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PROPOSED RESIDENTIAL
APARTMENT BUILDING DEVELOPMENT
276, 284 & 292 PLAINS ROAD EAST
CITY OF BURLINGTON

PROJECT No.: 20223

FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT

Prepared For:

BROOKLYN CONTRACTING

Prepared By:

The Odan/Detech Group Inc.

Original:	December 23rd, 2020
Revised:	April 16th, 2021
Revised:	April 22nd, 2021
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TABLE OF CONTENTS

DESCRIPTION page

1.0 INTRODUCTION....3
2.0 SCOPE OF WORK....3
3.0 WATER DISTRIBUTION ASSESSMENT....4
i) Existing Infrastructure....4
ii) Proposed Servicing....4
4.0 SANITARY SEWERS....8
i) Existing Infrastructure & Drainage....8
ii) Proposed Servicing....8
iii) Receiving Downstream Sanitary Sewer Capacity....10
5.0 STORM WATER MANAGEMENT....14
i) Existing Infrastructure & Drainage....14
ii) Stormwater Quantity Control Criteria....16
iii) Allowable Discharge Flow Rate....17
iv) Post Development Flow Analysis....17
v) Water Budget/Stormwater Retention....22
vi) Water Quality....24
6.0 GROUNDWATER....27
i) Introduction....27
ii) Long-Term Groundwater....27
iii) Short-Term (Construction) Groundwater....27
7.0 CONCLUSIONS....28
8.0 REFERENCES....29

LIST OF FIGURES

Figure 1 - Visual OTTHYMO Model (Showing Flows in 100-Year Storm) 21
Figure 2 - Excerpt from CETV Verification Statement for Stormceptor OGS 25
Figure 3 - Excerpt from CETV Verification Statement for Jellyfish Filter 26

LIST OF TABLES

TABLE 1 – Total Water Demand for Proposed Building 4
TABLE 2 – Post-Development Sanitary Flow 8
TABLE 3 – Allowable Flows (Pre-Development 5-year storm runoff) 17
TABLE 4 - Catchment Characteristics for the site Post-Development 19
TABLE 5 – Stage/Storage/Discharge Relationship for Stormwater Detention & Pumped Stormwater Detention Tank 21
TABLE 6 – Stormwater Management Summary 22
TABLE 7 - Summary 28

APPENDIX A

Existing Site	Aerial view of Site and surrounding area
Site Plan & Statistics	by Reinders + Law

APPENDIX B

Visual OTTHYMO Model Output (2-Year & 100-Year storms)
CETV Verification Statement – Imbrium Systems Inc. Stormceptor OGS
CETV Verification Statement – Imbrium Systems Inc. Jellyfish Filter

APPENDIX C

Email Correspondence from Regional Engineering Staff regarding Watermains and hydrant tests thereof

APPENDIX D

Functional Servicing Plan
Functional Grading Plan

1.0 INTRODUCTION

The subject of this report is a 0.51 Ha (1.25 acres) parcel of land bound by the following:

- To the east: an existing house of commercial use
- To the south: detached houses
- To the west: an existing house
- To the north: Plains Road East

Presently the site comprises two houses, one of which has been converted to commercial use (284 & 292 Plains Rd. E.), as well as a one-storey commercial building located at 276 Plains Rd. E.

Whereas the initial iterations of this report reflected that the Site comprised only 284 & 292 Plains Rd. E., the Site now also comprises 276 Plains Rd. E.

Refer to the Key Plan in Appendix A for the site's layout and adjacent properties.

It is proposed to construct a 6-Storey (plus mechanical penthouse) residential tower. The building will be constructed ontop of a one-storey below-grade parking structure.

Refer to the architectural site plan in Appendix A for the proposed development's layout.

The municipal addresses of the site comprise 276, 284 & 292 Plains Road East.

For detailed topography of the existing site conditions, as of February 2018, refer to the topographic survey prepared by David B. Searles Surveying Ltd.

This report evaluates the serviceability of the site with respect to sanitary waste water, water and storm water management (SWM) and will implement the City of Burlington's SWM requirements.

2.0 SCOPE OF WORK

THE ODAN/DETECH GROUP INC. was retained by **Brooklyn Contracting** to review the Site, collect data, evaluate the Site for the proposed use and present the findings in a Functional Servicing and Storm Water Management Report in support of a Rezoning application. The scope of work in brief involves the following:

a) Collecting existing servicing drawings from the CITY in order to establish availability and feasibility of Site servicing;
b) Meetings/conversations with CITY Engineers and Design Team.
c) Evaluation of the data and presentation of the findings in a Functional Servicing and Storm Water Management Report in support of the Rezoning application.

3.0 WATER DISTRIBUTION ASSESSMENT

i) Existing Infrastructure

The following watermains presently exist beneath the streets bordering the site. Refer to the Functional Servicing Plan.

- Plains Road East, south side: 400mm watermain
- Plains Road East, north side: 300mm watermain
- Plains Road East, beneath road centreline: 600mm watermain

ii) Proposed Servicing

The proposed building is proposed to be serviced by a 200mm fire service connection from the 400mm watermain beneath the south side of Plans Road with branch 150mm domestic water service.

The 600mm watermain is a trunk watermain and it is assumed it would thus not be permitted to connect-to.

It was feasible to conduct a hydrant flow test on the 300mm watermain on the north side of the street (provided below) – but not the 400mm watermain within the south side of Plains Rd., due to unavailability of hydrants. Regional engineering staff confirmed that conclusions may be drawn regarding system adequacy from the test on the 300mm watermain, nonetheless, as per the email correspondence provided here in Appendix C.

The building will be less than 85m in height, therefore OBC 2006 3.2.9.7 (4) does not require a second fire service

Refer to the Functional Servicing Plan for proposed service connections.

The water demand for the proposed building is as follows.

a) Average Day domestic demand - As per average sanitary flow 1.1 L/sec

b) Peak day demand - 2.25 x average demand 2.5 L/sec

c) Fire flow as per FUS 1999 manual calculation, below 133 L/sec

TABLE 1 – Total Water Demand for Proposed Building

	L/sec	USGM
Peak Day Demand	2.5	40
Fire Flow Demand	133	2113
Total Water Demand	136	2153
Available Flow at 20 PSI (Plains Rd. E. 300mm WM)	201	3182

Hydrant flow test results as stated above are provided below.

The following assumptions are made in the following Fire Underwriters' Survey fire flow calculation.

- The proposed building is of fire-resistive construction (reinforced concrete)
- The building will be sprinklered for fire protection and the sprinklers will be fully monitored according to NFPA 13
- The building's contents (residences) will be non-combustible in nature
- The building's areas in the calculation are as per the architectural floor areas provided in Appendix A

The hydrant flow test was conducted as per the NFPA 291 hydrant testing standard on a hydrant adjacent to 305 Plains Rd. E. Refer to the test report, below.

The available flow at 20 psi in the Plains Rd. E. 300mm watermain is 3182 USGM which is greater than the proposed development's total water demand (2153 USGM), therefore the existing watermain infrastructure is sufficient to service the proposed development and no watermain infrastructure upgrades are required.

WATER SUPPLY FOR PUBLIC FIRE PROTECTION , FIRE UNDERWRITERS SURVEY GUIDE
FOR DETERMINATION OF REQUIRED FIRE FLOWS

$F = 220 \times C \times \sqrt{A}$

Where:

F = required fire flow in liters per minute
C= Coefficient related to the type of construction
A = the total floor area in square meters (excluding basements) in the building considered

Coefficient related to type of construction	
1.5	Wood Frame
1	Ordinary
0.8	Non combustible
0.6	Fire Resistive

LOCATION: 276-292 Plains Rd. E., Burlington

OBC OCCUPANCY: Residential Apartment

BUILDING FOOT PRINT (m2): 1590

OF STOREYS: 6

PROJECT: 284-292 Plains Rd. E., Burlington

PROJECT No: 20223

Contents	Charge
Non-Combustible	-25%
limited Combustible	-15%
Combustible	0%
Free Burning	15%
Rapid Buring	25%

CONSTRUCTION CLASS: Fire Resistive

AUTOMATED SPRINKLER PROTECTION

		Credit	Total
NFPA 13 sprinkler standard	yes	10%	
Standard Water Supply	yes	30%	50%
Fully Supervised System	yes	10%	
		50%	

CONTENTS FACTOR: Non Combustible — CHARGE: -20%

EXPOSURE 1 (south) Ex. House	Distance to Exposure Building (m) / Length - Height	22	10%
EXPOSURE 2 (east) Ex. House	Distance to Exposure Building (m) / Length - Height	21	20%
EXPOSURE 3 (west) Ex House	Distance to Exposure Building (m) / Length - Height	16	15%
EXPOSURE 4 (north) Ex. Church	Distance to Exposure Building (m) / Length - Height	>45	0%
		Total:	45%

no more than 75%

Separation	Charge
0-3 m	25%
3.1 -10 m	20%
10.1 - 20 m	15%
20.1 - 30 m	10%
30.1 - 45	5%
> 45 m	0%

ARE BUILDINGS CONTIGUOUS: NO

FIRE RESISTANT BUILDING — Are vertical openings and exterior vertical communications protected with a minimum one (1) hr rating? No

CALCULATIONS

	C =	0.6		
	A =	7568	m2	(2 largest floors + 50% floors above)
	F =	11483	L/min	
Round to Nearest 1000 L/min	F =	11000	L/min	must be > 2000 L/min

STOREY AREAS m2	
1549	L1
2044	L2
2044	L3
1987	L4
1987	L5
1931	L6

CORRECTION FACTORS:

OCCUPANCY	-2200	L/min
FIRE FLOW ADJUSTED FOR OCCUPANCY	8800	L/min
REDUCTION FOR SPRINKLER	-4400	L/min
EXPOSURE CHARGE	3960	L/min

REQUIRED FIRE FLOW F = 8360 L/min

Round to Nearest 1000 L/min

F =	8000	L/min	2113 usgm
F =	133	L/sec	

FLOWMETRIX
INDU-TECH
PROCESS
WESTCAN

Fire Flow Testing Report

Residual Hydrant # **HY212735**
NFPA Colour Code **BLUE**

DATE	April 20, 2021
TIME	9:50 AM
ADDRESS	305 Plains Rd E
	Burlington, ON
	L7T 2C7
SIZE-inches/mm	12 300
MATERIAL	CI
CONTACT INFO	Brooklyn Contracting Inc.
	Marko A. Juricic
	P : 289-337-2707 ext: 210
	E : marko@brooklyncontract.com

RESIDUAL HYDRANT INFO.

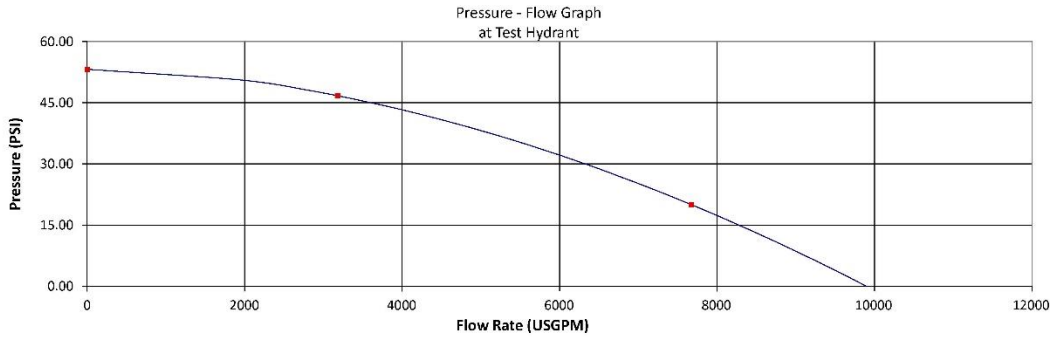
HYDRANT #	HY212735	
N.F.P.A. COLOUR CODE	BLUE	
STATIC PRESSURE	53.2	psi
RESIDUAL PRESSURE	46.7	psi
PRESSURE DROP	6.5	psi
% PRESSURE DROP	12.2	% psi

Flow on Water Main At Test Hydrant - 20 psi 7674 USGPM

FLOW HYDRANT(S) INFO.

HYDRANT ASSET ID	HYD. # PORTS	OUTLET DIAMETER (INCHES)	NOZZLE COEFFICIENT	DIFFUSER TYPE	DIFFUSER COEFFICIENT	PITOT READING (psi)	PITOT FLOW (USGPM)	FLOW METER (USGPM)
HY5748	2	2.5	Round	Swivel	1.00	**22.5**	1591	0
		2.5	Round	Swivel	1.00	**22.5**	1591	0
					Total Flow (USGPM)		3182	0
					Total Flow (USGPM)		3182	

FIRE FLOW CHART



COMMENTS

OPERATOR	FMX	Jordan Whitlock
OPERATOR	FMX	Denis Kriventsev
OPERATOR		Halton Region

Copy of Copy of Fire Flow Testing Report - Plains Rd

"If we don't measure it, how do you manage it?"

4.0 SANITARY SEWERS

i) Existing Infrastructure & Drainage

The following sanitary sewers are located within the streets bordering the subject site. Refer to the Functional Servicing Plan, for the layout of the existing sanitary sewers adjacent to the subject site.

- Plains Road East, south side: there is a 250mm sanitary sewer which commences near the site's northeast corner and flows westerly before continuing southerly beneath Shadeland Avenue to the west.

- Plains Road East, north side: there is a 250mm sanitary sewer flowing easterly.

ii) Proposed Servicing

It is proposed to service the proposed building using a proposed 200mm sanitary service connection to the 250mm sanitary sewer beneath Plains Rd. E.

Refer to the Functional Servicing Plan for the proposed sanitary service connection. Sanitary flows are calculated as follows.

The sanitary sewer design criteria and unit flow is provided in the Regional Municipality of Halton's *Water and Wastewater Linear Design Manual* (April 2019), as follows. The following information is provided in Tables 3-1 and 3-2 of the foregoing manual.

- Unit flow: q = average daily residential per capita dry weather unit flow = 0.275 m^3/cap/day
- I/I = Unit of peak inflow/infiltration = 0. 286 L/s/ha
- Apartment (over 6-storey): 285 p/Ha and 0.275 m^3/p/day or 0.003183 x 10^{-3} m^3/p/s
- Apartment (6-storeys or less): 135 p/Ha and 0.275 m^3/p/day or 0.003183 x 10^{-3} m^3/p/s
- Notwithstanding the above unit population, however, a unit population of 2.7 P/unit is assumed for the proposed apartment tower development because the Region standard 285 P/Ha unit population (apartments <6-storeys) would result in a unit population of approximately 0.3 Person/unit for the foregoing proposed statistics, which is unrealistic

The proposed sanitary flows are as follows. Refer to the detailed calculation on the following pages.

TABLE 2 – Post-Development Sanitary Flow

Component	Population (P)	Sanitary Flow (l/s)	Groundwater Discharge (l/s)	Inflow & Infiltration (l/s)	Total Flow (l/s)
Residential Flows	381	4.26	N/A	0.14	4.4

The 4.4 L/s total sanitary flow for the proposed building will be conveyed to the existing 250mm sanitary sewer beneath Plains Rd. E. by a proposed 200mm at 2.0% (43 L/s capacity) sewer connection. This sewer connection is adequately sized to convey the above flow. This connection is deliberately oversized anticipating that the mechanical engineer's OBC criteria for the mechanical pipe leaving the building will require a larger-size pipe than municipal criteria would require.

SANITARY & WATER FLOW CALCULATIONS

SCENARIO: PROPOSED DEVELOPMENT

This program calculates the sanitary discharge from various land use

FILL IN COLOURED CELLS AS REQUIRED

COMMERCIAL SITE AREA (ha) =

RESIDENTIAL SITE AREA (ha) = 0.504

TOTAL SITE AREA (ha) = 0.504

NOTE:

LAND USE	NUMBER OF UNITS	SITE AREA, (ha)	GROSS FLOOR AREA, m2	TOTAL POPULATION	TOTAL DAILY FLOW (LITERS)	AVERAGE DAILY FLOW l/sec	PEAKING FACTOR, M	TOTAL FLOW FROM LAND USE, l/sec
RESIDENTIAL Detached, using 55 person/site area				0	0	0.00	4.50	0.00
RESIDENTIAL Semi Houses, using 100 persons/site area				0	0	0.00	4.50	0.00
RESIDENTIAL Apartments (<6 st), using 135 persons/site area				0	0	0.00	4.50	0.00
RESIDENTIAL Apartments (>6 st), using 285 persons/site area				0	0	0.00	4.50	0.00
RESIDENTIAL Density 3, using 2.7 persons/unit	141			381	91368	1.06	4.03	4.26
COMMERCIAL, Using 90 persons/ha (Floor Ha)				0	0	0.00	3.60	0.00
COMMERCIAL, Using 0.60 L/sec per ha				0	0	0.00	2.50	0.00

TOTAL

V1= 91368

Q1=	4.26
Q2=	0.00
Qinfil	0.14
Qtot	4.41

Q = (MqP/86400) + A * I (L/sec)

Q1= total flow from Residential Land Use (L/sec)
Q2= total flow from Commercial Land Use (L/sec)
Qinfil = total flow from infiltration (L/sec)
Qtot = total flow (Land use + infiltration)

V1= Total Volume from Land Use in liters

where : P is population
q = 0.275 m3/d/p = 0.004 L/sec/person for residential and
q = 0.60 L/sec/ha for commercial and offices
A = gross site area
i = 0.286 L/sec/ha (infiltration rate)

Peaking Factor M = 1 + [14 / (4 + (P/1000,1/2)) (for residential)
Peaking Factor M = 0.8* {1 + [14 / (4 + (P/1000,1/2))]} (for Commercial)

iii) Receiving Downstream Sanitary Sewer Capacity

The proposed development is proposed to discharge into the existing 250mm sanitary sewer flowing westerly beneath Plains Rd. E. as discussed above. Consideration is required for this sewer's capacity to receive the proposed flows.

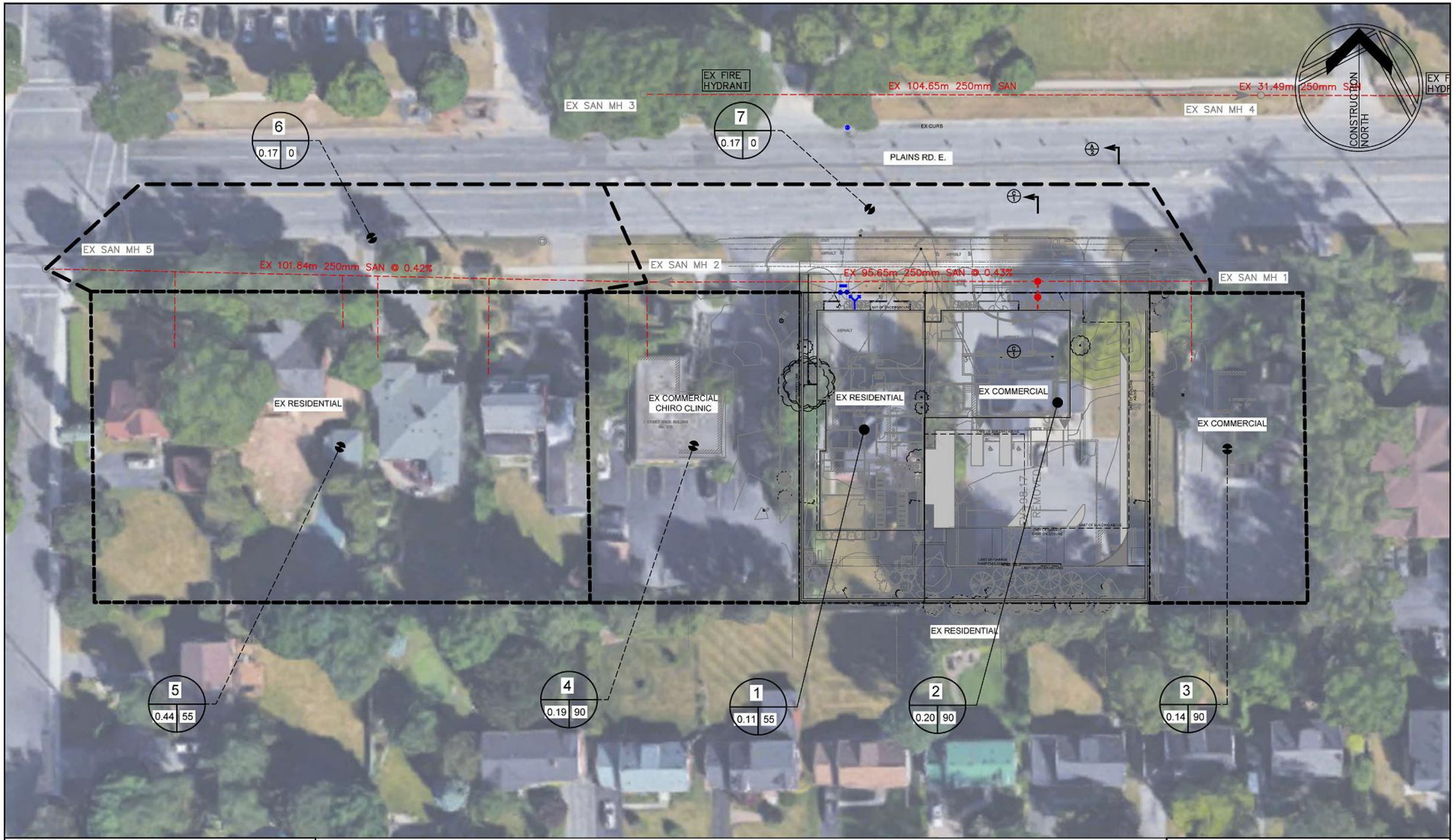
Region engineering staff have stated in the *Halton Region Servicing Comments* section of the preconsultation checklist dated March 10, 2021 that the FSR should incorporate a hydraulic analysis of the receiving wastewater mains. That memo states that this analysis should be based on either the Region's model or an original analysis prepared by the consultant in reference to the Region's latest Linear Infrastructure Design Manual. An original analysis is therefore provided herein in reference to the latest Linear Design Manual.

The analysis is provided on the following pages. The analysis comprises a catchment plan identifying the tributary catchment areas and statistics, on the following page, and two conventional sewer analysis sheets: pre-development and post-development.

This analysis has considered two downstream segments, until the outlet to a 450mm sanitary sewer flowing southerly beneath Shadeland Ave. to the west.

The following discussion and conclusions are drawn from the analysis on the following pages.

- In pre-development conditions the downstream pipes are flowing within capacity (less than 4% of the pipes' capacity)
- In post-development conditions, with the additional flows from the subject Development, the downstream pipes are flowing within capacity, up to 19% of the pipes' capacity
- Given that the downstream pipes are flowing well within their respective capacities – no downstream sanitary sewer system upgrades are necessary to accommodate the proposed development



LEGEND

CATCHMENT BOUNDARY – EXISTING BUILDING

TOTAL TRIBUTARY AREA (HA) — 1 / 0.03 50 — TRIBUTARY AREA ID / POPULATION DENSITY (P/Ha)

FIGURE 4

DATE:	PROJ. NO.:	SCALE:
APRIL 2020	20223	1 : 600

DRAWING : DOWNSTREAM SANITARY SEWER CATCHMENT PLAN

PROJECT : 284–292 PLAINS RD. E. PROPOSED RESIDENTIAL DEVELOPMENT

ODAN·DETECH CONSULTING ENGINEERS
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5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

SANITARY SEWER DESIGN SHEET - Existing Plains Rd. E. Sewer - Pre-Development Scenario

Site location: 284-292 Plains Road East, Burlington

Ref# PN 20223

				Inlet Flow																		Pipe						
Location				Industrial/Commercial					Residential Population				Inflow/ Infiltration	K(av) Factor	Industrial & Commercial P/F	Peak Industrial & Commercial Flow	Residential P/F	Peak Residential Sanitary Flow	Unit Inflow/ Infiltration	Segment Inflow/ Infiltration	Accumulative Inflow/ Infiltration	Accumulative Sanitary Flow	Length	Size	Slope	Shape	Full Flow Capacity	Full Flow Velocity	% Full
	Street Name	US Node	DS Node	Industrial (ha)	Commercial (ha)	Acc've Area (ha)	Population (Person)	Acc've Pop'n (Person)	Apartments (Ha)	SFD (Ha)	Population (Person)	Acc've Pop'n (Person)	Area (ha)	Kav	M	Q(p) (L/s)	M	Q(p) (275 L/c/d) (L/s)	I/I (0.28 L/Sec/ha)	Q(i) (L/s)	Q(i) (L/s)	Q(d) (L/s)	L (m)	D (mm)	S (%)		Qcap (L/s)	V (m/s)	Q(d)/Qcap
1 & 2 & 3	Plains Rd. E.	EX SAN MH 1	EX SAN MH 2	-	0.340	0.340	30.60	30.60	-	0.110	6.05	6.05	0.62	0.85	3.70	0.36	3.76	0.07	0.280	0.17	0.17	0.61	95.65	250	0.43	circle	39.00	0.79	1.55%
4 & 5	Plains Rd. E.	EX SAN MH 2	EX SAN MH 5	-	0.190	0.53	17.10	47.70	-	0.440	24.20	30.25	0.80	0.90	3.89	0.59	3.93	0.38	0.280	0.22	0.40	1.37	101.84	250	0.42	circle	38.54	0.79	3.55%

Flow Calculation Criteria

(Unit Flowfrom Table 3-1 and 3-2, Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2019)

q = average daily residential per capita dry weather unit flow = 0.275 m3/cap/d

q = average daily commercial dry weather unit flow = 24.750m3/ha/d or 0.28646 L/ha/s

I/I = Unit of peak inflow/infiltration allowance = 0.286 (L/s/ha)

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

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

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

Mannings Equation:

Qcap=(D/1000)^2.667*(S/100)^0.5/(3.211*n)*1000(L/s)

D: pipe size (mm)

S: slope (grade) of pipe (%)

n = Manning roughness coefficient = 0.013

(Unit Population from Table 3-1 and 3-2, Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2019)

Single Family Population Density	55 P/ha
Semi-detached/duplex/4-plex	100 P/ha
Townhouse, Maisonette	135 P/ha
Apartment (Over 6 Stories High)	285 P/ha
Commercial	90 P/ha

(Peaking Factor from Section 3.2.3., Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2019)

PEAKING FACTOR (All Land Uses)	M =Kav*[1 + 14/(4+(P/1000^0.5))]
K(av) factor	Kav=(Ar + 0.80x(Ai+Ac))/(Ar+Ai+Ac)
PEAK DESIGN FLOW,	Q(d) = Q(p) + Q(i) L / Sec.
PIPE ROUGHGNESS,	n = 0.013 For Manning's Equation

Vmin. = 0.6m/s and Vmax. = 3m/s

SANITARY SEWER DESIGN SHEET - Existing Plains Rd. E. Sewer - Post-Development Scenario

Site location: 284-292 Plains Road East, Burlington

Ref# PN 20223

Location				Inlet Flow																	Pipe						
				Industrial/Commercial					Residential Population				Inflow/ Infiltration	K(av) Factor	Industrial & Commercial P/F	Peak Industrial & Commercial Flow	Residential P/F	Peak Residential Sanitary Flow	Unit Inflow/ Infiltration	Segment Inflow/ Infiltration	Accumulative Inflow/ Infiltration	Pumped Groundwater/ Foundation Drains	Accumulative Sanitary Flow	Length	Size	Slope	Shape	Full Flow Capacity	Full Flow Velocity	% Full
	Street Name	US Node	DS Node	Industrial (ha)	Commercial (ha)	Acc've Area (ha)	Population (Person)	Acc've Pop'n (Person)	Apartments (Ha)	SFD (Ha)	Population (Person)	Acc've Pop'n (Person)	Area (ha)	Kav	M	Q(p) (L/s)	M	Q(p) (275 L/c/d) (L/s)	I/I (0.28 L/Sec/ha)	Q(i) (L/s)	Q(i) (L/s)	Q (gw) (L/s)	Q(d) (L/s)	L (m)	D (mm)	S (%)		Qcap (L/s)	V (m/s)	Q(d)/Qcap
1 & 2 & 3	Plains Rd. E.	EX SAN MH 1	EX SAN MH 2	-	0.140	0.140	12.60	12.60	0.504	-	381.00	381.00	0.62	0.96	4.21	0.17	3.86	4.68	0.280	0.17	0.17	2.00	7.02	95.65	250	0.43	circle	39.00	0.79	18.00%
4 & 5	Plains Rd. E.	EX SAN MH 2	EX SAN MH 5	-	0.190	0.33	17.10	29.70	-	0.440	24.20	405.20	0.80	0.92	4.01	0.38	3.70	4.77	0.280	0.22	0.40	-	7.55	101.84	250	0.42	circle	38.54	0.79	19.59%

Flow Calculation Criteria
(Unit Flow from Table 3-1 and 3-2, Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2019)
q = average daily residential per capita dry weather unit flow = 0.275 m3/cap/d
q = average daily commercial dry weather unit flow = 24.750m3/ha/d or 0.28646 L/ha/s
I/I = Unit of peak inflow/infiltration allowance = 0.286 (L/s/ha)
Q(p) = peak population flow (L/s)
Q(I) = peak extraneous flow (L/s)
Q(d) = peak design flow (L/s)

Mannings Equation:

Qcap=(D/1000)^2.667*(S/100)^0.5/(3.211*n)*1000(L/s)
D: pipe size (mm)
S: slope (grade) of pipe (%)
n = Manning roughness coefficient = 0.013

(Unit Population from Table 3-1 and 3-2, Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2019)

Single Family Population Density	55 P/ha
Semi-detached/duplex/4-plex	100 P/ha
Townhouse, Maisonette	135 P/ha
Apartment (Over 6 Stories High)	285 P/ha
Apartment (per unit)	2.7 P/unit
Commercial	90 P/ha

(Peaking Factor from Section 3.2.3., Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2019)

PEAKING FACTOR (All Land Uses)	M =Kav*[1 + 14/(4+(P/1000^0.5))]
K(av) factor	Kav=(Ar + 0.80x(Ai+Ac))/(Ar+Ai+Ac)
PEAK DESIGN FLOW,	Q(d) = Q(p) + Q(i) L / Sec.
PIPE ROUGHGNESS,	n = 0.013 For Manning's Equation

Vmin. = 0.6m/s and Vmax. = 3m/s

5.0 STORM WATER MANAGEMENT

i) Existing Infrastructure & Drainage

The following storm sewers presently existing beneath the streets adjacent to the site

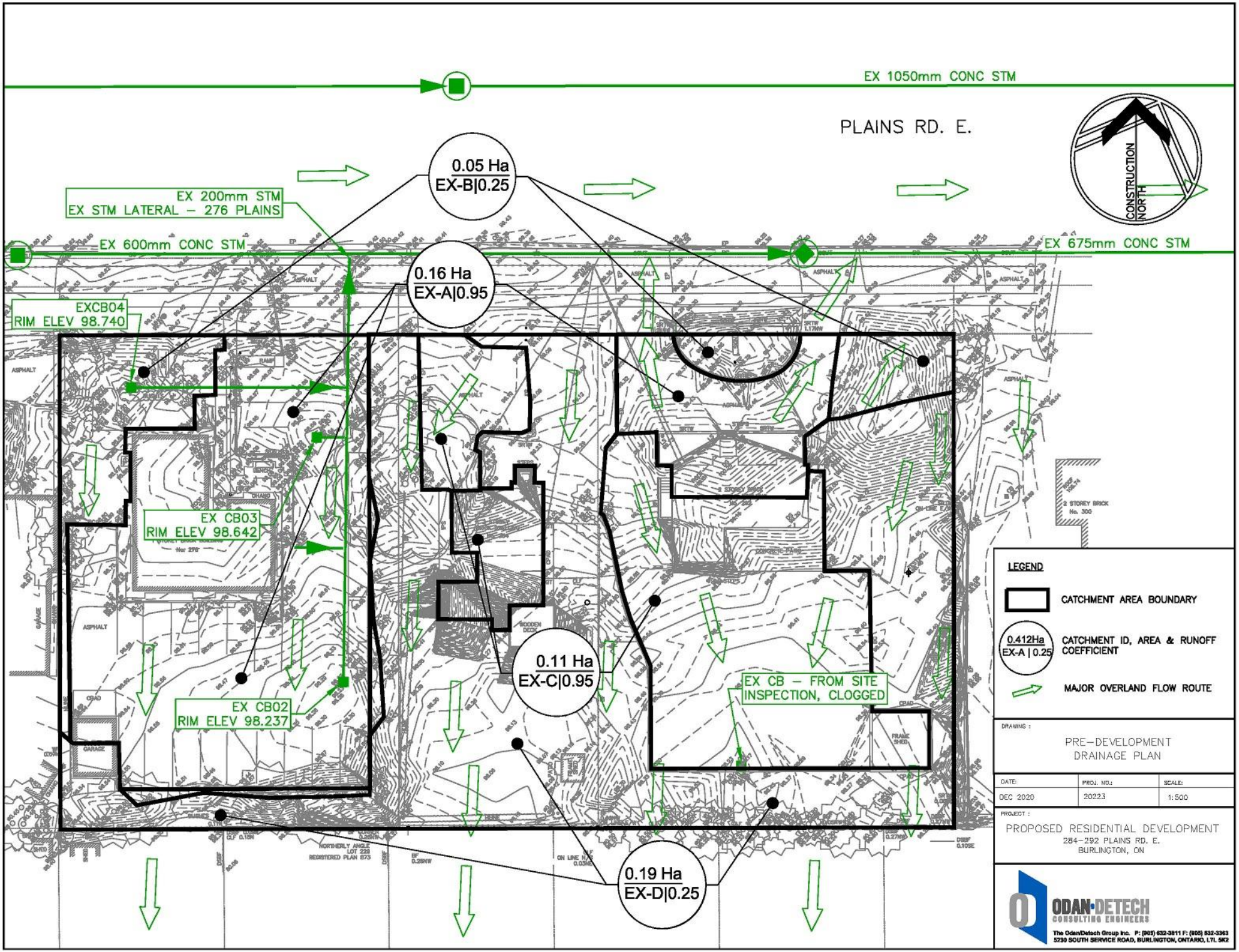
- Plains Road East, south side: there is an existing 600mm storm sewer flowing easterly, which continues southerly beneath Filmandale Road to the east.

- Plains Road East, north side: there is an existing 1050mm storm sewer flowing easterly which continues southerly beneath Filmandale Road to the east, where there is a confluence with the above sewer.

Refer to the Pre-Development Drainage Plan on the following page for existing drainage patterns and outlets.

Presently the north-east portion of 284 & 292 Plains Road, as well as the majority of 276 Plains Road (Catchment Areas EX-A and EX-B) drains towards Plains Road, where it then drains to the existing 600mm storm sewer. 284 & 292 Plains Road drain by overland sheet flow towards Plains road, whereas 276 Plains Road drains via three existing catchbasins and a storm sewer connection.

The portion of the existing site which does not drain to Plains Road – Catchment Areas EX-C and EX-D – drains southerly offsite towards the adjacent private properties to the south. A portion of the rear yard of 294 Plains drains towards an existing catchbasin, however inspection of this catchbasin showed that it's outlet is to the south and it is presently clogged with debris and is therefore not providing drainage outlet, therefore stormwater from this area drained southerly.


EX 1050mm CONC STM
PLAINS RD. E.
CONSTRUCTION NORTH
0.05 Ha
EX-B|0.25
EX 200mm STM
EX STM LATERAL – 276 PLAINS
EX 600mm CONC STM
EX 675mm CONC STM
0.16 Ha
EX-A|0.95
EXCB04
RIM ELEV 98.740
EX CB03
RIM ELEV 98.642
EX CB02
RIM ELEV 98.237
0.11 Ha
EX-C|0.95
EX CB – FROM SITE
INSPECTION, CLOGGED
0.19 Ha
EX-D|0.25
2 STOREY BRICK
No. 300
LEGEND
CATCHMENT AREA BOUNDARY
0.412Ha
EX-A | 0.25
CATCHMENT ID, AREA & RUNOFF COEFFICIENT
MAJOR OVERLAND FLOW ROUTE
DRAWING :
PRE–DEVELOPMENT
DRAINAGE PLAN
DATE:
DEC 2020
PROJ. NO.:
20223
SCALE:
1:500
PROJECT :
PROPOSED RESIDENTIAL DEVELOPMENT
284–292 PLAINS RD. E.
BURLINGTON, ON
ODAN·DETECH
CONSULTING ENGINEERS
The Odan/Detech Group Inc. P: (905) 632-3811 F: (905) 632-3363
5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

ii) Stormwater Quantity Control Criteria

Storm water management criteria is given in the City of Burlington's manual *Stormwater Management Design Guidelines* (May 21, 2020). Section 6.1 therein provides site-level stormwater management criteria, which applies to site developments such as this.

Section 6.1.1.1 *Design Criteria* states that where an 'end-of-pipe' stormwater management facility is not present and where a site discharges to a storm sewer, the criteria is to control the 100-year return period post-development storm to the allotted capacity of the storm sewer. The manual further states that where the capacity of the storm sewer is unknown, that the 100-year post-development storm should be controlled to the 5-year pre-development storm flow.

The above criteria (control 100-year post-development to 5-year pre-development flows) applies to this development, given that it is proposed to discharge to a storm sewer – the Plains Road East 600mm storm sewer - and that the capacity of this storm sewer is unknown.

City of Burlington IDF curves are provided in Appendix B of the above manual. They are provided as follows and will be used in the below Visual OTTHYMO hydraulic model.

IDF-Curve	***Intensity when t, time of concentration, is 10 minutes:***
$I_2 = 681.52 / (6 + t)^{0.780}$	$I_2 = 78.4$ mm/hr
$I_5 = 802.04 / (5 + t)^{0.764}$	$I_5 = 101.3$ mm/hr
$I_{10} = 918.28 / (5 + t)^{0.763}$	$I_{10} = 116.3$ mm/hr
$I_{25} = 1065.95 / (5 + t)^{0.762}$	$I_{25} = 135.4$ mm/hr
$I_{50} = 1172.34 / (5 + t)^{0.761}$	$I_{50} = 149.3$ mm/hr
$I_{100} = 1281.34 / (5 + t)^{0.761}$	$I_{100} = 163.2$ mm/hr

iii) Allowable Discharge Flow Rate

Allowable discharge flow rate is calculated to be 29.6 L/s as follows based on the above criteria, however there will be two uncontrolled catchment areas which drain directly to the Plains Road Right-of-Way (Catchment Areas D and E). The 100-year runoff flows from these catchment areas have been deducted from the allowable release rate, giving allowable release rate of ***23.4 L/s***, as follows.

TABLE 3 – Allowable Flows (Pre-Development 5-year storm runoff)

Location	Run-off Coefficient	Rainfall Intensity (mm/hr)	Catchment Area	Site Allowable (L/s)
Pre-Development 5-Year Storm Runoff to Plains Rd. E. 600mm Storm Sewer Catchment EX-A & EX-B	0.5 *(Actual Composite C=0.78)*	101.3	0.05 Ha + 0.16 Ha = 0.21 Ha	***29.6***

Allowable Discharge Flow Rate, Q_{All}, less uncontrolled catchment areas (Catchment Areas D and E):

$$Q_{All} = 29.6\ L/s - Q_{100\text{-}Y\text{-}D} - Q_{100\text{-}Y\text{-}E}$$

$$Q_{All} = 29.6\ L/s - 2.78*0.25*163.2mm/hr*0.017Ha - 2.78*0.95*163.2mm/hr*0.010Ha$$

$$Q_{All} = 29.6\ L/s - 1.9\ L/s - 4.3\ L/s$$

$$\boldsymbol{Q_{All} = 23.4\ L/s}$$

Pre-Development 5-Year Storm Runoff to Adjacent Properties to South Catchment EX-C, EX-D	0.5 *(Actual Composite C=0.51)*	101.3	0.11 Ha + 0.19 Ha = 0.30 Ha	***42.2***

iv) Post Development Flow Analysis

The proposed development will control the post development flows to the allowable flow rate calculated above; on-site storage will be required.

The adjacent properties have self-contained storm drainage and runoff from the adjacent properties do not enter the subject development based on pre-development drainage patterns. Refer to the Pre-Development Drainage Plan for pre-development drainage patterns.

- ***To the West:*** There is an existing detached house which appears to drain by overland flow to the south and does not drain into the subject site.
- ***To the East:*** There an existing house/commercial building which appears to drain to internal catchbasins and does not drain into the subject site
- ***To the North:*** There is Plains Road East which drains to the above-described existing storm sewers.
- ***To the South:*** There are existing detached houses and the topography is away from the subject site to the south.

The proposed development is almost entirely underlain by the proposed below-grade parking structure from property-line to property-line, therefore all private storm conveyances will be by mechanical storm piping. Storm area drains on the roofs and ground-levels of the development will drain uncontrolled by mechanical piping to a stormwater detention tank to be constructed in the below-grade parking levels, beneath the parking ramp. The stormwater detention tank will be of cast-in-place concrete construction. Refer to the Functional Servicing Plan for the tank's location.

The tank's bottom elevation is below the elevation of the downstream receiving storm sewer within Plains Rd. E., therefore the tank will be pumped into the storm sewer connection. The pump will provide stormwater quantity controls, pumping at no-more than the allowable release-rate of 23.4 L/s (above).

The 100-year stormwater detention tank will be sized, based on the following SWM quantity control analysis, to provide adequate storage to the 100-year post-development storm event below-grade – such that there is no surface ponding.

Foundation drainage water will be pumped into the storm control maintenance hole (on a long-term basis) at a maximum flow rate of 2.0 L/s. The site's controlled stormwater release rate is revised accordingly such that the total discharge from the site remains less than allowable, despite the foundation drainage flows. This is considered in Table 6, below, where it is shown that the controlled storm flows are no greater than allowable, considering the foundation drain flows.

Note that the site is graded such that the site's major overland flow route is directed towards Plains Rd. E. via the site's driveway apron. This requires a retaining wall at the site's rear, such that the grade at the site's rear (south) side is marginally higher than the grade at Plains Rd. E., with a minimum freeboard of 0.30m. This is in accordance with Section 5.1.1 *Site Grading* of the City of Burlington's manual which states that sites should be graded to direct runoff to an acceptable receiver. Given that the neighbouring private properties to the south are not an acceptable receiver, the site is graded to direct the major overland flow route to Plains Rd. E.

There are existing trees on or near the site's south, west and east boundaries with required Tree Protection Zones (TPZ) around them. It is not permitted to alter the grading/topography within or at the boundary of the TPZ's. Refer to the Grading Plan. As a result, the landscaped areas at the south, west and east sides of the site, which are largely in TPZ areas, must drain out of the site, as existing, following existing topography. It is not possible to install drainage measures in these areas because the topography is lower than that of the driveway areas of the site and because of the TPZ's. The allowable release rate criteria is nonetheless satisfied, because the pre-development 5-year storm flow rate from these areas (36.6 L/s – Table 3), is greater than the post-development 100-year storm flow rate from these areas (Catchment D) (8 L/s – Table 6).

Visual OTTHYMO 2.3.2. will be used to model and determine the detention volume required. For drainage areas with significant imperviousness the calculation of effective rainfall in Visual OTTHYMO is accomplished using the "Standhyd" method. This method is used in urban watersheds to simulate runoff by combining two parallel standard unit hydrographs resulting from the effective rainfall intensity over the pervious and impervious surfaces. For pervious surfaces, losses are calculated using the SCS modified CN method.

The following table summarizes the parameters used in Visual OTTHYMO to characterize the post development catchment areas. Refer to the Post-Development Visual OTTHYMO Model in Appendix B. The catchment areas appear in the Post-Development Catchment Plan, below.

TABLE 4 - Catchment Characteristics for the site Post-Development

Area I.D.	Area (ha)	Hydrograph Method	% impervious	imperviousness directly connected %	Loss Method for Pervious Area	CN for Pervious Area	Initial Abstraction for Pervious (mm)	Time to peak (T_p)
Catchment A: Building Roof Drains to Tank	0.216	StandHyd	99	99	SCS	80	1	-
Catchment B: Impervious paved surfaces & Landscape on-top of parking structure	0.176	StandHyd	99	99	SCS	80	1	-
Catchment C: Pervious Surfaces Drains Offsite to East	0.014	StandHyd	25	25	SCS	80	5	-
Catchment D: Pervious Surfaces Drains Offsite to Plains Rd.	*100-Year runoff flow rate from this uncontrolled catchment area is deducted from allowable release rate per Table 3, therefore this catchment area is not considered in stormwater detention calculations.*							
Catchment E: Impervious Surfaces Drains Offsite to Plains Rd.	*100-Year runoff flow rate from this uncontrolled catchment area is deducted from allowable release rate per Table 3, therefore this catchment area is not considered in stormwater detention calculations.*							
Catchment F: Pervious Surfaces Drains Offsite to South	0.072	StandHyd	25	25	SCS	80	1	-

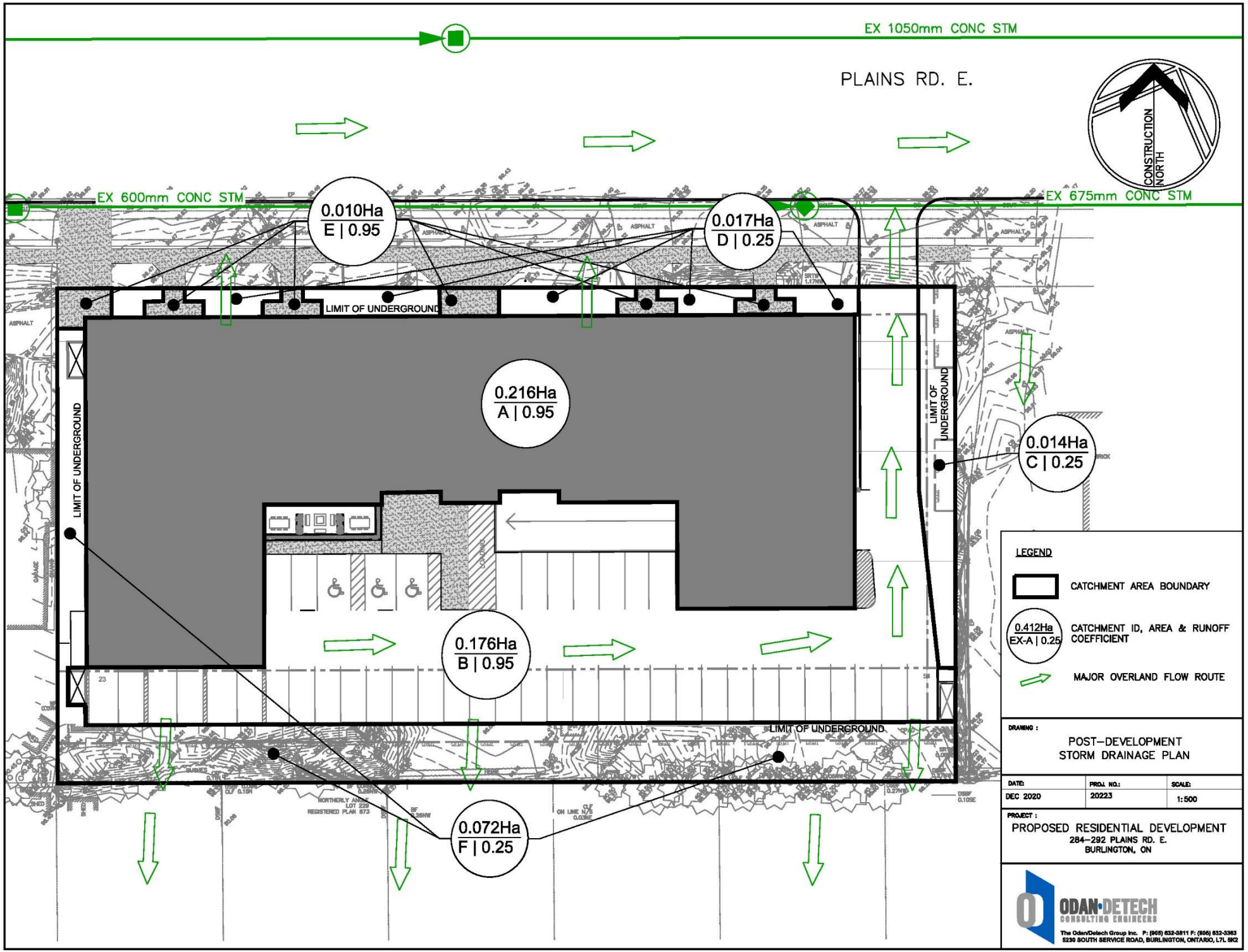
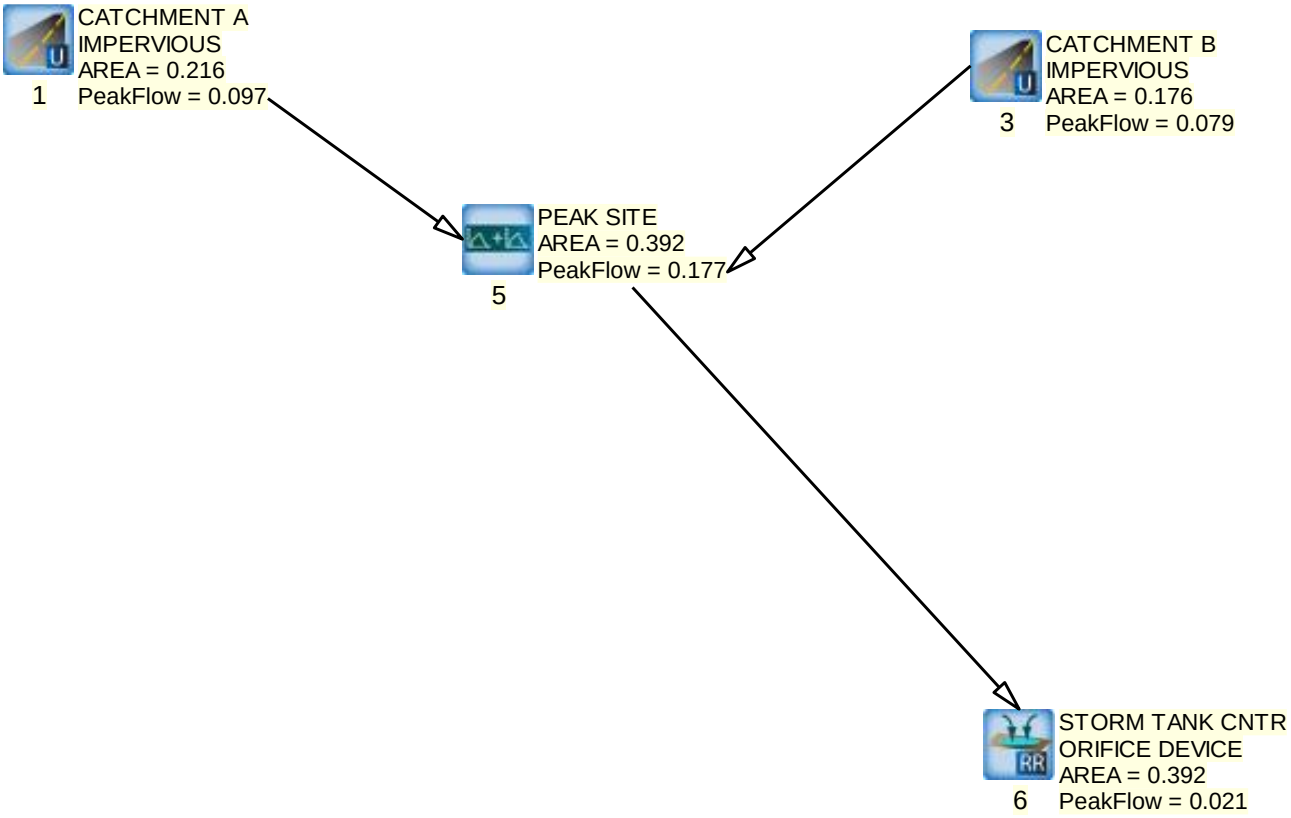

EX 1050mm CONC STM
PLAINS RD. E.
CONSTRUCTION NORTH
EX 600mm CONC STM
EX 675mm CONC STM
0.010Ha
E | 0.95
0.017Ha
D | 0.25
LIMIT OF UNDERGROUND
0.216Ha
A | 0.95
LIMIT OF UNDERGROUND
LIMIT OF UNDERGROUND
0.014Ha
C | 0.25
0.176Ha
B | 0.95
LIMIT OF UNDERGROUND
0.072Ha
F | 0.25
LEGEND
CATCHMENT AREA BOUNDARY
0.412Ha
EX-A | 0.25
CATCHMENT ID, AREA & RUNOFF COEFFICIENT
MAJOR OVERLAND FLOW ROUTE
DRAWING :
POST–DEVELOPMENT
STORM DRAINAGE PLAN
DATE:
DEC 2020
PROJ. NO.:
20223
SCALE:
1:500
PROJECT :
PROPOSED RESIDENTIAL DEVELOPMENT
284–292 PLAINS RD. E.
BURLINGTON, ON
ODAN·DETECH
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Figure 1 - Visual OTTHYMO Model (Showing Flows in 100-Year Storm)



The controlled discharge is thus provided as follows.

The stage-storage-discharge relationship for the proposed storm tank with Hydrovex Vortex 50SVHV-1 orifice control is as follows. The tank is a proposed 225m^2 footprint.

TABLE 5 – Stage/Storage/Discharge Relationship for Stormwater Detention & Pumped Stormwater Detention Tank

Head (m)	Elevation (m)	Provided Volume (m^3)	Discharge (l/s)	Description
0.00	93.85	0	0	Bottom of Tank
0.50	94.35	112.5	21.0	Tank
1.00	94.85	225	21.0	Tank
1.50	95.35	337.5	21.0	Tank
2.00	95.85	450	21.0	Tank
2.50	96.35	561.5	21.0	Tank
2.80	96.65	630	21.0	Tank Roof

Actual 100-year Discharge from Stormwater Tank Pump: 21.0 L/s in all scenarios (pumped)

The following is a summary of the Visual OTTHYMO results.

TABLE 6 – Stormwater Management Summary

	2 Yr. Storm	**5 Yr. Storm**	**10 Yr. Storm**	**25 Hr. Storm**	**50 Yr. Storm**	**100 Yr. Storm**
Total Flows Attenuated by Orifice Device	14 L/s	18 L/s	21 L/s	21 L/s	21 L/s	21 L/s
Max. Pumped Foundation Drain Flows	2 L/s	2 L/s	2 L/s	2 L/s	2 L/s	2 L/s
Total Flows	18 L/s	21 L/s	23 L/s	23 L/s	23 L/s	23 L/s
Allowable Flow Rate (to Plains Rd. – Table 3)			***23 L/s***			
Required 100-Year Volume	77 m^3	99 m^3	114 m^3	137 m^3	154 m^3	171 m^3
Provided Tank Volume			***675 m^3***			
Catchment C and F – Uncontrolled Flows offsite to adjacent private property	5 L/s	6 L/s	6 L/s	7 L/s	8 L/s	10 L/s
Allowable Flow Rate (to External Properties to South – Table 3)			***33 L/s***			

The proposed stormwater quantity control is such that the controlled discharge in a 100-year storm is equal to the allowable release rate. The stormwater storage is provided such that the volume provided is greater than the required 100-year storm volume. Stormwater quantity control criteria is therefore satisfied.

v) Water Budget/Stormwater Retention

The primary objective of the Water Balance/Budget Targets/Criteria is to capture and manage annual rainfall on the development site itself to preserve the pre-development hydrology (or “water balance”, which typically consists of three components: runoff, infiltration, and evapotranspiration) through a combination of infiltration, evapotranspiration, landscaping, rainwater reuse and/or other low impact development practices.

Storm water management water budget/retention criteria is given in the City of Burlington’s manual *Stormwater Management Design Guidelines* (May 21, 2020). Section 6.1.3 therein provides site-level stormwater management stormwater retention/water budget criteria, which would apply to this

development. That section states that proponents are required to maintain the water budget in accordance with a SWS, MDP or MESP that encompasses the subject site, or where not available, maintain pre-development water budget in the proposed development.

We are not aware of a SWS, MDP or MESP which would apply to this site, therefore the criteria given in the third paragraph of Section 6.1.3 has been adopted.

The third paragraph of Section 6.1.3 provides criteria for analysis of the water budget, stating that the proponent should retain runoff up to the 27mm event, or at a minimum, the 5mm event. We assume the criteria at this time is to retain the 5mm event.

The water balance target volume is as follows.

	Initial Abstraction (mm)	Area (m^2)	Volume (m^3)
5mm Volume Whole Site	5	5048	25.2
Less Impervious Surfaces initial abstraction – Catchment A, B & E	1	2160 + 1760 + 100 = 4020	4.0
Less Pervious Surfaces initial abstraction Catchment C, D & F	5	140 + 170 + 720 = 1030	5.2
Required Retention Volume			***16.0***

A cistern of minimum volume 16.0m^3 will be designed at the SPA Stage to retain the required volume of stormwater. The cistern will function such that in minor storm events, runoff draining from the site's mechanical drains will first drain into the cistern for storage and reuse.

In storm events greater than 5mm, the cistern will fill-up and stormwater will occupy the larger storage area allocated for 100-year storm storage before draining via the site's orifice control. This will be detailed on SPA drawings.

Various alternatives are considered by which the foregoing water balance target might be achieved on this site, as follows.

1) ***Infiltration Gallery (Percolation)***: The proposed development is entirely comprised of the proposed below-grade parking structure below-grade. Infiltration is not feasible because the design criteria – *MOE Stormwater Management Planning & Design* Manual, 2003, as well as the OBC – requires such an infiltration gallery to be located with a minimum 4.00m horizontal separation from proposed buildings. There is no such location on this site in which to locate an infiltration gallery. Further, on a small site such as this it is generally not feasible to install an infiltration gallery because if the infiltration gallery were provided with the required horizontal setback from the building (min. 5m) and the footprint required to retain the volume of water (say, approx. 20m^2 with conservative percolation rate assumptions), the infiltration gallery would take up an impracticably large portion of the lot footprint which would otherwise be used for the below-grade parking structure.

2) ***Irrigation***: Irrigation shall be explored in future as there are landscaped surfaces including trees at-grade – the capacity to uptake water by irrigation will be determined at the SPA

stage. It is anticipated that irrigation alone will not consume the required volume, therefore greywater reuse is also investigated as follows.

3) ***Greywater reuse***: Greywater reuse is feasible, and may be required depending on the volume of water that can be reused by irrigation. Greywater reuse may be feasible as toilet flushing in the commercial or amenity areas of the proposed tower.

vi) Water Quality

Storm water management quality control criteria is given in the City of Burlington's manual *Stormwater Management Design Guidelines* (May 21, 2020). Section 6.1.4 *Quality Control* provides criteria which would apply to developments. That section states that a 'treatment train' approach to stormwater quality is required comprising at least two forms of water quality treatment in series to achieve 80% TSS removal, or satisfying the MECP 'Enhanced' criteria.

The site was divided according to surface conditions and the effective TSS removal for each surface condition was considered based on the treatment it would receive. The general basis of the effective TSS removal rates are as follows:

1. Rooftop areas are subject only to airborne particles and insignificant amounts of sediment transported by foot traffic. As such, an effective removal efficiency of 80% is utilized on a conventional roof to reflect the inherent runoff quality from a conventional roof.

2. Balconies and sodded areas are subject to insignificant amounts of sediment transport by foot traffic. An effective removal rate of 80% is used as it is the City limit for roofs.

3. Driving and ground-level pedestrian surfaces which are open-to-above will be subject to winter maintenance, therefore they are assumed to have an effective removal efficiency of 0% and stormwater quality treatment is thus required.

It is proposed to provide a Jellyfish Stormwater filter to satisfy the 80% TSS removal criteria. We appreciate that this is a single-stage stormwater quality solution, rather than treatment-train approach as per the manual, however we provide the following rationale as to why this should be acceptable for this development and request City consideration.

The Burlington manual recognizes conventional Oil-Grit Separators as providing 50% TSS Removal. Section 6.1.4.4 of the City's manual states that the Canadian Environmental Technology Verification (CA-ETV) is the governing authority for storm filters/quality devices.

The CETV certification for the *Stormceptor STC*-model conventional Oil-Grit Separator by Imbrium Systems Inc. concurs with the town manual that the conventional OGS provides 50% TSS removal. Figure 2, as follows, is an excerpt from the CETV *Verification Statement – Imbrium Systems Inc. Stormceptor … Oil-Grit Separators* (November 2017).

Figure 2 shows that the Stormceptor conventional OGS provides typically 50-70% TSS removal, therefore the conclusion holds that a conventional OGS does not satisfactorily address the stormwater quality criteria for this site because it does not provide 80% TSS.

Figure 2 - Excerpt from CETV Verification Statement for Stormceptor OGS

Table 1. Removal efficiencies (%) of the EF4 at specified surface loading rates

Particle size fraction (µm)	Surface loading rate (L/min/m²)						
	40	80	200	400	600	1000	1400
>500	90	58	58	100*	86	72	100*
250 - 500	100*	100*	100	100*	100*	100*	100*
150 - 250	90	82	26	100*	100*	67	90
105 - 150	100*	100*	100*	100*	100*	100*	100
75 - 105	100*	92	74	82	77	68	76
53 - 75	Undefined [a]	56	100*	72	69	50	80
20 - 53	54	100*	54	33	36	40	31
8 - 20	67	52	25	21	17	20	20
5 – 8	33	29	11	12	9	7	19
<5	13	0	0	0	0	0	4
All particle sizes by mass balance	**70.4**	**63.8**	**53.9**	**47.5**	**46.0**	**43.7**	**49.0**

TSS Removal efficiency is ~50%

[a] An outlier in the feed sample sieve data resulted in a negative removal efficiency for this size fraction.
* Removal efficiencies were calculated to be above 100%. Calculated values ranged between 101 and 171% (average 128%).
See text and Bulletin # CETV 2016-11-0001 for more information.

The CETV also provides certification for stormwater quality filters that provide TSS removal by mechanical filtration through a filter media. One such model is the Jellyfish Filter by Imbrium Systems. The CETV verification statement for the Jelllyfish Filter by Imbrium Systems states that it provides a minimum 80% TSS Removal. Figure 3, as follows, is an excerpt from the CETV *Verification Statement – Imbrium Systems Inc. Jellyfish Filter …* (August 2017).

Figure 3 shows that the Jellyfish Filter provides 80% TSS Removal, therefore it addresses the City criteria for 80% TSS Removal. The entire Jellyfish Filter CETV Verification Statement is provided here in Appendix B.

It is accordingly proposed to provide a Jellyfish Filter by Imbrium to address City criteria for 80% TSS Removal. This is lieu of providing a conventional OGS with a treatment-train approach to providing 80% TSSR. This strategy of using a manufactured treatment device/filter is more practical on a development like this because it is not feasible to install permeable pavers/bioswales etc. in small infill developments such as this which are entirely underlain by the underground parking structure.

A Jellyfish Filter will be specified to provide 80% TSS Removal at the SPA stage. Refer to Appendix B for the design report for the Jellyfish Filter

Figure 3 - Excerpt from CETV Verification Statement for Jellyfish Filter

Table 4. Summary statistics for influent and effluent event mean concentrations for selected constituents

Water Quality Variable	Sampling Location	Min	Max	Median	Range	Mean	SD	Load based removal efficiency (%)
TSS	Influent (mg/L)	16.30	261.00	79.30	244.70	86.26	51.37	87.2
	Effluent (mg/L)	3.20	21.70	11.80	18.50	10.99	4.79	
SSC	Influent (mg/L)	78.20	1401.70	444.50	1323.50	482.26	338.34	98.6
	Effluent (mg/L)	2.80	18.10	7.30	15.30	7.88	3.77	
TP	Influent (µg/L)	887.00	8793.00	3063.00	7906.00	3550.20	1914.50	64.2
	Effluent (µg/L)	472.00	4769.00	1480.00	4297.00	1688.08	1059.98	
TN	Influent (µg/L)	1170.00	10479.00	3110.00	9309.00	3519.32	2161.47	46.3
	Effluent (µg/L)	553.00	6579.00	1610.00	6026.00	2091.76	1613.61	
Zn	Influent (µg/L)	0.005	7600.00	1500.00	7600.00	1792.00	1852.91	76.1
	Effluent (µg/L)	0.005	2760.00	450.00	2760.00	561.64	594.70	
Cu	Influent (µg/L)	0.001	880.40	79.50	880.40	171.28	229.33	92.1
	Effluent (µg/L)	0.001	51.30	6.90	51.30	14.36	17.22	
Oil and Grease	Influent (mg/L)	0.20	4.06	0.93	3.86	1.07	0.82	46.4
	Effluent (mg/L)	0.00	2.32	0.35	2.32	0.50	0.60	

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6.0 GROUNDWATER

i) Introduction

It is proposed to discharge any water collected by the building's foundation drains into the storm control maintenance hole as discussed in Section 5.0, above.

A *Hydrogeological Investigation* (dated 2023-07-22) has been prepared for this development by Exp Services to qualitatively and quantitatively characterize the groundwater with respect to City of Burlington and Halton Region guidelines.

The report concludes in Table 4-3 that the short-term construction dewatering volume will be up to 454,000 L/day (including safety factor and considering storm/precipitation flows).

The report concludes in Table 4-4 that the long-term flows into the building's foundation drains will be 24,500 L/day (including safety factor and considering 1 Level of underground parking).

The report concludes in Section 5.4 that the groundwater quality complies with the criteria for discharge into Halton Region's sanitary sewer/wastewater main (although this information is not pertinent because Halton Region does not permit discharge of foundation drainage into their sewer) and that there are quality exceedances for discharge into the City of Burlington's storm sewers, therefore treatment/filtration would be required.

ii) Long-Term Groundwater

The hydrogeological investigation states as above that the peak daily volume entering the foundation drains will be 24,500 L/day, which equates to flows of 0.3 L/s. It is intended to collect this water in a sump installed in the proposed buildings below-grade levels and pump it into the proposed Building's storm control maintenance hole, thereby draining to the storm sewer connection. It is assumed that a peaking factor will be applied in the design of the pump such that the pumped groundwater discharge rate is 2.0 L/s. This is considered in the controlled storm flows compliance, as outlined in Table 6, above.

The owner intends to review, in future, the feasibility of filtering the water to address the noted exceedances for storm sewer discharge, relative to the City criteria. If, pending consideration, it is unfeasible to filter the water to address such criteria, the building's foundations may then instead be 'bathtubbed' such that there is no groundwater discharge.

iii) Short-Term (Construction) Groundwater

Groundwater collected during the site's excavation and construction by either sumps or wellpoints will need to be discharged to facilitate the excavation-for and construction-of the proposed building.

The short-term groundwater flow rate identified by the Hydrogeological Assessment totals up to 454,000 L/d (5.3 L/s). It is proposed to discharge this flow rate into the existing Plains Road storm sewer. This temporary groundwater flow rate is less than the allowable storm flow rate (23 L/s, as per Table 3), therefore short-term/construction groundwater flows are within the allowable flow rate.

7.0 CONCLUSIONS

From the foregoing investigation, the site is serviceable utilizing existing sanitary/wastewater sewers and watermain infrastructure adjacent to the site. Storm water management can be accommodated with on-site storage as described in this report.

The following table summarizes the SWM and Servicing components of the proposed development.

TABLE 7 - Summary

	Proposed Building
Peak Sanitary Discharge (L/s)	4.4 L/s
Proposed Sanitary Service	200mm @ 2.0%
Receiving Sanitary Sewer	Plains Rd. E. 250mm sanitary sewer (south side)
Development Water Demand (Fire + Domestic)	2153 USGM
Available Flow Rate	3182 USGM
Proposed Fire Service	200mm
Proposed Domestic Service	Branch 150mm
Allowable release rate from site (L/s)	23.0 L/s
Proposed release rate from site (L/s) (100 year storm)	23.0 L/s
Stormwater Quality	Jellyfish Stormwater Filter
Quantity Control	To be designed at SPA stage

8.0 REFERENCES

1. City of Burlington *Stormwater Management Design Guidelines* (May 21, 2020).

2. Storm water Management Planning and Design Manual, Ontario Ministry of the Environment, March 2003.

3. Halton Region *Water and Wastewater Linear Design Manual* (April 2019)

4. New Jersey Storm Water Best Management Practices Manual, April 2004.

5. Visual OTTHYMO v2.0 Reference Manual, July 2002

Respectfully Submitted;
The Odan Detech Group Inc.



Mark Harris
Digitally signed by Mark Harris
DN: cn=Mark Harris, o=The Odan/Detech Group Inc., ou, email=mark@odandetech.com, c=CA
Date: 2023.08.22 16:12:24 -04'00'

Paul Hecimovic, P.Eng.

Mark Harris, Dipl.Tech.
Senior Project Manager

APPENDIX A

Existing Site	Aerial view of Site and surrounding area
Site Plan & Statistics	by Reinders + Law



LEGEND

PROPERTY LINE

DRAWING :

KEY PLAN

DATE:	PROJ. NO.:	SCALE:
DEC 2020	20223	N.T.S.

PROJECT :

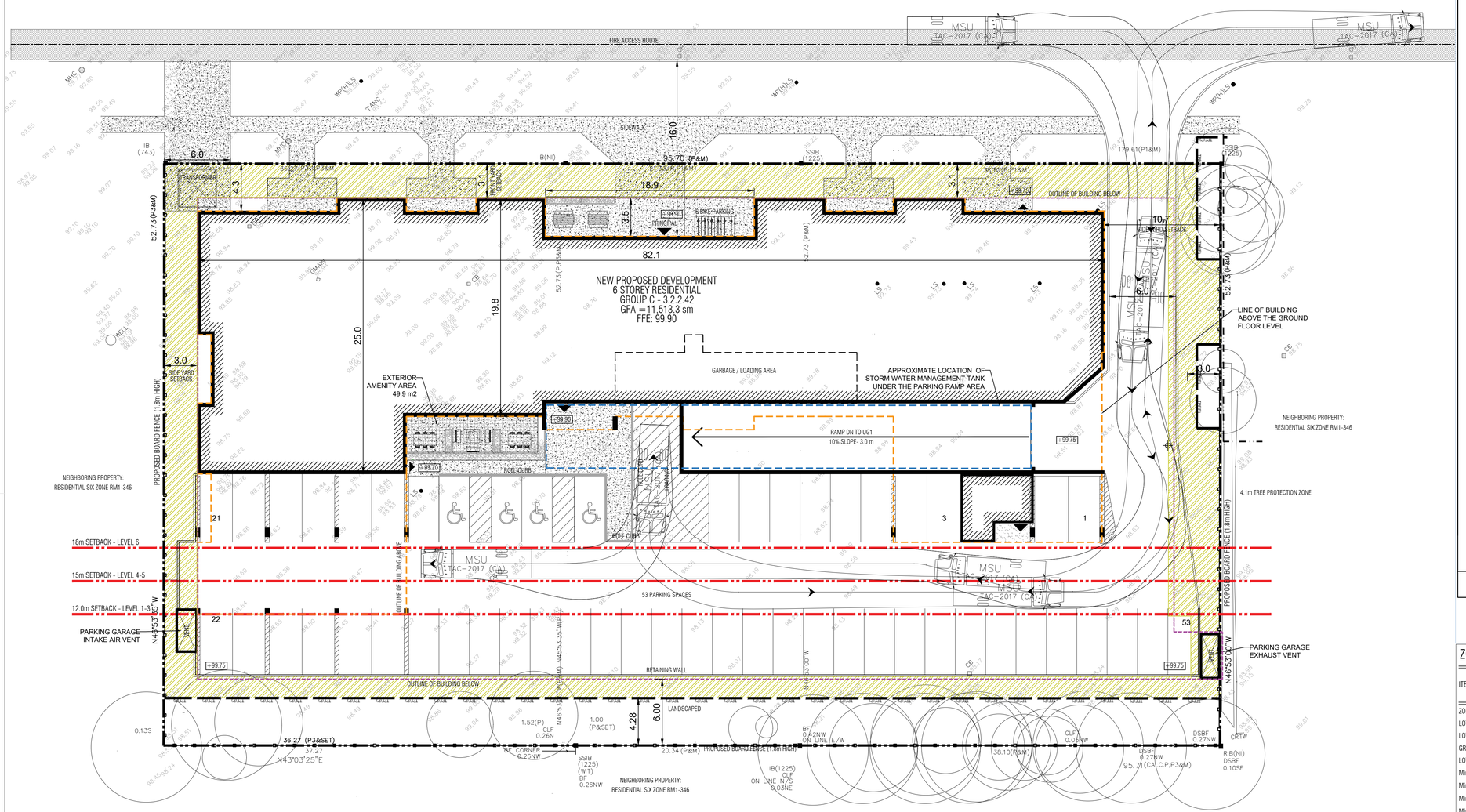
PROPOSED RESIDENTIAL DEVELOPMENT
284–292 PLAINS RD. E.
BURLINGTON, ON

ODAN·DETECH
CONSULTING ENGINEERS
The Odan/Detech Group Inc. P: (905) 632-3811 F: (905) 632-3353
5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

0 10 20 30 40 50 60 70 80 90 100

PLAINS ROAD EAST

(BY ORDER IN COUNCIL 4000/69, FORMERLY THE KING'S HIGHWAY No. 2)
(ROAD ALLOWANCE BETWEEN BROKEN FRONT CONCESSION AND CONCESSION 1)
(PART 3, INST. 1431 DEPARTMENT OF HIGHWAYS PLAN P-1957-44)
PIN 07113-0002(LT)



LEGEND:

- PROPERTY LINE
- BUILDING SETBACK LINE
- TREE PROTECTION ZONE
- OUTLINE OF BUILDING ABOVE
- STORMWATER MANAGEMENT TANK BELOW
- PROPOSED FIRE ROUTE (USING HEAVY DUTY PAVING)
- HEAVY DUTY PAVING
- LIGHT DUTY PAVING
- CONCRETE PAD OR SIDEWALK
- PROPOSED CONCRETE CURB
- C/C OR D/C — PROPOSED CONCRETE CURB WITH CURB CUT OR DEPRESSED CURB
- NEW BUILDING AREA (FOOTPRINT)
- EXISTING CHAIN LINK FENCE
- PROPOSED BOARD FENCE
- PRINCIPAL ENTRANCE TO THE BUILDING
- SECONDARY ENTRANCES TO, OR EGRESSES FROM THE BUILDING
- FRS — FIRE ROUTE SIGNS
- BFS — BARRIER FREE SIGNS (RB93)
- BARRIER FREE LOGO (PAVED ON ASPHALT)
- TACTILE STRIPS ON CONCRETE OR ASPHALT SURFACES (AODO COMPLIANT)
- EXISTING CATCHBASIN AND STRUCTURE
- PROPOSED CATCHBASIN AND STRUCTURE
- PROPOSED DOUBLE CATCHBASIN AND STRUCTURE
- EXISTING MANHOLE
- PROPOSED MANHOLE
- EXISTING CATCHBASIN AND MANHOLE PROPOSED CATCHBASIN AND MANHOLE
- PROPOSED SIAMESE CONNECTION TO SPRINKLER SYSTEM
- HP — PROPOSED HIGH POINT
- LP — PROPOSED LOW POINT
- PROPOSED DRAINAGE FLOW
- DRIVE WAY VISIBILITY TRIANGLE
- DC — DEPRESSED CURB
- H — FIRE HYDRANT (EXISTING)
- TPFP — TREE PROTECTION FENCE - (POLY MESH) - MIN 1.2M HT
- AREA OF TREE PROTECTION ZONE ENCROACHMENT EXCAVATION WHICH MAY REQUIRE ROOT PRUNING ARBORIST TO BE PRESENT DURING ANY EXCAVATION IN THESE AREAS.

ZONING STATISTICS

ITEM	EXISTING ZONING BYLAW REQUIREMENTS	PROPOSAL
ZONING CATEGORY	MXG	MXG
LOT AREA (sm)	1,000	5,043
LOT WIDTH (sm)	25 m	95.6 m
GROUND FLOOR AREA (sm)	N/A	1590.5 m²
LOT COVERAGE	N/A	40.1 %
Minimum Front Yard	3.0 m	2.8* m
Minimum Rear Yard	3.0 m	6.45 m
Minimum Rear Yard (Level 1-3)	12.0 m	12.0 m
Minimum Rear Yard (Level 4-5)	15.0 m	15.0 m
Minimum Rear Yard (Level 6)	18.0 m	18.0 m
Minimum Side Yard Abutting a Street	0 m	3.0 m
Minimum Side Yard	0 m	3.0 m
NUMBER OF PARKING SPACES RESIDENTIAL - 1 PARKING SPACE PER UNIT VISITOR - 0.25 PARKING SPACE UNIT	177 (1.25/unit) 141 36 (0.25/unit)	153 (1.08/unit) 141 12 (0.08/unit)
NUMBER OF ACCESSIBLE PARKING SPACES 2 + 2% of total required parking	6	6
PARKING STALL DIMENSIONS (m)	2.75 X 6.0	2.75 X 6.0
PARKING AISLE DIMENSIONS (m)	6.0	6.0
ACCESSIBLE PARKING STALL DIMENSIONS (m)	2.75 X 6.0	2.75 X 6.0
ACCESSIBLE PARKING PATHWAY WIDTH (m)	2.0	2.0
LOADING SPACE SIZE	10.7 X3.0	10.7 X3.0
LOADING SPACE REQUIRED	1	1
BIKE PARKING 0.5 SPACES PER UNIT	71	71
DENSITY (FSI)	1.5	2.3
AMENITY SPACE 15 sm PER 1 BED 10 (EFFICIENCY UNITS) X 15 = 150 25 sm PER 1 BED 79 (1 BED UNITS) X 35 = 2765 35 sm PER 2 + BED 52 (2+ BED UNITS) X 35 = 1820	4,735 150 2765 1820	2,784.6*

TOTAL UNITS

1 BED (EFFICIENCY)	10
1 BED	67
1 BED + D	17
2 BED	48
2 BED + D	4
3 BED	1
TOTAL UNITS	141

AMENITY SPACE

GR. FL. AMENITY	345.0 sq.m.	
FRONT YARD PATIOS	158.6 sq.m.	
BALCONIES/TERRACES	1,057.2 sq.m.	
ROOFTOP - INDOOR	576.7 sq.m.	
ROOFTOP - OUTDOOR	650.1 sq.m.	
TOTAL	2,784.6 sq.m.	19.75 sq.m./Unit

TRUE NORTH

CONSTRUCTION NORTH

no.	revisions	date	init.

GENERAL NOTE:
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cad file	23027_SP1
date plotted	06/20/2023 plot scale 1:1
drawn	JY
designed	KR
reviewed	KR
date	06/20/2023
scale	1:200

project
272-296 PLAINS RD. DEVELOPMENT

272-296 PLAINS RD. BURLINGTON, ON

drawing
ARCHITECTURAL SITE PLAN

REINDERS +LAW
ARCHITECTURE.ENGINEERING
REINDERS + LAW LTD.
ARCHITECTURE.ENGINEERING
64 ONTARIO STREET NORTH
MILTON, ON L9T 2T1
T. 905.457.1618 F. 905.457.8852
EMAIL@REINDERS.CA WWW.REINDERS.CA

drawing no. 23027_SP1 rev. no.

SHEET SIZE: A1 609.6mm X 914.4mm

BUILDING DATA - Revised Proposal

BUILDING AREAS				UNIT COUNT						
PARKING	**METRIC (sq.m.)**	**IMPERIAL(sq.ft.)**	**PARKING**							
P1 - UNDERGROUND PARKING	3839.9	41332	100							
LEVEL 1 - MAIN LEVEL PARKING	2100.7	22612	53							
RESIDENTIAL BUILDING	**METRIC (sq.m.)**	**IMPERIAL(sq.ft.)**		**1 BED (Efficiency)**	**1 BED**	**1 BED+D**	**2 BED**	**2BED+D**	**3BED**	**TOTAL**
LEVEL 1 - ENTRY LOBBY/AMENITY/CORES	773.5	8326								
LEVEL 1 - RESIDENTIAL	775.0	8342		0	1	4	2	1	1	9
LEVEL 2 - RESIDENTIAL	2043.6	21997		2	14	1	9	0	1	27
LEVEL 3 - RESIDENTIAL	2043.6	21997		2	14	1	9	0	1	27
LEVEL 4 - RESIDENTIAL	1987.2	21390		2	12	2	10	0	0	26
LEVEL 5 - RESIDENTIAL	1987.2	21390		2	12	2	10	0	0	26
LEVEL 6 - RESIDENTIAL	1931.2	20787		2	14	2	8	0	0	26
TOTAL GFA - ABOVE GRADE	**11541.3**	**124229**	**153**	**10**	**67**	**12**	**48**	**1**	**3**	**141**
FSR (5043 sq.m. LOT AREA)	**2.3**					Parking Spaces/Unit				1.085106

APPENDIX B

Visual OTTHYMO Model Output (2-Year & 100-Year storms)

CETV Verification Statement – Imbrium Systems Inc. Stormceptor OGS

CETV Verification Statement – Imbrium Systems Inc. Jellyfish Filter

Visual OTTHYMO Output (2, 5, 10, 25, 50 & 100-year storm)

```
================================================================================================================================
================================================================================================================================

       V    V   I    SSSSS  U   U    A    L
       V    V   I    SS     U   U   A A   L
        V  V    I     SS    U   U  AAAAA  L
        V  V    I      SS   U   U  A   A  L
         VV     I    SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O
        OOO     T      T    H   H    Y    M   M   OOO

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                  *****  D E T A I L E D   O U T P U T *****


  Input   filename: C:\VO Dongle Driver\Visual OTTHYMO 2.3.3\voin.dat
  Output  filename: P:\2020\20223\Design and Reports\Computer Analysis\Rev 3.0 WIP\VO2\Post-Development.out
  Summary filename: P:\2020\20223\Design and Reports\Computer Analysis\Rev 3.0 WIP\VO2\Post-Development.sum


DATE: 8/3/2023                            TIME: 2:26:10 PM

USER:


COMMENTS: _____________________________________________________


--------------------------------------------------------------------------------------------------------------------------------
  ****************************
  ** SIMULATION NUMBER:   1 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 681.520
| Ptotal= 37.19 mm |                          B=   6.000
--------------------                          C=    .780
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  4.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =   .33

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .17    2.54 |  1.17   19.72 |  2.17    5.98 |  3.17    3.02
                 .33    2.91 |  1.33   78.39 |  2.33    5.09 |  3.33    2.81
                 .50    3.42 |  1.50   25.74 |  2.50    4.45 |  3.50    2.62
                 .67    4.20 |  1.67   13.74 |  2.67    3.96 |  3.67    2.46
                 .83    5.53 |  1.83    9.48 |  2.83    3.58 |  3.83    2.33
                1.00    8.38 |  2.00    7.31 |  3.00    3.27 |  4.00    2.21


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0001) |   Area    (ha)=    .22
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .21          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      37.90        40.00
     Mannings n             =     .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                             ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
```

hrs	mm/hr	hrs	mm/hr	hrs	mm/hr	hrs	mm/hr
.083	2.54	1.083	19.72	2.083	5.98	3.08	3.02
.167	2.54	1.167	19.72	2.167	5.98	3.17	3.02
.250	2.91	1.250	78.39	2.250	5.09	3.25	2.81
.333	2.91	1.333	78.39	2.333	5.09	3.33	2.81
.417	3.42	1.417	25.74	2.417	4.45	3.42	2.62
.500	3.42	1.500	25.74	2.500	4.45	3.50	2.62
.583	4.20	1.583	13.74	2.583	3.96	3.58	2.46
.667	4.20	1.667	13.74	2.667	3.96	3.67	2.46
.750	5.53	1.750	9.48	2.750	3.58	3.75	2.33
.833	5.53	1.833	9.48	2.833	3.58	3.83	2.33
.917	8.38	1.917	7.31	2.917	3.27	3.92	2.21
1.000	8.38	2.000	7.31	3.000	3.27	4.00	2.21

Max.Eff.Inten.(mm/hr)=	78.39	38.49	
over (min)	5.00	5.00	
Storage Coeff. (min)=	1.57 (ii)	2.81 (ii)	
Unit Hyd. Tpeak (min)=	5.00	5.00	
Unit Hyd. peak (cms)=	.33	.28	
			TOTALS
PEAK FLOW (cms)=	.05	.00	.047 (iii)
TIME TO PEAK (hrs)=	1.33	1.33	1.33
RUNOFF VOLUME (mm)=	36.19	13.14	35.95
TOTAL RAINFALL (mm)=	37.19	37.19	37.19
RUNOFF COEFFICIENT =	.97	.35	.97

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

(i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
CN* = 80.0 Ia = Dep. Storage (Above)
(ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
THAN THE STORAGE COEFFICIENT.
(iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

CALIB			
STANDHYD (0003)	Area (ha)=	.18	
ID= 1 DT= 5.0 min	Total Imp(%)=	99.00	Dir. Conn.(%)= 99.00

		IMPERVIOUS	PERVIOUS (i)
Surface Area	(ha)=	.17	.00
Dep. Storage	(mm)=	1.00	5.00
Average Slope	(%)=	1.00	2.00
Length	(m)=	34.30	40.00
Mannings n	=	.013	.250

Max.Eff.Inten.(mm/hr)=	78.39	19.24	
over (min)	5.00	5.00	
Storage Coeff. (min)=	1.48 (ii)	2.72 (ii)	
Unit Hyd. Tpeak (min)=	5.00	5.00	
Unit Hyd. peak (cms)=	.33	.29	
			TOTALS
PEAK FLOW (cms)=	.04	.00	.038 (iii)
TIME TO PEAK (hrs)=	1.33	1.33	1.33
RUNOFF VOLUME (mm)=	36.19	10.83	35.93
TOTAL RAINFALL (mm)=	37.19	37.19	37.19
RUNOFF COEFFICIENT =	.97	.29	.97

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

(i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
CN* = 80.0 Ia = Dep. Storage (Above)
(ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
THAN THE STORAGE COEFFICIENT.
(iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

ADD HYD (0005)				
1 + 2 = 3	AREA (ha)	QPEAK (cms)	TPEAK (hrs)	R.V. (mm)
ID1= 1 (0001):	.22	.047	1.33	35.95
+ ID2= 2 (0003):	.18	.038	1.33	35.93
ID = 3 (0005):	.39	.085	1.33	35.94

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

```
--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0000    |    .0214       .0450
                          .0211      .0113    |    .0215       .0562
                          .0212      .0225    |    .0216       .0630
                          .0213      .0338    |    .0000       .0000

                                AREA      QPEAK       TPEAK       R.V.
                                (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0005)      .392       .085        1.33       35.94
     OUTFLOW: ID= 1 (0006)      .392       .014        1.67       35.62

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 17.02
                   TIME SHIFT OF PEAK FLOW         (min)= 20.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0077

--------------------------------------------------------------------------------
  ****************************
  ** SIMULATION NUMBER:   2 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 802.040
| Ptotal= 47.95 mm |                          B=   5.000
--------------------                          C=    .764
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  =  4.00 hrs
                         Storm time step    = 10.00 min
                         Time to peak ratio =   .33

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .17    3.43 |  1.17   24.57 |  2.17    7.80 |  3.17    4.05
                 .33    3.91 |  1.33  101.31 |  2.33    6.69 |  3.33    3.77
                 .50    4.57 |  1.50   31.90 |  2.50    5.88 |  3.50    3.54
                 .67    5.56 |  1.67   17.28 |  2.67    5.26 |  3.67    3.33
                 .83    7.23 |  1.83   12.11 |  2.83    4.77 |  3.83    3.15
                1.00   10.76 |  2.00    9.44 |  3.00    4.38 |  4.00    2.99


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0001) |   Area    (ha)=    .22
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .21          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      37.90        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                             ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                .083    3.43 | 1.083   24.57 | 2.083    7.80 |  3.08    4.05
                .167    3.43 | 1.167   24.57 | 2.167    7.80 |  3.17    4.05
                .250    3.91 | 1.250  101.31 | 2.250    6.69 |  3.25    3.77
                .333    3.91 | 1.333  101.31 | 2.333    6.69 |  3.33    3.77
                .417    4.57 | 1.417   31.90 | 2.417    5.88 |  3.42    3.54
                .500    4.57 | 1.500   31.90 | 2.500    5.88 |  3.50    3.54
                .583    5.56 | 1.583   17.28 | 2.583    5.26 |  3.58    3.33
                .667    5.56 | 1.667   17.28 | 2.667    5.26 |  3.67    3.33
                .750    7.23 | 1.750   12.11 | 2.750    4.77 |  3.75    3.15
                .833    7.23 | 1.833   12.11 | 2.833    4.77 |  3.83    3.15
                .917   10.76 | 1.917    9.44 | 2.917    4.38 |  3.92    2.99
               1.000   10.76 | 2.000    9.44 | 3.000    4.38 |  4.00    2.99

     Max.Eff.Inten.(mm/hr)=     101.31        38.28
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.42 (ii)    2.54 (ii)
```

```
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .29
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .06          .00           .060 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      46.95        19.96         46.67
     TOTAL RAINFALL   (mm)=      47.95        47.95         47.95
     RUNOFF COEFFICIENT   =        .98          .42           .97

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0003) |   Area    (ha)=    .18
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                           IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .17          .00
     Dep. Storage     (mm)=       1.00         5.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      34.30        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     101.31        31.48
               over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.34 (ii)    2.45 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .30
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .05          .00           .049 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      46.95        17.33         46.65
     TOTAL RAINFALL   (mm)=      47.95        47.95         47.95
     RUNOFF COEFFICIENT   =        .98          .36           .97

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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

--------------------
| ADD HYD   (0005) |
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)      (mm)
         ID1= 1 (0001):       .22     .060     1.33     46.67
       + ID2= 2 (0003):       .18     .049     1.33     46.65
         ================================================
         ID = 3 (0005):       .39     .110     1.33     46.66

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------       (cms)     (ha.m.)   |    (cms)     (ha.m.)
                          .0000      .0000     |    .0214       .0450
                          .0211      .0113     |    .0215       .0562
                          .0212      .0225     |    .0216       .0630
                          .0213      .0338     |    .0000       .0000

                                AREA      QPEAK       TPEAK       R.V.
                                (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0005)      .392       .110        1.33       46.66
     OUTFLOW: ID= 1 (0006)      .392       .018        1.67       46.34

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 16.88
```

```
                    TIME SHIFT OF PEAK FLOW          (min)= 20.00
                    MAXIMUM  STORAGE   USED        (ha.m.)=   .0099

--------------------------------------------------------------------------------
  ****************************
  ** SIMULATION NUMBER:   3 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 918.280
| Ptotal= 55.20 mm |                          B=   5.000
--------------------                          C=    .763
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  4.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =   .33

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .17    3.96 |  1.17   28.29 |  2.17    9.00 |  3.17    4.68
                 .33    4.51 |  1.33  116.31 |  2.33    7.72 |  3.33    4.36
                 .50    5.28 |  1.50   36.70 |  2.50    6.78 |  3.50    4.09
                 .67    6.42 |  1.67   19.91 |  2.67    6.07 |  3.67    3.85
                 .83    8.35 |  1.83   13.96 |  2.83    5.51 |  3.83    3.64
                1.00   12.41 |  2.00   10.89 |  3.00    5.06 |  4.00    3.46


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--------------------
| CALIB            |
| STANDHYD  (0001) |   Area    (ha)=    .22
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .21          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      37.90        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                            ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                .083    3.96 | 1.083   28.29 | 2.083    9.00 |  3.08    4.68
                .167    3.96 | 1.167   28.29 | 2.167    9.00 |  3.17    4.68
                .250    4.51 | 1.250  116.31 | 2.250    7.72 |  3.25    4.36
                .333    4.51 | 1.333  116.31 | 2.333    7.72 |  3.33    4.36
                .417    5.28 | 1.417   36.70 | 2.417    6.78 |  3.42    4.09
                .500    5.28 | 1.500   36.70 | 2.500    6.78 |  3.50    4.09
                .583    6.42 | 1.583   19.91 | 2.583    6.07 |  3.58    3.85
                .667    6.42 | 1.667   19.91 | 2.667    6.07 |  3.67    3.85
                .750    8.35 | 1.750   13.96 | 2.750    5.51 |  3.75    3.64
                .833    8.35 | 1.833   13.96 | 2.833    5.51 |  3.83    3.64
                .917   12.41 | 1.917   10.89 | 2.917    5.06 |  3.92    3.46
               1.000   12.41 | 2.000   10.89 | 3.000    5.06 |  4.00    3.46

     Max.Eff.Inten.(mm/hr)=     116.31        48.50
                 over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.34 (ii)    2.40 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .30
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .07          .00           .069 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      54.20        24.96         53.90
     TOTAL RAINFALL   (mm)=      55.20        55.20         55.20
     RUNOFF COEFFICIENT   =        .98          .45           .98

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
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--------------------
| CALIB            |
| STANDHYD  (0003) |   Area    (ha)=    .18
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .17          .00
     Dep. Storage     (mm)=       1.00         5.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      34.30        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     116.31        41.43
               over (min)         5.00         5.00
     Storage Coeff.  (min)=       1.27 (ii)    2.32 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .30
                                                      *TOTALS*
     PEAK FLOW       (cms)=        .06          .00          .057 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33         1.33
     RUNOFF VOLUME    (mm)=      54.20        22.16        53.87
     TOTAL RAINFALL   (mm)=      55.20        55.20        55.20
     RUNOFF COEFFICIENT   =        .98          .40          .98

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| ADD HYD   (0005) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0001):      .22     .069     1.33    53.90
        + ID2= 2 (0003):      .18     .057     1.33    53.87
          ================================================
          ID = 3 (0005):      .39     .126     1.33    53.89

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |    (cms)     (ha.m.)
                           .0000      .0000   |    .0214       .0450
                           .0211      .0113   |    .0215       .0562
                           .0212      .0225   |    .0216       .0630
                           .0213      .0338   |    .0000       .0000

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0005)       .392       .126        1.33       53.89
     OUTFLOW: ID= 1 (0006)       .392       .021        1.67       53.56

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 16.76
                  TIME SHIFT OF PEAK FLOW         (min)= 20.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0114

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  ****************************
  ** SIMULATION NUMBER:   4 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1065.950
| Ptotal= 64.43 mm |                          B=   5.000
--------------------                          C=    .762
                       used in:   INTENSITY =  A / (t + B)^C

                       Duration of storm  =  4.00 hrs
                       Storm time step    = 10.00 min
                       Time to peak ratio =   .33
```

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.17	4.64	1.17	33.01	2.17	10.53	3.17	5.48
.33	5.29	1.33	135.38	2.33	9.03	3.33	5.11
.50	6.18	1.50	42.80	2.50	7.94	3.50	4.79
.67	7.52	1.67	23.26	2.67	7.11	3.67	4.51
.83	9.77	1.83	16.31	2.83	6.45	3.83	4.27
1.00	14.51	2.00	12.73	3.00	5.92	4.00	4.05

CALIB	
STANDHYD (0001)	Area (ha)= .22
ID= 1 DT= 5.0 min	Total Imp(%)= 99.00 Dir. Conn.(%)= 99.00

		IMPERVIOUS	PERVIOUS (i)
Surface Area	(ha)=	.21	.00
Dep. Storage	(mm)=	1.00	1.00
Average Slope	(%)=	1.00	2.00
Length	(m)=	37.90	40.00
Mannings n	=	.013	.250

NOTE: RAINFALL WAS TRANSFORMED TO 5.0 MIN. TIME STEP.

---- TRANSFORMED HYETOGRAPH ----

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.083	4.64	1.083	33.01	2.083	10.53	3.08	5.48
.167	4.64	1.167	33.01	2.167	10.53	3.17	5.48
.250	5.29	1.250	135.38	2.250	9.03	3.25	5.11
.333	5.29	1.333	135.38	2.333	9.03	3.33	5.11
.417	6.18	1.417	42.80	2.417	7.94	3.42	4.79
.500	6.18	1.500	42.80	2.500	7.94	3.50	4.79
.583	7.52	1.583	23.26	2.583	7.11	3.58	4.51
.667	7.52	1.667	23.26	2.667	7.11	3.67	4.51
.750	9.77	1.750	16.31	2.750	6.45	3.75	4.27
.833	9.77	1.833	16.31	2.833	6.45	3.83	4.27
.917	14.51	1.917	12.73	2.917	5.92	3.92	4.05
1.000	14.51	2.000	12.73	3.000	5.92	4.00	4.05

Max.Eff.Inten.	(mm/hr)=	135.38	62.48	
over	(min)	5.00	5.00	
Storage Coeff.	(min)=	1.26 (ii)	2.26 (ii)	
Unit Hyd. Tpeak	(min)=	5.00	5.00	
Unit Hyd. peak	(cms)=	.33	.30	
				TOTALS
PEAK FLOW	(cms)=	.08	.00	.081 (iii)
TIME TO PEAK	(hrs)=	1.33	1.33	1.33
RUNOFF VOLUME	(mm)=	63.43	31.70	63.11
TOTAL RAINFALL	(mm)=	64.43	64.43	64.43
RUNOFF COEFFICIENT	=	.98	.49	.98

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

(i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
CN* = 80.0 Ia = Dep. Storage (Above)
(ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
THAN THE STORAGE COEFFICIENT.
(iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

CALIB	
STANDHYD (0003)	Area (ha)= .18
ID= 1 DT= 5.0 min	Total Imp(%)= 99.00 Dir. Conn.(%)= 99.00

		IMPERVIOUS	PERVIOUS (i)
Surface Area	(ha)=	.17	.00
Dep. Storage	(mm)=	1.00	5.00
Average Slope	(%)=	1.00	2.00
Length	(m)=	34.30	40.00
Mannings n	=	.013	.250

Max.Eff.Inten.	(mm/hr)=	135.38	55.17
over	(min)	5.00	5.00
Storage Coeff.	(min)=	1.19 (ii)	2.19 (ii)
Unit Hyd. Tpeak	(min)=	5.00	5.00

```
     Unit Hyd. peak  (cms)=        .33          .31
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .07          .00          .066 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      63.43        28.73         63.08
     TOTAL RAINFALL   (mm)=      64.43        64.43         64.43
     RUNOFF COEFFICIENT   =        .98          .45           .98

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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

--------------------
| ADD HYD   (0005) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0001):      .22    .081     1.33    63.11
        + ID2= 2 (0003):      .18    .066     1.33    63.08
          ================================================
          ID = 3 (0005):      .39    .147     1.33    63.09

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .0000      .0000    |    .0214       .0450
                           .0211      .0113    |    .0215       .0562
                           .0212      .0225    |    .0216       .0630
                           .0213      .0338    |    .0000       .0000

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0005)       .392       .147        1.33       63.09
     OUTFLOW: ID= 1 (0006)       .392       .021        1.75       62.77

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 14.40
                   TIME SHIFT OF PEAK FLOW         (min)= 25.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0137

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  ****************************
  ** SIMULATION NUMBER:   5 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1172.340
| Ptotal= 71.25 mm |                          B=   5.000
--------------------                          C=    .761
                         used in:   INTENSITY =  A / (t + B)^C

                         Duration of storm  =  4.00 hrs
                         Storm time step    = 10.00 min
                         Time to peak ratio =   .33

                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .17    5.16 |  1.17   36.50 |  2.17   11.67 |  3.17    6.08
                 .33    5.87 |  1.33  149.30 |  2.33   10.01 |  3.33    5.67
                 .50    6.86 |  1.50   47.31 |  2.50    8.81 |  3.50    5.31
                 .67    8.34 |  1.67   25.73 |  2.67    7.89 |  3.67    5.01
                 .83   10.83 |  1.83   18.06 |  2.83    7.16 |  3.83    4.74
                1.00   16.07 |  2.00   14.10 |  3.00    6.57 |  4.00    4.50


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0001) |   Area    (ha)=    .22
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
```

```
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .21          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      37.90        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                             ---- TRANSFORMED HYETOGRAPH ----
               TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
               .083    5.16 | 1.083   36.50 | 2.083   11.67 |  3.08    6.08
               .167    5.16 | 1.167   36.50 | 2.167   11.67 |  3.17    6.08
               .250    5.87 | 1.250  149.30 | 2.250   10.01 |  3.25    5.67
               .333    5.87 | 1.333  149.30 | 2.333   10.01 |  3.33    5.67
               .417    6.86 | 1.417   47.31 | 2.417    8.81 |  3.42    5.31
               .500    6.86 | 1.500   47.31 | 2.500    8.81 |  3.50    5.31
               .583    8.34 | 1.583   25.73 | 2.583    7.89 |  3.58    5.01
               .667    8.34 | 1.667   25.73 | 2.667    7.89 |  3.67    5.01
               .750   10.83 | 1.750   18.06 | 2.750    7.16 |  3.75    4.74
               .833   10.83 | 1.833   18.06 | 2.833    7.16 |  3.83    4.74
               .917   16.07 | 1.917   14.10 | 2.917    6.57 |  3.92    4.50
              1.000   16.07 | 2.000   14.10 | 3.000    6.57 |  4.00    4.50

     Max.Eff.Inten.(mm/hr)=     149.30        73.31
               over (min)         5.00         5.00
     Storage Coeff.  (min)=       1.22 (ii)    2.17 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .31
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .09          .00          .089 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      70.25        36.90         69.91
     TOTAL RAINFALL   (mm)=      71.25        71.25         71.25
     RUNOFF COEFFICIENT   =        .99          .52           .98

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0003) |   Area    (ha)=    .18
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .17          .00
     Dep. Storage     (mm)=       1.00         5.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      34.30        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     149.30        65.91
               over (min)         5.00         5.00
     Storage Coeff.  (min)=       1.15 (ii)    2.10 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .31
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .07          .00          .073 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      70.25        33.83         69.88
     TOTAL RAINFALL   (mm)=      71.25        71.25         71.25
     RUNOFF COEFFICIENT   =        .99          .47           .98

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| ADD HYD   (0005) |
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0001):      .22    .089     1.33    69.91
        + ID2= 2 (0003):      .18    .073     1.33    69.88
          ================================================
          ID = 3 (0005):      .39    .162     1.33    69.90

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0000    |    .0214       .0450
                          .0211      .0113    |    .0215       .0562
                          .0212      .0225    |    .0216       .0630
                          .0213      .0338    |    .0000       .0000

                               AREA      QPEAK       TPEAK       R.V.
                               (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0005)      .392       .162        1.33       69.90
     OUTFLOW: ID= 1 (0006)      .392       .021        1.75       69.57

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 13.07
                  TIME SHIFT OF PEAK FLOW         (min)= 25.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0154

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  ****************************
  ** SIMULATION NUMBER:   6 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1281.340
| Ptotal= 77.88 mm |                          B=   5.000
--------------------                          C=    .761
                       used in:   INTENSITY =  A / (t + B)^C

                       Duration of storm  =  4.00 hrs
                       Storm time step    = 10.00 min
                       Time to peak ratio =   .33

               TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                .17    5.63 |  1.17   39.89 |  2.17   12.75 |  3.17    6.65
                .33    6.41 |  1.33  163.18 |  2.33   10.94 |  3.33    6.20
                .50    7.49 |  1.50   51.70 |  2.50    9.62 |  3.50    5.81
                .67    9.11 |  1.67   28.13 |  2.67    8.62 |  3.67    5.47
                .83   11.83 |  1.83   19.74 |  2.83    7.83 |  3.83    5.18
               1.00   17.56 |  2.00   15.41 |  3.00    7.18 |  4.00    4.92


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0001) |   Area    (ha)=    .22
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .21          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      37.90        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                            ---- TRANSFORMED HYETOGRAPH ----
               TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
               .083    5.63 | 1.083   39.89 | 2.083   12.75 |  3.08    6.65
               .167    5.63 | 1.167   39.89 | 2.167   12.75 |  3.17    6.65
               .250    6.41 | 1.250  163.18 | 2.250   10.94 |  3.25    6.20
               .333    6.41 | 1.333  163.18 | 2.333   10.94 |  3.33    6.20
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                .417    7.49 | 1.417   51.70 | 2.417    9.62 |  3.42    5.81
                .500    7.49 | 1.500   51.70 | 2.500    9.62 |  3.50    5.81
                .583    9.11 | 1.583   28.13 | 2.583    8.62 |  3.58    5.47
                .667    9.11 | 1.667   28.13 | 2.667    8.62 |  3.67    5.47
                .750   11.83 | 1.750   19.74 | 2.750    7.83 |  3.75    5.18
                .833   11.83 | 1.833   19.74 | 2.833    7.83 |  3.83    5.18
                .917   17.56 | 1.917   15.41 | 2.917    7.18 |  3.92    4.92
               1.000   17.56 | 2.000   15.41 | 3.000    7.18 |  4.00    4.92

     Max.Eff.Inten.(mm/hr)=     163.18        84.44
               over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.17 (ii)    2.10 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .31
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .10          .00          .097 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33         1.33
     RUNOFF VOLUME    (mm)=      76.88        42.10        76.52
     TOTAL RAINFALL   (mm)=      77.88        77.88        77.88
     RUNOFF COEFFICIENT   =        .99          .54          .98

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0003) |   Area    (ha)=    .18
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .17          .00
     Dep. Storage     (mm)=       1.00         5.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      34.30        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     163.18        76.98
               over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.11 (ii)    2.03 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .31
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .08          .00          .079 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33         1.33
     RUNOFF VOLUME    (mm)=      76.88        38.94        76.49
     TOTAL RAINFALL   (mm)=      77.88        77.88        77.88
     RUNOFF COEFFICIENT   =        .99          .50          .98

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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

--------------------
| ADD HYD   (0005) |
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0001):       .22     .097     1.33    76.52
        + ID2= 2 (0003):       .18     .079     1.33    76.49
          ================================================
          ID = 3 (0005):       .39     .177     1.33    76.51

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0006) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
```

```
--------------------      (cms)      (ha.m.)   |   (cms)      (ha.m.)
                          .0000       .0000    |    .0214       .0450
                          .0211       .0113    |    .0215       .0562
                          .0212       .0225    |    .0216       .0630
                          .0213       .0338    |    .0000       .0000

                                AREA       QPEAK        TPEAK        R.V.
                                (ha)       (cms)        (hrs)        (mm)
     INFLOW : ID= 2 (0005)      .392        .177         1.33        76.51
     OUTFLOW: ID= 1 (0006)      .392        .021         1.83        76.18

                    PEAK   FLOW   REDUCTION [Qout/Qin](%)= 11.96
                    TIME SHIFT OF PEAK FLOW         (min)= 30.00
                    MAXIMUM  STORAGE   USED       (ha.m.)=   .0171

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

 FINISH
================================================================================================================================
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ETV Canada Verified

The Stormceptor® STC

Technology Fact Sheet for Imbrium Systems Inc.

Performance Claim

The Stormceptor® STC is capable of removing the following pollutants from stormwater runoff when designed in accordance with the PCSWMM for Stormceptor:

- Total Suspended Solids (TSS) overall loading removal range from 76% to 94%
- Total Kjeldahl Nitrogen (TKN) overall loading removal range from 43% to 65%

The TSS claim is based on three overall loading tests performed at three geographically different sites. Site 1 included eight rain events, site 2 had three rain events and site 3 had four rain events. The rain events varied in intensity and duration.

The TKN claim is based on two overall loading tests performed at two geographically different sites. Site 1 included eight rain events and site 3 had four rain events. The rain events varied in intensity and duration.

Simulations produced by the PCSWMM for Stormceptor are based on runoff that is generated from a stabilized catchment with all areas covered by vegetation, concrete, asphalt, structures and/or other non-erodible surfaces.

Technology Application

The patented Stormceptor® STC is a stormwater quality treatment device that can be installed in place of a conventional maintenance hole in a storm drainage system.

The Stormceptor® STC is a vertically oriented precast concrete cylindrical chamber that is separated into upper and lower compartments by a fiberglass insert.

Technology Operation

Stormceptor® STC flows into the upper by-pass chamber from the sewer. Inflows less than the design flow rate are diverted by a weir and orifice/drop pipe-assembly through the fibreglass insert into the lower treatment chamber. The drop pipe discharges water parallel to the circular chamber wall to increase detention time and inhibit mixing. From the treatment chamber, water flows up through the riser pipe into the by-pass chamber on the downstream side of the weir and is discharged into the storm sewer.

The water velocity slows when it enters the treatment chamber. Oil or other liquids with a specific gravity less than water will rise and become trapped beneath the fiberglass insert. These pollutants are retained in the treatment chamber because the entrance to the outlet riser pipe is submerged. Sediment will settle to the bottom of the chamber by gravity.

Flows in excess of the orifice/drop pipe capacity will flow over the weir and into the downstream sewer. This action prevents high flows from entering the lower treatment chamber and ensures that captured pollutants are not resuspended.

Performance Claim Conditions

The conditions for this performance claim are as follows:

St. Paul, MN, COMO PARK - SITE 1 0.4 ha

	3 Aug 98	7 Aug 98	27 Aug 98	19 Sep 98	23 Sep 98	7 Sep 99	11 Sep 99	19 Sep 99	**OVERALL**
TSS in, kg	5.22	19.47	1.35	1.42	0.72	0.25	14.59	0.13	43.15
TSS out, kg	1.30	3.61	0.40	1.70	0.89	0.21	2.31	0.03	10.45
TSS removed, kg	3.92	15.86	0.95	-0.28	-0.17	0.04	12.28	0.10	**32.70**
removal % mass	75	81	70	-19	-24	16	84	77	**76**
TKN in, kg	0.188	0.141	0.011	0.153	0.011	0.013	0.486	0.002	1.005
TKN out, kg	0.166	0.055	0.012	0.066	0.011	0.001	0.091	0.001	0.345
TKN removed, kg	0.08	0.09	0.00	0.09	0.00	0.01	0.40	0.00	**0.66**
removal % mass	44	61	-9	57	0	92	81	50	**65**

Boston, MA, Westwood - SITE 2 0.3 ha

	5 Aug 97	21 Aug 97	29 Sep 97	**OVERALL**
TSS in, kg	0.185	0.099	0.120	0.404
TSS out, kg	0.002	0.008	0.013	0.023
TSS removed, kg	0.183	0.091	0.107	**0.381**
removal % mass	99	92	89	**94**
TKN in, kg	-	-	-	-
TKN out, kg				
TKN removed, kg	-	-	-	-
removal % mass	-	-	-	-

Seattle, WA, Seatac - SITE 3 0.4 ha

	13 Mar 99	25 Apr 99	3 May 99	28 Oct 99	**OVERALL**
TSS in, kg	1.891	0.699	0.296	7.401	10.287
TSS out, kg	0.658	0.315	0.093	0.308	1.373
TSS removed, kg	1.233	0.384	0.203	7.093	**8.914**
removal % mass	65	55	69	96	**87**
TKN in, kg	0.099	0.024	0.028	0.083	0.234
TKN out, kg	0.033	0.024	0.024	0.052	0.133
TKN removed, kg	0.066	0.000	0.004	0.031	**0.101**
removal % mass	67	0	14	37	**43**

The performance claim is based on the above data from three field studies conducted at three geographically different locations, comprising fourteen storm events of varying intensity (1 to 131 mm/hr, 1 to 24 hrs duration).

Verification

Testing was done by the following: Service Environmental & Engineering (St. Paul, MN site); Environmental Sampling Technology (Boston, MA site); Associated Earth Sciences Inc. (Seattle, WA site). The evaluation was conducted by Pollutech Group of Companies Inc. following the Canadian ETV Program's General Verification Protocol (March 2000).

What is the ETV Program?

The Canadian Environmental Technology Verification (ETV) Program is delivered by The Bloom Centre for Sustainability (BLOOM) under a license agreement from Environment Canada. The Canadian ETV Program is designed to support Canada's environment industry by providing credible and independent verification of technology performance claims.

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Limitation of Verification
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Printed March 2000 Revised March 2009 Revised May 2012 Expires March 2015

◆ Verification Statement ◆ Verification Statement ◆ Verification Statement ◆

VERIFICATION STATEMENT

GLOBE Performance Solutions

Verifies the performance of

Jellyfish® Filter JF4-2-1

Developed by Imbrium Systems, Inc.,
Whitby, Ontario, Canada

In accordance with

ISO 14034:2016

Environmental management —
Environmental technology verification (ETV)

John D. Wiebe, PhD
Executive Chairman
GLOBE Performance Solutions

August 3, 2017
Vancouver, BC, Canada

Verification Body
GLOBE Performance Solutions
404 – 999 Canada Place | Vancouver, B.C | Canada |V6C 3E2

Technology description and application

The Jellyfish® Filter is an engineered stormwater quality treatment technology designed to remove a variety of stormwater pollutants including floatable trash and debris, oil, coarse and fine suspended sediments, and particulate-bound pollutants such as nutrients, heavy metals, and hydrocarbons. The Jellyfish Filter combines gravitational pre-treatment (sedimentation and floatation) and membrane filtration in a single compact structure. The system utilizes membrane filtration cartridges comprised of multiple pleated filter elements ("filtration tentacles") that provide high filtration surface area with the associated advantages of high flow rate, high sediment capacity, and low filtration flux rate.

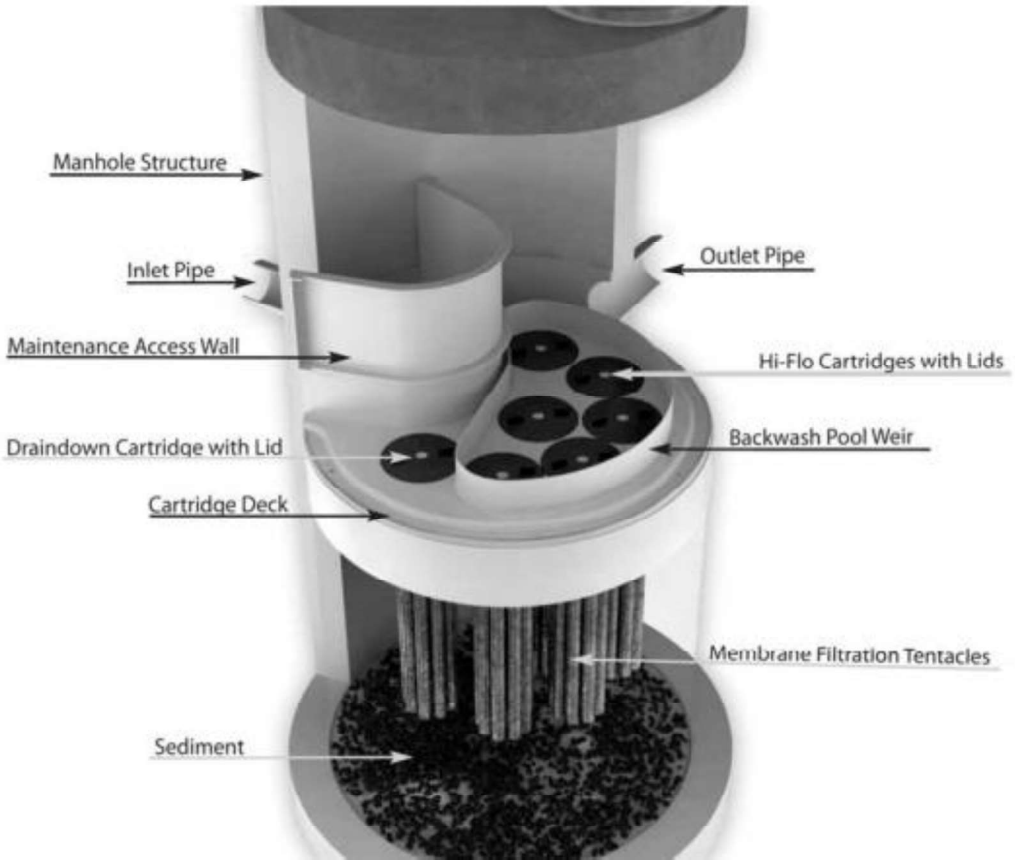


Figure 1. Cut-away graphic of a Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge

Figure 1 depicts a cut-away graphic of a typical 6-ft diameter Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge (JF6-6-1). Stormwater influent enters the system through the inlet pipe and builds a pond behind the maintenance access wall, with the pond elevation providing driving head. Flow is channeled downward into the lower chamber beneath the cartridge deck. A flexible separator skirt (not shown in the graphic) surrounds the filtration zone where the filtration tentacles of each cartridge are suspended, and the volume between the vessel wall and the outside surface of the separator skirt comprises a pretreatment channel. As flow spreads throughout the pretreatment channel, floatable pollutants accumulate at the surface of the pond behind the maintenance access wall and also beneath the cartridge deck in the pretreatment channel, while coarse sediments settle to the sump. Flow proceeds under the separator skirt and upward into the filtration zone, entering each filtration tentacle and depositing fine suspended sediment and associated particulate-bound pollutants on the outside surface of the membranes. Filtered water proceeds up the center tube of each tentacle, with the flow from each tentacle combining under the cartridge lid, and discharging to the top of the

cartridge deck through the cartridge lid orifice. Filtered effluent from the hi-flo cartridges enters a pool enclosed by a 15-cm high weir, and if storm intensity and resultant driving head is sufficient, filtered water overflows the weir and proceeds across the cartridge deck to the outlet pipe. Filtered effluent discharging from the draindown cartridge(s) passes directly to the outlet pipe, and requires only a minimal amount of driving head (2.5 cm) to provide forward flow. As storm intensity subsides and driving head drops below 15 cm, filtered water within the backwash pool reverses direction and passes backward through the hi-flo cartridges, and thereby dislodges sediment from the membranes which subsequently settles to the sump below the filtration zone. During this passive backwashing process, water in the lower chamber is displaced only through the draindown cartridge(s). Additional self-cleaning processes include gravity, as well as vibrational pulses emitted when flow exits the orifice of each cartridge lid, and these combined processes significantly extend the cartridge service life and maintenance cleaning interval. Sediment removal from the sump by vacuum is required when sediment depths reach 30 cm, and cartridges are typically removed, externally rinsed, and recommissioned on an annual basis, or as site-specific maintenance conditions require. Filtration tentacle replacement is typically required every 3 – 5 years.

Performance conditions

The data and results published in this Technology Fact Sheet were obtained from a field monitoring program conducted on a Jellyfish® Filter JF4-2-1 (4-ft diameter manhole with 2 hi-flo cartridges and 1 draindown cartridge), in accordance with the provisions of the TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements—Amendments to TARP Tier II Protocol (NJDEP, 2009). Testing was completed by researchers led by Dr. John Sansalone at the University of Florida's Engineering School of Sustainable Infrastructure and Environment. The drainage area providing stormwater runoff to the test unit varied between 502 m^2 and 799 m^2 (5400 ft^2 to 8600 ft^2) depending on storm intensity and wind direction. The unit was monitored for a total of 25 TARP qualifying storm events (i.e. $\geq$ 2.5 mm of rainfall) contributing cumulative rainfall of 381 mm (15 in) over the 13-month period between May 28, 2010 and June 27, 2011. Only TARP-qualified storms were routed through the unit, and maintenance was not required during the testing period based on sediment accumulation less than the depth indicated for maintenance, and also based on hydraulic testing performed on the system after the conclusion of monitoring.

Table 1 shows the specified and achieved amended TARP criteria for storm selection and sampling.
Table 2 shows the observed ranges of operational conditions that occurred over the testing period.

Table 1. Specified and achieved amended TARP criteria for storm selection and sampling

Description	Criteria value	Achieved value
Total rainfall	$\geq$ 2.5 mm (0.1 in)	> 2.5 mm (0.1 in)
Minimum inter-event period	6 hrs	10 hrs
Minimum flow-weighted composite sample storm coverage	70% including as much of the first 20% of the storm	100%
Minimum influent/effluent samples	10, but a minimum of 5 subsamples for composite samples	Minimum of 8 subsamples for composite samples
Total sampled rainfall	Minimum 381 mm (15 in)	384 mm (15.01 in)
Number of storms	Minimum 20	25

Table 2. Observed operational conditions for events monitored over the study period

Operational condition	Observed range
Storm durations	26 – 691 min
Previous dry hours	10 - 910 hrs
Rainfall depth	3 – 50 mm
Initial rainfall to runoff lag time	1 – 34 min
Runoff volume	206 – 13,229 L
Peak rainfall intensity	5 – 137 mm/hr
Peak runoff flow rate	0.5 – 14.3 L/s
Event median flow rate	0.01 – 5.5 L/s

The 4-ft diameter test unit has sedimentation surface area of 1.17 m^2 (12.56 ft^2). Each of the three filter cartridges employed in the test unit uses filtration tentacles of 137 cm (54 in) length, with filter surface area of 35.4 m^2 (381 ft^2) per cartridge, and total filter surface area of 106.2 m^2 (1143 ft^2) for the three cartridges combined. The design treatment flow rate is 5 L/s (80 gal/min) for each of the two hi-flo cartridges and 2.5 L/s (40 gal/min) for the single draindown cartridge, for a total design treatment flow rate of 12.6 L/s (200 gal/min) at design driving head of 457 mm (18 in). This translates to a filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m^2 (0.21 gal/min/ft^2) for each hi-flo cartridge and 0.07 L/s/m^2 (0.11 gal/min/ft^2) for the draindown cartridge. The design flow rate for each cartridge is controlled by the sizing of the orifice in the cartridge lid. The distance from the bottom of the filtration tentacles to the sump is 61 cm (24 in).

Performance claims

The Jellyfish® Filter demonstrated the removal efficiencies indicated in **Table 3** for respective constituents during field monitoring of 25 TARP qualified storm events with cumulative rainfall of 381 mm, conducted in accordance with the provisions of the TARP Tier II Protocol (TARP, 2003) and New Jersey Tier II Stormwater Test Requirements—Amendments to TARP Tier II Protocol (NJDEP, 2009), and using the following design parameters:

- System hydraulic loading rate (system treatment flow rate per unit of sedimentation surface area) of 10.8 L/s/m^2 (15.9 gal/min/ft^2) or lower
- Filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m^2 (0.21 gal/min/ft^2) or lower for each hi-flo cartridge and 0.07 L/s/m^2 (0.11 gal/min/ft^2) or lower for each draindown cartridge
- Distance from the bottom of the filtration tentacles to the sump of 61 cm (24 in) or greater
- Driving head of 457 mm (18 in) or greater

Table 3. Mean, median and 95% confidence interval (median) for removal efficiencies of selected stormwater constituents

Parameter	Mean	Median	Median - 95% Lower Limit	Median - 95% Upper Limit
TSS	84.7	85.6	82.8	89.8
SSC	97.5	98.3	97.1	98.7
Total phosphorus	48.8	49.1	43.3	60.1
Total nitrogen	37.9	39.3	31.2	54.6
Zinc	55.3	69	39	75
Copper	83.0	91.7	75.1	98.9
Oil and grease	60.1	60	42.7	100

N.B. As with any field test of stormwater treatment devices, removal efficiencies will vary based on pollutant influent concentrations and other site specific conditions.

Performance results

The frequency of rainfall depths monitored during the study is presented in **Figure 2**. The median and 90th percentile rainfall depths were 11 mm and 31.7 mm, respectively. These values represent the depth of rainfall that is not exceeded in 50 and 90 percent of the monitored rainfall events.

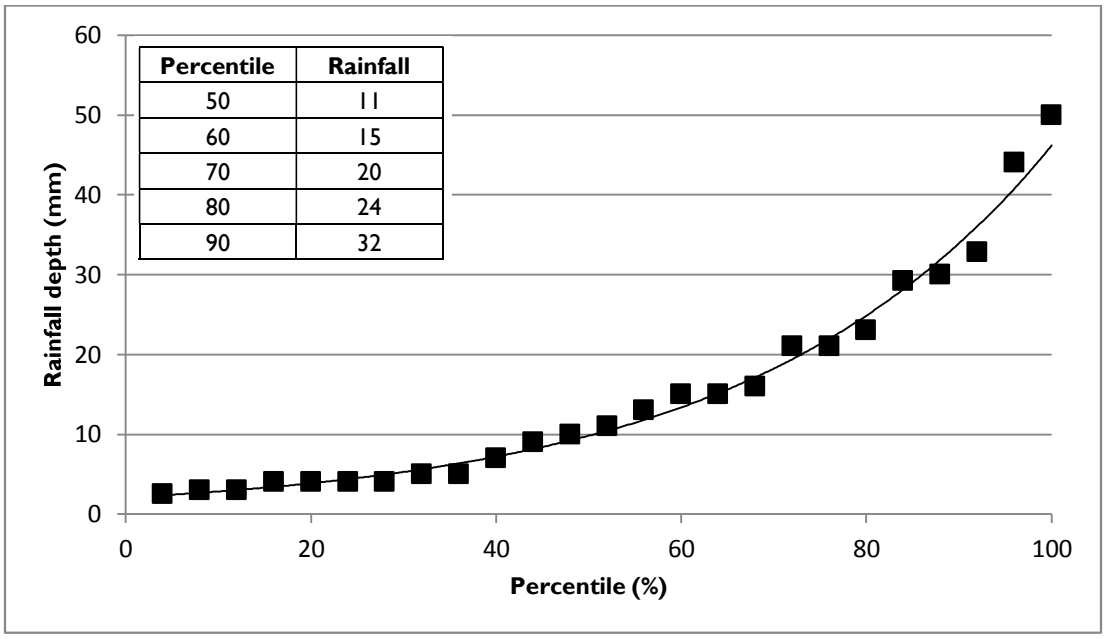


Figure 2. Rainfall depth frequency curve

Sediment removal performance was assessed by measuring the event mean concentration and mass of suspended sediment entering and leaving the unit during runoff events. This involved sampling the full cross-section of influent and effluent flows manually at 2 - 10 minute intervals for the full duration of each storm event and combining discrete samples into flow-weighted composites. Comparing the theoretical mass recovery from the sump calculated by the difference between the influent and effluent mass to the actual dry weight of the recovered sump mass showed an overall mass balance recovery of 94.5% over the study period.

The median d50 particle size (i.e. 50th percentile particle size) of the influent and effluent was 82 and 3 µm, respectively (**Figure 3**). The median influent particles sizes ranged between 22 and 263 µm, whereas median effluent particle sizes ranged between 1 and 11 µm.

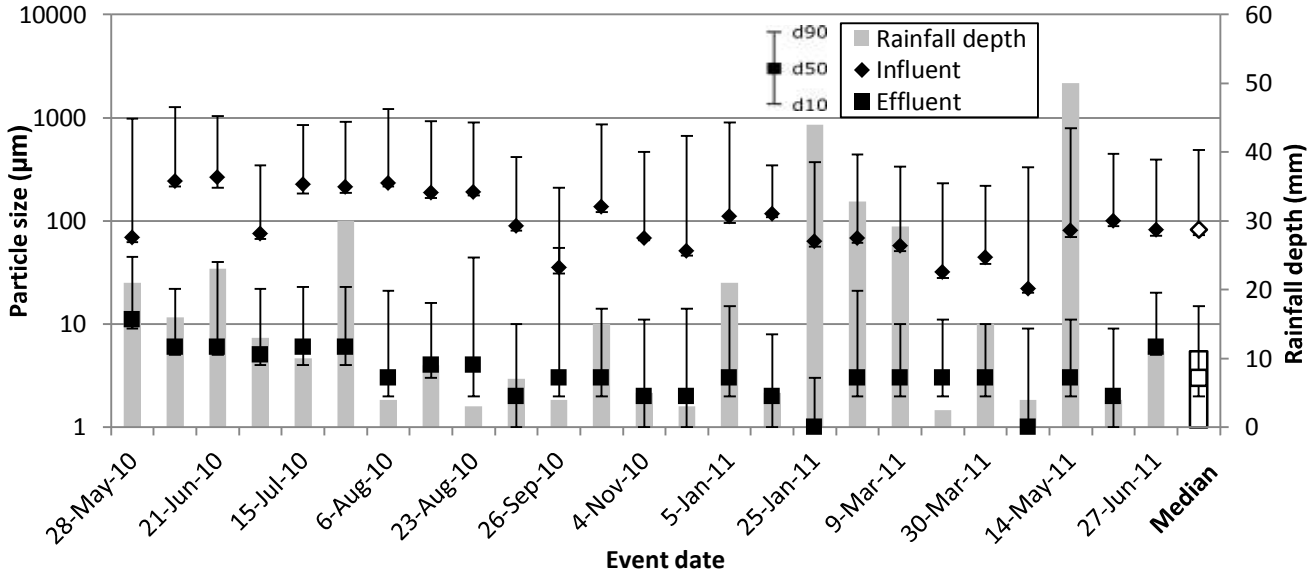


Figure 3. The rainfall depth and d10, d50, and d90 particle sizes of the influent and effluent composite samples for each monitored storm event over the 13-month testing period

Sampling of flows into and out of the Jellyfish Filter over the testing period showed statistically significant reductions ($p < 0.05$; Wilcoxon signed-rank test) in influent event mean concentrations for all selected stormwater constituents (**Table 4** and **Figure 4**). Effluent event mean Suspended Sediment Concentrations (SSC) were below 19 mg/L during all monitored events. Load-based removal rates were also calculated based on the sum of loads over the study period. These removal rages ranged from 46.3 for Total Nitrogen to 98.6 for SSC (**Table 4**).

Table 4. Summary statistics for influent and effluent event mean concentrations for selected constituents

Water Quality Variable	Sampling Location	Min	Max	Median	Range	Mean	SD	Load based removal efficiency (%)
TSS	Influent (mg/L)	16.30	261.00	79.30	244.70	86.26	51.37	87.2
	Effluent (mg/L)	3.20	21.70	11.80	18.50	10.99	4.79	
SSC	Influent (mg/L)	78.20	1401.70	444.50	1323.50	482.26	338.34	98.6
	Effluent (mg/L)	2.80	18.10	7.30	15.30	7.88	3.77	
TP	Influent (µg/L)	887.00	8793.00	3063.00	7906.00	3550.20	1914.50	64.2
	Effluent (µg/L)	472.00	4769.00	1480.00	4297.00	1688.08	1059.98	
TN	Influent (µg/L)	1170.00	10479.00	3110.00	9309.00	3519.32	2161.47	46.3
	Effluent (µg/L)	553.00	6579.00	1610.00	6026.00	2091.76	1613.61	
Zn	Influent (µg/L)	0.005	7600.00	1500.00	7600.00	1792.00	1852.91	76.1
	Effluent (µg/L)	0.005	2760.00	450.00	2760.00	561.64	594.70	
Cu	Influent (µg/L)	0.001	880.40	79.50	880.40	171.28	229.33	92.1
	Effluent (µg/L)	0.001	51.30	6.90	51.30	14.36	17.22	
Oil and Grease	Influent (mg/L)	0.20	4.06	0.93	3.86	1.07	0.82	46.4
	Effluent (mg/L)	0.00	2.32	0.35	2.32	0.50	0.60	

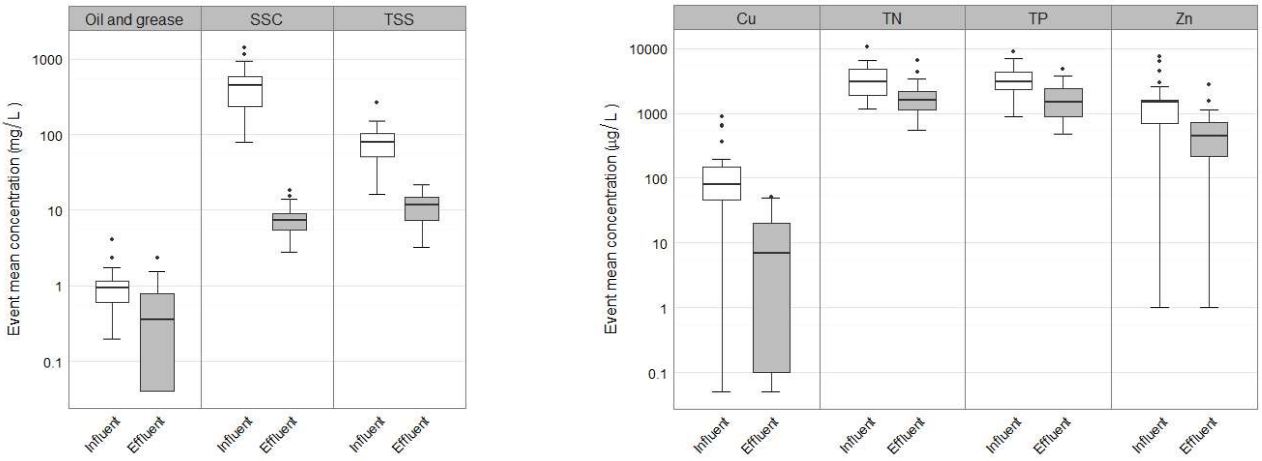


Figure 4. Boxplots showing the distribution of influent and effluent event mean concentrations (EMC) for selected stormwater constituents over the study period

Verification

The verification was completed by the Verification Expert, Toronto and Region Conservation Authority, contracted by GLOBE Performance Solutions, using the International Standard ***ISO 14034:2016 Environmental management -- Environmental technology verification (ETV)***. Data and information provided by Imbrium Systems to support the performance claim included the performance monitoring report prepared by University of Florida, Engineering School of Sustainable Infrastructure and Environment, and dated November 2011. This report is based on testing completed in accordance with the Technology Acceptance Reciprocity Partnership (TARP) Tier II Protocol (2003) and New Jersey Tier II Stormwater Test Requirements--Amendments to TARP Tier II Protocol (NJDEP, 2009).

What is ISO14034:2016 Environmental management – Environmental technology verification (ETV)?

ISO 14034:2016 specifies principles, procedures and requirements for environmental technology verification (ETV), and was developed and published by the *International Organization for Standardization (ISO)*. The objective of ETV is to provide credible, reliable and independent verification of the performance of environmental technologies. An environmental technology is a technology that either results in an environmental added value or measures parameters that indicate an environmental impact. Such technologies have an increasingly important role in addressing environmental challenges and achieving sustainable development.

For more information on the Jellyfish® Filter please contact:

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For more information on ISO 14034:2016 / ETV please contact:

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Tel: 604-695-5018 / Toll Free: 1-855-695-5018
etv@globeperformance.com

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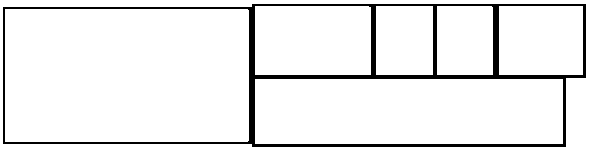
APPENDIX C

Email Correspondence from Regional Engineering Staff regarding Watermains and hydrant tests thereof

From: Kisneris, John [mailto:John.Kisneris@halton.ca]
Sent: Thursday, July 21, 2022 9:09 AM
To: mark@odandetech.com
Subject: RE: 292 Plains Road East - Burlington - (284 & 292 PRdE), 505-07/21, 520-008/21

Hi Mark. For information, in this case only, if you have already undertaken fireflow tests off of the 300mm watermain and have the results, the Region will not be asking for fireflow tests off of the 400mm watermain.
Question. What does the Mechanical Engineer need from you in order to design the building's water systems ?
Does he/she just need the static pressure, which is something that you should easily be able to undertake/verify ?

John Kisneris
Development Project Manager
Planning Services
Legislative & Planning Services
Halton Region
905-825-6000, ext. 7742 | 1-866-442-5866



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From: Mark Harris - Odan Detech Group <mark@odandetech.com>
Sent: Thursday, July 21, 2022 8:35 AM
To: Kisneris, John <John.Kisneris@halton.ca>
Subject: RE: 292 Plains Road East - Burlington -

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Sorry, with 4 hydrants shown.

Thank you,

Mark Harris, Dipl. Tech. | Senior Project Manager
The Odan/Detech Group Inc.

P : (905) 632-3811 ext.122 | **F :** (905) 632-3363 | **C : (**289) 439-7281
5230, SOUTH SERVICE ROAD, UNIT 107 | BURLINGTON, ONTARIO | L7L 5K2
www.odandetech.com | mark@odandetech.com

Vacation Alert – I will be on vacation July 7th returning Monday July 11th.
Vacation Alert – I will be on vacation August 5th returning Monday August 22nd.
I will respond to your email upon my return.
If this is urgent please forward your email to detech@odandetech.com

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From: Mark Harris - Odan Detech Group [mailto:mark@odandetech.com]
Sent: Thursday, July 21, 2022 8:34 AM
To: John Kisneris (John.Kisneris@halton.ca) <John.Kisneris@halton.ca>
Subject: 292 Plains Road East - Burlington -
Importance: High

John,

With regards to our site located at 292 Plains Road the Region requested that connection be made to the 400mm dia. water main and not the 300mm dia. water main.
We had originally proposed connection to the 300mm and complete flow tests of this water main which were sufficient.
As additional costs will be incurred by the Owner to test the 400mm water main would the Region be able to provide flow testing in this area related to the 400mm water main?
Based on the OPS mapping attached I have identified 4 hydrants, 2 on the 400mm main and 2 on side streets with 150mm water mains.

Please let me know if the Region can provide flow information on these hydrants for our use to avoid additional testing.

Thank you,

Mark Harris, Dipl. Tech. | Senior Project Manager
The Odan/Detech Group Inc.

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APPENDIX D

Functional Servicing Plan
Functional Grading Plan