1609514
ONTARIO
CORP.

Proposed Townhouse Development

1020, 1024, 1028, 1032 and 1042 Sixth Line Road

Oakville, Ontario

Functional Servicing

and

Stormwater Management Report

(in Support of a Re-Zoning and Official Plan Amendment Application)

Prepared by:

JSW+ associates

Consulting Engineers and Landscape Architects
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44 East Beaver Creek, Unit 1
Richmond Hill, Ontario, L4B 1G6

April 24th, 2023
Project No. 16-29

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Summary of Issues/Revisions

No.	Date	Description
1	April 24th, 2023	ZBA/OPA Submission

Johnson Sustronk Weinstein + Associates
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1.0 INTRODUCTION

1.1 Objectives

This report has been prepared as supporting documentation for a re-zoning and official plan amendment application for the property at 1020, 1024, 1028, 1032 and 1042 Sixth Line Road. The development consists of seven (7) townhouse blocks, one (1) semi-detached block, and one (1) existing single-family unit as indicated on the Architectural Site Plan prepared by Infinity Architecture and Design.

1.2 Scope of Report

The servicing of the development is addressed with respect to storm and sanitary drainage. Water demand for domestic and fire flow requirements are presented. Stormwater management requirements are based on the Town of Oakville's criteria for stormwater management.

In more detail the Functional Servicing sections of this report will address:

- Anticipated domestic water consumption based on Halton Region's criteria.
- Anticipated fire flow demand based on the nature of the development and the requirements of the Fire Underwriter's Survey, 2020 (FUS).
- Review of existing watermains servicing the area.
- Pressure and flow test of existing watermain(s).
- Determine if upgrades are required to the existing watermain infrastructure.
- Anticipated sanitary discharge from the development.
- Current capacity of existing sanitary sewer to which the development will connect.
- Impact to the existing sanitary sewer as a result of the new demand will be assessed.
- Determine if upgrades are required to the existing sanitary sewer infrastructure.
- Storm drainage from the development will be controlled to the 1:5-year pre-development run-off levels storm event up to and including the 1:100-year event as per the Conservation Halton's criteria.
- The existing storm sewer capacity will be determined.

- Impact to the existing storm sewer as a result of the new controlled flow from the development will be assessed.
- Determine if upgrades are required to the existing storm sewer infrastructure.

With regards to stormwater management the following are addressed:

- The Grading Plan and Servicing Plan enclosed with the report indicate direction of surface drainage.
- Emergency overland flow route is indicated on the Grading Plan.
- Storage volumes are provided for detention for events up to and including the 100-year storm.
- Water quality calculation indicating 80% TSS removal.
- Water balance for the site.

1.3 Study Area

The proposed subject site is located in the Town of Oakville and will consist of the comprehensive redevelopment of the properties at 1020, 1024, 1028, 1032, and 1042 Sixth Line Road. These properties are all single-family dwellings and are situated just northeast of the Ministry of Transportation (M.T.O.) Lands which runs parallel to a service road and to Highway 403 (Q.E.W.). The site is also situated to the southwest of Sixth Line Road which traverses the site in a northeast-southwest direction. Additionally, the site is bound by Sixteen Mile Creek to the south and a series of single-family residential dwellings along Sunnycrest Lane to the west. The surveyed site area was found to be a 3.5041-hectare parcel of land. However, a significant portion of the overall site area is located within Halton Region's property on the south side of the site (Sixteen Mile Creek). Sixteen Mile Creek sits approximately +/- 23 meters below the top of the bank, where the properties of 1020, 1024, 1028, 1032, and 1042 Sixth Line Road are located. Therefore, the site will be analyzed based on the general development area of 1.5037-hectares, which is the site area north of a 15-meter setback from the top of the stable slope line as identified in the geotechnical report prepared by Soil Engineers, dated May 29th, 2018 (see the geotechnical report submitted as a stand-alone document). The site presently consists of all single-family dwellings, where the existing residential dwellings at 1020, 1024, 1028, and 1032 will be decommissioned and demolished to facilitate the development of a new residential subdivision.

Figure 1 – Key Plan

2.0 DEVELOPMENT CONCEPT

2.1 <u>Development</u>

The proposed residential development consists of eight (8) residential blocks with a total of 57 proposed units, in which Block 'A' – Block 'F' and Block 'H' are to be townhouse residential blocks for a total of 55 townhouse units. Block 'G' is to be a semi-detached block for a total of two (2) units. The existing single-family residential dwelling at 1042 Sixth Line Road is to remain and is to be incorporated in the post-development condition. There are two (2) proposed access points proposed for the subject site, where one access is along Sunnycrest Lane, while the other will be off of Sixth Line Road. The proposed development will possess 11,602.63 m^2 of total GFA. The parking requirements of the site will be satisfied by providing garage parking for each of the proposed individual units, where the individual units in Block ‘C’ – Block ‘F’ will also consist of personal driveway parking. Additionally, the site will also possess eight (8) visitor parking spaces.

3.0 SANITARY SEWERS

3.1 Existing Sanitary Sewers

Presently, there is an existing 450mm diameter sanitary sewer that is located within Sixth Line Road. The aforementioned sewer is situated along the centerline of Sixth Line Road and traverses the site in a west to east direction, where the existing 450mm diameter sewer slopes at approximately 1.27%. To determine the pipe’s full flow capacity, Manning’s Formula was used (see **Equation 1.0**).

$$Q = \frac{1}{n} * R^{2/3} * S^{1/2} * A \qquad \textbf{[Equation 1.0]}$$

Where,

$Q = Design\ Flow\ \left({m^3}/{sec}\right)$

$n = Manning's\ Coefficient\ of\ Roughness$

$n = 0.013$

$R = Hydraulic\ Radius\ (m)$

$S = Slope\ ({m}/{m})$

$A = Section\ Area\ of\ Flow\ (m^2)$

Based on the existing conditions of the sanitary sewer that fronts the subject site (450mm sewer sloping at 1.27%), the full flow capacity of this sewer was calculated, using **Equation 1.0**, to be 336 L/sec. The existing contributing sanitary drainage area upstream of the subject site is approximately 200.19 hectares (see **FIG-2** and **FIG-3** in *Appendix ‘A’* for reference).

In order to determine the actual existing flows in the 450mm public sanitary sewer that traverses the subject site, the contributing tributary area upstream of the aforementioned sanitary sewer pipe was analyzed based on the Regional Municipality of Halton Water and Wastewater Linear Design Manual criteria (Section 3.2 of the design manual). The design manual states that the sanitary flows can be estimated based on **Equation 2.0** below:

Sanitary Design Flow	=	Average Dry Weather Flow	*	Average Peak Wastewater Flow Factor	+	Inflow / Infiltration Allowance	**[Equation 2.0]**

The average dry weather flow can be calculated using Table 3-1 and Table 3-2 of the design manual (Section 3.2.2). The average peak wastewater flow factor can be calculated using the modified Harmon Formula equation (using **Equation 2.1** and **Equation 2.2**) for all tributary land use mixes. The design manual also states that for all drainage areas that do not include existing developed areas, the inflow/infiltration allowance shall be 0.286 x 10^{-3} m^3/ha/s.

$$M = K_{av} * \left(1 + \frac{14}{4+\sqrt{P_e}}\right) \quad \textbf{[Equation 2.1]}$$

Where,

$$K_{av} = \left(\frac{A_R+(0.8*(A_I+A_C)}{A_R+A_I+A_C}\right) \quad \textbf{[Equation 2.2]}$$

And:

$M = ratio\ of\ peak\ flow\ to\ average\ flow$

$P_e = equivalent\ tributary\ population\ in\ thousands$

$A_R = residential\ land\ use\ area\ (ha)$

$A_I = industrial\ land\ use\ area\ (ha)$

$A_C = commercial\ land\ use\ (ha)$

Note, the minimum permissible peaking factor (M) is to be used as 2.0. Additionally, land use determinations that were used were based on the planned land use designation. Tributary populations are the product of land use area (in hectares) and equivalent population density (persons per hectare) as described in Table 3-1 and Table 3-2 in Section 3.2.2 of the design manual.

Based on the aforementioned criteria, the existing contributing sanitary drainage area was analyzed using **Equation 2.0**, **Equation 2.1**, and **Equation 2.2**. Firstly, a review of the existing contributing sanitary drainage area found that it primarily consists of a mix of single-family dwellings, semi-detached dwellings, and townhome dwellings. As per Table 3-1 and 3-2 in the Regional Municipality of

Halton Water and Wastewater Linear Design Manual, the equivalent population densities for single-family units, semi-detached units, and townhouse units are 55 persons/hectare, 100 persons/hectare, and 135 persons/hectare, respectively. Since a large majority of the contributing area consists of single-family dwellings, the population density of 55 persons/hectare was utilized in the population approximation. As per **Figure 2** and **Figure 3**, through a tributary area of 200.19 hectares, the total estimated population contributing to this existing sewer is 11,011 persons (see **Table 1** below). Based on the aforementioned, the existing (pre-development) average dry weather flow for the existing contributing sanitary drainage area can be found in **Table 2** below.

Table 1 – Pre-Development Sanitary Population Estimation

Existing Contributing Sanitary Drainage Area	Area (ha)	Pe, (Equivalent Population Density – persons/ha)	P, (Population - persons)
Tributary Area Upstream of Subject Site [1]	200.19	55	11,011

[1] *See FIG-2 and FIG-3 in Appendix 'C' for details.*

Table 2 – Pre-Development Sanitary Average Dry Weather Flow Table

Area (ha)	P, (Population - persons)	Unit Sewage Flow (m^3pcd) [2]	Average Dry Weather Flow (L/sec)	Average Peak Wastewater Flow Factor (M)	Inflow / Infiltration Allowance (L/sec)	Sanitary Design Flow (L/sec)
200.19	11,011	0.275	**35.05**	**2.91**	**57.26**	**159.26**

[2] *m^3pcd = cubic meters per capita per day.*

As illustrated above, the average dry weather flow was determined to be 35.05 L/sec in the pre-development condition for the existing contributing sanitary drainage area assuming an equivalent population density of 55 persons/hectare. Additionally, using the data from **Table 1**, the average peak wastewater flow factor (M) was calculated to be 2.91 using **Equation 2.1** and **Equation 2.2** using an equivalent population density of 55 persons/hectare. The inflow/infiltration allowance was also calculated, based on the existing contributing sanitary drainage area, to be 57.26 L/sec [(0.286 x 10^{-3} m^3/ha/s) * (200.19 ha) * (1000L/m^3)]. This means that, in the pre-development condition, the existing maximum day sanitary flow in the existing 450mm sewer fronting the site can conservatively be estimated as 159.26 L/sec using **Equation 2.0** [(35.05 L/sec * 2.91) + 57.26 L/sec]. Therefore, since the full flow capacity in the existing sewer is 336 L/sec, the remaining capacity is 176.74 L/sec.

3.2 Proposed Sanitary Sewer Design

The proposed development's sanitary sewer system was designed with a minimum 200mm PVC DR-35 sanitary pipe throughout the development, where the minimum slope on the first leg of each sanitary sewer is 2.0%. Connection to the existing sanitary sewer within Sixth Line Road is to be made by a 12.7-meter long 200mm PVC DR-35 sanitary pipe sloping at 2% towards proposed Manhole 1A (see the *Servicing Plan*, ***S-1***, for details). Manhole 1A will be connected directly to the sanitary sewer main on Sixth Line Road as referenced in ***Section 3.1***.

Using the same methodology described in ***Section 3.1***, the post-development sanitary design flow can be determined. As per Table 3-1 design manual, the proposed development (1020-1042 Sixth Line Road) can be categorized into three (3) different development classifications. The existing development at 1042 Sixth Line Road is to remain in the post-development condition and will not be redeveloped. As such, 1042 Sixth Line Road is classified as a single-family development but will not be considered in the post-development sanitary design flows because the design flow was already estimated as part of the existing contributing sanitary drainage area as outlined in ***Section 3.1***. The proposed Development Blocks A-F and Development Block H can all be classified as townhouse developments. Development Block G can be classified as a semi-detached development. Based on these development classifications stated in Section 3.2 of the design manual, as well as the known site area, a population estimation can be determined for the site area (see **Table 3** for equivalent population densities below). Based on this calculation, it is estimated that the proposed population of the site is 201 persons.

Additionally, **Table 4** below summarizes the post-development average dry weather flow and peak sanitary design flow. Subsequently, the average dry weather flow was determined to be 0.64 L/sec in the post-development condition.

Additionally, using the data from **Table 4**, the average peak wastewater flow factor (M) was calculated to be 4.15 using **Equation 2.1** and **Equation 2.2**. The inflow/infiltration allowance was also calculated based on the developable site area to be 0.43L/sec. Subsequently, using **Equation 2.0**, the sanitary wastewater demand for the subject site in the post-development condition is 3.08L/sec. Therefore, it is evident (as summarized in **Table 5**), that approximately 173.66L/sec of capacity remains in the existing 450mm sanitary sewer main located within Sixth Line Road.

Table 3 – Post-Development Sanitary Population Estimation

Existing Contributing Sanitary Drainage Area	Area (ha)	Pe, (Equivalent Population Density – persons/ha)	P, (Population - persons)
Block A – F + Block H (Townhouse)	1.4156	135	192
Block G (Semi-Detached)	0.0881	100	9
Total	**1.5037**		**201**

Table 4 – Post-Development Sanitary Average Dry Weather Flow Table

Area (ha)	P, (Population – persons)	Unit Sewage Flow (m^3pcd) [2]	Average Dry Weather Flow (L/sec)	Average Peak Wastewater Flow Factor (M)	Inflow / Infiltration Allowance (L/sec)	Sanitary Design Flow (L/sec)
1.5037	**201**	0.275	**0.64**	**4.15**	**0.43**	**3.08**

[2] m^3pcd = cubic meters per capita per day.

Table 5 – Sanitary Demand Comparison Summary Table

Existing Full Flow Sanitary Discharge Rate in Connecting Sewer (L/s)	Proposed Peak Sanitary Discharge Rate from Development (L/s)	Post-Development Full Flow Sanitary Discharge in Connecting Sewer (L/s)	Full Flow Capacity of Connecting Sewer (L/s)	Post-Development Available Capacity in Connecting Sewer (L/s)
159.26	3.08	162.34	336	**173.66**

Based on the results summarized in **Table 5**, the existing sanitary sewer in the right-of-way is adequately sized to receive sanitary effluent from the proposed development and as such, no adverse impacts on the sanitary system should occur due to the proposed development.

4.0 WATER DISTRIBUTION

4.1 Existing Watermains

There is an existing 300 mm watermain along Sixth Line Road that will act as the supply line for the development. Presently, the properties at 1020-1042 Sixth Line Road are being serviced by the aforementioned watermain within Sixth Line Road. All the aforementioned service connections for the properties at 1020-1042 Sixth Line Road will be decommissioned and removed during construction, where the property at 1042 Sixth Line Road will be connected to the proposed watermain upon completion of the construction of the installation the newly proposed watermain.

4.2 Proposed Watermains

The proposed development will be supplied by a 150mm watermain network that will be drawing from the existing 300mm watermain within Sixth Line Road in two (2) locations, where one location is at the proposed entrance driveway along Sixth Line Road, and the other is at the most easterly location of the site along Sixth Line Road (see the *Servicing Plan*, ***S-1***, for details). Each connection to the existing 300mm watermain within Sixth Line Road will be made as per Halton Region Standard RH-409.010. This proposed system will service the domestic and fire demand requirements within the development. Each unit in each development block will be supplied by a 25mm diameter copper service connection branching off the proposed 150mm watermain, where each unit will be equipped with a shut off valve at the respective branch locations. Additionally, each unit in each development block will possess a 25mm water meter to be installed as per Halton Region Standard RH-500.010 and 500.011.

The site will also posses three (3) proposed fire hydrants within the site and will be installed at a maximum of 90m spacing between each hydrant. Another parameter considered in the placement of the hydrants (as illustrated on the *Servicing Plan*, ***S-1***) was to ensure that a 90m maximum distance was maintained from any existing/proposed dwellings within the site to any one of the existing/proposed hydrants. All proposed hydrants will be installed as per Halton Region Standard RH-407.010.

4.2.1 Domestic Demand

The watermain design must accommodate the demand under the greater of the two conditions, the maximum daily demand plus fire flow and the maximum hourly demand. As per the Region of Halton Water and Wastewater Linear Design Manual the proposed development (1020-1042 Sixth Line Road) can be categorized into three (3) different development classifications. The existing development at 1042 Sixth Line Road is to remain in the post-development condition and will not be re-developed. As such, development Blocks A-F and Development Block H is classified as townhouse developments. Development Block G is classified as a semi-detached development (see **Section 3.2** for additional details). Based on the development classification, the domestic demand is summarized for average daily demand, maximum daily demand, and maximum hourly demand, based on peaking factors stated in the design manual (Section 2.4), in **Table 6** as follows. The development demand for the average day is calculated by multiplying 0.275 m^3/capita by the estimated population in each block, which is estimated by the approximate site area each block covers. The remaining parameters are calculated by multiplying the peaking factor by the average daily domestic demand.

Table 6 – Domestic Demand

	Estimated Population	Average Daily Demand (L/min)	Maximum Daily Demand (L/min)	Maximum Hourly Demand (L/min)
		Peaking Factors (Residential)		
		1.0	2.25	4.00
Block A – F, Block H (Townhouse)	192	36.67	82.50	146.67
Block G (Semi-Detached)	9	1.72	3.87	6.88
Total	**201**	**38.39**	**86.37**	**153.54**

4.2.2 Fire Demand

Based on the Fire Underwriters Survey, Water Supply for Fire Protection (2020), an estimate of the fire flow required is given by **Equation 3.0**:

$$RFF = 220C\sqrt{A}$$ **[Equation 3.0]**

where: $RRF = required\ fire\ flow\ in\ litres/minute$

$C = coefficient\ relating\ to\ type\ of\ construction$

$A = floor\ area\ in\ square\ metres$

The calculations were completed based on analyzing three (3) development blocks (Block 'A', Block 'B' and Block 'E') which would have the highest population density and the lowest separation exposure between proposed development blocks and/or existing developments neighbouring the subject site. Based on the calculations it was determined that Development Block 'B' would be the site's worst-case scenario in being affected by the design fire. The required fire flow was found to be 20,000 L/min (see Fire Demand Calculations in *Appendix 'B'*).

4.2.3 Total Demand

The total demand is the greater of the two conditions, the maximum daily demand plus fire flow and the maximum hourly demand. Thus:

$$Total\ Demand = 86.37 + 20{,}000$$
$$Total\ Demand = 20{,}087\ {}^{L}/_{min}$$

Hydrant flow tests were undertaken for this site by Corix Water Services on dated November 3rd, 2016. The available flow rate from the Sixth Line Road hydrant at 20 psi has been determined by using the data from the hydrant flow test within the Hazen-Williams Equation (see the hydrant flow test and analysis in *Appendix 'B'*). Additionally, the flow and pressure relationship of this fire hydrant was graphed on N-1.85 paper, as shown in *Appendix 'B'*. Based on the aforementioned, the flow rates from the hydrant located on Sixth Line Road at 20 psi is approximately 30,261 liters/minute

Therefore, it is evident that the site's water demand will not exceed the capacity within the municipal watermain. This is because the total demand required for the proposed development (20,087 L/min) is less than the available flow rate within Sixth Line Road (30,261 L/min).

5.0 STORM SEWERS

5.1 Existing Site Drainage and Storm Sewer System

Currently the site slopes from the southwest corner towards the northeast corner of the property (towards Sixth Line Road). Additionally, the development is located at the top of the bank of Sixteen Mile Creek. Sixteen Mile Creek sits approximately +/- 23 meters below the top of the bank, where the properties of 1020, 1024, 1028, 1032, and 1042 Sixth Line Road are situated. For the purposes of this residential development, and to ensure there are no adverse impacts to Sixteen Mile Creek from the overland flow drainage of the site, the proposed development plans were to be setback 15m north of the long-term stable slope as identified in the geotechnical report prepared by Soil Engineers, dated May 29th, 2018 (see the geotechnical report submitted as a stand-alone document). There is no uncontrolled overland flow drainage proposed to Sixteen Mile Creek from the subject site.

The existing development does not contain any stormwater management infrastructure. Any excess stormwater runoff is being sheet drained from the southwest side of the site towards Sixth Line Road. At present, there is no stormwater management being implemented on the site and the storm runoff is being discharged at an uncontrolled rate. The stormwater runoff is being released to catch basins on Sixth Line Road. There is an existing 525mm diameter storm sewer that is located within Sixth Line Road. The aforementioned sewer is situated just to the northeast of the centerline of Sixth Line Road and traverses the site in a west to east direction, where the sewer slopes at approximately 1.20%. To determine the pipe's full flow capacity, Manning's Formula was used (see ***Section 3.1*** and **Equation 1.0**). Based on the existing conditions of the storm sewer fronts the subject site (525mm sewer sloping at 1.20%), the full flow capacity of this sewer was calculated, using **Equation 1.0**, to be 490.50 L/sec.

5.2 Proposed Storm Drainage and Storm Sewer Design

It is proposed to discharge storm flows from the development to the existing storm sewer located within Sixth Line Road utilizing a proposed minor system (see ***Section 5.2.1*** and ***Section 6.0*** for details). Any storm in excess to the 1:100-year storm event will be directed to Sixth Line Road using the overland flow route of the major system.

5.2.1 Proposed Storm Flows

Based on the Town of Oakville's design criteria, the allowable outflow from the site will be limited to the 5-year pre-development Rational Method peak flow. Based on a time of concentration of 10 minutes, a pre-development runoff coefficient of 0.34 and a site area of 1.5037 hectares, it is determined that the allowable outflow from the site will be 166.99 L/sec **–** this accounts for the uncontrolled flows (see **FIG-4** and detailed calculations in *Appendix 'C'* and ***Section 6.6*** for details).

5.2.2 Storm Connection

In order to service the proposed residential development, a service connection will be required to discharge the captured stormwater runoff from the site. As per ***Section 5.1***, the site is presently discharging to Sixth Line Road at an uncontrolled rate. In order to preserve the pre-existing drainage pattern, the site will continue to discharge to the existing storm sewers located within Sixth Line Road in the post-development condition.

The receiving sewer will have adequate capacity to service the proposed development based the fact that the site will now control the 100-year storm event to the 5-year pre-development condition, as opposed to the pre-construction condition, where the site did not implement any stormwater management techniques.

6.0 STORM DRAINAGE AND STORMWATER MANAGEMENT

6.1 Drainage Parameters

The general requirements for drainage and stormwater management are specified in the Town of Oakville's Development Engineering Procedures and Guidelines, Conservation Halton Guidelines for Stormwater Management Engineering Submissions, and the MECP Stormwater Management Planning and Design Manual:

i. TSS Removal:
Long term average removal of 80% of total suspended solids.

ii. Water Balance:
The criterion is based on the MECP Stormwater Management Planning and Design Manual that stipulates that the site is to retain the runoff from a 5mm storm by infiltration, evapotranspiration or reuse.

iii. Allowable Site Discharge Rate Criteria:
The peak allowable outflow from the site must be limited to the peak runoff rate generated in the pre-development condition by a 5-year design rate using a Time of Concentration of 10 minutes and a maximum runoff coefficient equal to 0.50, or the existing capacity of the receiving storm sewer, whichever is less.

6.2 Water Quality Control

Water quality for the site must satisfy 80% Total Suspended Solids (TSS) removal. It is assumed that the removal ratio for an impervious roof and soft landscaping surfaces is 80%. No removal rate was assigned to the hard-landscaped areas. Based on the types of surface areas on the site, the following water quality table was prepared to summarize the Total Suspended Solids (T.S.S.) removal.

Table 7 below indicates that 57.4% of T.S.S. is removed from the site during the initial T.S.S. Treatment (see *Figure 6* enclosed in *Appendix 'C'* for details). Therefore, a Jellyfish Filtration Unit will be installed in an offline configuration to remove the remaining T.S.S. in accordance with the 80% minimal removal rate.

Table 7 – Stormwater Quality from Site

Surface	Fractional Area (ha) [3]	Percentage of Total Area	TSS Removal for Site Area	TSS Removal Over Total Site
Hard Landscape	0.4259	28.32%	0%	0%
Soft Landscape	0.6388	42.48%	80%	34.0%
Bare Roof	0.4390	29.09%	80%	23.4%
TOTAL	**1.5037**	**100%**		**57.4%**

[3] **See *FIG-5* enclosed in *Appendix 'C'* for details.**

6.3 Water Balance

The MECP Stormwater Management Planning and Design Manual requires that 5mm of every storm event be retained on site. Therefore, the surface treatments in the post-development condition were analyzed to affect a water balance on the site. As per **Table 8** below, the site will abstract 2.72mm of water through evapotranspiration and infiltration, for every rainfall event; this generates a shortfall of 2.28mm from every storm event to adhere to the water balance criteria.

In order to achieve the water balance for the site, retention storage volume will be proposed in conjunction with the stormwater management detention storage tank (see ***Section 6.5*** and ***Section 6.5.1*** for additional details). The retention storage is proposed as an infiltration trench in order to recharge the subgrade with stormwater. The retention storage is proposed in addition to required detention volume for stormwater management purposes as indicated in ***Section 6.5***. The detention and retention storage volumes for the site will consist of a series of interconnected stormwater management chambers (plastic blocks or segments), as manufactured by Greenstorm (see *Appendix 'C'* for details). Greenstorm ST stormwater management blocks can provide stormwater detention storage but also provides a potential alternative to a clear stone infiltration trench as the Greenstorm ST stormwater management block is manufactured with a 96% porosity. This allows for a more efficient infiltration trench storage footprint as it allows for a larger storage volume without occupying large, site area as compared to a traditional clear stone infiltration trench with a porosity of 40%.

Table 8 – Water Balance of Site

Developable Area	Area (ha) [5]	Fraction of Area	Initial Abstraction (mm)	Initial Abstraction over Area (mm)
Hard Landscape	0.4259	28.32%	1	0.29
Soft Landscape	0.6388	42.48%	5	2.13
Bare Roof	0.4390	29.09%	1	0.30
TOTAL	**1.5037**	**100%**		**2.72**

[1] See *Figure 5* enclosed in *Appendix 'C'* for details

Based on the shortfall of 2.28mm of initial abstraction, the required volume to be stored in the infiltration trench is to be a minimum of 35m^3. This retention volume was calculated by multiplying the initial abstraction shortfall over the entire site area (0.00228m x 15,037m^2 = 35m^3). This retention volume will be achieved by providing approximately 36m^3 of storage over an area of 55.68m^2 (at 0.66m deep) using the Greenstorm ST stormwater management blocks. The retention tank will be situated northwest of the proposed entrance driveway located at Sixth Line Road and just north of the Development Block 'C' (see ***Section 6.5.1*** for additional details).

As discussed in the geotechnical report prepared by Soil Engineers, dated May 29th, 2018 (see the geotechnical report submitted as a stand-alone document), Borehole 8 (BH-8) was drilled in the approximate area on which the proposed infiltration tank will be installed. Based on the observations from this borehole a groundwater table was not observed, and the borehole was terminated at a depth of 6.3m (elevation of 102.90m) as the auger encountered bedrock. The Ontario Building Code (OBC) stipulates that the underside of any infiltration trench must be a minimum of 1.0 meter above any groundwater table elevation or bedrock elevation. The underside of the proposed retention tank (infiltration trench) will be situated at ±106.84m, which leaves approximately 3.94m of vertical separation under the infiltration trench to the bedrock.

Based on the aforementioned existing subgrade conditions, it will be permissible to install an infiltration trench. In accordance with the referenced geotechnical report, the site's stratigraphy in the location where the proposed SWM tank is situated mainly consists of silty clay till. Therefore, it is assumed that the approximate soil infiltration rate is 7mm/hr), in which a retention (infiltration) volume of $36m^3$ will have a 48-hour drawdown period using Darcy's Law (see Appendix 'C' for drawdown time calculations). Based on the site's proposed retention storage volume of $36m^3$ dedicated for infiltration, the site's abstraction increases by 2.39mm, resulting in a total abstraction of 5.11mm. Therefore, the water balance requirement has been achieved for this development.

6.4 External Drainage

There are no external drainage areas contributing storm runoff to the subject property. It is to be noted that there are two properties to the west of the subject site (1048 and 1052 Sixth Line Road) that roughly 50% of their respective areas drain into the proposed development site. The existing overland flow route from the properties at 1048 and 1052 Sixth Line Road will be redirected towards Sixth Line Road as a proposed retaining wall will be installed in a north-south direction along the west side of the proposed landscaping feature (see the *Grading Plan*, ***G-1***, enclosed at the back of this report). Therefore, no additional flow will be discharging into the proposed development from the overland flow drainage from the sites at 1048 and 1052 Sixth Line Road.

6.5 Quantity Control

The quantity control of the site was established by approximately controlling the 100-year post-development flow rate to the 5-year pre-development flow rate. This volume was determined by using the Modified Rational Method (see *Appendix 'C'* for details).

The site in the pre-development condition consisted mostly of soft landscaping, ultimately generating a pre-development runoff coefficient of 0.35. The total area of the subject site is 1.5037 hectares (see ***Section 5.1*** for details). Based on the pre-development runoff coefficient of 0.34, the peak 5-year allowable runoff rate for the site is 166.99 L/sec.

However, in the post-development condition, there is a total uncontrolled area of 0.0471 hectares and an uncontrolled post-development runoff coefficient of 0.35. Based on the aforementioned, the proposed controlled post-development site area is 1.4566 hectares. Based on the controlled site area, the controlled post-development site runoff coefficient is 0.63. Therefore, the peak allowable controlled discharge rate was calculated as 157.80 L/sec.

In accordance with the orifice sizing calculations (see *Appendix 'C'* for details), an orifice tube will be required on this site to attenuate the storm discharge. The orifice tube diameter was determined using **Equation 4.0**, below.

$$A = \frac{Q}{C\sqrt{2gh}} \qquad \textbf{[Equation 4.0]}$$

where:

$Q = Peak\ Allowable\ Controlled\ Discharge\ Rate$

$C = 0.82\ (Orifice\ Tube\ Coefficient)$

$g = 9.81\ {}^{m}/_{s^2}\ (gravitational\ constant)$

$h = depth\ of\ 1{:}100\ year\ stom\ event$

$A = Area\ of\ orifice,\ m^2$

Based on the stormwater management calculations (see *Appendix 'C'* for details), the depth of the 1:100-year storm event was determined at an elevation of 108.66m assuming a 289m^3 storage capacity. Based the aforementioned factors and using **Equation 4.0**, the maximum permissible orifice diameter was calculated as 223mm. Since orifice tubes come in nominal sizes, the orifice tube diameter was rounded down to the nearest PVC DR-35 pipe size, which was determined to be a 200mm orifice pipe. Thus, the stormwater discharge flow rate was adjusted to 126.04 L/sec for a 200mm orifice tube and will require a minimum of 252 m^3 of detention volume storage.

The detention storage volume requirement of 252 m^3 is to be achieved by implementing a series of interconnected stormwater management chambers (plastic blocks or segments), as manufactured by Greenstorm (see ***Section 6.5.1*** and *Appendix 'C'* for additional details). The stormwater detention tank will span an approximate surface area of 227.84m^2 and will be 1.32m deep (289m3 with a 96% porosity in the Greenstorm system). Additionally, one of the advantages to implementing a Greenstorm stormwater management tank is that the system can infiltrate the captured stormwater to recharge the subgrade with stormwater. Due to site constraints, mainly the detention tank's proximity to the proposed development Block 'C,' all Greenstorm blocks within five (5) meters of Development Block 'C' will be wrapped with an impermeable liner on the bottom and the sides of the tank such that no infiltration will occur (see *Servicing Plan*, ***S-1***, for details). The area of the tank within 5m of the of Development Block 'C' is 108.16m^2 (47.5% of the total tank volume is to be impermeable tank to be wrapped in the impermeable liner on the bottom and the sides of the tank). The remaining area of the tank (approximately 119.68 m^2 or 52.5% of the tank) outside of the 5m offset from Development Block 'C' will not be wrapped in the impermeable liner. This means that there is approximately 152m^3 of available detention storage volume that will be permitted to infiltrate into the surrounding soils. In order to provide a conservative approach in addressing the discharge rate for the subject site, the available detention storage that is permissible to infiltrate was not considered in the discharge rate of the detention volume in the stormwater management calculations. The infiltration rate into the surrounding soils was not considered because the infiltration of the stormwater detention volume will occur over a minimum of a 48-hour drawdown period, where the discharge rate that is controlled by the orifice tube will discharge the entire detention volume in a few hours. This ultimately provides a higher designed discharge rate than what will realistically occur on site.

The stormwater flows for this site can be managed without proposing any surface storage (ponding), therefore no surface storage is proposed for this development. Lastly, since the site is not presently utilizing a stormwater management system and is discharging at an uncontrolled rate, it is anticipated that the receiving system (an existing 525mm diameter storm sewer) will not be subjected to any adverse affects from the aforementioned discharge rate. This is because the post-development condition is an improvement to the discharge rate, in relation to the pre-development condition.

6.5.1 <u>Site Storage</u>

In the post-development condition, the site will release the captured stormwater runoff at a controlled rate of 126.04 L/sec (see ***Section 6.5*** for details). Based on the flow attenuation, the development will require a stormwater management tank to provide temporary detention storage prior to the release of the runoff.

The proposed stormwater management tank will be located on the north side of the site just to the west of the proposed entrance to the subject site. This system consists of a series of interconnected stormwater management chambers. As the stormwater management tank is comprised of plastic blocks or segments (as manufactured by Greenstorm), the segments will be referred to as the Greenstorm blocks throughout this report. The Greenstorm blocks are provided by the manufacturer called Stormcon and the product is a Greenstorm ST stormwater management block (see *Appendix 'C'* for details).

Each Greenstorm block possesses a segment dimension of 0.8m (L) x 0.8m (W) x 0.66m (D) and 0.406 m^3 of storage per unit as the system has been specified with 96% porosity (air voids). Furthermore, the Greenstorm blocks are also stackable in layers in order to accommodate required detention volumes. The subject site's stormwater management tank will posses two (2) full layers of stacked Greenstorm blocks spanning an approximate surface area of 228.5 m^2 x 1.32m deep.

Additionally, one of the advantages to implementing a Greenstorm block stormwater management tank is that the system can infiltrate the captured stormwater to recharge the subgrade with stormwater. Due to the location of the proposed stormwater management tank, mainly its proximity to the proposed development Block 'C,' all Greenstorm blocks within five (5) meters of Development Block 'C' will be wrapped with an impermeable liner such that no infiltration will occur (see *Servicing Plan*, ***S-1***, for details). The area of the tank to be wrapped in the impermeable liner will be approximately 108.16m^2 (47.3% of the total tank volume is to be impermeable). This will prevent any excess stormwater drainage from being collected by the foundation drainage collection system (weeping tile) and/or having stormwater back up into the basement level located within Block 'C.' The tank areas located greater than five (5) meters from Block

‘C’ will not be wrapped in the impermeable layer and will allow infiltration to occur.

It is to be noted that the site is proposing to utilize infiltration in order to meet the water balance requirements for the development. In order to do so, a full third layer of Greenstorm blocks will be proposed beneath the stormwater detention storage volume Greenstorm Blocks. This third layer of blocks will be dedicated as retention storage volume for a total of 36m^3, which is to be drawn down over 48 hours (see ***Section 6.3***). The retention storage volume is not accredited in the overall detention storage volume provided of 280m^3. Therefore, the quantity control for the site is satisfied for the detention storage of 252 m^3 and retention storage of 35 m^3 based on the proposed storage of 289 m^3 and 36m^3 for detention and retention storage, respectively, for a total stormwater storage volume of 325m^3 (see ***Section 6.3*** and ***Section 6.5*** for details).

7.0 EROSION AND SEDIMENT CONTROL

An effective Erosion and Sediment Control (ESC) Plan is essential for minimizing the potentially adverse environmental effects originating from a construction site. A multi-barrier approach is to be considered to limit the adverse effects, such as to prevent erosion during the construction process to deal with the suspended sediment at the source and to minimize sediment transport from leaving the construction site. The following measures will be implemented onsite during construction to minimize sediment transport downstream of the site, in accordance with the Greater Golden Horseshoe Area Conservation Authorities Erosion & Sediment Control Guidelines for Urban Construction in addition to Halton Region requirements:

- Temporary silt fencing will be installed along site boundary prior to any construction / grading activities (as illustrated on CMESC-1, CMESC-2, and CMESC-3 as per Halton Region Standard Drawing RH-200.040).
- Construction access mud mats will be installed (as illustrated on CMESC-1 as per as per Halton Region Standard Drawing RH-200.040).
- Any and all future grassed areas will be seeded as soon as possible.

- All erosion and sediment control measures shall be installed prior to commencement of site construction works and will remain in place through the duration of construction.
- Conduct inspections of all erosion and sediment control measures after significant rainfall and snowmelt events.

The contractor is to ensure that all involved parties on site are familiar with ESC practices and are trained in the ECS Plan, implementation, inspections, maintenance, and repairs. During construction, the site inspector shall monitor the ECS measures and shall be maintained by the contractor.

8.0 CONCLUSIONS AND SUMMARY

a) The capacity of the sanitary sewer network that services the property is adequate to accommodate post-development flows from the proposed development.

b) Peak sanitary flows from the proposed development are expected to be 3.08 litres/second.

c) A hydrant flow test was completed by Corix Water Services (dated November 3rd, 2016) and it was found that the available flow is 30,261 litres/minute for the existing hydrant on Sixth Line Road.

d) The required fire flow for the proposed development is 20,000 litres/minute, which is less than the available flow measured on Sixth Line Road.

e) The required domestic flow for the proposed development is 20,087 litres/minute, in accordance with the maximum daily demand plus fire flow criteria.

f) The receiving stormwater system (Sixth Line Road) has adequate capacity under the 1 in 100-year storm event to accommodate peak storm outflows from the proposed development.

g) Quantity control requirements must limit peak site outflows from the 1 in 100-year storm event to less than 126.04 liters/second and store a total of 252 m^3 of runoff. This is achieved by implementing a Greenstorm ST stormwater management system, providing 289 m^3 of detention storage (in conjunction with a 200mm orifice tube).

h) The Greenstorm ST stormwater management system will provide detention storage volume for the site and will consist of a series of two (2) full layers of Greenstorm blocks. The Greenstorm block's dimensions are 0.88 m (L) x 0.88 m (W) x 0.66 m (D) (1.32m overall depth).

i) Water quality target of 80% T.S.S. removal will be met by implementing a Jellyfish Filtration Unit.

j) Water balance targets have been achieved by providing 5.11mm of abstraction on site by managing the requirement of 35m^3 of retention storage (by means of an infiltration trench) on site using the Greenstorm ST stormwater management system. The Greenstorm ST stormwater management system will provide retention storage volume of 36m^3.

Table 9 – Stormwater Management Summary Table

Item	Value
Actual Release Rate	126.04 L/sec
Required Detention Storage	252 m^3
Provided Detention Storage	289 m^3
Required Retention Storage	35 m^3
Provided Retention Storage	36 m^3
Total Required Stormwater Storage	287 m^3
Total Provided Stormwater Storage	325 m^3
Orifice Size	200mm
High Water Level at 100-Year Storm Event	108.66m
Water Balance Required	5.00 mm
Water Balance Provided	5.11 mm
TSS % Removal	80%

Johnson Sustronk Weinstein + Associates

Prepared by:

LICENSED PROFESSIONAL ENGINEER
16-29
Apr. 24 / 2023
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PROVINCE OF ONTARIO

Stephan Starcevic, P.Eng.

Checked By:

Michael Mikhail, P.Eng.

APPENDIX ‘A’:

SANITARY DESIGN SHEET

FIG-2 – EXISTING SANITARY DRAINAGE AREA

FIG-3 – EXISTING SANITARY DRAINAGE AREA

JSW+ associates

Sanitary Drainage Design Sheet

Post-Development Sanitary Flows

Population Equivalents:

1. Townhouse, Pe = 135 persons/ha
2. Semi-detached, Pe = 100 persons/ha
3. Single-Family, Pe = 55 persons/ha

Street	Up Stream MH	Up Stream Invert	Down Stream MH	Down Stream Invert	Increment $Area_{SD}$ (ha)	Increment A_{TH} (ha)	Increment P	Cumulative p	Cumulative Areas Ha.	M	q_{dry} (L/s)	A Road (ha)	A Gross (ha)	INF (L/s)	Q Total (L/s)	Pipe L (m)	Pipe Actual Size [T] (mm)	Pipe Nominal Size (mm)	Pipe Grade (%)	Pipe Cap. Flow (l/s)	Pipe Cap. Vel. (m/s)	Pipe Act. Vel. (m/s)	Pipe % Full	Cross-Sectional Area of Pipe [A] m^2	Hydraulic Mean Depth [h_m] m	Froude Number [Fr]	Type of Flow	NOTE
Block H (3 Units - Townhouse)	MH7A	108.60	MH6A	108.31	0.0000	0.0926	13	13	0.0926	4.40	0.04	-	0.0926	0.026	0.21	14.3	203	200	2.0	48.60	1.50	0.08	0.43	0.0324	0.1594	0.06	Sub-critical	1,2
	MH6A	108.26	MH10A	108.17	0.0000	0.0000	0	13	0.0926	4.40	0.04	-	0.0926	0.026	0.21	4.6	203	200	2.0	47.74	1.47	0.08	0.44	0.0324	0.1594	0.06	Sub-critical	1,2
Block G (2 Units - Semi-Detached)	MH14A	108.50	MH10A	108.17	0.0881	0.0000	9	9	0.0881	4.42	0.03	-	0.0881	0.025	0.15	16.7	203	200	2.0	47.97	1.48	0.06	0.32	0.0324	0.1594	0.05	Sub-critical	1,2
Block F (4 Units - Townhouse)	MH10A	108.12	MH5A	107.76	0.0000	0.1241	17	39	0.3048	4.34	0.12	-	0.3048	0.087	0.63	30.3	203	200	1.2	37.20	1.15	0.19	1.68	0.0324	0.1594	0.15	Sub-critical	1,2
Block F (4 Units - Townhouse)	MH5A	107.71	MH4A	107.41	0.0000	0.1241	17	56	0.4289	4.30	0.18	-	0.4289	0.123	0.89	24.6	203	200	1.2	37.69	1.16	0.23	2.36	0.0324	0.1594	0.19	Sub-critical	1,2
Block E (1 Unit - Townhouse)	MH11A	107.96	MH9A	107.84	0.0000	0.0310	4	4	0.0310	4.45	0.01	-	0.0310	0.009	0.07	13.3	203	200	0.9	32.42	1.00	0.05	0.20	0.0324	0.1594	0.04	Sub-critical	1,2
Block E (7 Units - Townhouse)	MH9A	107.79	MH8A	107.51	0.0000	0.2172	29	33	0.2482	4.35	0.11	-	0.2482	0.071	0.53	30.2	203	200	0.9	32.86	1.02	0.18	1.61	0.0324	0.1594	0.15	Sub-critical	1,2
	MH8A	107.46	MH4A	107.41	0.0000	0.0000	0	33	0.2482	4.35	0.11	-	0.2482	0.071	0.53	5.3	203	200	0.9	33.15	1.02	0.18	1.59	0.0324	0.1594	0.14	Sub-critical	1,2
Block D & C (7 & 1 Units - Townhouse)	MH4A	107.36	MH3A	106.96	0.0000	0.2496	34	123	0.9267	4.22	0.39	-	0.9267	0.265	1.92	42.9	203	200	0.9	32.95	1.02	0.42	5.82	0.0324	0.1594	0.34	Sub-critical	1,2
Block A & B (12 & 11 Units - Townhouse)	MH13A	107.85	MH3A	106.96	0.0000	0.4184	57	57	0.4184	4.30	0.18	-	0.4184	0.120	0.90	58.8	203	200	1.5	41.99	1.30	0.22	2.14	0.0324	0.1594	0.18	Sub-critical	1,2
Block C (5 Units - Townhouse)	MH3A	106.91	MH2A	106.61	0.0000	0.1586	21	201	1.5037	4.15	0.64	-	1.5037	0.430	3.08	31.2	203	200	0.9	33.24	1.03	0.57	9.28	0.0324	0.1594	0.46	Sub-critical	1,2
	MH2A	106.56	MH1A	106.31	0.0000	0.0000	0	201	1.5037	4.15	0.64	-	1.5037	0.430	3.08	12.5	203	200	2.0	48.26	1.49	0.45	6.39	0.0324	0.1594	0.36	Sub-critical	1,2

A_{site} = Site Area (ha)

P = Population Equivalent * A_{Site}

Q Total = (q_{dry} * M) + INF (L/s)

INF = (0.286 L/ha/s)* A_{Site} (L/s)

INF = Inflow/Infiltration Allowance

q_{dry}= (0.003183 L/capita/sec) * P

$Area_{SD}$ = Semi-detached site area (ha)

M = K_{av} * (1 + (14 /(4 + (Pe^0.5)))) = Ratio of Peak Flow to Average Flow

The minimum permissible peaking factor is M = 2.0

Pe = equivalent tributary population in thousands

K_{av} = (A_R+ (0.8 * (A_I + A_C)))/(A_R + A_I + A_C) = 1.0 (Residential Land Use Only)

A_R = Residential Land Use Area (ha) = 1.4940ha; A_I = Industrial Land Use Area (ha) = 0ha; A_C = Commercial Land Use Area (ha) = 0ha

$Area_{TH}$ = Townhouse site area (ha)

h_m = A/T = hydraulic mean depth (m)

A = cross-section area (m^2)

T = diameter of conduit (m)

Fr = $v/(g\ h_m)^{1/2}$

Fr = Froude's number

v = flow velocity (m/s)

g = acceleration of gravity (9.81 m/s^2)

NOTE: 1) The site area is 1.5037ha and the GFA of all combined proposed buildings is 124,889.62 m^2

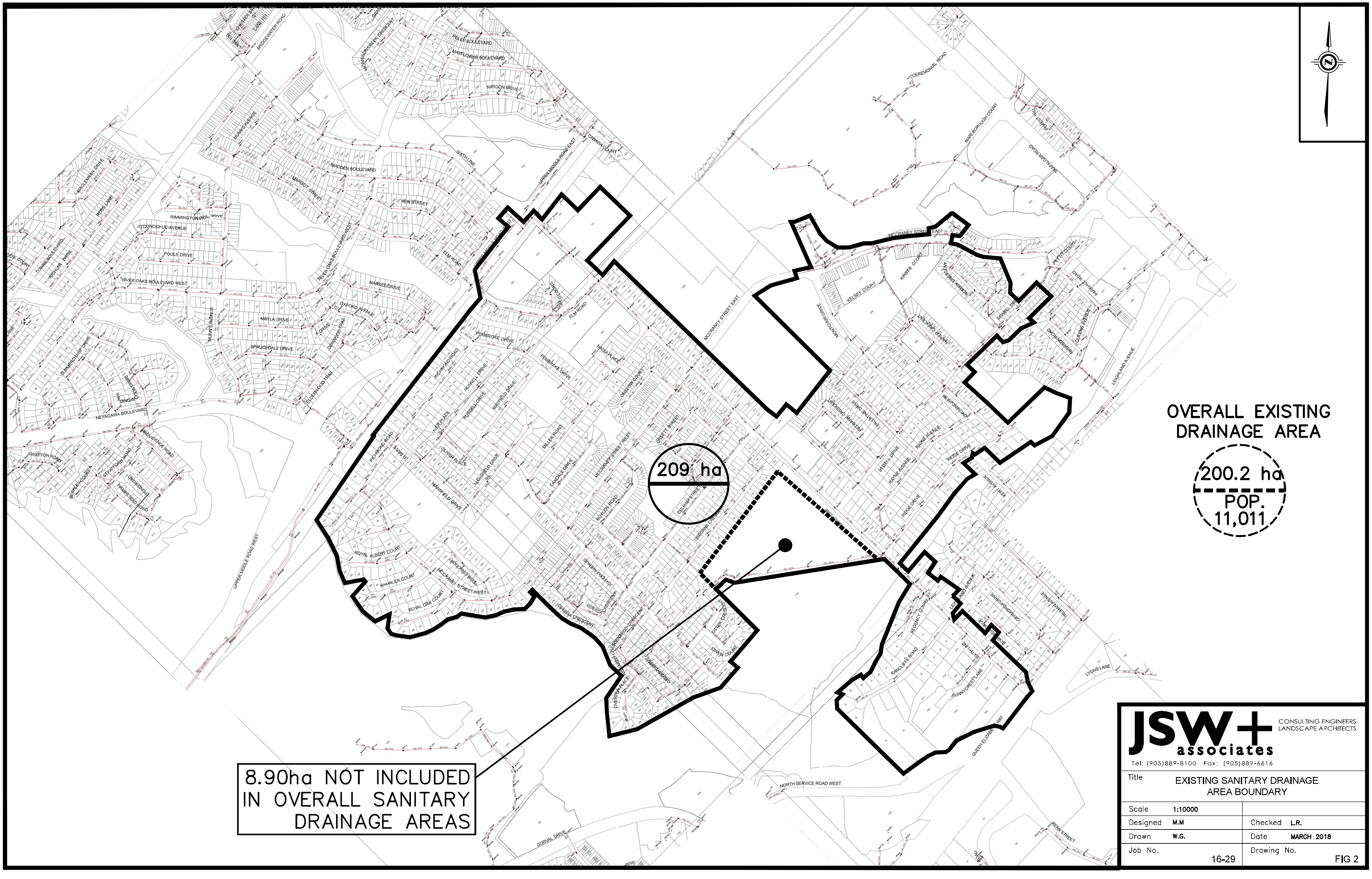
NOTE: 2) When Fr =1 flow is critical, Fr < 1 flow is subcritical, Fr > 1 flow is supercritical

1020, 1024, 1028, 1032, 1042 Sixth Line Road

Residential Development

Designed: **S.S.** Job No: 16-29

Checked: **M.M.** Date: 4/13/2023 Sheet : 1 of 1


OVERALL EXISTING
DRAINAGE AREA
200.2 ha
POP.
11,011
209 ha
8.90ha NOT INCLUDED
IN OVERALL SANITARY
DRAINAGE AREAS
JSW+
associates
CONSULTING ENGINEERS
LANDSCAPE ARCHITECTS
Tel: (905)889-8100 Fax: (905)889-6616
Title
EXISTING SANITARY DRAINAGE
AREA BOUNDARY
Scale 1:10000
Designed M.M
Checked L.R.
Drawn W.G.
Date MARCH 2018
Job No. 16-29
Drawing No. FIG 2

JSW+ associates
CONSULTING ENGINEERS
LANDSCAPE ARCHITECTS
Tel: (905)889-8100 Fax: (905)889-6616

Title	EXISTING SANITARY DRAINAGE AREA BOUNDARY (AERIAL)		
Scale	1:10000		
Designed	M.M	Checked	L.R.
Drawn	W.G.	Date	MARCH 2018
Job No.	16-29	Drawing No.	FIG 3

APPENDIX ‘B’:

FIRE DEMAND CALCULATIONS

HYDRANT FLOW TEST

HYDRANT FLOW TEST ANALYSIS

Fire Demand Calculations Based on Fire Underwriters Survey

1. *Fire Flow Estimate for a Given Area*

Table 1 – Fire Flow Estimate for a Given Area

Proposed Development Block	Construction Coefficient [C] [1]	Total GFA (m^2) [2, 3]	*RFF* (L/min) [4]	*RFF* (L/min) [5]
Block ‘A’	1.5	2,141.50	15,271.19	**16,000**
Block ‘B’	1.5	1,965.03	14,628.46	**15,000**
Block ‘E’	1.5	1,756.36	13,829.95	**14,000**

Note:

[1] *Since the building is constructed of wood frame construction and combustible material, the ‘C’ coefficient is to be selected as 1.5 (Fire Underwriters Survey [2020], Page 20).*

[2] *The total floor area was taken from the architectural site plans supplied by Dunpar. The fire flow calculations considered ALL above ground floors because the total floor area is to be taken as the largest floor area that would be affected by the design fire (in square meters). Additionally, for a building classified with the ‘C’ coefficient between 1.0 and 1.5, 100% of all floor areas are considered in determining the Total Effective Area to be used in the formula (Fire Underwriters Survey [2020], Page 22).*

[3] *Total floor area is not including the underground basements, since the area is at least 50% below grade and is not considered within the fire flow estimate (Fire Underwriters Survey [2020], Page 23).*

[4] $\left[RFF = 220C\sqrt{A}\right]$ *(Fire Underwriters Survey [2020], Page 19)*

[5] *The fire flow calculated must be rounded to the nearest 1000 L/min (Fire Underwriters Survey [2020], Page 19)*

2. *Occupancy Surcharge or Reduction*

Table 2 – Fire Flow Occupancy Reduction

Proposed Development Block	*RFF* (L/min) [5]	Occupancy Reduction Factor [6]	*RFF* (L/min) [6]
Block ‘A’	16,000	0.85	**13,600**
Block ‘B’	15,000	0.85	**12,750**
Block ‘E’	14,000	0.85	**11,900**

Note:

[6] *Since the building possesses a low contents fire hazard, a reduction of -15% (0.85) can be applied to the estimated fire flow (Fire Underwriters Survey [2020], Page 24-25).*

3. *Sprinkler Protection*

Note:

[7] The building will not contain a sprinkler system conformed to NFPA 13, therefore no reduction is granted to the Fire Flow from Part 2. Occupancy Surcharge or Reduction (Fire Underwriters Survey [2020], Page 27).

4. *Separation Exposure*

Table 3 – Separation Exposure Summary

Direction	Block 'A'		Block 'B'		Block 'E'	
	Separation	Charge	Separation	Charge	Separation	Charge
West	10.1 to 20m	15%	10.1 to 20m	15%	3.1 to 10m	20%
East	>30m	0%	>30m	0%	>30m	0%
North	>30m	0%	3.1 to 10m	20%	10.1 to 20m	15%
South	3.1 to 10m	20%	10.1 to 20m	15%	>30m	0%
Total		**35%**		**50%**		**35%**

Table 4 – Separation Exposure Charge

Proposed Development Block	*RFF* (L/min) [6]	Separation Exposure Reduction Factor [8]	*RFF* (L/min) [8]
Block 'A'	13,600	0.35	**4,760**
Block 'B'	12,750	0.50	**6,375**
Block 'E'	11,900	0.35	**4,165**

Note:

[8] The separation charge is applied the Fire Flow from Part 2. Occupancy Surcharge or Reduction (Fire Underwriters Survey [2020], Page 30).

5. *Adjusted Fire Flow*

Table 5 – Adjusted Fire Flow

Proposed Development Block	*RFF* (L/min) [6]	*RFF* (L/min) [7]	*RFF* (L/min) [8]	*RFF* (L/min) [9]	*RFF* (L/min) [5]
Block 'A'	13,600	0	4,760	18,360	19,000
Block 'B'	**12,750**	**0**	**6,375**	**19,125**	**20,000**
Block 'E'	11,900	0	4,165	16,065	17,000

Note:

[9] $\left[RFF_{Adjusted} = RFF_6 - RFF_7 + RFF_8\right]$ *(Fire Underwriters Survey [2020], Page 19)*

corix® Water Services

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SITE NAME: JSW + Associates, 16-29 DATE: 11.03.2016

LOCATION: 1020 Sixth Line, Oakville

TEST DATA TIME OF TEST: 8:45am

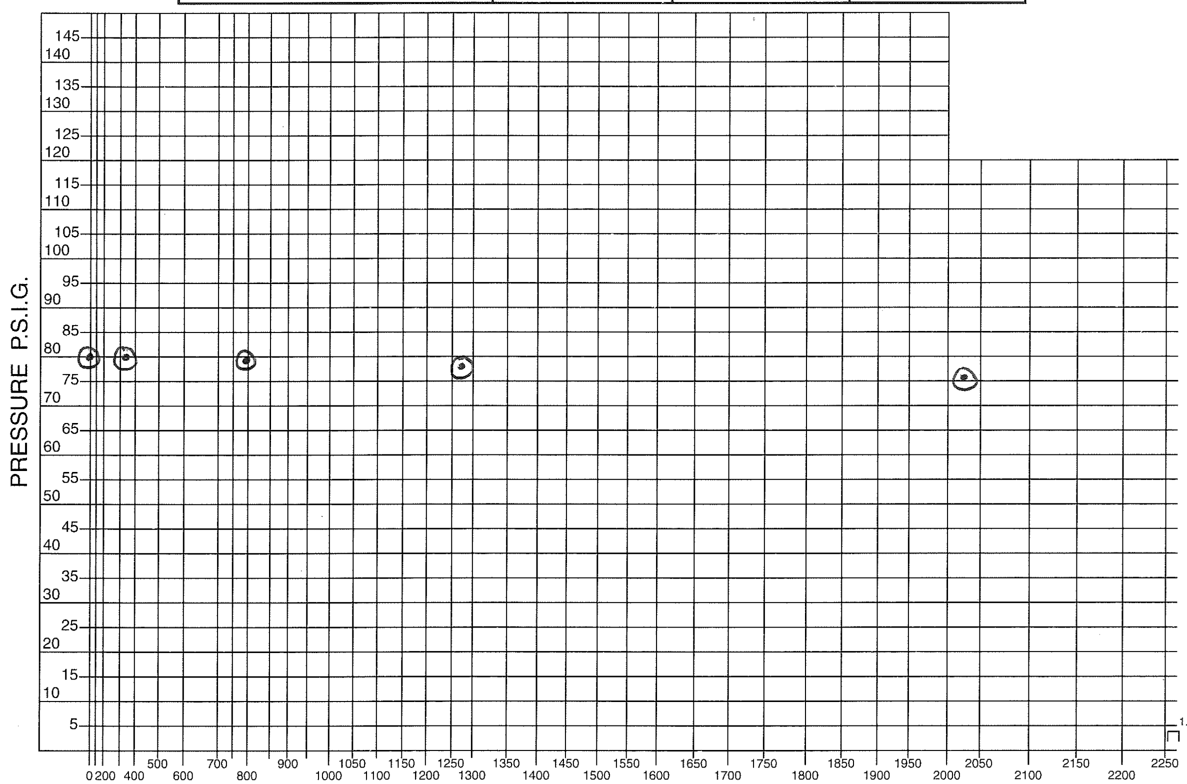
LOCATION OF TEST: (FLOW) 1020 Sixth Line, C.V. 3port

(RESIDUAL) C.V. hydrant @ Sunnycrest Lane

MAIN SIZE: 300mm

STATIC PRESSURE: 80psi

	NUMBER OF OUTLETS & ORIFICE SIZE	PITOT PRESSURE	FLOW (U.S. G.P.M.)	RESIDUAL PRESSURE
# 1	1 x 1 1/8	82	339	80
# 2	1 x 1 3/4	76	792	79
# 3	1 x 2 1/2	58	1274	78
# 4	2 x 2 1/2	37	2036	76



COMMENTS: Performed one (1) complete NFPA 291 flow test.

Authorized Signature ____________ Corix Water Services Signature J. Cowan

MAIN SIZE: 300 mm

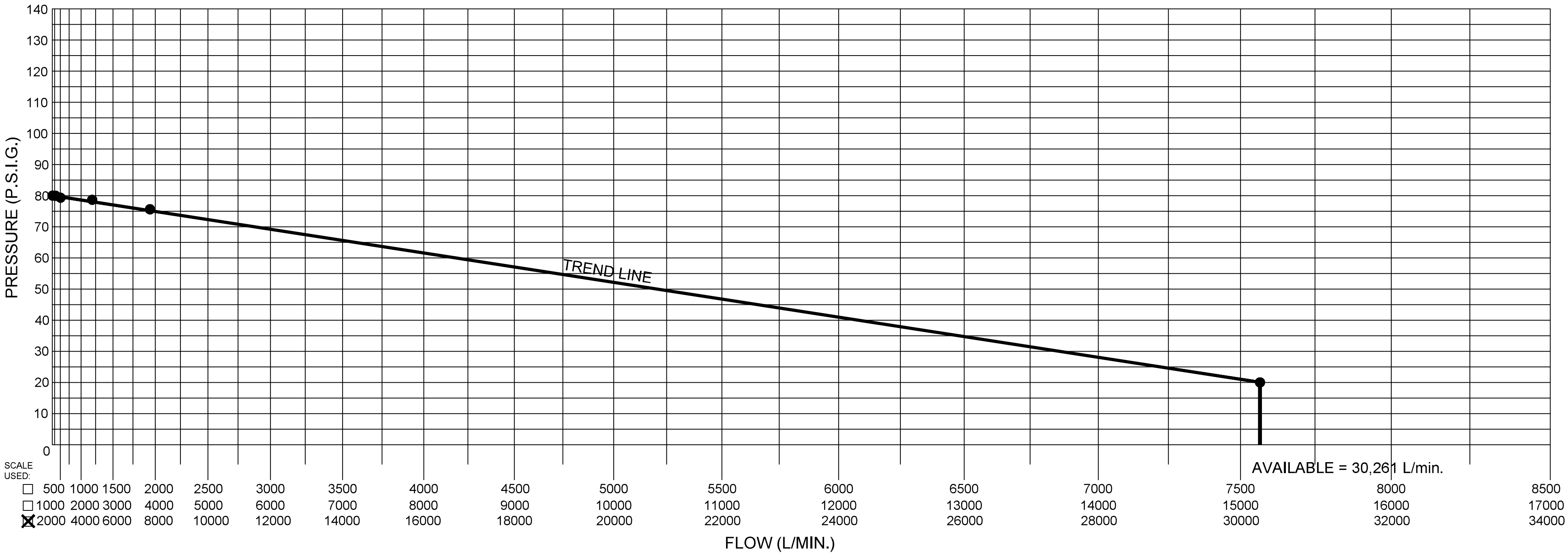
STATIC / BASELINE PRESSURE: 80 P.S.I. / 20 P.S.I.

Hr (STATIC - BASELINE): (80 - 20) = 60 P.S.I.

Hf (STATIC - RESIDUAL): (80 - 78) = 2 P.S.I.

	NUMBER OF OUTLETS & ORIFICE SIZE	PITOT PRESSURE (P.S.I.)	Q_F (FLOW [U.S. G.P.M.])	Q_F (FLOW [L/MIN.])[2]	RESIDUAL PRESSURE (P.S.I.)
#1	1 x 1 $\frac{1}{8}$	82	339	1,011	80
#2	1 x 1 $\frac{3}{4}$	76	792	2,381	79
#3	1 x 2 $\frac{1}{2}$	58	1,274	3,467	78
#4	2 x 2 $\frac{1}{2}$	37	2,036	6,461	76
#5			Q_R = 7,995 [1]	*Q_R = 30,261 [1]	20

[1] Q_R = RATED FLOW CAPACITY AT 20 P.S.I. = Q_F x (Hr / Hf)^0.54
[2] 1 U.S.G.P.M. = 3.785 L/MIN.



NOTE: THE TREND LINE IS A LINE OF BEST FIT BASED ON THE DATA FROM THE HYDRANT FLOW TEST.

JOB NUMBER: 16-29

FLOW TEST DATE: NOVEMBER 3rd, 2016

JOB LOCATION: 1020 SIXTH LINE, OAKVILLE, ON

PLOT DATE: APRIL 6th, 2023

FLOW: 1020 SIXTH LINE, C.V. 3-PORT

RESIDUAL PRESSURE: C.V. HYDRANT @ SUNNYCREST LANE

TEST LOCATION: NO. 1

JSW+ associates
CONSULTING ENGINEERS
LANDSCAPE ARCHITECTS
Tel: (905)889-8100 Fax: (905)889-6616

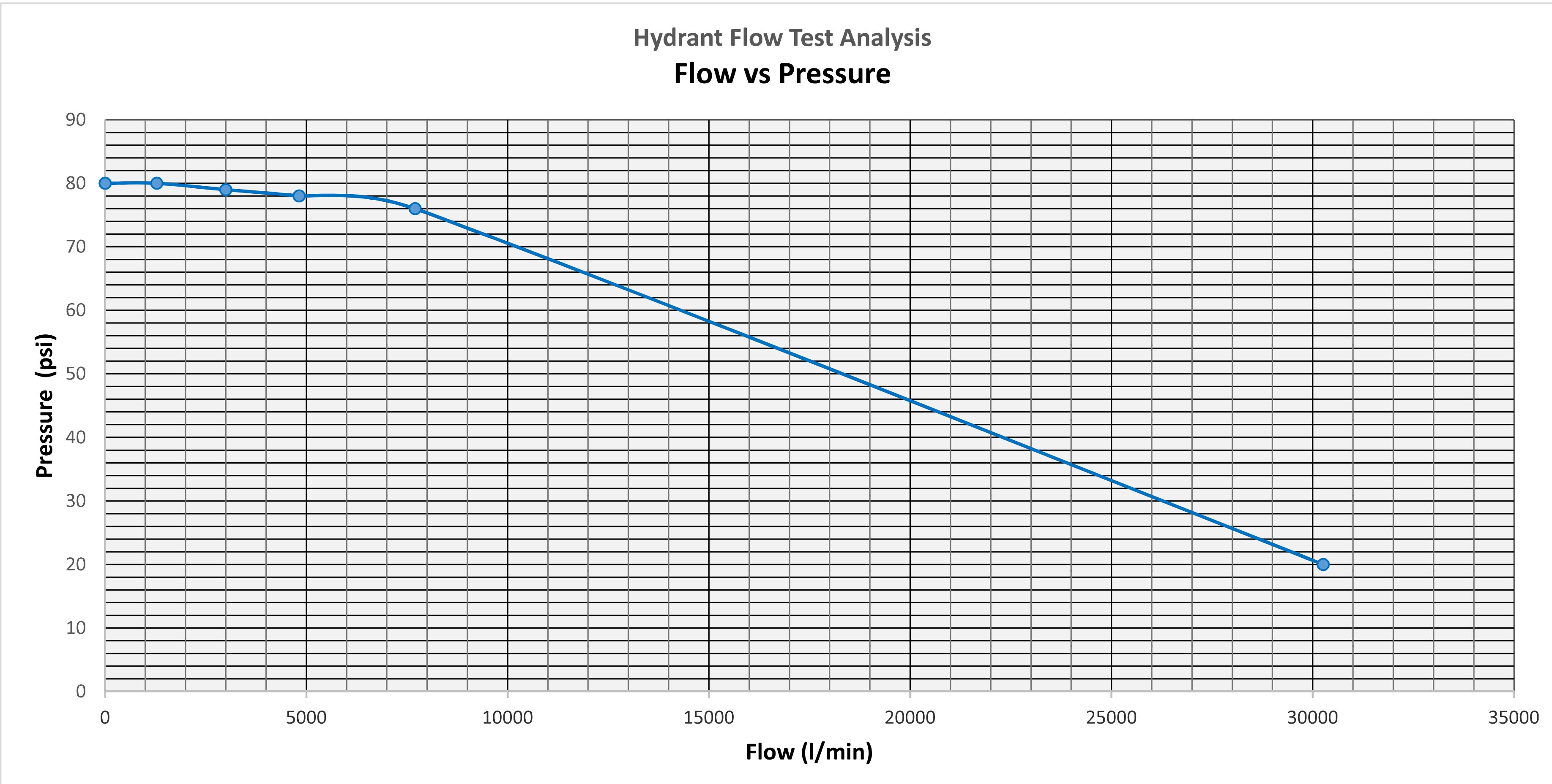
Sixth Line - Oakville
Hydrant Flow Test Analysis

Graph 1

Test Location No. 1

Pitot (psi)	Q_F (US GPM)	Residual (psi)
82	339	80
76	792	79
58	1274	78
37	2036	76

Static (psi)	Baseline (psi)	h_r (psi)	h_f (psi)	Q_R (US GPM)	Q_R (lpm)
80	20	60	2	7,995	30,261



APPENDIX ‘C’:

STORM DESIGN SHEET

STM-1 – POST-DEVELOPMENT STORM DRAINAGE AREA PLAN

FIG-4 – PRE-DEVELOPMENT HARD AND SOFT PLAN

FIG-5 – POST-DEVELOPMENT HARD AND SOFT PLAN

STORMWATER MANAGEMENT CALCULATIONS

GREENSTORM ST MODULAR DETENTION TANK DETAILS

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Storm Drainage Design Sheet

Street	Up Stream		Down Stream		No. of Hectares		Storm Co-efficient						Total A*C					5 yr	100 yr	100 - 5 yr	Cont.	Total	Pipe Data								
																I-5	I-100	Flow	Flow	Flow	Flow	Design Flow	L	Actual Size [2]	Nominal Size	Grade	Cap.	Vel. at Cap	Time	% Pipe	
	MH	Invert	MH	Invert	Contributing Area	Cum. Total	0.25	0.50	0.60	0.70	0.80	0.90	Contributing Area*C	Cum	T_c	(mm/hr)	(mm/hr)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m^3/s)	(m)	(mm)	(mm)	(%)	(m^3/s)	(m/s)	(min.)	Full	Remarks
	MH10	109.49	MH8	108.94	0.058	0.058	0.046				0.013		0.023	0.023	10.00	114.21	200.80	0.007	0.013	0.006		0.013	27.9	254	250	2.0	0.087	1.72	0.27	15	
	MH9	109.05	MH8	108.94	0.148	0.148	0.042		0.064		0.042		0.089	0.089	10.00	114.21	200.80	0.028	0.049	0.021		0.049	8.7	254	250	1.3	0.070	1.38	0.11	71	
	MH8	108.89	MH18	108.82	0.000	0.206							0.000	0.112	10.27	112.59	197.87	0.035	0.061	0.026		0.061	6.4	299	300	1.2	0.103	1.47	0.07	60	
	MH18	108.77	MH7	108.45	0.149	0.356				0.102	0.048		0.114	0.226	10.34	112.16	197.10	0.070	0.124	0.053		0.124	27.2	366	375	1.2	0.178	1.69	0.27	69	
	MH7	108.40	MH5	108.11	0.068	0.424					0.068		0.055	0.281	10.61	110.62	194.31	0.086	0.152	0.065		0.152	24.6	366	375	1.2	0.178	1.70	0.24	85	
	MH6	108.50	MH5	108.11	0.114	0.114	0.058	0.056					0.029	0.029	10.00	114.21	200.80	0.009	0.016	0.007		0.016	25.7	254	250	1.5	0.076	1.51	0.28	21	
	MH5	108.06	MH16	108.02	0.000	0.538							0.000	0.310	10.85	109.26	191.87	0.094	0.165	0.071		0.165	6.5	448	450	0.6	0.221	1.40	0.08	75	
	MH13	108.48	MH11	108.31	0.184	0.184	0.051	0.055		0.051	0.026		0.092	0.092	10.00	114.21	200.80	0.029	0.051	0.022		0.051	29.2	299	300	0.6	0.073	1.04	0.47	70	
	MH11	108.26	MH12	108.10	0.085	0.268					0.085		0.067	0.159	10.47	111.44	195.80	0.049	0.087	0.037		0.087	29.0	366	375	0.6	0.124	1.18	0.41	70	
	MH12	108.05	MH16	108.02	0.000	0.268							0.000	0.159	10.88	109.13	191.62	0.048	0.085	0.036		0.085	5.0	366	375	0.5	0.116	1.10	0.08	73	
	MH16	107.97	MH4	107.78	0.124	0.930					0.124		0.099	0.567	10.93	108.84	191.10	0.172	0.301	0.130		0.301	40.6	594	600	0.5	0.409	1.48	0.46	74	
	MH14	108.52	CB.MH8	108.41	0.009	0.009						0.009	0.008	0.008	10.00	114.21	200.80	0.002	0.004	0.002		0.004	8.4	254	250	1.3	0.071	1.40	0.10	6	
	CB.MH8	108.41	CB.MH7	108.25	0.026	0.035					0.018	0.009	0.008	0.016	10.10	113.61	199.71	0.005	0.009	0.004		0.009	12.0	254	250	1.3	0.072	1.41	0.14	12	
	CB.MH7	108.25	CB.MH6	108.08	0.049	0.085					0.024	0.025	0.023	0.038	10.24	112.76	198.18	0.012	0.021	0.009		0.021	13.3	254	250	1.3	0.070	1.38	0.16	30	
	CB.MH6	108.08	CB.MH5	107.91	0.049	0.133					0.024	0.024	0.022	0.060	10.40	111.82	196.48	0.019	0.033	0.014		0.033	12.8	254	250	1.3	0.071	1.41	0.15	46	
	CB.MH5	107.91	MH4	107.78	0.049	0.182					0.024	0.024	0.022	0.082	10.55	110.95	194.91	0.025	0.045	0.019		0.045	9.9	254	250	1.3	0.071	1.40	0.12	63	
	MH4	107.73	MH3	107.66	0.125	1.237				0.106	0.010	0.009	0.095	0.745	11.39	106.39	186.67	0.220	0.387	0.166		0.387	25.0	762	750	0.3	0.615	1.35	0.31	63	1
	MH15	107.95	MH3	107.66	0.061	0.061		0.021	0.040				0.028	0.028	10.00	114.21	200.80	0.009	0.016	0.007		0.016	43.9	254	250	0.7	0.050	1.00	0.74	31	
	MH3	107.61	Dispersion MH	107.57	0.014	1.312	0.014						0.004	0.777	11.70	104.80	183.81	0.226	0.397	0.171		0.397	12.5	762	750	0.3	0.615	1.35	0.15	65	1
	Dispersion MH	107.55	SWM TANK	107.54	0.000	1.312							0.000	0.259	11.85	104.02	182.42	0.075	0.131	0.056		0.131	1.2	366	375	0.8	0.150	1.43	0.01	88	
	Dispersion MH	107.55	SWM TANK	107.54	0.000	1.312							0.000	0.259	11.85	104.02	182.42	0.075	0.131	0.056		0.131	1.2	366	375	0.8	0.150	1.43	0.01	88	
	Dispersion MH	107.55	SWM TANK	107.54	0.000	1.312							0.000	0.259	11.85	104.02	182.42	0.075	0.131	0.056		0.131	1.2	366	375	0.8	0.150	1.43	0.01	88	

Q = 0.00278 x A x C x I
C = Runoff Coefficient
5 Year Rainfall Intensity = 1170 / ([t_d + 5.8] ^ 0.843)

A = Area (Hectares)
n = Pipe Roughness = 0.013
100 Year Rainfall Intensity = 1250 / ([t_d + 5.7] ^ 0.861) (in mm/hr)
$t_d = T_c$ (in min)

1020, 1024, 1028, 1032, 1042 Sixth Line Road
Residential Development

Designed:	S.S.	Job No:	16-29		
Checked:	M.M.	Date:	4/17/2023	Sheet:	1 of 2

Notes: 1 - Reinforced Concrete Pipe to be used. All other sewer legs are assumed to be PVC DR-35 Pipe unless otherwise noted.
2 - Actual diameters of PVC DR-35 Pipe taken from IPEX Municipal Servicing Pipes Catalogue.

JSW+ associates

Storm Drainage Design Sheet

Street	Up Stream MH	Up Stream Invert	Down Stream MH	Down Stream Invert	No. of Hectares Contributing Area	No. of Hectares Cum. Total	Storm Co-efficient 0.25	0.50	0.60	0.70	0.80	0.90	Total A*C Contributing Area*C	Total A*C Cum	T_c	I-5 (mm/hr)	I-100 (mm/hr)	5 yr Flow (m^3/s)	100 yr Flow (m^3/s)	100 - 5 yr Flow (m^3/s)	Cont. Flow (m^3/s)	Total Design Flow (m^3/s)	Pipe Data L (m)	Actual Size [2] (mm)	Nominal Size (mm)	Grade (%)	Cap. (m^3/s)	Vel. at Cap (m/s)	Time (min.)	% Pipe Full	Remarks
	CB.MH2	109.10	CB.MH3	108.71	0.017	0.017		0.017					0.008	0.008	10.00	114.21	200.80	0.003	0.005	0.002		0.005	20.0	254	250	2.0	0.087	1.71	0.19	5	
	CB.MH3	108.71	CB.MH4	107.96	0.018	0.035			0.018				0.011	0.019	10.19	113.04	198.68	0.006	0.011	0.005		0.011	41.5	254	250	1.8	0.083	1.65	0.42	13	
	CB.MH4	107.96	MH17	107.63	0.008	0.043			0.008				0.005	0.025	10.62	110.59	194.27	0.008	0.013	0.006		0.013	17.4	254	250	1.9	0.085	1.69	0.17	16	
	MH17	107.58	SWM TANK	107.54	0.000	0.043							0.000	0.025	10.79	109.63	192.52	0.008	0.013	0.006		0.013	2.0	254	250	2.0	0.088	1.73	0.02	15	
	Control 100 year Flow to 126.04 L/s																														
	SWM TANK	107.54	Bypass MH1	107.53	0.101	1.457	0.101						0.026	0.828	11.87	103.96	182.29	0.239	0.420	0.180	0.126	0.126	1.8	201	200	0.7	0.027	0.86	0.04	464	
	Bypass MH1	107.48	Filtration Unit	107.46	0.000	1.457							0.000	0.828	11.90	103.78	181.98	0.239	0.419	0.180	0.126	0.126	3.0	366	375	0.7	0.134	1.28	0.04	94	
	Filtration Unit	107.31	MH2	107.28	0.000	1.457							0.000	0.828	11.94	103.59	181.63	0.238	0.418	0.180	0.126	0.126	3.2	366	375	0.9	0.159	1.51	0.04	79	
	Bypass MH1	107.63	MH2	107.34	0.000	1.457							0.000	0.828	10.00	114.21	200.80	0.263	0.462	0.199	0.126	0.126	5.4	366	375	5.4	0.381	3.62	0.02	33	
	MH2	107.23	Control MH1	107.20	0.000	1.457							0.000	0.828	11.98	103.41	181.32	0.238	0.417	0.179	0.126	0.126	4.6	366	375	0.7	0.133	1.26	0.06	95	
	Control MH1	107.15	MH1	106.90	0.000	1.457							0.000	0.828	12.04	103.12	180.79	0.237	0.416	0.179	0.126	0.126	12.2	366	375	2.0	0.235	2.24	0.09	54	

$Q = 0.00278 \times A \times C \times I$
C = Runoff Coefficient
5 Year Rainfall Intensity = 1170 / ([t_d + 5.8] ^ 0.843)

A = Area (Hectares)
n = Pipe Roughness = 0.013
100 Year Rainfall Intensity = 1250 / ([t_d + 5.7] ^ 0.861) (in mm/hr)
$t_d = T_c$ (in min)

1020, 1024, 1028, 1032, 1042 Sixth Line Road
Residential Development

Designed:	**S.S.**	Job No:	16-29		
Checked:	**M.M.**	Date:	4/11/2023	Sheet:	2 of 2

Notes: 1 - Reinforced Concrete Pipe to be used. Actual Sizes taken from Con Cast Concrete Pipe (Perfect Pipe). All other sewer legs are assumed to be PVC DR-35 Pipe unless otherwise noted.
2 - Actual diameters of PVC DR-35 Pipe taken from IPEX Municipal Servicing Pipes Catalogue.