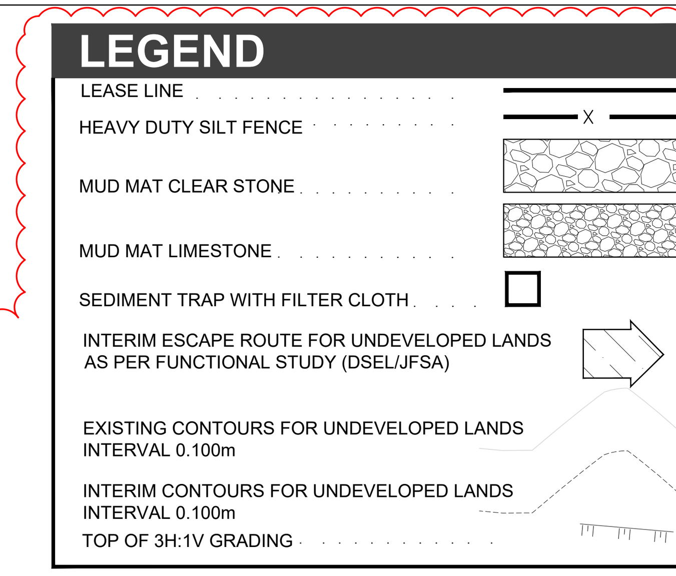
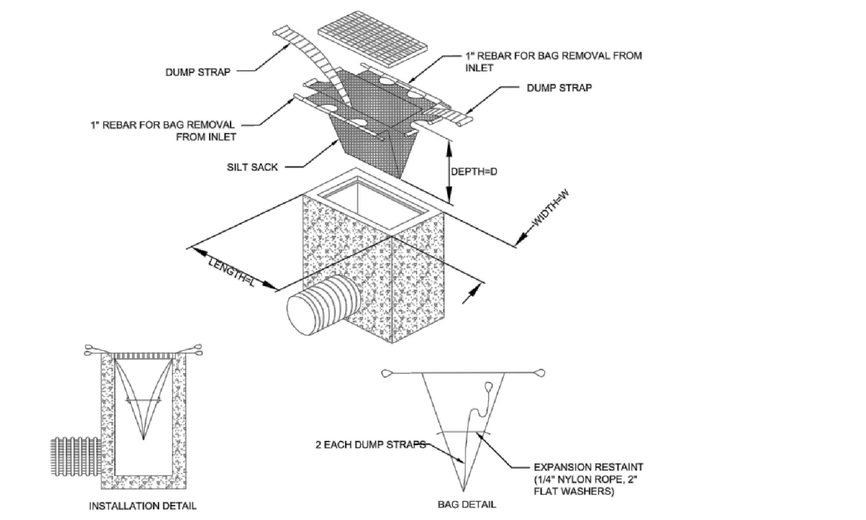


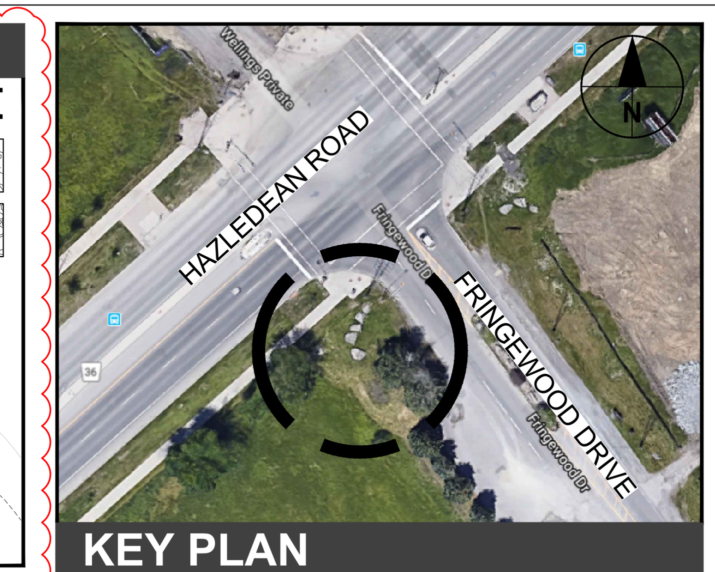
1 C101.0 **SEDIMENT & EROSION CONTROL PLAN**
SCALE: 1:200







2 C101.0 SEDIMENT TRAP WITH FILTER CLOTH IN CATCHBASIN
SCALE:NTS



KEY PLAN

SITE INFORMATION

MUNICIPAL ADDRESS
5 ORCHARD DR
STITTSVILLE, ONTARIO

ZONING
ARTERIAL MAINSTREET SUBZONE 9 - AM9

GENERAL NOTES

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.

EROSION AND SEDIMENT CONTROL RESPONSIBILITIES

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

AECOM

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CTM Design Services Ltd.
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REGISTRATION

LEGAL DESCRIPTION
PART OF BLOCK 21 OF DRAFT PLAN OF SUBDIVISION OF PARTS OF LOTS 26 AND 27 OF REGISTERED PLAN 4R2506 PART 3, CONCESSION 11, GEOGRAPHIC TOWNSHIP OF GOULBOURN (CITY OF OTTAWA)

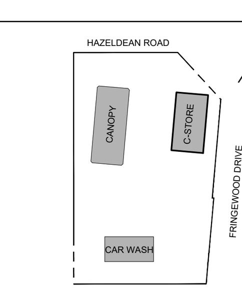
ISSUE/REVISION

I/R	DATE	DESCRIPTION
C	2020-09-30	RE-ISSUED FOR SPA
B	2020-07-07	REVISED PER SPA COMMENTS
A	2020-03-31	ISSUED FOR SPA

DRAWN BY

NAS

KEY PLAN



GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

SITE
SEDIMENT AND EROSION CONTROL PLAN

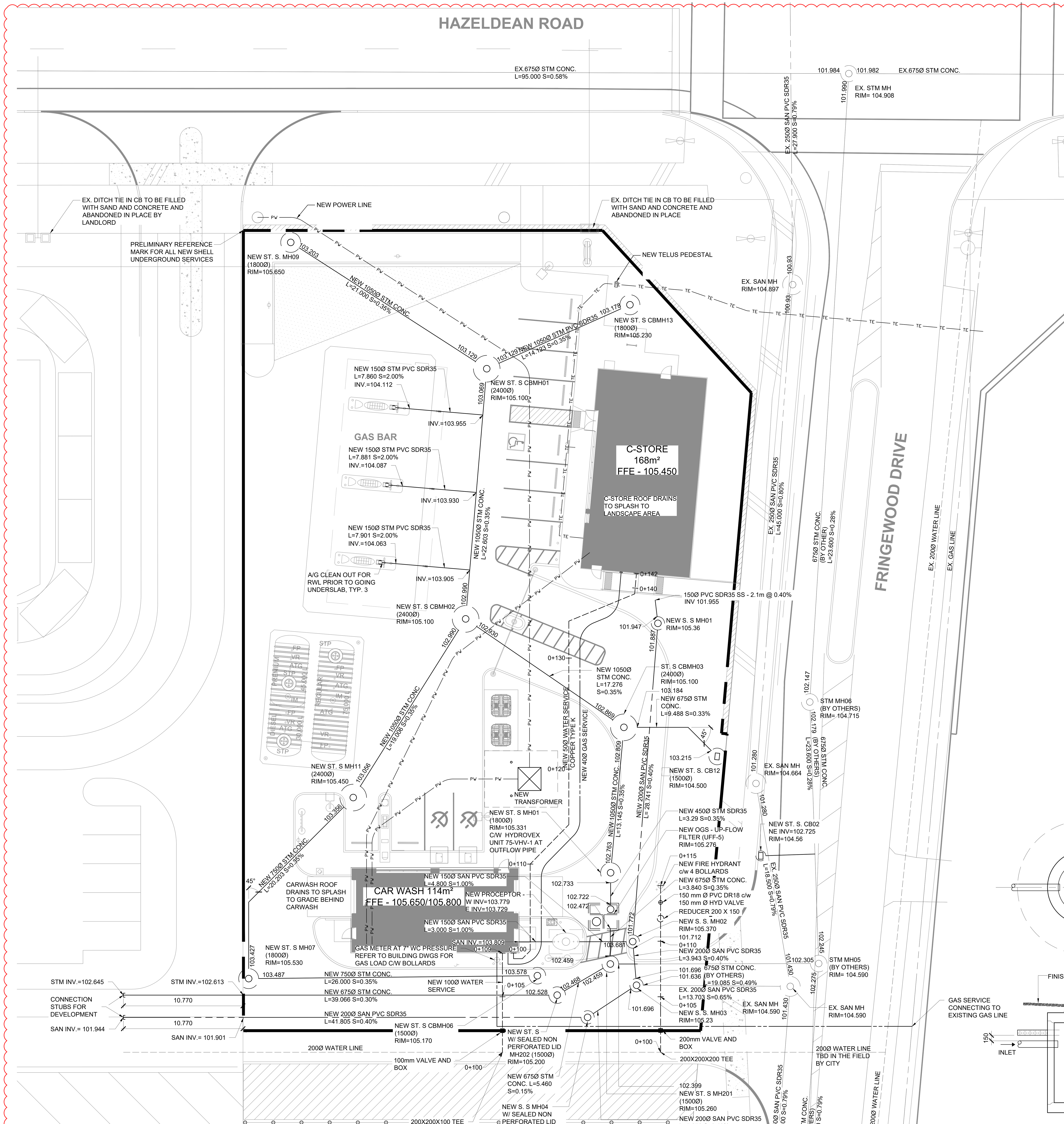
CTM DESIGN FILE NAME

2020072_C101.0

SHEET NUMBER

C101.0

CITY FILE NO. D07-12-20-0043
CITY PLAN NO. 18131



1 SITE SERVICING PLAN

C103.0 SCALE: 1:200

0 5 10 m

1:200

NOTES

1. FOR GENERAL NOTES SEE DRAWING C001.0

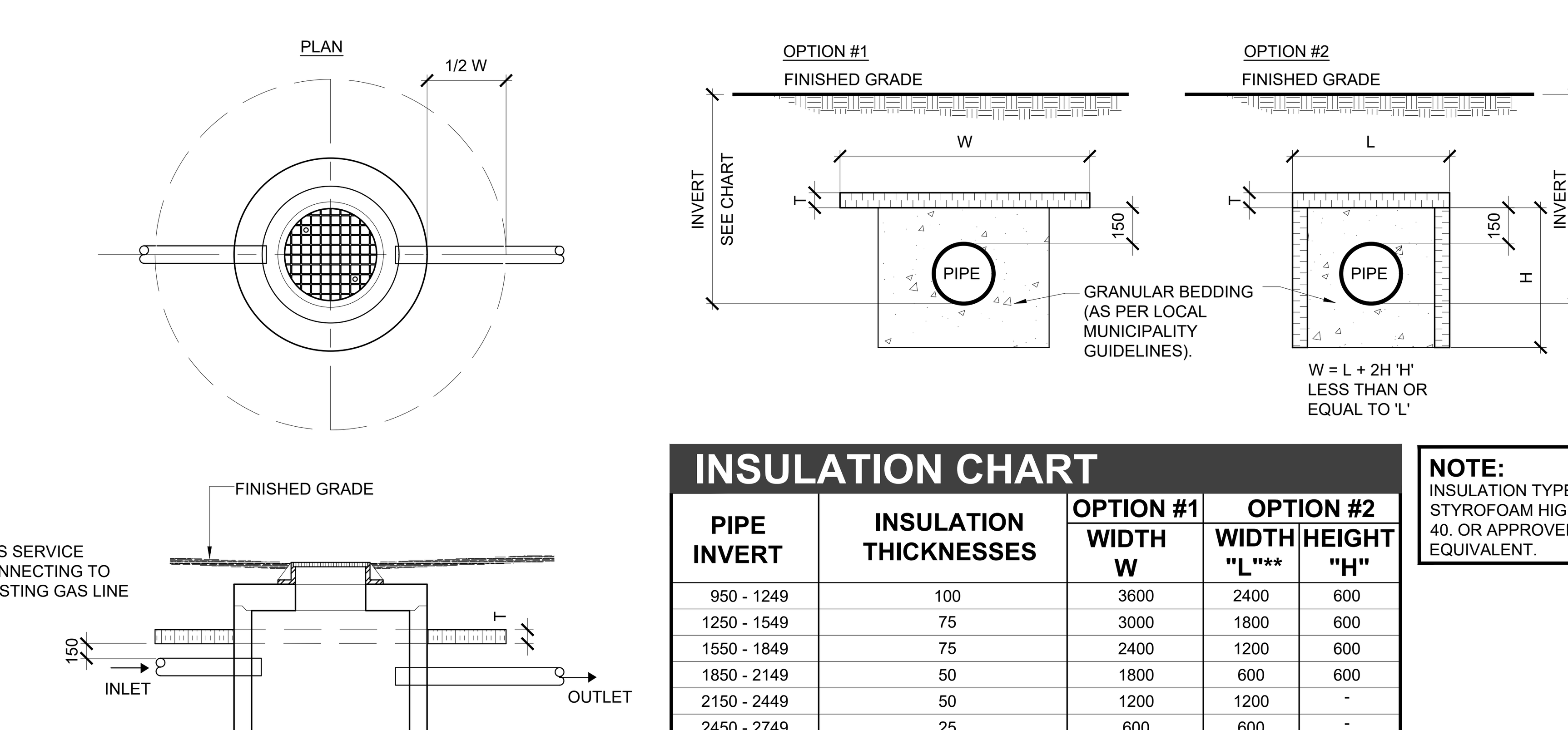
LEGEND

MANHOLE	MH
CATCH BASIN MANHOLE	CBMH
CATCH BASIN	CB
OIL/GRIT SEPARATOR	OGS
STORM SEWER	ST. S.
SANITARY SEWER	S. S.
NEW TELECOM	TE
NEW POWER LINE	PV
NEW WATER LINE	
EXISTING WATER LINE	
NEW STORM LINE	
EXISTING STORM LINE	
NEW SANITARY LINE	
EXISTING SANITARY LINE	
NEW GAS LINE	
EXISTING GAS LINE	
GAS METER	
WATER VALVE	

SERVICE LOCATIONS

ITEMS	SOUTH	EAST
OGS-UF7	61.783	32.440
ST. S. MH201	66.042	33.049
ST. S. MH202	68.843	28.262
ST. S. MH01	57.826	33.074
ST. S. MH07	67.318	0.478
ST. S. MH09	1.075	4.264
ST. S. MH11	51.055	9.893
ST. S. CBMH01	12.448	21.918
ST. S. CBMH02	34.971	20.085
ST. S. CBMH03	44.695	34.270
ST. S. CBMH06	67.094	26.476
ST. S. CBMH13	6.691	34.815
ST. S. CB12	47.314	42.673
PROCEPTOR	64.035	29.120
S. S. MH01	35.369	37.335
S. S. MH02	64.021	35.069
S. S. MH03	67.950	35.408
S. S. MH04	70.803	31.002

ALL NEW UNDERGROUND SERVICES ARE BASED OFF OF THE REFERENCE MARK LOCATED AT THE NORTH WEST CORNER OF SITE WHERE THE LEASE LINE AND P/L INTERSECT AND THE PLAN NORTH



INSULATION CHART

PIPE INVERT	INSULATION THICKNESSES	OPTION #1	OPTION #2	
		WIDTH W	WIDTH "L"**	HEIGHT "H"
950 - 1249	100	3600	2400	600
1250 - 1549	75	3000	1800	600
1550 - 1849	75	2400	1200	600
1850 - 2149	50	1800	600	600
2150 - 2449	50	1200	1200	-
2450 - 2749	25	600	600	-
2750 - 2850	25	600	600	-

*DESIGN INSTALL THICKNESS IS A PRODUCT OF CURRENT SUPPLY AND DEMAND AVAILABILITY OF PRODUCT, 2400L X 600W X 50 THICK.
** MINIMUM WIDTHS TAKEN FROM 600 WIDE SHEET BEING AVAILABLE FOR SIDEWALLING TO DEPTH "H"

NOTE:
INSULATION TYPE TO BE STYROFOAM HIGH LOAD 40. OR APPROVED EQUIVALENT.

2 FROST PROTECTION DETAIL

C103.0 SCALE: NTS

AECOM

PROJECT

Shell Canada Products
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ctm Design Services Ltd

REGISTRATION

LEGAL DESCRIPTION

PART OF BLOCK 21 OF DRAFT PLAN OF SUBDIVISION OF PARTS OF LOTS 26 AND 27 OF REGISTERED PLAN 4R2506 PART 3, CONCESSION 11, GEOGRAPHIC TOWNSHIP OF GOULBOURN (CITY OF OTTAWA)

ISSUE/REVISION

I/R	DATE	DESCRIPTION
D	2020-09-30	RE-ISSUED FOR SPA
C	2020-09-15	ISSUED TO CLIENT
B	2020-08-14	ISSUED FOR SWM ANALYSIS
A	2020-03-31	ISSUED FOR SPA

DRAWN BY

JNT

KEY PLAN

GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

SITE
SITE SERVICING PLAN

CTM DESIGN FILE NAME

2020072_C103.0

SHEET NUMBER

C103.0

CITY PLAN NO. 18131

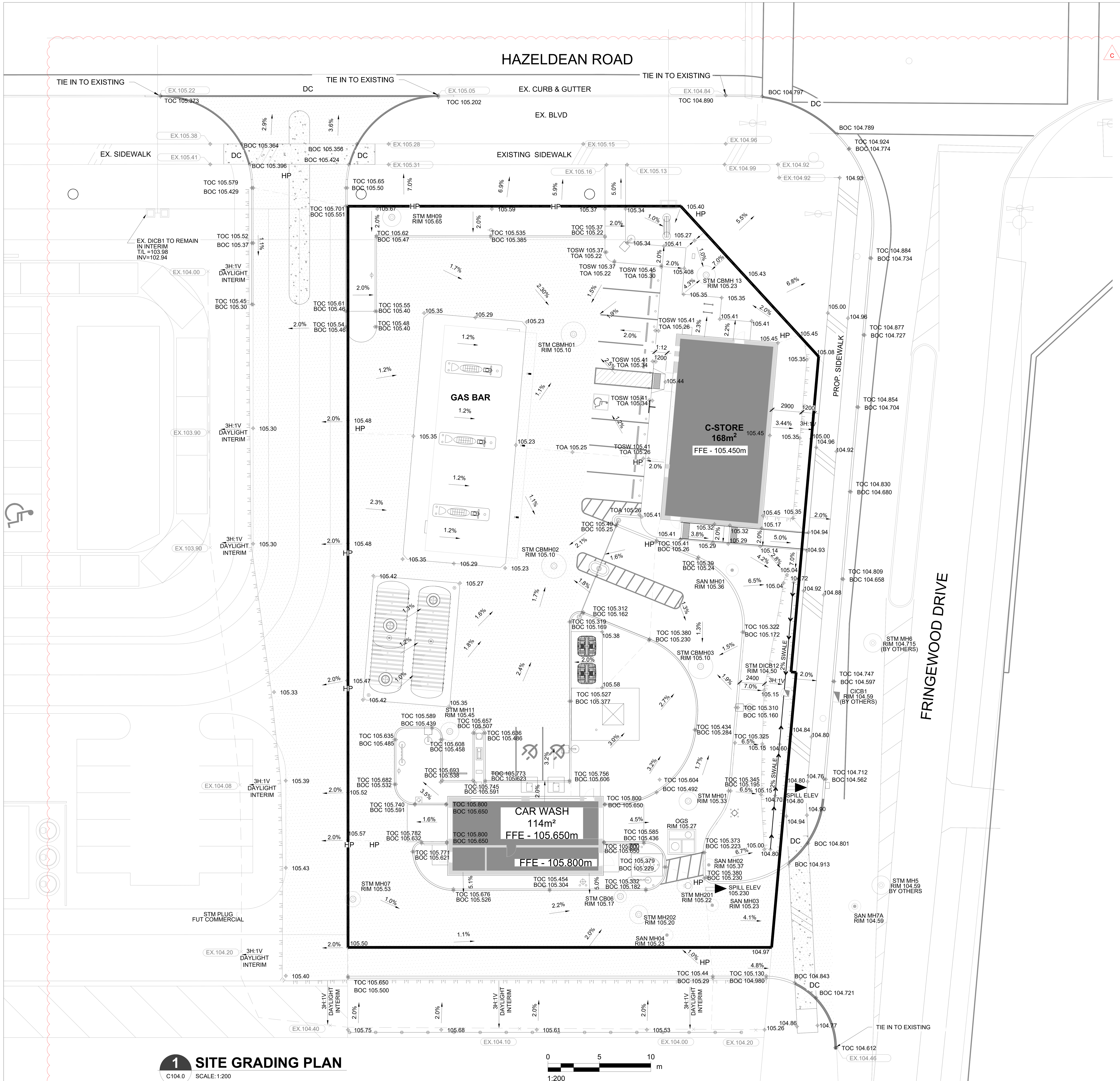
CITY FILE NO. D07-12-20-0043

ANSI D 864mm x 559mm

Project Management Initials: Designer: PE Checked: JP/DJ Approved:

Last Plotted: 2020-09-23

Filename: 2020072_C103.0.DWG



1 SITE GRADING PLAN
C104.0 SCALE: 1:200

NOTES

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

LEGEND

PROPOSED GRADE	105.24
TOP OF CURB PROPOSED GRADE	TOC 105.15
BOTTOM OF CURB PROPOSED GRADE	BOC 105.00
EXISTING SURFACE GRADE	EX.105.15
PROPOSED GRADE BREAK (HP)	
TOP OF 3H:1V GRADING	
DAYLIGHT LINE (TOE OF SLOPE)	
OVERLAND FLOW ROUTE	
SLOPE DIRECTION	
FINISH FLOOR LEVEL	FFL
TOP OF ASPHALT	TOA
TOP OF CONCRETE SLAB	TOCS
TOP OF CURB	TOC
TOP OF ISLAND	TOI
TOP OF SIDEWALK	TOSW
HIGH POINT	HP
PROPOSED LEASE LINE	
PROPOSED STORMWATER / CATCH BASIN MANHOLE / MANHOLE	STM CBMH/ STM MH
PROPOSED SANITARY MANHOLE	SAN MH
PROPOSED STORMWATER CATCH BASIN	STM.CB
PROPOSED OGS - VAULT WITH UP-FLOW FILTER	
PROPOSED FIRE HYDRANT	FH
PROPOSED LIGHT STANDARD	
EXISTING HYDRANT	
PROPOSED DEPRESSED CURB (AS PER SC7.1)	DC

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

AECOM

PROJECT

Shell Canada Products
HAZELDEAN RD. & FRINGEWOOD DR. NTI

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REGISTRATION

LEGAL DESCRIPTION
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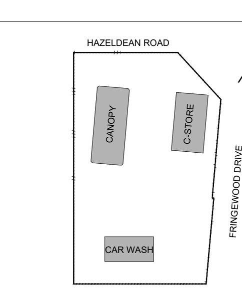
I/R	DATE	DESCRIPTION
C	2020-09-30	RE-ISSUED FOR SPA
B	2020-09-15	ISSUED FOR CLIENT REVIEW
A	2020-03-31	ISSUED FOR SPA

DRAWN BY

ED

KEY PLAN



GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

SITE
SITE GRADING PLAN

CTM DESIGN FILE NAME

2020072_C104.0

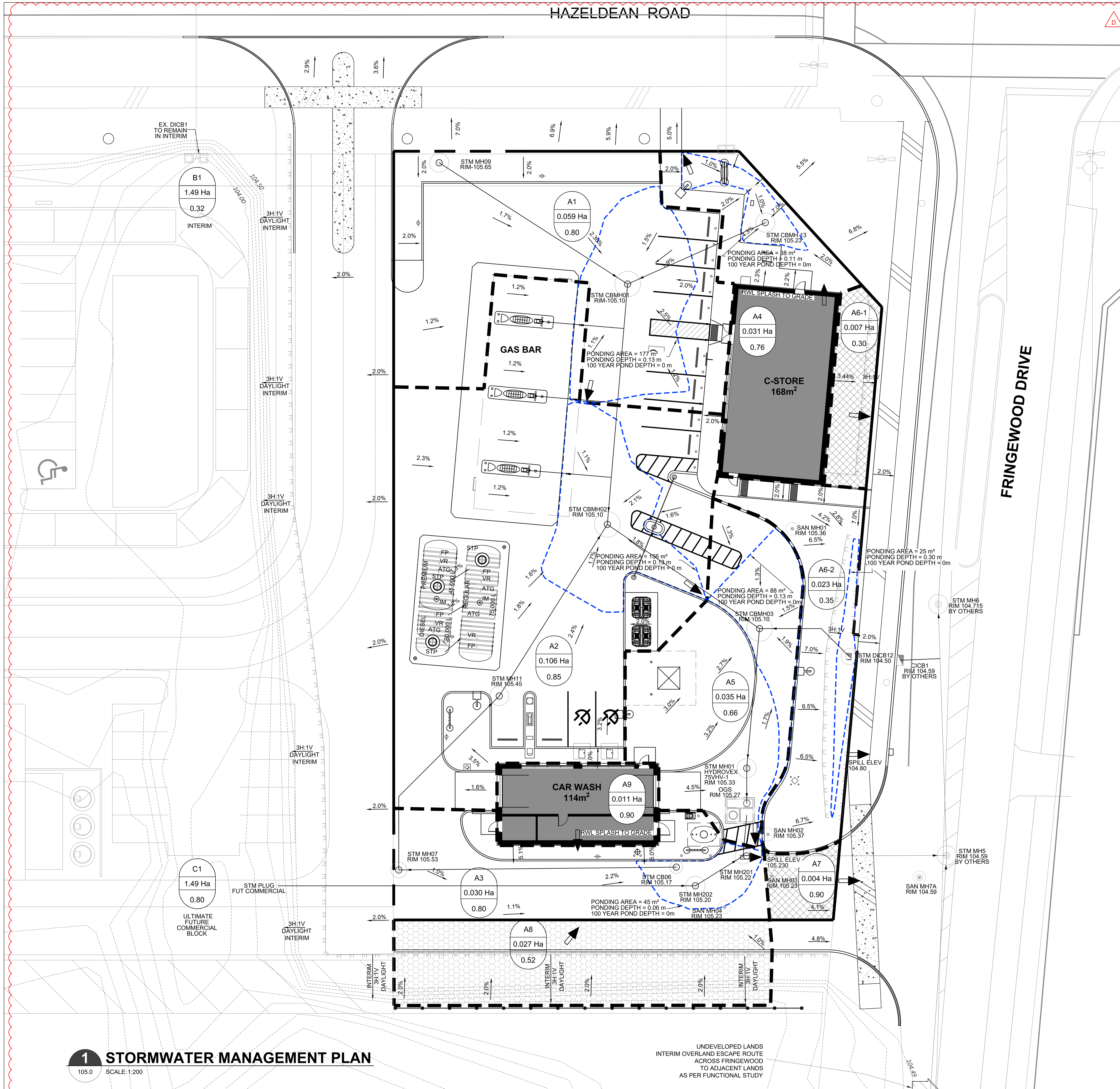
SHEET NUMBER

C104.0

CITY FILE NO. D07-12-20-0043

CITY PLAN NO. 18131

ANSI D 864mm x 559mm
Approved: Checked: YF Designer: ED Project Management Initials:
Last Plotted: 2020-09-23
Last Plotted: NSAROGLU(2020-09-21)
Filename: 2020072_C104.0.DWG



1 STORMWATER MANAGEMENT PLAN
105.0 SCALE: 1:200

NOTES

1. FOR GENERAL NOTES SEE DRAWING C001.0

LEGEND

- PROPOSED LEASE AND PROPERTY LINE
- PROPOSED STORMWATER CATCH BASIN MANHOLE / MANHOLE WITH STORM SEWER & FLOW DIRECTION — STM CBMH/ STM MH
- PROPOSED SANITARY MANHOLE — SAN MH
- PROPOSED STORMWATER CATCH BASIN — STM.CB
- PROPOSED OGS - VAULT WITH UP-FLOW FILTER BY HYDRO-INTERNATIONAL
- PROPOSED SUB-CATCHMENT BOUNDARIES
- PROPOSED LIMIT OF PONDING
- OVERLAND FLOW ROUTE
- INTERIM ESCAPE ROUTE FOR UNDEVELOPED LANDS AS PER FUNCTIONAL STUDY (DSEL/JFSA)
- INTERIM CONTOURS FOR UNDEVELOPED LANDS INTERVAL 0.100m
- TOP OF 3H:1V GRADING
- POST - DEVELOPMENT AREA ID — A1
- POST - DEVELOPMENT DRAINAGE AREA (Ha) — 0.058 Ha
- 1:5 YEAR WEIGHTED RUNOFF COEFFICIENT — 0.87
- UNCONTROLLED STORMWATER FLOW AREA RUNOFF
- EXTERNAL STORMWATER FLOW AREA RUNON

AREA STATEMENT (IN HECTARES)

Sub Catchment	PAVED AREA (ha)	LANDSCAPE AREA (ha)	ROOF TOP AREA (ha)	TOTAL AREA (ha)
A1	0.0491	0.0099		0.0590
A2	0.0780	0.0080	0.0200	0.1060
A3	0.0250	0.0050		0.0300
A4	0.0070	0.0070	0.0170	0.0310
A5	0.0210	0.0140		0.0350
A6-1	0.0000	0.0070		0.0070
A6-2	0.0020	0.0210		0.0230
A7	0.0040			0.0040
A8 (External Area)	0.0100	0.0170		0.0270
A9			0.011	0.0110

AECOM

PROJECT

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B	2020-08-14	ISSUED FOR SWM ANALYSIS
A	2020-03-31	ISSUED FOR SPA

DRAWN BY

NAS

KEY PLAN

GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

SITE
STORMWATER MANAGEMENT PLAN

CTM DESIGN FILE NAME

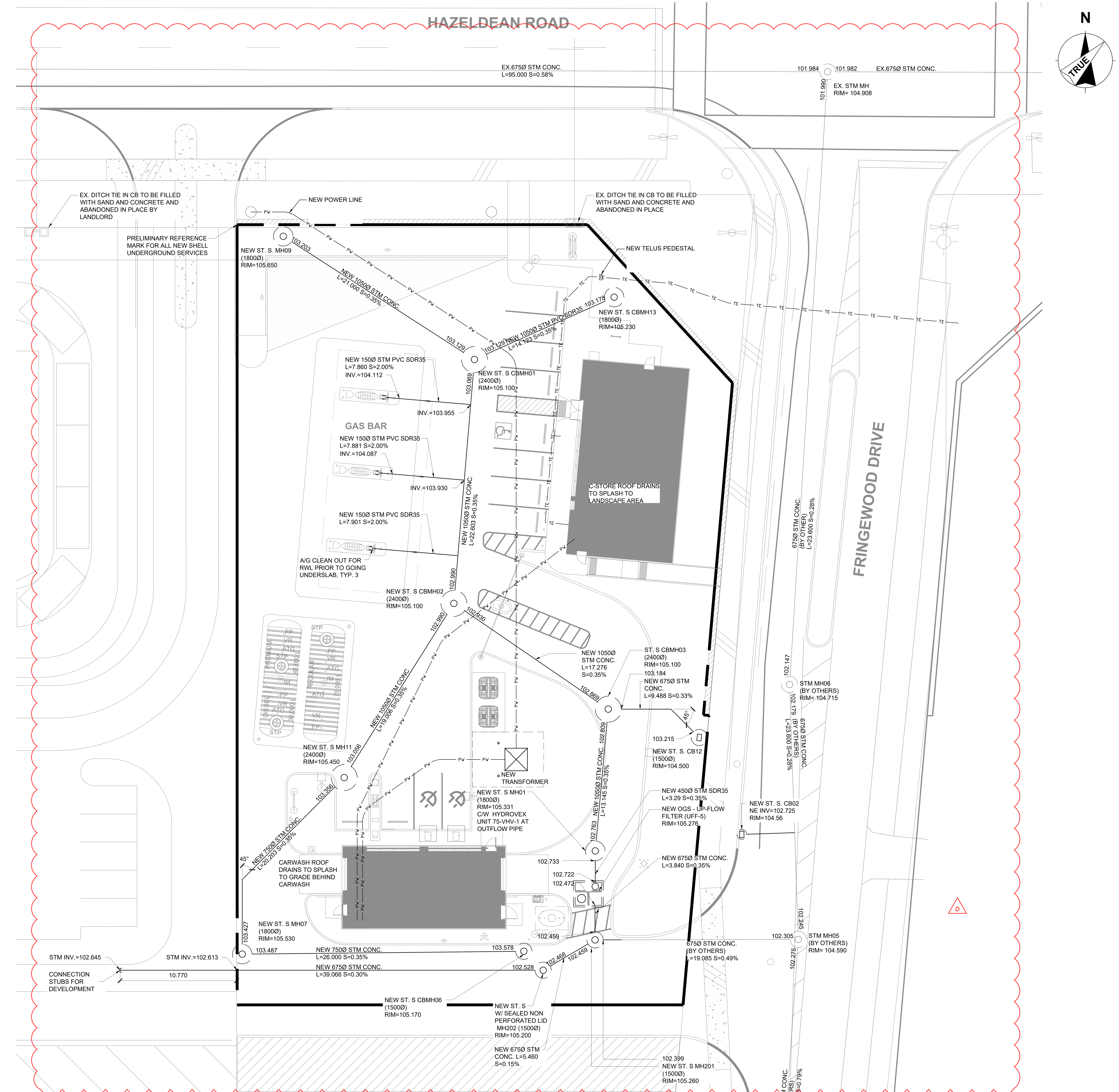
2020072_C105.0

SHEET NUMBER

C105.0

CITY PLAN NO. 18131
CITY FILE NO. D07-12-20-0043

ANSI D 864mm x 559mm
Approved:
Checked: AG
Designer: YF
Project Management Initials:
Last Plotted: 2020-09-23
Last Plotted: NSARIOGLU(2020-09-21)
Filename: 2020072_C105.0.DWG



1 STORM SERVICING PLAN

C803.0 SCALE: 1:200

0 5 10 m
1:200

LEGEND

MANHOLE	MH
CATCH BASIN MANHOLE	CBMH
CATCH BASIN	CB
OIL/GRIT SEPARATOR	OGS
STORM SEWER	ST. S.
NEW TELECOM	TE
NEW POWER LINE	PV
NEW STORM LINE	
EXISTING STORM LINE	

ANSI D 864mm x 559mm

Approved:

Checked: JP/DJ

Designer: PE

Project Management Initials:

Last Plotted: 2020-09-23

Filename: 2020072_C103.0.DWG

Last Plotted: NSARIOGLU(2020-09-23)

AECOM

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C	2020-09-15	ISSUED TO CLIENT
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A	2020-03-31	ISSUED FOR SPA

DRAWN BY

JNT

KEY PLAN

GLOBAL PROJECT ID NUMBER

CAN01444

SHEET TITLE

SITE
STORM SERVICING PLAN

CTM DESIGN FILE NAME

2020072_C103.0

SHEET NUMBER

C803.0 D

CITY FILE NO. D07-12-20-0043

CITY PLAN NO. 18131

Appendix H

PCSWMM Output

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


*********************************************************
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 ............ 00:01:00
Dry Time Step ............ 00:01:00
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 ......         0.078        42.540
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         0.008         4.642
Surface Runoff ...........         0.067        36.552
Final Storage ............         0.003         1.375
Continuity Error (%) .....        -0.067


**************************        Volume        Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.067         0.666
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         0.065         0.655
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 ......         0.001         0.012
Continuity Error (%) .....        -0.116
```

****
Time-Step Critical Elements
****
Link P13 (29.72%)

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

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

Minimum Time Step	:	2.18 sec
Average Time Step	:	4.23 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	7.08	34.28	0.02	0.0
A2	42.54	0.00	0.00	5.31	35.97	0.04	0.0
A3	42.54	0.00	0.00	7.08	34.29	0.01	0.0
A4	42.54	0.00	0.00	8.49	32.97	0.01	0.0
A5	42.54	0.00	0.00	12.05	29.56	0.01	0.0
A6-1	42.54	0.00	0.00	24.76	17.48	0.00	0.0
A6-2	42.54	0.00	0.00	23.01	19.11	0.00	0.0
A7	42.54	0.00	0.00	3.53	37.71	0.00	0.0
A8	42.54	0.00	0.00	17.06	24.75	0.01	0.0
A9	42.54	0.00	0.00	3.54	37.66	0.00	0.0
Commercial_Area	42.54	0.00	0.00	3.60	37.53	0.56	0.3

****
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
Connect	JUNCTION	0.07	0.21	102.85	0 02:12	0.21
J2	JUNCTION	0.23	0.49	103.19	0 02:14	0.49
MH01-N	JUNCTION	0.03	0.11	102.85	0 02:12	0.11
MH202	JUNCTION	0.13	0.38	102.85	0 02:12	0.38
OGS	JUNCTION	0.11	0.37	102.85	0 02:12	0.37
MH05-DSEL	OUTFALL	0.22	0.60	102.84	0 02:12	0.60
CBMH01	STORAGE	0.18	0.55	103.62	0 02:04	0.55
CBMH02	STORAGE	0.24	0.69	103.62	0 02:06	0.69

CBMH03	STORAGE	0.30	0.81	103.62	0	02:08	0.81
CBMH06	STORAGE	0.01	0.09	103.67	0	01:10	0.09
CBMH13	STORAGE	0.13	0.45	103.62	0	02:04	0.45
DICB12	STORAGE	0.11	0.41	103.62	0	02:08	0.41
J1	STORAGE	0.13	0.39	103.19	0	02:11	0.39
MH01	STORAGE	0.32	0.86	103.62	0	02:08	0.86
MH07	STORAGE	0.04	0.20	103.62	0	02:04	0.20
MH09	STORAGE	0.12	0.42	103.62	0	02:01	0.42
MH11	STORAGE	0.18	0.57	103.62	0	02:06	0.57
MH201	STORAGE	0.15	0.45	102.85	0	02:12	0.45

****
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
Connect	JUNCTION	0.000	0.025	0	02:14	0	0.547	0.0
J2	JUNCTION	0.000	0.052	0	04:16	0	0.563	0.2
MH01-N	JUNCTION	0.000	0.004	0	02:08	0	0.105	0.2
MH202	JUNCTION	0.000	0.025	0	02:10	0	0.547	0.0
OGS	JUNCTION	0.000	0.004	0	02:29	0	0.105	-0.0
MH05-DSEL	OUTFALL	0.003	0.029	0	02:29	0.00273	0.655	0.0
CBMH01	STORAGE	0.016	0.035	0	01:08	0.0202	0.047	0.0
CBMH02	STORAGE	0.029	0.042	0	01:09	0.0381	0.0973	0.0
CBMH03	STORAGE	0.009	0.028	0	01:05	0.0103	0.105	-0.0
CBMH06	STORAGE	0.018	0.018	0	01:10	0.0211	0.0211	0.4
CBMH13	STORAGE	0.008	0.011	0	01:06	0.0102	0.0116	-0.0
DICB12	STORAGE	0.005	0.005	0	01:10	0.00439	0.00491	0.3
J1	STORAGE	0.349	0.349	0	01:10	0.559	0.573	-0.1
MH01	STORAGE	0.000	0.011	0	01:05	0	0.104	-0.9
MH07	STORAGE	0.000	0.018	0	01:10	0	0.0211	0.7
MH09	STORAGE	0.000	0.011	0	01:08	0	0.00847	0.4
MH11	STORAGE	0.000	0.018	0	01:10	0	0.0217	-0.8
MH201	STORAGE	0.000	0.029	0	02:25	0	0.652	0.0

****
Node Surcharge Summary
****

No nodes were surcharged.

****
Node Flooding Summary
****

No nodes were flooded.

****
Storage Volume Summary
****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maxi Outf

CBMH01	0.001	9	0	0	0.003	27	0	02:04	0.
CBMH02	0.001	11	0	0	0.003	32	0	02:06	0.
CBMH03	0.001	13	0	0	0.004	36	0	02:08	0.
CBMH06	0.000	0	0	0	0.000	6	0	01:10	0.
CBMH13	0.000	6	0	0	0.001	22	0	02:04	0.
DICB12	0.000	9	0	0	0.001	32	0	02:08	0.
J1	0.133	4	0	0	0.386	13	0	02:11	0.
MH01	0.001	12	0	0	0.002	34	0	02:08	0.
MH07	0.000	2	0	0	0.000	9	0	02:04	0.
MH09	0.000	5	0	0	0.001	17	0	02:01	0.
MH11	0.001	8	0	0	0.003	24	0	02:06	0.
MH201	0.000	5	0	0	0.001	16	0	02:12	0.

****
Outfall Loading Summary
****

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
MH05-DSEL	98.48	0.011	0.029	0.655
System	98.48	0.011	0.029	0.655

****
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
C1	CONDUIT	0.052	0	04:16	0.26	0.03	0.42
P1	CONDUIT	0.008	0	01:07	0.46	0.01	0.45
P10	CONDUIT	0.004	0	02:29	0.48	0.02	0.26
P11	CONDUIT	0.007	0	01:20	0.39	0.01	0.56
P12	CONDUIT	0.025	0	02:10	0.60	0.05	0.39
P13	CONDUIT	0.025	0	02:25	0.47	0.07	0.57
P14	CONDUIT	0.029	0	02:29	0.50	0.05	0.73
P2	CONDUIT	0.011	0	01:08	0.21	0.01	0.44
P3	CONDUIT	0.013	0	01:10	0.47	0.01	0.56
P4	CONDUIT	0.018	0	01:11	0.47	0.01	0.69
P5	CONDUIT	0.004	0	01:08	0.34	0.01	0.63
P6	CONDUIT	0.018	0	01:10	0.64	0.03	0.12
P7	CONDUIT	0.018	0	01:10	0.62	0.03	0.31
P8	CONDUIT	0.016	0	01:10	0.41	0.01	0.57
P9	CONDUIT	0.011	0	01:05	0.68	0.01	0.80
OR1_1	ORIFICE	0.025	0	02:14			1.00
Orifice	DUMMY	0.004	0	02:08			

****
Flow Classification Summary
****

	Adjusted /Actual	---------- Fraction of Time in Flow Class ---------- Up	Down	Sub	Sup	Up	Down	Norm	Inlet

Conduit	Length	Dry	Dry	Dry	Crit	Crit	Crit	Crit	Ltd	Ctrl
C1	1.00	0.02	0.00	0.00	0.98	0.00	0.00	0.00	0.02	0.00
P1	1.00	0.02	0.00	0.00	0.42	0.00	0.00	0.56	0.66	0.00
P10	1.00	0.02	0.00	0.00	0.21	0.00	0.00	0.77	0.00	0.00
P11	1.00	0.02	0.00	0.00	0.38	0.00	0.00	0.60	0.00	0.00
P12	1.00	0.03	0.00	0.00	0.34	0.00	0.00	0.62	0.06	0.00
P13	1.00	0.03	0.00	0.00	0.37	0.00	0.00	0.60	0.00	0.00
P14	1.00	0.02	0.00	0.00	0.51	0.00	0.00	0.48	0.08	0.00
P2	1.00	0.58	0.01	0.00	0.41	0.00	0.00	0.00	0.67	0.00
P3	1.00	0.01	0.00	0.00	0.46	0.00	0.00	0.52	0.02	0.00
P4	1.00	0.01	0.00	0.00	0.48	0.00	0.00	0.51	0.01	0.00
P5	1.00	0.01	0.00	0.00	0.40	0.00	0.00	0.58	0.68	0.00
P6	1.00	0.01	0.00	0.00	0.25	0.00	0.00	0.74	0.07	0.00
P7	1.00	0.02	0.00	0.00	0.33	0.00	0.00	0.65	0.13	0.00
P8	1.00	0.02	0.00	0.00	0.46	0.00	0.00	0.52	0.02	0.00
P9	1.00	0.01	0.00	0.00	0.98	0.00	0.00	0.00	0.00	0.00

****
Conduit Surcharge Summary
****

No conduits were surcharged.

Analysis begun on: Fri Sep 18 02:03:16 2020
Analysis ended on: Fri Sep 18 02:03:17 2020
Total elapsed time: 00:00:01

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


*********************************************************
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 ............ 00:01:00
Dry Time Step ............ 00:01:00
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 ......         0.131        71.708
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........         0.010         5.617
Surface Runoff ...........         0.118        64.779
Final Storage ............         0.003         1.376
Continuity Error (%) .....        -0.090


**************************        Volume        Volume
Flow Routing Continuity        hectare-m      10^6 ltr
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......         0.118         1.179
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........         0.117         1.168
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 ......         0.001         0.012
Continuity Error (%) .....        -0.048
```

****
Time-Step Critical Elements
****
Link P13 (44.87%)

****
Highest Flow Instability Indexes
****
Link Orifice (2)
Link C1 (2)
Link P4 (2)
Link P2 (1)
Link P9 (1)

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

Minimum Time Step	:	2.09 sec
Average Time Step	:	3.86 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	71.71	0.00	0.00	8.62	62.00	0.04	0.0
A2	71.71	0.00	0.00	6.47	64.08	0.07	0.0
A3	71.71	0.00	0.00	8.62	62.02	0.02	0.0
A4	71.71	0.00	0.00	10.34	60.42	0.02	0.0
A5	71.71	0.00	0.00	14.66	56.23	0.02	0.0
A6-1	71.71	0.00	0.00	30.16	41.51	0.00	0.0
A6-2	71.71	0.00	0.00	28.02	43.46	0.01	0.0
A7	71.71	0.00	0.00	4.31	66.21	0.00	0.0
A8	71.71	0.00	0.00	20.72	50.36	0.01	0.0
A9	71.71	0.00	0.00	4.31	66.14	0.01	0.0
Commercial_Area	71.71	0.00	0.00	4.34	65.97	0.98	0.6

****
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
Connect	JUNCTION	0.09	0.21	102.86	0	02:08	0.21
J2	JUNCTION	0.34	0.80	103.50	0	02:06	0.80
MH01-N	JUNCTION	0.04	0.12	102.85	0	02:08	0.12
MH202	JUNCTION	0.15	0.38	102.85	0	02:08	0.38

OGS	JUNCTION	0.13	0.38	102.85	0	02:08	0.38
MH05-DSEL	OUTFALL	0.25	0.60	102.84	0	02:08	0.60
CBMH01	STORAGE	0.46	1.09	104.16	0	02:24	1.09
CBMH02	STORAGE	0.55	1.23	104.16	0	02:22	1.23
CBMH03	STORAGE	0.64	1.35	104.16	0	02:22	1.35
CBMH06	STORAGE	0.17	0.58	104.16	0	02:20	0.58
CBMH13	STORAGE	0.39	0.98	104.16	0	02:24	0.98
DICB12	STORAGE	0.37	0.94	104.16	0	02:22	0.94
J1	STORAGE	0.24	0.70	103.50	0	02:06	0.70
MH01	STORAGE	0.67	1.40	104.16	0	02:22	1.40
MH07	STORAGE	0.24	0.73	104.16	0	02:23	0.73
MH09	STORAGE	0.37	0.96	104.16	0	02:24	0.96
MH11	STORAGE	0.47	1.10	104.16	0	02:23	1.10
MH201	STORAGE	0.17	0.45	102.85	0	02:08	0.45

****
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
Connect	JUNCTION	0.000	0.042	0 02:06	0	0.969	0.0
J2	JUNCTION	0.000	0.044	0 01:39	0	0.97	0.0
MH01-N	JUNCTION	0.000	0.005	0 02:22	0	0.193	0.1
MH202	JUNCTION	0.000	0.042	0 02:10	0	0.969	0.0
OGS	JUNCTION	0.000	0.005	0 02:27	0	0.193	-0.0
MH05-DSEL	OUTFALL	0.005	0.047	0 02:13	0.00555	1.17	0.0
CBMH01	STORAGE	0.029	0.049	0 01:04	0.0366	0.0792	-0.0
CBMH02	STORAGE	0.052	0.067	0 01:04	0.0679	0.187	0.0
CBMH03	STORAGE	0.017	0.040	0 01:05	0.0197	0.192	-0.0
CBMH06	STORAGE	0.032	0.032	0 01:10	0.0394	0.0416	1.1
CBMH13	STORAGE	0.015	0.015	0 01:10	0.0187	0.019	0.0
DICB12	STORAGE	0.010	0.010	0 01:10	0.00999	0.0101	-0.0
J1	STORAGE	0.664	0.664	0 01:10	0.982	0.982	-0.0
MH01	STORAGE	0.000	0.014	0 01:05	0	0.192	-0.4
MH07	STORAGE	0.000	0.035	0 01:07	0	0.0492	-0.0
MH09	STORAGE	0.000	0.017	0 01:09	0	0.0126	0.3
MH11	STORAGE	0.000	0.032	0 01:04	0	0.0606	-0.4
MH201	STORAGE	0.000	0.046	0 02:13	0	1.16	0.0

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Node Surcharge Summary
****

No nodes were surcharged.

****
Node Flooding Summary
****

No nodes were flooded.

****
Storage Volume Summary
****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days	Time of Max Occurrence hr:min	Maxi Outf
CBMH01	0.002	23	0	0	0.005	54	0	02:24	0.
CBMH02	0.003	26	0	0	0.006	57	0	02:22	0.
CBMH03	0.003	28	0	0	0.006	59	0	02:22	0.
CBMH06	0.000	11	0	0	0.001	37	0	02:20	0.
CBMH13	0.001	19	0	0	0.002	48	0	02:24	0.
DICB12	0.001	28	0	0	0.002	74	0	02:22	0.
J1	0.243	8	0	0	0.697	23	0	02:06	0.
MH01	0.002	26	0	0	0.004	54	0	02:22	0.
MH07	0.001	12	0	0	0.002	35	0	02:23	0.
MH09	0.001	15	0	0	0.002	39	0	02:24	0.
MH11	0.002	20	0	0	0.005	46	0	02:23	0.
MH201	0.000	6	0	0	0.001	16	0	02:08	0.

****
Outfall Loading Summary
****

Outfall Node	Flow Freq Pcnt	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr
MH05-DSEL	99.08	0.019	0.047	1.167
System	99.08	0.019	0.047	1.167

****
Link Flow Summary
****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days	Time of Max Occurrence hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
C1	CONDUIT	0.044	0	01:39	0.17	0.02	0.71
P1	CONDUIT	0.009	0	01:09	0.47	0.01	0.96
P10	CONDUIT	0.005	0	02:27	0.52	0.03	0.27
P11	CONDUIT	0.005	0	02:37	0.41	0.01	0.57
P12	CONDUIT	0.042	0	02:10	0.69	0.09	0.40
P13	CONDUIT	0.042	0	02:13	0.54	0.12	0.57
P14	CONDUIT	0.046	0	02:13	0.56	0.08	0.73
P2	CONDUIT	0.017	0	01:09	0.14	0.01	0.95
P3	CONDUIT	0.016	0	01:04	0.46	0.01	1.00
P4	CONDUIT	0.019	0	01:05	0.46	0.01	1.00
P5	CONDUIT	0.008	0	01:10	0.34	0.02	1.00
P6	CONDUIT	0.035	0	01:07	0.74	0.05	0.84
P7	CONDUIT	0.032	0	01:04	0.69	0.05	0.99
P8	CONDUIT	0.023	0	01:09	0.42	0.01	1.00
P9	CONDUIT	0.014	0	01:05	0.69	0.01	1.00
OR1_1	ORIFICE	0.042	0	02:06			1.00
Orifice	DUMMY	0.005	0	02:22			

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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
C1	1.00	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.02	0.00
P1	1.00	0.01	0.00	0.00	0.65	0.00	0.00	0.34	0.44	0.00
P10	1.00	0.01	0.00	0.00	0.21	0.00	0.00	0.78	0.00	0.00
P11	1.00	0.01	0.00	0.00	0.47	0.00	0.00	0.52	0.00	0.00
P12	1.00	0.02	0.00	0.00	0.38	0.00	0.00	0.59	0.06	0.00
P13	1.00	0.02	0.00	0.00	0.43	0.00	0.00	0.54	0.00	0.00
P14	1.00	0.01	0.00	0.00	0.64	0.00	0.00	0.35	0.02	0.00
P2	1.00	0.35	0.01	0.00	0.64	0.00	0.00	0.00	0.45	0.00
P3	1.00	0.01	0.00	0.00	0.69	0.00	0.00	0.31	0.03	0.00
P4	1.00	0.01	0.00	0.00	0.70	0.00	0.00	0.29	0.01	0.00
P5	1.00	0.01	0.00	0.00	0.63	0.00	0.00	0.36	0.46	0.00
P6	1.00	0.01	0.01	0.00	0.50	0.00	0.00	0.48	0.64	0.00
P7	1.00	0.41	0.00	0.00	0.56	0.00	0.00	0.03	0.56	0.00
P8	1.00	0.01	0.00	0.00	0.69	0.00	0.00	0.30	0.02	0.00
P9	1.00	0.01	0.00	0.00	0.99	0.00	0.00	0.00	0.00	0.00

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Conduit Surcharge Summary
****

Conduit	Hours Full			Hours Above Full Normal Flow	Hours Capacity Limited
	Both Ends	Upstream	Dnstream		
P3	1.43	1.43	2.28	0.01	0.01
P4	2.88	2.88	3.49	0.01	0.01
P5	3.80	3.80	4.15	0.01	0.01
P7	0.01	0.01	1.60	0.01	0.01
P8	1.60	1.60	2.28	0.01	0.01
P9	4.15	4.15	4.67	0.01	0.01

Analysis begun on: Fri Sep 18 02:04:22 2020
Analysis ended on: Fri Sep 18 02:04:23 2020
Total elapsed time: 00:00:01

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