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PROPOSED RETIREMENT RESIDENCE DEVELOPMENT
2380 LAKESHORE ROAD WEST
OAKVILLE, ONTARIO

PROJECT No. : 18219

FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT

Prepared For:

SUCCESSION DEVELOPMENT CORPORATION

Prepared By:

The Odan/Detech Group Inc.

Original:	July 5th, 2018
Revised:	January 22nd, 2019
Revised:	September 6th, 2019
Revised:	January 26th, 2023
Revised:	June 30th, 2023

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CONSULTING ENGINEERS

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HydroWorks HD-5

CETV Verification Statement

1.0 INTRODUCTION

The property under study is a 0.385 Ha (0.95 acre) site located at 2380 Lakeshore Road West in Oakville, Ontario. The site is presently occupied by the following:

- A three-storey commercial building with associated paved parking areas surrounding, located at the east side of the site
- A one-storey detached house building with associated driveway located at the west side of the site

The site is bound by the following:

- To the north: Lakeshore Road West
- To the east: Existing commercial/residential townhouse development
- To the south: Existing residential apartment development
- To the west: Existing commercial development and associated parking lot

For detailed topography of the existing site conditions, as of May 30, 2018, refer to the topographic survey prepared by Cunningham McConnell Limited.

It is proposed to construct a mixed-use six-storey commercial and retirement home development on the property. There is a below-grade parking structure proposed beneath the entire site. A driveway access is proposed from Lakeshore Road West from the site's north property line.

A 2.94m road widening of Lakeshore Road West is proposed in the subject development – refer to the architectural Site Plan and Servicing/Grading Plans. The site's area post-development will be 0.366 Ha (0.90 acres).

Refer to the Site Plan by Michael Spaziani Architect Inc. in Appendix A for the site's layout.

This report will evaluate the serviceability of the site with respect to sanitary waste water, water and storm water management (SWM) and will implement the Servicing criteria identified by Region engineering staff in prior correspondence.

2.0 SCOPE OF WORK

THE ODAN/DETECH GROUP INC. was retained by **Succession Development Corporation.** 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 Site Plan 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 FSR and Storm Water Management Report in support of the Site Plan Application.

3.0 WATER DISTRIBUTION

Design Considerations

There is an existing 300mm PVC watermain beneath the north side of Lakeshore Road West, opposite the subject site.

One 150mm fire service with branch 100mm branch domestic water service connection are proposed to the above main. Refer to the Site Servicing Plan.

The unit rate and peaking factors of water consumption, minimum pipe size and allowable pressure in line were established from the City Design Manual Standards. The pressures and volumes must be sufficient for peak hour conditions and under fire conditions as established by the Ontario Building Code 2006. The minimal residual pressure under fire conditions is 140 kpa. (or 20.3 psi).

Fire flow demand is calculated using the Fire Underwriters' Survey Fire Flow calculation, on the following page.

The allowable pressures are as follows:

Condition	**Allowable Pressures (kpa)**	
	min.	**max.**
1) Min. Hour	275	700
2) Peak Hour	275	700
3) Peak Day + Fire Flow	140	700

The water demand for redeveloped Building is calculated as follows:

a)	Average Day domestic demand -		0.86 L/s
b)	Peak day demand -	2.25 x average daily demand	1.94 L/s
c)	Fire flow as per FUS 1999 manual		200 L/s

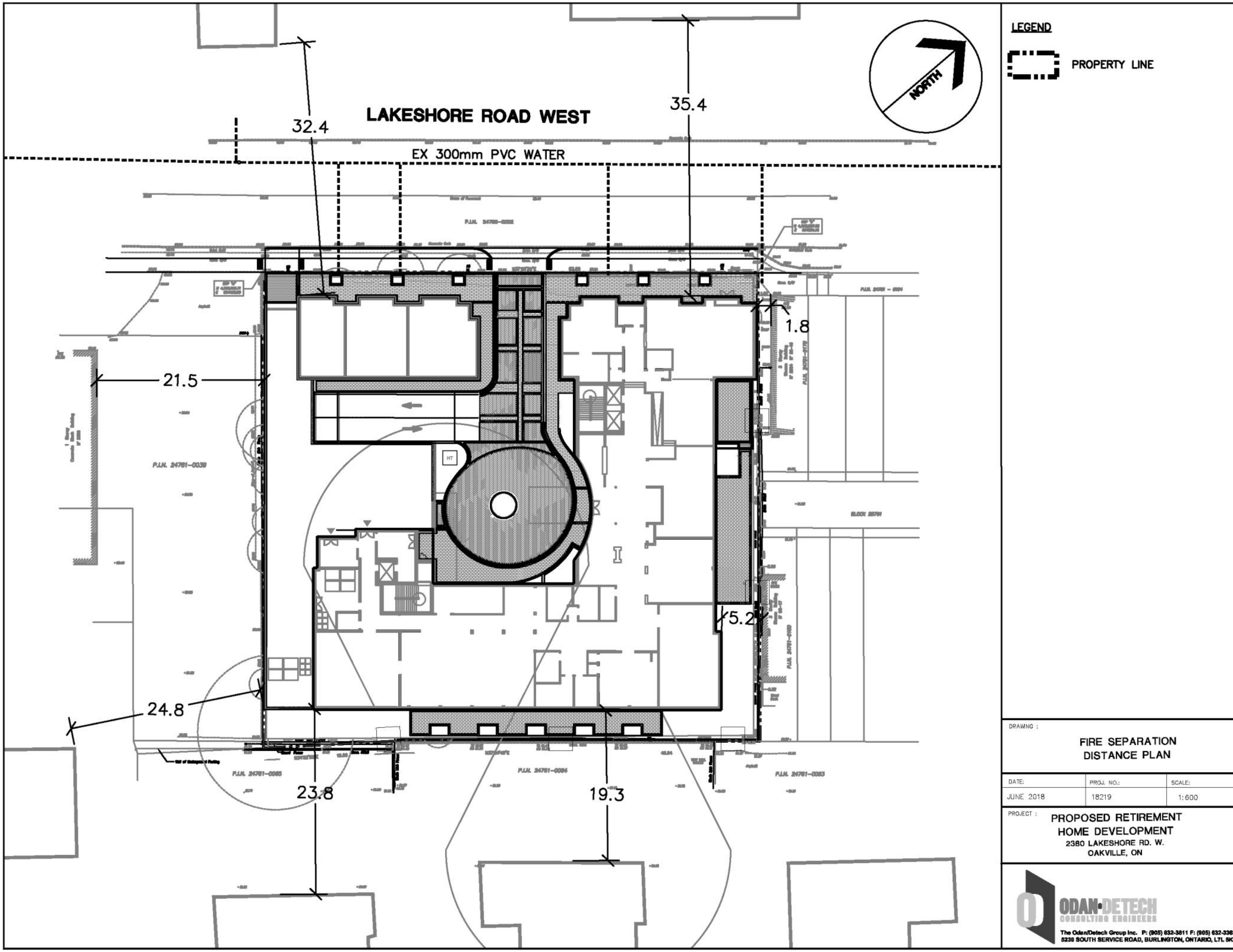
TABLE 1 – Total Water Demand

	L/sec	USGM
Peak Domestic Flow Demand	1.9	30
Fire Flow Demand (FUS)	200	3170
Total Water Demand	202	3200
Available Flow at 20 PSI Residual Pressure	383	6068

The following assumptions were made in the Fire Underwriters' Survey fire flow calculation:

- The building will be of fire-resistive (reinforced concrete) construction
- The contents will be non-combustible (residences)
- The building will be sprinklered as per NFPA 13 and the sprinklers fully monitored
- Horizontal separation from adjacent buildings as shown on the following *Fire Separation Distance Plan*

A hydrant flow test was prepared by Jackson Water Works to the NFPA 291 standard. The flow test reports are included on the following pages. The hydrant flow test shows that there is a flow rate of rate of 6068 USGM available at residual pressure 20 psi, which is greater than the development's water demand (3200 USGM) therefore it follows that ***the existing main is sufficient to provide fire protection to the subject development and no infrastructure improvements are necessary to service the subject development.***



DRAWING :

FIRE SEPARATION DISTANCE PLAN

DATE:	PROJ. NO.:	SCALE:
JUNE 2018	18219	1:600

PROJECT : PROPOSED RETIREMENT HOME DEVELOPMENT
2380 LAKESHORE RD. W.
OAKVILLE, ON

ODAN-DETECH CONSULTING ENGINEERS
The Odan/Detech Group Inc. P: (905) 632-3811 F: (905) 632-3363
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WATER SUPPLY FOR PUBLIC FIRE PROTECTION , FIRE UNDERWRITERS SURVEY
GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOWS

F = 220 x C x √ 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: Oakville

OBC OCCUPANCY: Residential & Commercial

BUILDING FOOT PRINT (m2): 2642

OF STOREYS: 6

PROJECT: 2380 Lakeshore Road West Retirement Home

PROJECT No: 18219

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	30%	
Standard Water Supply	yes	10%	50%
Fully Supervised System	yes	10%	
		50%	

CONTENTS FACTOR: Limited Combustible CHARGE: -15%

EXPOSURE 1 (south) Ex Apartments	Distance to Exposure Building (m) Length - Height	19.3	15%
EXPOSURE 2 (east) Existing Townhouses	Distance to Exposure Building (m) Length - Height	1.8	25%
EXPOSURE 3 (west) Existing Comm	Distance to Exposure Building (m) Length - Height	21.5	10%
EXPOSURE 4 (north) Existing House	Distance to Exposure Building (m) Length - Height	32.4	5%
		Total:	55%

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: Yes

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

CALCULATIONS

	C =	0.6		Fire Resistive
	A =	8932	m2	(2 Largest floors + 50% of floors above)
	F =	12475	L/min	
Round to Nearest 1000 L/min	F =	13000	L/min	must be > 2000 L/min

STOREY AREAS m2	
1921	1
2305	2
2353	3
2353	4
2353	5
2353	6

CORRECTION FACTORS:

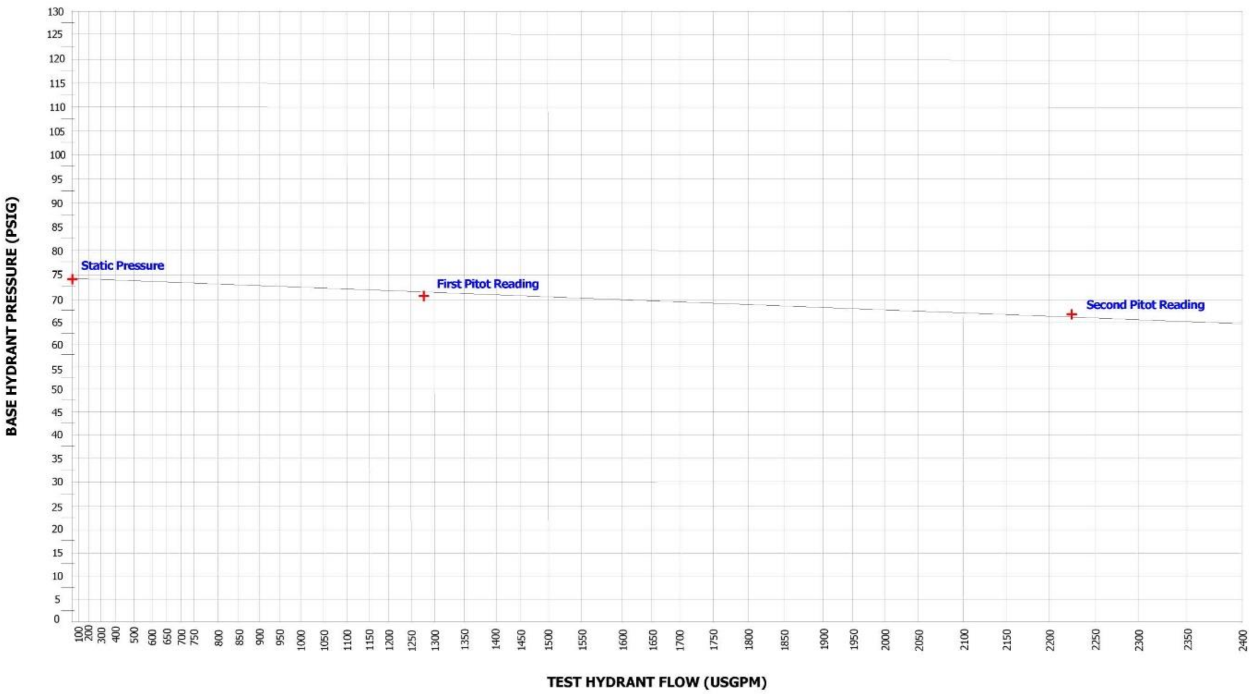
OCCUPANCY	-1950	L/min
FIRE FLOW ADJUSTED FOR OCCUPANCY	11050	L/min
REDUCTION FOR SPRINKLER	-5525	L/min
EXPOSURE CHARGE	6077.5	L/min

REQUIRED FIRE FLOW	**F =**	11603	L/min		
Round to Nearest 1000 L/min	**F =**	12000	L/min	3170	usgm
	F =	200	L/sec		

JACKSON WATERWORKS JW
Telephone: (905) 547-6770
Toll Free: (800)-734-5732
E-mail: jww@bellnet.ca
Website: www.jacksonwaterworks.ca

FIRE HYDRANT FLOW TEST RESULTS

TEST #1 of 1



No. of Ports Open	Port Dia. (in)	Pitot Reading (psig)	Pitot Conversion (usgpm) Conversion Factor = 0	Residual Pressure (psig)
1	2.50	58	1278	71
2	2.50	44/44	2226	68
THEORETICAL FLOW @ 20psi			6086	

Test Date	25 May 2018
Test Time	11:30am
Pipe Diameter (in)	12
Static Pressure (psig)	74

Site Information		
Site Name or Developer Name	Southbound Developments Inc.	Engineer: Odan Detech Group Inc.
Site Address/Municipality	2380 Lakeshore Road West, Oakville	
Location of Test Hydrant	In Front of 2381 Lakeshore Road West	
Location of Base Hydrant	Lakeshore Road West, 1st West of Jones Street	
Comments	Testing has been completed in accordance with NFPA-291 guidelines wherever and whenever possible and practical. Conversion factors for pitot tube readings have been used depending on hose nozzle internal design and installation profile. Refer to attached cover letter for additional information.	
Verified By	Mark Schmidt	

221 Sherman Avenue North, Hamilton, Ontario L8L 6N2

4.0 SANITARY SEWERS

i) Available & Existing Infrastructure

The following sewers presently exist beneath the streets bordering the subject site. Refer to the Servicing Plan for the layout of the sewers bordering the subject site.

- Lakeshore Road West – there is a 300mm sanitary sewer flowing easterly adjacent to the site's north frontage. There is a high point in the sewer at the site's frontage from which the sewer flows east and west.

ii) Proposed Sanitary Servicing

It is proposed to drain the subject development to the 300mm Lakeshore Road sanitary sewer with a 200mm @ 2.00% sanitary service connection.

The sanitary sewer design criteria and unit flow is provided in the Regional Municipality of Halton's *Water and Wastewater Linear Design Manual* (October 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
- Light Commercial 90 p/Ha or 24.75 m^3/ha/day
- 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 (less than 6-storey): 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 retirement home development because the Region standard 135 P/Ha unit population would result in a unit population of approximately 0.5 P/unit for the proposed statistics, which is unrealistic

Peaking Factor (Residential)

$$M = 1 + \frac{4}{4+\sqrt{P}}$$

The peak sanitary flow from the proposed development is thus calculated as follows, in Table 2.

TABLE 2 – Proposed Sanitary Flows

	Population (P)	Average Flow (l/s)	Peak Factor	Peak Sanitary (l/s)	Infiltration Allowance (l/s)	Total Flow (l/s)
Retirement Home Units	421	1.17	4.01	4.69	0.11	**4.85**
Retail	5	0.01	3.55	0.05		

A 150mm @ 2.0% sanitary sewer connection is proposed to the 300mm sanitary sewer beneath Lakeshore Road. The pipe has a capacity of 22 L/s, which is adequate to convey the above post-development sanitary flow.

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) =		NOTE:
RESIDENTIAL SITE AREA (ha) =	0.37	
TOTAL SITE AREA (ha) =	0.385	

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	156			421	101088	1.17	4.01	4.69
COMMERCIAL, Using 90 persons/ha (Floor Ha)		0.05		5	1238	0.01	3.55	0.05
COMMERCIAL, Using 0.60 L/sec per ha				0	0	0.00	2.50	0.00

TOTAL	V1=	102326	Q1=	4.69	
			Q2=	0.05	
Q = (MqP/86400) + A * I (L/sec)			Qinfil	0.11	
			Qtot	4.85	

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) Downstream Sanitary Sewer Capacity

Region engineering staff have stated that an independent downstream sanitary sewer analysis is required to confirm the capacity of the receiving sanitary sewers to receiving flows from the subject development.

The following downstream sanitary sewer analysis shows that the receiving sanitary sewers have capacity for the proposed development and no offsite infrastructure improvements are necessary to accommodate the flows from the proposed development.

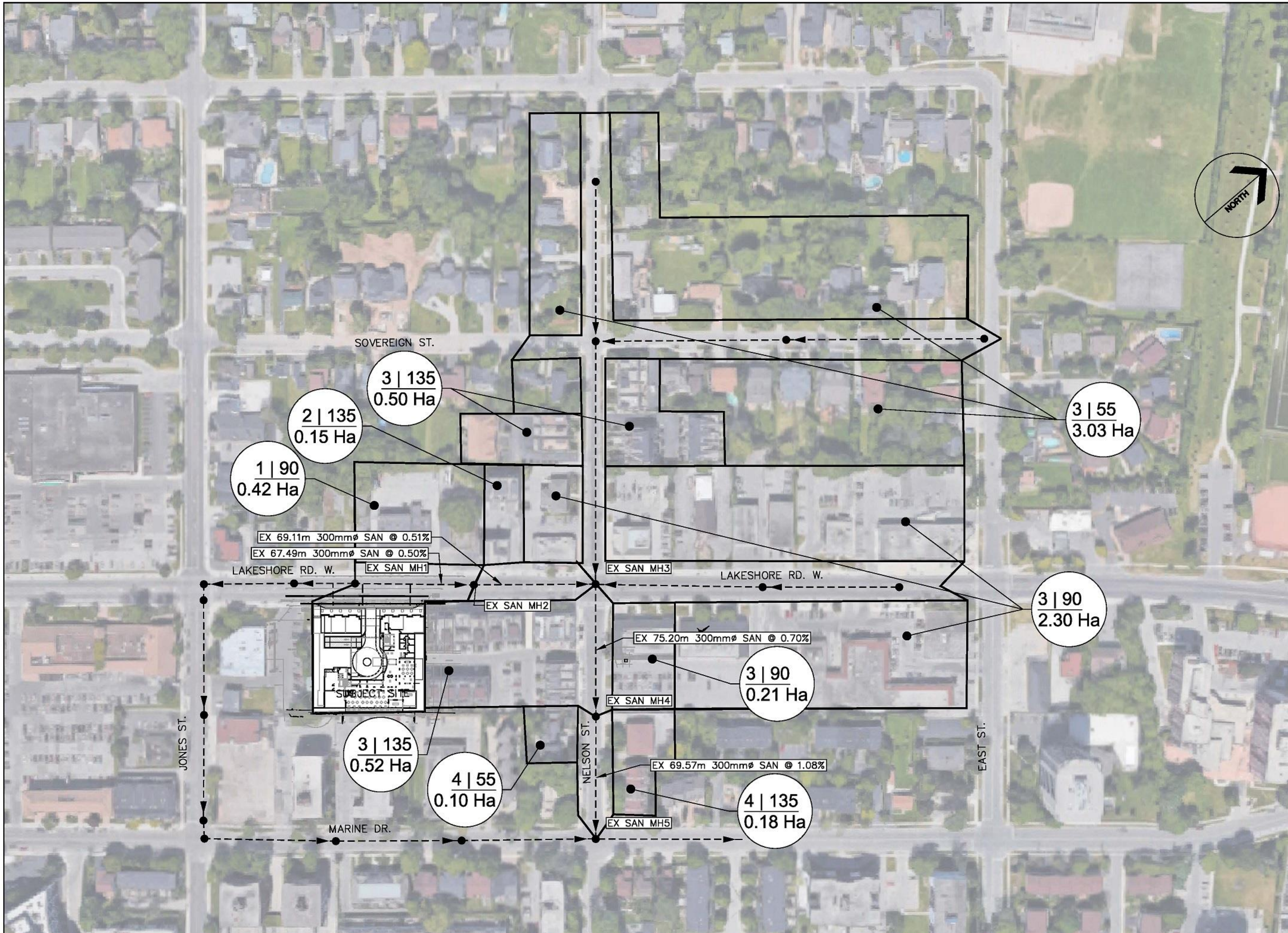
Region staff stated that an independent analysis of the immediate downstream segments is acceptable. That is, it is not necessary to analyze all segments to the trunk discharge point.

The Odan/Detech Group subsequently prepared an original analysis. The methodology for the analysis is as follows.

1) The downstream sewer catchment plan on the following page was prepared to show the tributary catchment areas for sanitary flow, path of the sewer pipe, etc.
2) Catchment areas as shown on the downstream sewer catchment plan were delineated by original research by the Odan/Detech Group.
3) Population density and unit flow was established as given in Tables 3-1 and 3-2 in the Regional Municipality of Halton’s Water and Wastewater Linear Design Manual (October 2019).
4) Downstream sewer design sheets were prepared in pre-development and post-development scenarios as shown on the following pages.
5) The slopes, pipe diameters, as-built information was taken from as-built plan & profile drawings provided by the Region of Halton as well as inverts surveyed in the survey prepared for this development by Cunningham McConnell (May 2018).

We provide the following discussion on the downstream sanitary sewer analysis.

1) Pre-development, no pipes are flowing at more than 13.9% of their respective capacity.
2) Post-development, with the additional flow from the subject development, no pipes are flowing at more than 21.4% of their capacity. This is acceptable, therefore no improvements are necessary to the downstream sanitary sewer network on account of the proposed development.
3) There is a high point in the receiving local sanitary sewers adjacent to the site’s north frontage to Lakeshore Road West. That is, the site is tributary to two different tributary branches of the downstream sanitary network. Note that the two legs ultimately converge at a bifurcation located at Marine Drive and Nelson Street. The site is proposed to drain to the easterly leg because that is a more direct path to the downstream outlet. Refer to the downstream sanitary sewer catchment plan on the following page for the layout of the receiving sewer network. The westerly leg also serves a much larger catchment area originating on Jones Street, therefore it stands to reason that the easterly leg is a sounder outlet for the proposed development.
4) There is a discrepancy in the Region’s sanitary sewer public works information operating maps. The maps show that EX SAN MH1 adjacent to the site’s north frontage slopes up to the east. That is, in this case, the sanitary sewers adjacent to the site’s north frontage flow westerly across the site’s entire north frontage. The as-built plan & profile as well as the inverts surveyed by Cunningham McConnel in this sewer, on the other hand, show that the pipe slopes down to the east away from EX SAN MH1. This is reflected in this analysis.



LEGEND

DENOTES CATCHMENT AREA ID

DENOTES POPULATION DENSITY (P/Ha)

3 | 55
3.03 Ha

DENOTES CATCHMENT AREA AREA

DRAWING :

DOWNSTREAM SANITARY SEWER CATCHMENT PLAN

DATE:	PROJ. NO.:	SCALE:
JUNE 2018	18219	1:2000

PROJECT :

PROPOSED RETIREMENT HOME DEVELOPMENT
2380 LAKESHORE ROAD WEST
OAKVILLE, ON

ODAN·DETECH
CONSULTING ENGINEERS

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5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

DOWNSTREAM SANITARY SEWERS (Pre-Development)

Site location: Subject Proposed Development - 2380 Lakeshore Road West

Ref# PN 18219

Location				Existing Industrial/Commercial					Existing Residential Population					
Segment SAN Trib ID	Street Name	US Node	DS Node	Commercial (ha)	School (ha)	Acc've Area (ha)	Population (Person)	Acc've Pop'n (Person)	Apartments (>6 St)(Ha)	Apartments (<6 St)(Ha)	Townhouse (Ha)	Detached (Ha)	Population (Person)	Acc've Pop'n (Person)
1 (Trib of MH 1)	Lakeshore Rd W	EX SAN MH1	EX SAN MH2	0.420		0.420	37.80	37.80	-	-	-	-	-	-
2 (Trib of MH 2)	Lakeshore Rd W	EX SAN MH2	EX SAN MH3			0.420	-	37.80	-	-	0.150	-	20.25	20.25
3 (Trib of MH 3)	Nelson St	EX SAN MH3	EX SAN MH4	2.300	-	2.720	207.00	244.80	-	-	1.020	3.030	304.35	324.60
4 (Trib of MH 4)	Nelson St	EX SAN MH4	EX SAN MH5			2.720	-	244.80	-	0.180	-	0.100	29.80	354.40

Segment SAN Trib ID	Inflow/ Infiltration Area (ha)	Residential P/F M	Peak Residential Sanitary Flow Q(p) (275 L/c/d) (L/s)	Commercial P/F M	Peak Commercial Sanitary Flow Q (p) (24.75 m3/ha/d) (L/s)	Unit Inflow/ Infiltration I/I (0.28 L/Sec/ha)	Segment Inflow/ Infiltration Q(i) (L/s)	Accumulative Inflow/ Infiltration Q(i) (L/s)	Accumulative Sanitary Flow Q(d) (L/s)	Pipe Length L (m)	Size D (mm)	Slope S (%)	Shape	Full Flow Capacity Qcap (L/s)	Full Flow Velocity V (m/s)	% Full Q(d)/Qcap
1 (Trib of MH 1)	0.98	4.50	-	3.47	0.42	0.280	0.27	0.27	0.69	67.49	300	0.50	circle	68.38	0.97	1.01%
2 (Trib of MH 2)	0.26	4.38	0.28	3.47	0.42	0.280	0.07	0.35	1.05	69.11	300	0.51	circle	69.06	0.98	1.52%
3 (Trib of MH 3)	8.27	4.06	4.20	3.29	2.56	0.280	2.32	2.66	9.43	75.20	300	0.49	circle	67.69	0.96	13.93%
4 (Trib of MH 4)	0.41	4.05	4.56	3.29	2.56	0.280	0.11	2.78	9.91	69.57	300	1.08	circle	100.49	1.42	9.86%

Flow Calculation Criteria

(Unit Flowfrom Table 3-1 and 3-2, Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2015)
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 2015)

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
Community Services (School)	40 P/ha

(Peaking Factor from Section 3.2.3., Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2015)

PEAKING FACTOR (Residential)	M =1 + 14/(4+(P/1000^0.5))
PEAKING FACTOR (Commercial)	M =0.8[1 + 14/(4+(P/1000^0.5))]
PEAK DESIGN FLOW,	Q(d) = Q(p) + Q(i) L / Sec.
PIPE ROUGHNESS,	n = 0.013 For Manning's Equation

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

DOWNSTREAM SANITARY SEWERS (Post-Development)

Site location: Subject Proposed Development - 2380 Lakeshore Road West

Ref# PN 18219

Location				Existing + Proposed Industrial/Commercial					Existing + Proposed Residential Population						
Segment SAN Trib ID	Street Name	US Node	DS Node	Commercial (ha)	School (ha)	Acc've Area (ha)	Population (Person)	Acc've Pop'n (Person)	Apartments (2.7PPU) (U)	Apartments (>6 St)(Ha)	Apartments (<6 St)(Ha)	Townhouse (Ha)	Detached (Ha)	Population (Person)	Acc've Pop'n (Person)
1 (Trib of MH 1)	Lakeshore Rd W	EX SAN MH1	EX SAN MH2	0.467		0.467	42.03	42.03	156.000	-	-	-	-	421.20	421.20
2 (Trib of MH 2)	Lakeshore Rd W	EX SAN MH2	EX SAN MH3			0.467	-	42.03	-	-	-	0.150	-	20.25	441.45
3 (Trib of MH 3)	Nelson St	EX SAN MH3	EX SAN MH4	2.300	-	2.767	207.00	249.03	-	-	-	1.020	3.030	304.35	745.80
4 (Trib of MH 4)	Nelson St	EX SAN MH4	EX SAN MH5			2.767	-	249.03	-	-	0.180	-	0.100	29.80	775.60

Segment SAN Trib ID	Inflow/ Infiltration Area (ha)	Residential P/F M	Peak Residential Sanitary Flow Q(p) (275 L/c/d) (L/s)	Commercial P/F M	Peak Commercial Sanitary Flow Q (p) (24.75 m3/ha/d) (L/s)	Unit Inflow/ Infiltration I/I (0.28 L/Sec/ha)	Segment Inflow/ Infiltration Q(i) (L/s)	Accumulative Inflow/ Infiltration Q(i) (L/s)	Accumulative Sanitary Flow Q(d) (L/s)	Pipe: Length L (m)	Pipe: Size D (mm)	Pipe: Slope S (%)	Pipe: Shape	Pipe: Full Flow Capacity Qcap (L/s)	Pipe: Full Flow Velocity V (m/s)	Pipe: % Full Q(d)/Qcap
1 (Trib of MH 1)	0.98	4.01	5.38	3.46	0.46	0.280	0.27	0.27	6.12	67.49	300	0.50	circle	68.38	0.97	8.9%
2 (Trib of MH 2)	0.26	4.00	5.62	3.46	0.46	0.280	0.07	0.35	6.43	69.11	300	0.51	circle	69.06	0.98	9.3%
3 (Trib of MH 3)	8.27	3.88	9.21	3.29	2.61	0.280	2.32	2.66	14.48	75.20	300	0.49	circle	67.69	0.96	21.4%
4 (Trib of MH 4)	0.41	3.87	9.55	3.29	2.61	0.280	0.11	2.78	14.93	69.57	300	1.08	circle	100.49	1.42	14.9%

Flow Calculation Criteria
(Unit Flow from Table 3-1 and 3-2, Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2015)
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 2015)

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
Community Services (School)	40 P/ha

(Peaking Factor from Section 3.2.3., Regional Municipality of Halton, Water and Wastewater Linear Design Manual, April 2015)

PEAKING FACTOR (Residential)	M =1 + 14/(4+(P/1000^0.5))
PEAKING FACTOR (Commercial)	M =0.8[1 + 14/(4+(P/1000^0.5))]
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) Terms of Reference & Available Infrastructure

There is an existing 375mm storm sewer beneath the north side of Lakeshore Road West. The following criteria is assumed based on the Town of Oakville's *Development Engineering Procedures and Guidelines Manual* (May 2005).

1) Quantity Control: Control 100-year post-development storm events to 5-year pre-development storm events
2) Quality Control: 50% TSS Removal by an Oil/Grit Separator or other such measure

Design storm data for the Town of Oakville 5-year and 100-year storms are shown below.

$$I_5 = 1170 / (5.8 + t)^{0.843}$$
$$I_{100} = 2150 / (5.7 + t)^{0.861}$$

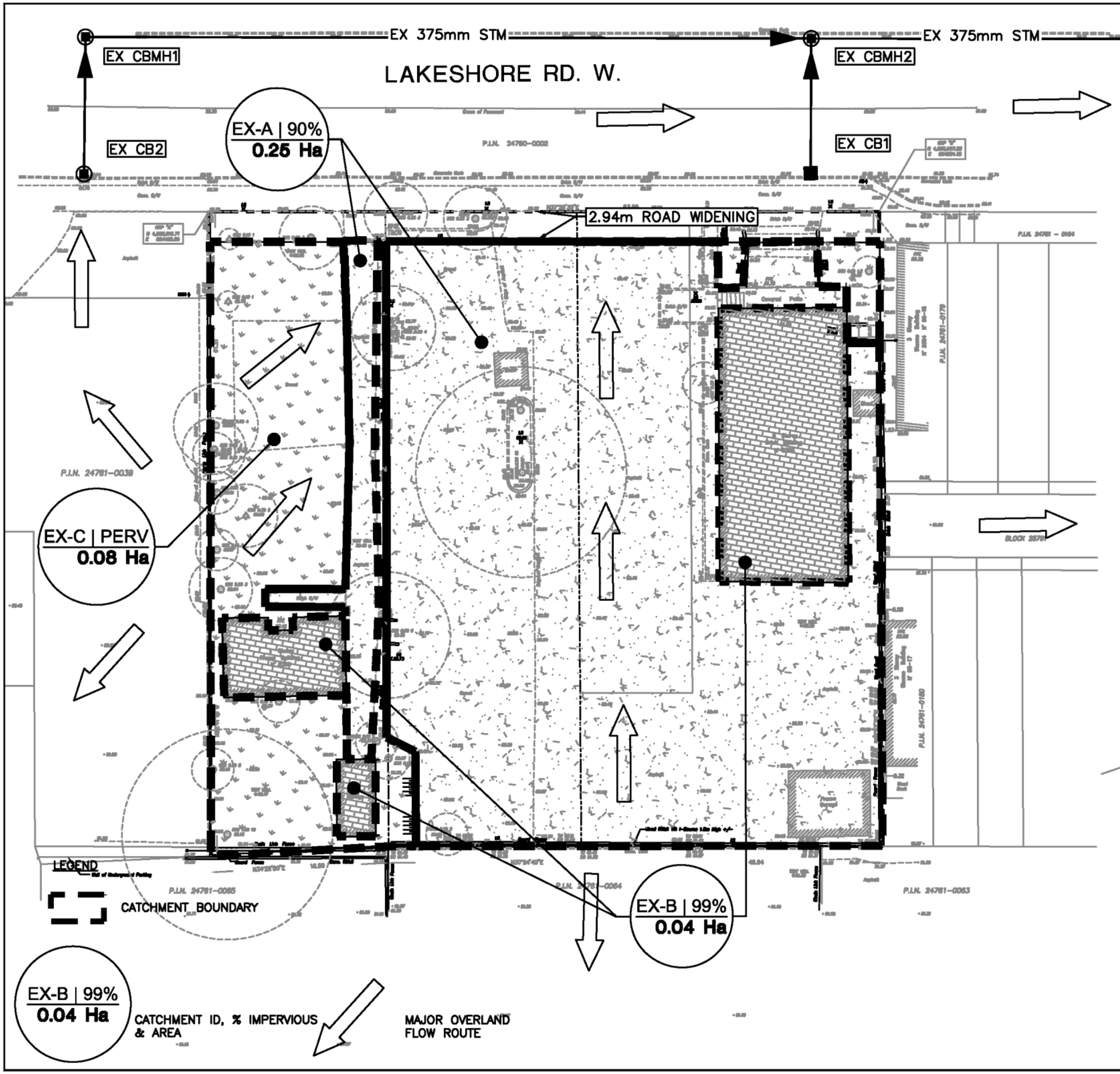
When time of concentration, *t*, is 10 minutes, the 5-year and 100-year rainfall intensities are as follows.

$$I_5 = 114.2 \text{ mm/hr}$$
$$I_{100} = 200.8 \text{ mm/hr}$$

ii) Allowable & Pre-Development Discharge Rate

The subject site drained in pre-development conditions as shown on the *Pre-Development Drainage Plan* on the following page. Pre-development catchment areas are delineated on that plan.

The site drained predominantly northerly to Lakeshore Road West in pre-development conditions. There is no evidence that any of the adjacent properties drained onto the subject site. All adjacent properties are graded to drain internally and appear to have internal catchbasins etc.



DRAWING :

PRE-DEVELOPMENT DRAINAGE PLAN

DATE:	PROJ. NO.:	SCALE:
JUNE 2018	18219	1:500

PROJECT :

PROPOSED RETIREMENT HOME DEVELOPMENT
2380 LAKESHORE RD. W.
OAKVILLE, 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

The site was modelled in the pre-development scenario using Visual OTTHYMO 2.3.2 to determine the pre-development 2-year storm runoff flow rates. Note that the pre-development catchment areas used to establish the allowable release rate conservatively considers the post-development 2.94m Lakeshore Road West road widening.

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.

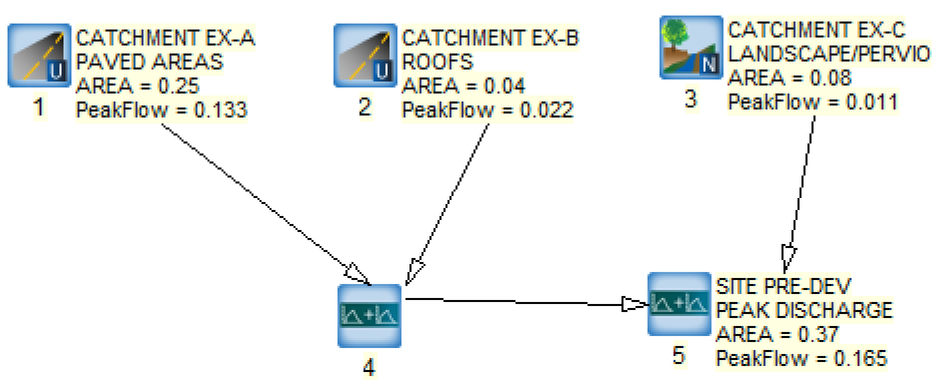
The catchment area statistics in the pre-development scenario are as follows.

TABLE 3 - Catchment Characteristics for Site, Pre-Development

Area I.D.	Area (ha)	Hydrology Method	% impervious	imperviousness directly connected %	Loss Method for Pervious Area	CN for Pervious Area	Initial Abstraction for Pervious (mm)	Time to peak (T_p)
EX-A Paved Surfaces	0.25	StandHyd	90	90	SCS	80	1	-
EX-B Roofs	0.04	StandHyd	99	99	SCS	80	1	-
EX-C Landscape/ Pervious	0.08	NashHyd	-	-	SCS	80	5	0.11

The pre-development Visual OTTHYMO Model is as follows. Refer to the output in Appendix B.

Figure 1 - Pre-Development Visual OTTHYMO Model (100-Year Storm)



The peak pre-development 2-year storm discharge rate on the 375mm storm sewer beneath Lakeshore Road West is as follows. Refer to the pre-development visual OTTHYMO output in Appendix B. These flow rates form the basis for the allowable release rate.

TABLE 4 – Pre-Development (allowable) Discharge

Outlet Location	5-Year Storm	100-Year Storm
Flow to Lakeshore Road West	90 L/s	165 L/s

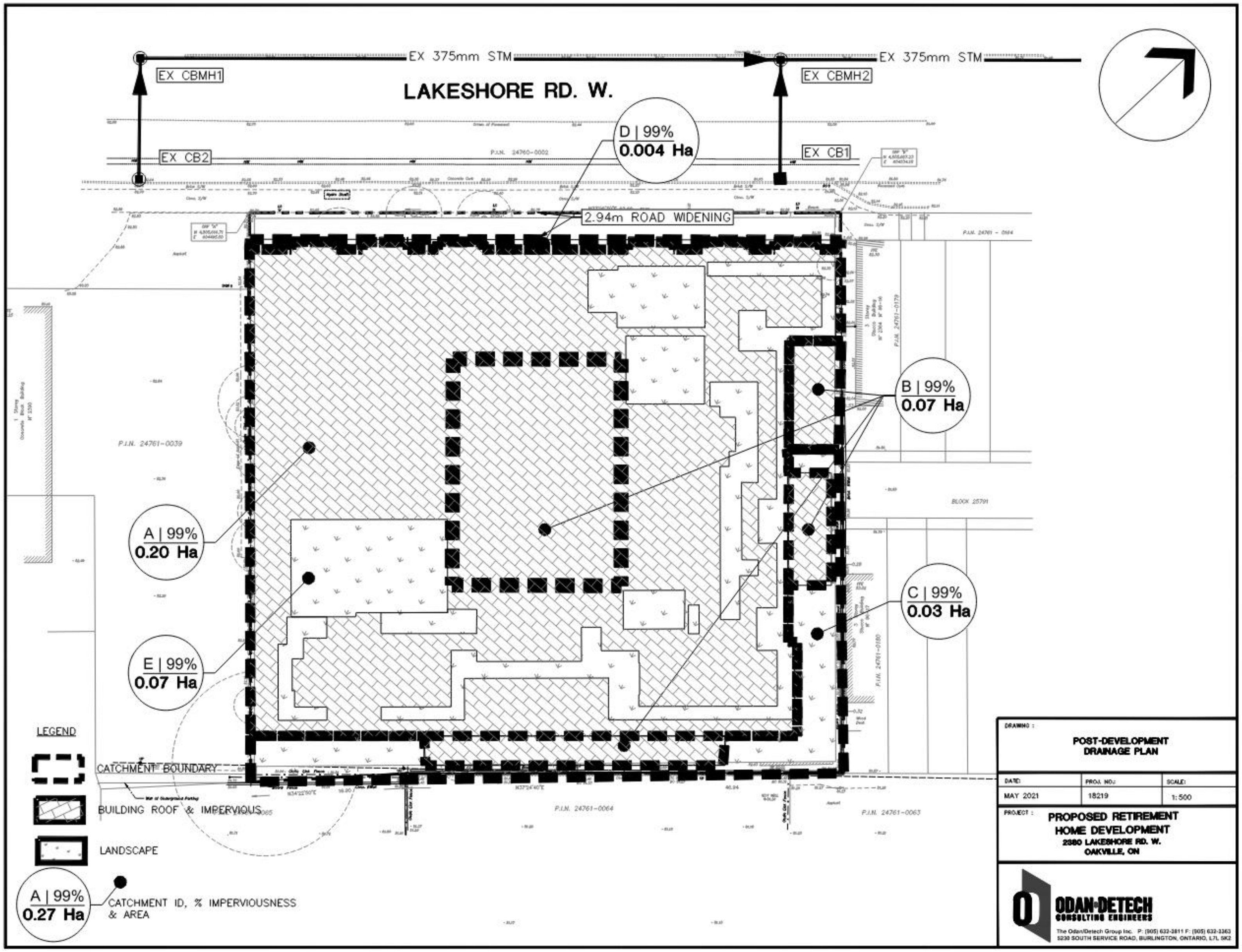
iii) Proposed Conditions & Post-Development Flow Analysis

City staff have not provided preconsultation comments regarding stormwater management quantity control criteria, therefore it is proposed to control storm flows 100-Year-Post-Development to 5-year-Pre-Development based on the pre-development flows established in Table 4, above.

The following table summarizes the parameters used in Visual OTTHYMO to characterize the post development catchment areas. Refer to the Post-Development Drainage Plan on the following page and the Post-Development Visual OTTHYMO Model thereafter.

TABLE 5 - Catchment Characteristics for Post-Developed Site

Area I.D.	Area (ha)	Hydrology Method	% impervious	imperviousness directly connected %	Loss Method for Pervious Area	CN for Pervious Area	Initial Abstraction for Pervious (mm)	Time to peak (T_p)
A – Tower Rooftop	0.20	StandHyd	99	99	SCS	80	1	-
B – Ground Level Paved	0.07	StandHyd	99	99	SCS	80	1	-
C – Landscape Areas	0.03	NashHyd	-	-	SCS	80	5	0.11
D – Uncontrolled Ground Level Paved	0.004	StandHyd	99	99	SCS	80	1	-
E – Tower Green Roof	0.07	StandHyd	60	60	SCS	80	1	-

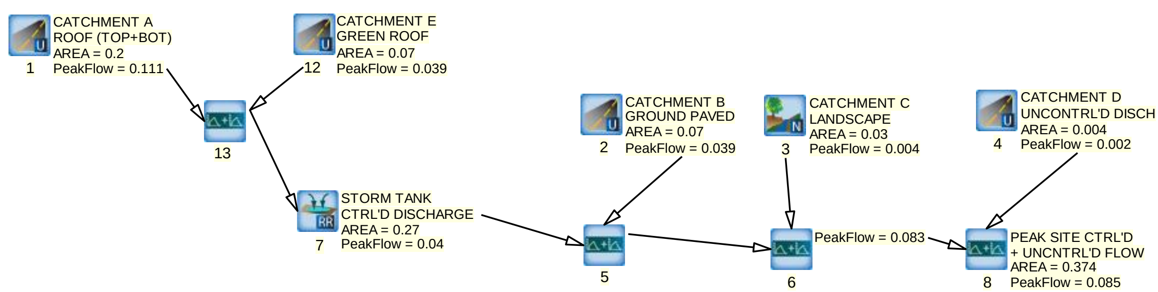

EX 375mm STM
EX 375mm STM
EX CBMH1
EX CBMH2
LAKESHORE RD. W.
EX CB2
EX CB1
D | 99%
0.004 Ha
2.94m ROAD WIDENING
B | 99%
0.07 Ha
A | 99%
0.20 Ha
C | 99%
0.03 Ha
E | 99%
0.07 Ha
LEGEND
CATCHMENT BOUNDARY
BUILDING ROOF & IMPERVIOUS
LANDSCAPE
A | 99%
0.27 Ha
CATCHMENT ID, % IMPERVIOUSNESS & AREA
DRAWING :
POST-DEVELOPMENT DRAINAGE PLAN
DATE: MAY 2021
PROJ. NO.: 18219
SCALE: 1:500
PROJECT : PROPOSED RETIREMENT HOME DEVELOPMENT
2380 LAKESHORE RD. W.
OAKVILLE, ON
ODAN-DETECH
CONSULTING ENGINEERS

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 foregoing catchment areas appear in the post-development Visual OTTHYMO Model, as follows. The model shows flows in a 100-year storm. Refer to the detailed Visual OTTHYMO Output in Appendix B for detailed results for both 5-year and 100-year storms.

Stormwater quantity controls will be provided in the basement (via storm tank) of the proposed building to provide controlled release

Figure 2 – Post-Development Visual OTTHYMO Model (100Y Storm Flows)



As evident above, the discharge to the Lakeshore Road West 375mm storm sewer is 85 L/s, which is less than the 5-year pre-development flow rate (90 L/s – Table 4), therefore the development is in compliance with the stormwater quantity control criteria identified above.

The following is a description of the SWM quantity control system via storm tank:

- storm tank will be a cast in place concrete structure located in the basement underneath the underground parking entrance ramp and detailed by Structural.
- Using a combination of impervious roof and pervious green roof, runoff form the building's roof will be directed (uncontrolled) via mechanical storm drains to the storm tank
- A mechanical sump pump will be installed in the storm tank, pumping at a release rate of 40 L/s to the Control MH
- The mechanical sump pump requires submersion in water to remain operational. A float valve will be used to activate the pump once incoming storm flows raise the water level above the pumps baseline.
- 100 year flows will result in an depth of approximately 1.50m within the storm water tank. The total height of the tank is 3.20m. Approximately 68 cu.m. is required for the tank volume. Total available volume in the tank is 136 cu.m.

Adequate stormwater storage is provided to the 100-year storm in the basement of the building based on the foregoing storm tank as follows in Table 6.

Catchment Areas B and C (Ground-level areas) are to drain directly to the Jellyfish Filter, uncontrolled – as shown in Figure 2, those catchment areas do not contribute to the 100-Year storm tank.

TABLE 6 – Stormwater Storage

	5 Yr. Storm (m^3)	**100 Yr. Storm** (m^3)
Required Storage Volume	28	68
Provided Storage Volume via Storm Tank	136	

The controlled and uncontrolled discharge from the site is as follows based on the Visual OTTHYMO Model. The site's peak storm flow in the 100-year storm (85 L/s) is less than the 5-year pre-development storm flow (90 L/s – Table 4) therefore the development meets the stormwater quantity control criteria.

TABLE 7 - Summary of Discharge from Site

	5 Yr. Storm (L/s)	**100 Yr. Storm** (L/s)
Controlled flow from Storm Tank (Catchment A&E)	40	40
Flow from ground-level paved areas (Catchment B)	22	39
Flow from ground-level landscape areas (Catchment C)	2	4
Flow from ground level paved (overland to Lakeshore Rd) (Catchment D)	1	2
Entire Development Peak Flow (Controlled + Uncontrolled)*	***65***	***85***

**Note: The entire development's Peak Discharge Rate to Lakeshore Rd. W. is not the sum of the peak flows from all tributary areas within the site. Rather, the peak discharge considers the different peaking time between the attenuated/controlled discharge and the uncontrolled drainage areas based on "overlaying" the hydrographs in the Visual OTTHYMO Model.*

iv) Erosion Control

Erosion and sediment control will be implemented on-site prior to construction and be maintained through the entire duration of construction. Erosion control measures to be implemented are:

- silt fence around the entire site
- sediment socks within existing and proposed catchbasins
- an entrance mud mat for trucks
- daily cleaning and weekly washing of roads

v) Stormwater Quality Control

The City of Oakville's *Development Engineering Procedures & Guidelines Manual* states in Section 6.2.4, *Storm Drainage Criteria*, that:

> *Quality treatment of stormwater is required. The level of treatment is to be determined per the receiving system (see Halton Conservation). Wet Ponds, Oil/Grit Separators and Landscape Filter Strips are acceptable methods.*

It is accordingly proposed to provide a Hydrowork HD-5 Oil/Grit Separator which is certified by the Canadian Environmental Technology Verification (CETV) program and sized to provide 80% TSS Removal with a percent imperviousness of 85%. The Oil/Grit design report and CETV verification statement is provided here in Appendix B. Refer to the Site Servicing Plan for the location of the OGS.

Alternatively any OGS that provides the same level of treatment and is able to be located in the available space allocated for this unit will be considered for use on the site during tender and construction. If any change is made the Town will be notified accordingly.

Town engineering review staff stated in the SPA review memorandum of October 3, 2018 that stormwater quality control criteria is to provide 80% TSS Removal for the whole development. 88% TSS Removal is provided for the whole development using the strategy below and based on an area-weighted approach as follows, therefore the quality control criteria is satisfied.

Catchment ID	Area (Ha)	Area (% of total)	TSS Removal	Weighted TSS Removal (%) (=%A x %TSSR)
Catchment 'A'&'E' – Reg. Roofs	0.27	73%	90%	66%
Catchment 'B' – Paved areas subject to winter maintenance – drains to Stormceptor OGS	0.06	15%	80%	12%
Catchment 'C' – Landscape Surfaces	0.04	11%	90%	10%
Catchment 'D' – Paved areas – drain uncontrolled to Lakeshore Road	0.004	1%	0%	0%
Total	***0.37***			***88%***

6.0 CONCLUSIONS

From the foregoing investigation, the site is serviceable utilizing existing sanitary, storm and watermain infrastructure within and 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 8 - Summary

Peak Sanitary Discharge (L/s)	3.6 L/s (City criteria)
Proposed Sanitary Service	150mm at 2.00%
Receiving Sanitary Sewer	Lakeshore Rd. W. 300mm Sanitary
Development Water Demand (Fire + Domestic)	3200 USGM
Proposed Fire Service	150mm Fire Service
Proposed Domestic Service	Branch 100mm Domestic
Allowable release rate from site	90 L/s (5-Y Pre-Development)
Proposed release rate from site to (100 year storm)	85 L/s (100-Y Post-Development)
Quantity Control	Stormwater Management Tank

7.0 REFERENCES

1. Town of Oakville *Development Engineering Procedures and Guidelines Manual* (May 2005).

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

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

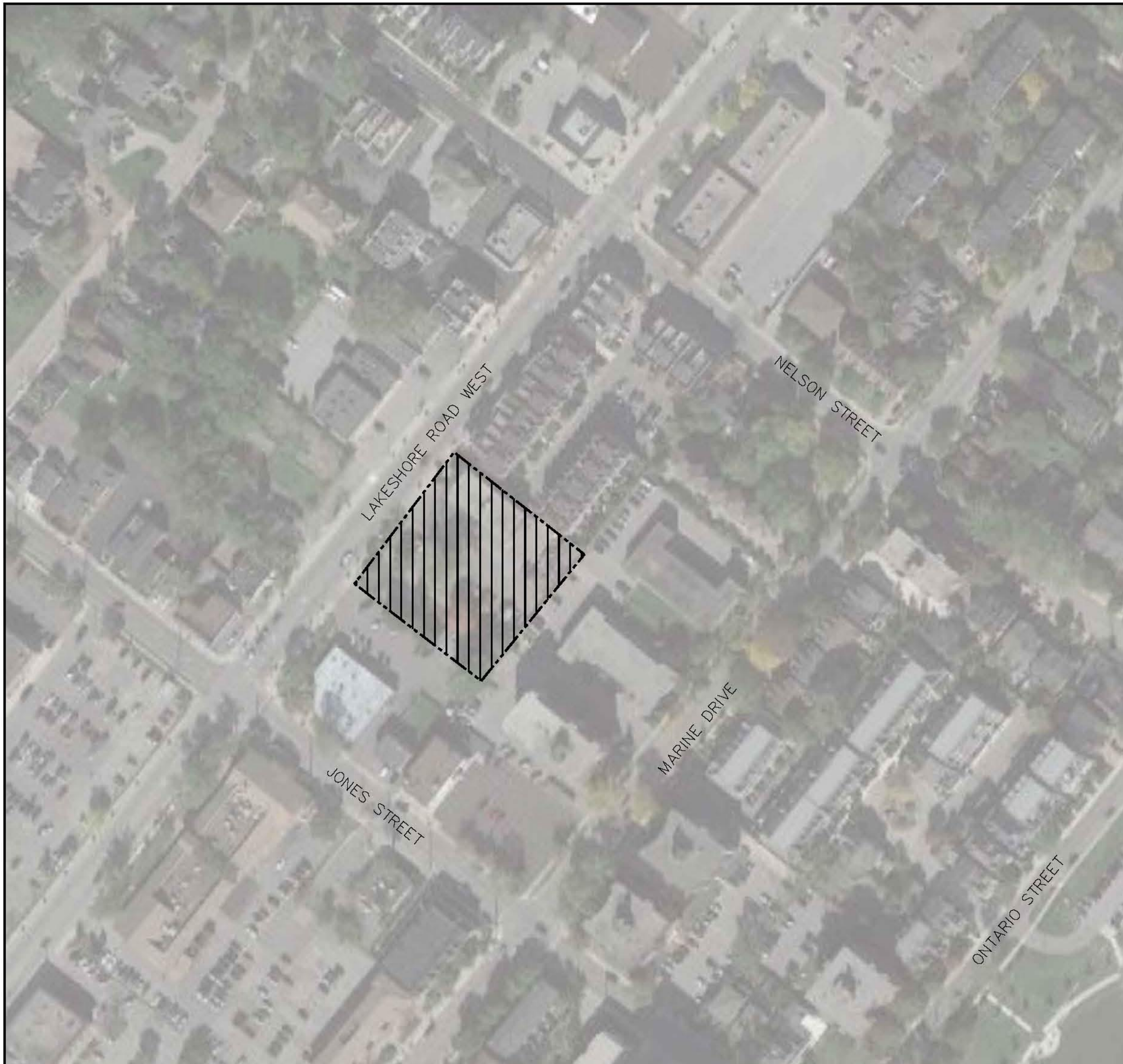
Respectfully Submitted;
The Odan Detech Group Inc.

June 30, 2023

Paul Hecimovic, P.Eng.	Mark Harris, Dipl. Tech.

APPENDIX A

Existing Site	Aerial view of Site and surrounding areas
Site Plan	by Michael Spaziani Architect Inc.
Development statistics	by Michael Spaziani Architect Inc.



LEGEND

PROPERTY LINE

DRAWING :

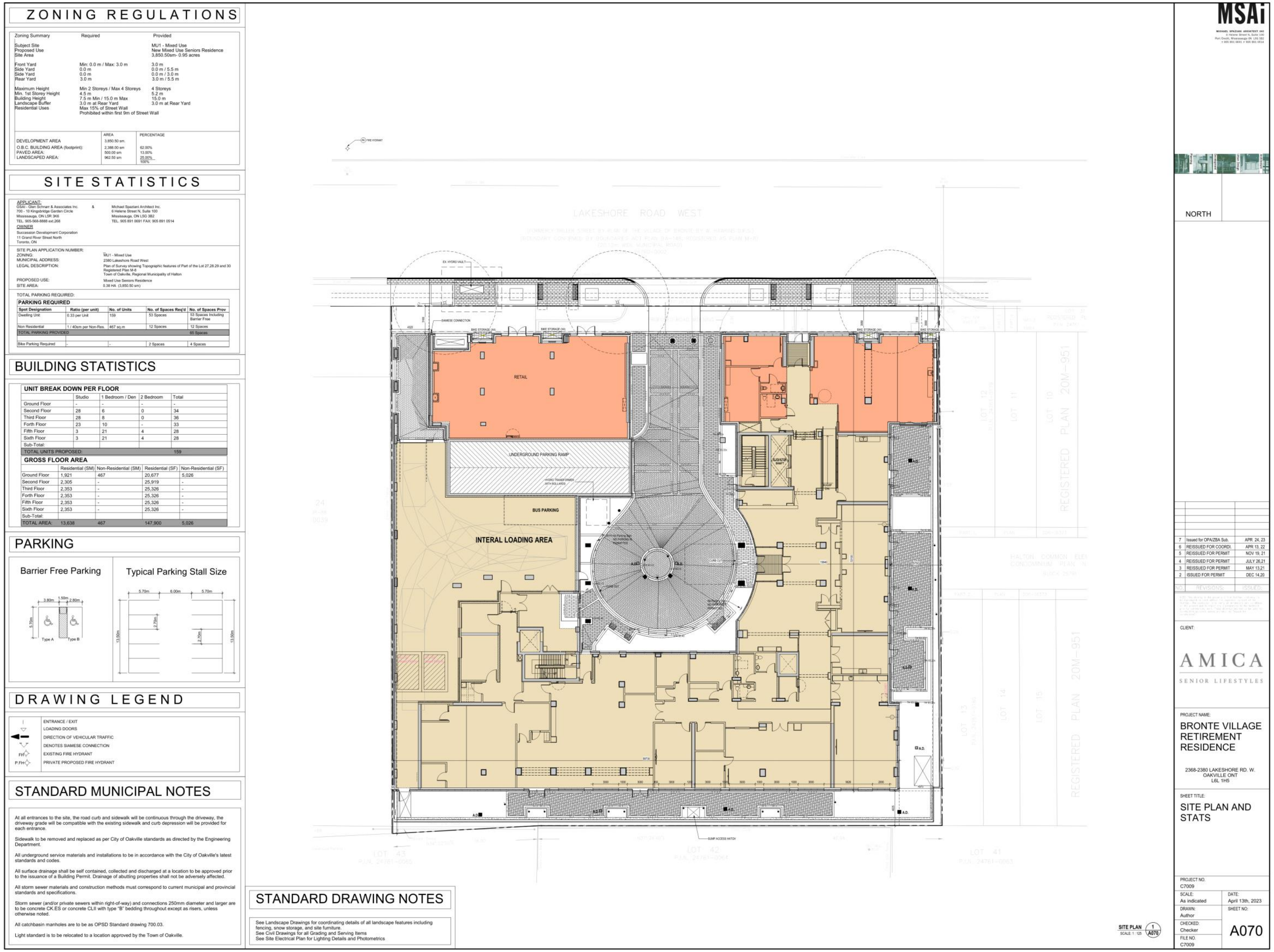
KEY PLAN

DATE:	PROJ. NO.:	SCALE:
JUNE 2018	18219	N.T.S.

PROJECT :

PROPOSED RETIREMENT RESIDENCE
2380 LAKESHORE ROAD WEST
OAKVILLE, 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


ZONING REGULATIONS
SITE STATISTICS
BUILDING STATISTICS
PARKING
Barrier Free Parking
Typical Parking Stall Size
DRAWING LEGEND
STANDARD MUNICIPAL NOTES
STANDARD DRAWING NOTES
INTERAL LOADING AREA
MSAi
NORTH
AMICA
SENIOR LIFESTYLES
BRONTE VILLAGE
RETIREMENT
RESIDENCE
SITE PLAN AND
STATS
A070

APPENDIX B

Pre-Development Visual OTTHYMO Model Output 5-year storm & 100-year storm

Post-Development Visual OTTHYMO Model Output 5-year storm & 100-year storm

HydroWorks – HD 5 Sizing Report

CETV Verification Statement

Pre-Development Visual OTTHYMO Output (5-year & 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:\Program Files (x86)\Visual OTTHYMO 2.3.3\voin.dat
  Output  filename: P:\2018\18219\Visual OTTHYMO\Rev1\18219 VO2\Pre-Dev.out
  Summary filename: P:\2018\18219\Visual OTTHYMO\Rev1\18219 VO2\Pre-Dev.sum


DATE: 7/2/2019                             TIME: 10:14:03 AM

USER:


COMMENTS: ____________________________________________________________

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

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1170.000
| Ptotal= 45.17 mm |                          B=   5.800
--------------------                          C=    .843
                        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    2.32 |  1.17   24.01 |  2.17    6.09 |  3.17    2.81
                  .33    2.70 |  1.33  114.21 |  2.33    5.07 |  3.33    2.59
                  .50    3.24 |  1.50   32.30 |  2.50    4.35 |  3.50    2.40
                  .67    4.08 |  1.67   15.74 |  2.67    3.82 |  3.67    2.24
                  .83    5.57 |  1.83   10.30 |  2.83    3.41 |  3.83    2.10
                 1.00    8.96 |  2.00    7.65 |  3.00    3.08 |  4.00    1.98


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0003) |   Area    (ha)=    .08   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .20

     Unit Hyd Qpeak  (cms)=    .015

     PEAK FLOW       (cms)=    .004 (i)
     TIME TO PEAK    (hrs)=   1.500
     RUNOFF VOLUME    (mm)=  15.155
     TOTAL RAINFALL   (mm)=  45.171
     RUNOFF COEFFICIENT   =    .335

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0002) |   Area    (ha)=    .04
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
```

```
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .04          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      16.30        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    2.32 | 1.083   24.01 | 2.083    6.09 |  3.08    2.81
                 .167    2.32 | 1.167   24.01 | 2.167    6.09 |  3.17    2.81
                 .250    2.70 | 1.250  114.21 | 2.250    5.07 |  3.25    2.59
                 .333    2.70 | 1.333  114.21 | 2.333    5.07 |  3.33    2.59
                 .417    3.24 | 1.417   32.30 | 2.417    4.35 |  3.42    2.40
                 .500    3.24 | 1.500   32.30 | 2.500    4.35 |  3.50    2.40
                 .583    4.08 | 1.583   15.74 | 2.583    3.82 |  3.58    2.24
                 .667    4.08 | 1.667   15.74 | 2.667    3.82 |  3.67    2.24
                 .750    5.57 | 1.750   10.30 | 2.750    3.41 |  3.75    2.10
                 .833    5.57 | 1.833   10.30 | 2.833    3.41 |  3.83    2.10
                 .917    8.96 | 1.917    7.65 | 2.917    3.08 |  3.92    1.98
                1.000    8.96 | 2.000    7.65 | 3.000    3.08 |  4.00    1.98

     Max.Eff.Inten.(mm/hr)=     114.21        42.13
                over (min)        5.00         5.00
     Storage Coeff.  (min)=        .82 (ii)    1.88 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .32
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .01          .00          .013 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      44.17        18.12         43.91
     TOTAL RAINFALL   (mm)=      45.17        45.17         45.17
     RUNOFF COEFFICIENT   =        .98          .40           .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  (0001) |   Area    (ha)=    .25
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .22          .03
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      40.80        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     114.21        42.13
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.41 (ii)    4.19 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .24
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .07          .00          .074 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      44.17        18.12         41.56
     TOTAL RAINFALL   (mm)=      45.17        45.17         45.17
     RUNOFF COEFFICIENT   =        .98          .40           .92

***** 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   (0004) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0002):      .04     .013     1.33    43.91
        + ID2= 2 (0001):      .25     .074     1.33    41.56
```

```
          ==================================================
          ID = 3 (0004):      .29    .087     1.33    41.88

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0005) |
|   1 +  2 =  3    |        AREA    QPEAK    TPEAK     R.V.
--------------------        (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0003):      .08    .004     1.50    15.15
        + ID2= 2 (0004):      .29    .087     1.33    41.88
          ==================================================
          ID = 3 (0005):      .37    .090     1.33    35.97

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=2150.000
| Ptotal= 75.20 mm |                          B=   5.700
--------------------                          C=    .861
                        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    3.49 |  1.17   39.75 |  2.17    9.50 |  3.17    4.26
                  .33    4.08 |  1.33  200.80 |  2.33    7.85 |  3.33    3.91
                  .50    4.93 |  1.50   54.01 |  2.50    6.70 |  3.50    3.62
                  .67    6.26 |  1.67   25.55 |  2.67    5.85 |  3.67    3.37
                  .83    8.66 |  1.83   16.41 |  2.83    5.19 |  3.83    3.15
                 1.00   14.21 |  2.00   12.04 |  3.00    4.68 |  4.00    2.96


-------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0003) |   Area    (ha)=    .08   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .20

     Unit Hyd Qpeak  (cms)=    .015

     PEAK FLOW       (cms)=    .011 (i)
     TIME TO PEAK    (hrs)=   1.500
     RUNOFF VOLUME    (mm)=  35.894
     TOTAL RAINFALL   (mm)=  75.204
     RUNOFF COEFFICIENT   =    .477

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0002) |   Area    (ha)=    .04
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .04          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      16.30        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    3.49 | 1.083   39.75 | 2.083    9.50 |  3.08    4.26
                 .167    3.49 | 1.167   39.75 | 2.167    9.50 |  3.17    4.26
                 .250    4.08 | 1.250  200.80 | 2.250    7.85 |  3.25    3.91
                 .333    4.08 | 1.333  200.80 | 2.333    7.85 |  3.33    3.91
                 .417    4.93 | 1.417   54.01 | 2.417    6.70 |  3.42    3.62
                 .500    4.93 | 1.500   54.01 | 2.500    6.70 |  3.50    3.62
                 .583    6.26 | 1.583   25.55 | 2.583    5.85 |  3.58    3.37
                 .667    6.26 | 1.667   25.55 | 2.667    5.85 |  3.67    3.37
```

```
               .750    8.66 | 1.750   16.41 | 2.750    5.19 |  3.75    3.15
               .833    8.66 | 1.833   16.41 | 2.833    5.19 |  3.83    3.15
               .917   14.21 | 1.917   12.04 | 2.917    4.68 |  3.92    2.96
              1.000   14.21 | 2.000   12.04 | 3.000    4.68 |  4.00    2.96

     Max.Eff.Inten.(mm/hr)=     200.80       210.66
                over (min)        5.00         5.00
     Storage Coeff.  (min)=        .65 (ii)    1.50 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .33
                                                           *TOTALS*
     PEAK FLOW       (cms)=        .02          .00           .022 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33           1.33
     RUNOFF VOLUME    (mm)=      74.20        39.99          73.86
     TOTAL RAINFALL   (mm)=      75.20        75.20          75.20
     RUNOFF COEFFICIENT   =        .99          .53            .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  (0001) |   Area    (ha)=    .25
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .22          .03
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      40.80        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     200.80       103.62
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.13 (ii)    3.35 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .26
                                                           *TOTALS*
     PEAK FLOW       (cms)=        .13          .01           .133 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33           1.33
     RUNOFF VOLUME    (mm)=      74.20        39.99          70.77
     TOTAL RAINFALL   (mm)=      75.20        75.20          75.20
     RUNOFF COEFFICIENT   =        .99          .53            .94

***** 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   (0004) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0002):      .04    .022     1.33    73.86
        + ID2= 2 (0001):      .25    .133     1.33    70.77
          ===============================================
          ID = 3 (0004):      .29    .155     1.33    71.20

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0005) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0003):      .08    .011     1.50    35.89
        + ID2= 2 (0004):      .29    .155     1.33    71.20
          ===============================================
          ID = 3 (0005):      .37    .165     1.33    63.37

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
 FINISH
```

Post-Development Visual OTTHYMO Output (5-year & 100-year Storms)

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

       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:\2018\18219\Visual OTTHYMO\Rev4\18219 VO2\Post-Dev.out
  Summary filename: P:\2018\18219\Visual OTTHYMO\Rev4\18219 VO2\Post-Dev.sum

DATE: 6/30/2023                           TIME: 9:44:55 AM

USER:

COMMENTS: ____________________________________________________________

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

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1170.000
| Ptotal= 45.17 mm |                          B=   5.800
--------------------                          C=    .843
                        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    2.32 |  1.17   24.01 |  2.17    6.09 |  3.17    2.81
                  .33    2.70 |  1.33  114.21 |  2.33    5.07 |  3.33    2.59
                  .50    3.24 |  1.50   32.30 |  2.50    4.35 |  3.50    2.40
                  .67    4.08 |  1.67   15.74 |  2.67    3.82 |  3.67    2.24
                  .83    5.57 |  1.83   10.30 |  2.83    3.41 |  3.83    2.10
                 1.00    8.96 |  2.00    7.65 |  3.00    3.08 |  4.00    1.98

-------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0003) |   Area    (ha)=    .03   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .20

     Unit Hyd Qpeak  (cms)=    .006

     PEAK FLOW       (cms)=    .002 (i)
     TIME TO PEAK    (hrs)=   1.500
     RUNOFF VOLUME    (mm)=  15.137
     TOTAL RAINFALL   (mm)=  45.171
     RUNOFF COEFFICIENT   =    .335

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0012) |   Area    (ha)=    .07
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
```

```
     Surface Area     (ha)=        .07          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      21.60        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    2.32 | 1.083   24.01 | 2.083    6.09 |  3.08    2.81
                 .167    2.32 | 1.167   24.01 | 2.167    6.09 |  3.17    2.81
                 .250    2.70 | 1.250  114.21 | 2.250    5.07 |  3.25    2.59
                 .333    2.70 | 1.333  114.21 | 2.333    5.07 |  3.33    2.59
                 .417    3.24 | 1.417   32.30 | 2.417    4.35 |  3.42    2.40
                 .500    3.24 | 1.500   32.30 | 2.500    4.35 |  3.50    2.40
                 .583    4.08 | 1.583   15.74 | 2.583    3.82 |  3.58    2.24
                 .667    4.08 | 1.667   15.74 | 2.667    3.82 |  3.67    2.24
                 .750    5.57 | 1.750   10.30 | 2.750    3.41 |  3.75    2.10
                 .833    5.57 | 1.833   10.30 | 2.833    3.41 |  3.83    2.10
                 .917    8.96 | 1.917    7.65 | 2.917    3.08 |  3.92    1.98
                1.000    8.96 | 2.000    7.65 | 3.000    3.08 |  4.00    1.98

     Max.Eff.Inten.(mm/hr)=     114.21       259.04
                over (min)        5.00         5.00
     Storage Coeff.  (min)=        .97 (ii)    2.03 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .31
                                                             *TOTALS*
     PEAK FLOW       (cms)=        .02          .00            .022 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33            1.33
     RUNOFF VOLUME    (mm)=      44.17        18.12           43.91
     TOTAL RAINFALL   (mm)=      45.17        45.17           45.17
     RUNOFF COEFFICIENT   =        .98          .40             .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  (0001) |   Area    (ha)=    .20
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .20          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      36.50        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     114.21       129.52
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.32 (ii)    2.39 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .30
                                                             *TOTALS*
     PEAK FLOW       (cms)=        .06          .00            .063 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33            1.33
     RUNOFF VOLUME    (mm)=      44.17        18.12           43.91
     TOTAL RAINFALL   (mm)=      45.17        45.17           45.17
     RUNOFF COEFFICIENT   =        .98          .40             .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  (0002) |   Area    (ha)=    .07
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .07          .00
     Dep. Storage     (mm)=       1.00         1.00
```

```
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      21.60        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     114.21        64.76
                over (min)        5.00         5.00
     Storage Coeff.  (min)=        .97 (ii)    2.03 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .31
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .02          .00          .022 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      44.17        18.12         43.91
     TOTAL RAINFALL   (mm)=      45.17        45.17         45.17
     RUNOFF COEFFICIENT   =        .98          .40           .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  (0004) |   Area    (ha)=    .00
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .00          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=       5.20        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     114.21        42.13
                over (min)        5.00         5.00
     Storage Coeff.  (min)=        .41 (ii)    1.48 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .33
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .00          .00          .001 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      44.17        18.12         36.01
     TOTAL RAINFALL   (mm)=      45.17        45.17         45.17
     RUNOFF COEFFICIENT   =        .98          .40           .80

***** 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   (0013) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0012):      .07    .022     1.33    43.91
        + ID2= 2 (0001):      .20    .063     1.33    43.91
          ================================================
          ID = 3 (0013):      .27    .085     1.33    43.91

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0007) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0001    |    .0401      .0136
                          .0400      .0002    |    .0000      .0000

                                  AREA      QPEAK       TPEAK       R.V.
                                  (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0013)        .270       .085        1.33       43.91
     OUTFLOW: ID= 1 (0007)        .270       .040        1.42       43.55

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 47.00
                  TIME SHIFT OF PEAK FLOW         (min)=  5.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0028
```

```
--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0005) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0007):      .27    .040     1.42    43.55
        + ID2= 2 (0002):      .07    .022     1.33    43.91
          ==================================================
          ID = 3 (0005):      .34    .062     1.33    43.62

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0006) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0003):      .03    .002     1.50    15.14
        + ID2= 2 (0005):      .34    .062     1.33    43.62
          ==================================================
          ID = 3 (0006):      .37    .063     1.33    41.31

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0008) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0006):      .37    .063     1.33    41.31
        + ID2= 2 (0004):      .00    .001     1.33    36.01
          ==================================================
          ID = 3 (0008):      .37    .065     1.33    41.25

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=2150.000
| Ptotal= 75.20 mm |                          B=   5.700
--------------------                          C=    .861
                        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    3.49 |  1.17   39.75 |  2.17    9.50 |  3.17    4.26
                  .33    4.08 |  1.33  200.80 |  2.33    7.85 |  3.33    3.91
                  .50    4.93 |  1.50   54.01 |  2.50    6.70 |  3.50    3.62
                  .67    6.26 |  1.67   25.55 |  2.67    5.85 |  3.67    3.37
                  .83    8.66 |  1.83   16.41 |  2.83    5.19 |  3.83    3.15
                 1.00   14.21 |  2.00   12.04 |  3.00    4.68 |  4.00    2.96


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0003) |   Area    (ha)=    .03   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .20

     Unit Hyd Qpeak  (cms)=    .006

     PEAK FLOW       (cms)=    .004 (i)
     TIME TO PEAK    (hrs)=   1.500
     RUNOFF VOLUME    (mm)=  35.892
     TOTAL RAINFALL   (mm)=  75.204
     RUNOFF COEFFICIENT   =    .477

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0012) |   Area    (ha)=    .07
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
```

```
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .07          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      21.60        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    3.49 | 1.083   39.75 | 2.083    9.50 |  3.08    4.26
                 .167    3.49 | 1.167   39.75 | 2.167    9.50 |  3.17    4.26
                 .250    4.08 | 1.250  200.80 | 2.250    7.85 |  3.25    3.91
                 .333    4.08 | 1.333  200.80 | 2.333    7.85 |  3.33    3.91
                 .417    4.93 | 1.417   54.01 | 2.417    6.70 |  3.42    3.62
                 .500    4.93 | 1.500   54.01 | 2.500    6.70 |  3.50    3.62
                 .583    6.26 | 1.583   25.55 | 2.583    5.85 |  3.58    3.37
                 .667    6.26 | 1.667   25.55 | 2.667    5.85 |  3.67    3.37
                 .750    8.66 | 1.750   16.41 | 2.750    5.19 |  3.75    3.15
                 .833    8.66 | 1.833   16.41 | 2.833    5.19 |  3.83    3.15
                 .917   14.21 | 1.917   12.04 | 2.917    4.68 |  3.92    2.96
                1.000   14.21 | 2.000   12.04 | 3.000    4.68 |  4.00    2.96

     Max.Eff.Inten.(mm/hr)=     200.80       103.62
                over (min)        5.00         5.00
     Storage Coeff.  (min)=        .77 (ii)     1.62 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .32
                                                             *TOTALS*
     PEAK FLOW       (cms)=        .04          .00            .039 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33            1.33
     RUNOFF VOLUME    (mm)=      74.20        39.99           73.86
     TOTAL RAINFALL   (mm)=      75.20        75.20           75.20
     RUNOFF COEFFICIENT   =        .99          .53             .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  (0001) |   Area    (ha)=    .20
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .20          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      36.50        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     200.80       103.62
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.06 (ii)     1.91 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .32
                                                             *TOTALS*
     PEAK FLOW       (cms)=        .11          .00            .111 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33            1.33
     RUNOFF VOLUME    (mm)=      74.20        39.99           73.86
     TOTAL RAINFALL   (mm)=      75.20        75.20           75.20
     RUNOFF COEFFICIENT   =        .99          .53             .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  (0002) |   Area    (ha)=    .07
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
```

```
     Surface Area     (ha)=        .07          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      21.60        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     200.80       103.62
                over (min)        5.00         5.00
     Storage Coeff.  (min)=        .77 (ii)    1.62 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .32
                                                         *TOTALS*
     PEAK FLOW       (cms)=        .04          .00         .039 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33         1.33
     RUNOFF VOLUME    (mm)=      74.20        39.99        73.86
     TOTAL RAINFALL   (mm)=      75.20        75.20        75.20
     RUNOFF COEFFICIENT   =        .99          .53          .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  (0004) |   Area    (ha)=    .00
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .00          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=       5.20        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     200.80       103.62
                over (min)        5.00         5.00
     Storage Coeff.  (min)=        .33 (ii)    1.18 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .34
                                                         *TOTALS*
     PEAK FLOW       (cms)=        .00          .00         .002 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33         1.33
     RUNOFF VOLUME    (mm)=      74.20        39.99        65.29
     TOTAL RAINFALL   (mm)=      75.20        75.20        75.20
     RUNOFF COEFFICIENT   =        .99          .53          .87

***** 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   (0013) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0012):      .07    .039     1.33    73.86
        + ID2= 2 (0001):      .20    .111     1.33    73.86
          ===============================================
          ID = 3 (0013):      .27    .150     1.33    73.86

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0007) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .0000      .0001   |    .0401       .0136
                           .0400      .0002   |    .0000       .0000

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0013)       .270       .150        1.33       73.86
     OUTFLOW: ID= 1 (0007)       .270       .040        1.50       73.51

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 26.70
```

```
                   TIME SHIFT OF PEAK FLOW         (min)= 10.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0068

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

--------------------
| ADD HYD   (0005) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0007):      .27    .040     1.50    73.51
        + ID2= 2 (0002):      .07    .039     1.33    73.86
          ==================================================
          ID = 3 (0005):      .34    .079     1.33    73.58

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0006) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0003):      .03    .004     1.50    35.89
        + ID2= 2 (0005):      .34    .079     1.33    73.58
          ==================================================
          ID = 3 (0006):      .37    .083     1.33    70.52

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0008) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0006):      .37    .083     1.33    70.52
        + ID2= 2 (0004):      .00    .002     1.33    65.29
          ==================================================
          ID = 3 (0008):      .37    .085     1.33    70.47

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

 FINISH
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```

Hydroworks Sizing Summary

Retirement Home
2380 Lakeshore Rd W, Oakville

06-30-2023

Recommended Size: HydroDome HD 5

A HydroDome HD 5 is recommended to provide 80 % annual TSS removal based on a drainage area of .37 (ha) with an imperviousness of 85 % and Toronto Central, Ontario rainfall for the ETV/NJDEP particle size distribution.

The recommended HydroDome HD 5 treats 100 % of the annual runoff and provides 81 % annual TSS removal for the Toronto Central rainfall records and ETV/NJDEP particle size distribution.

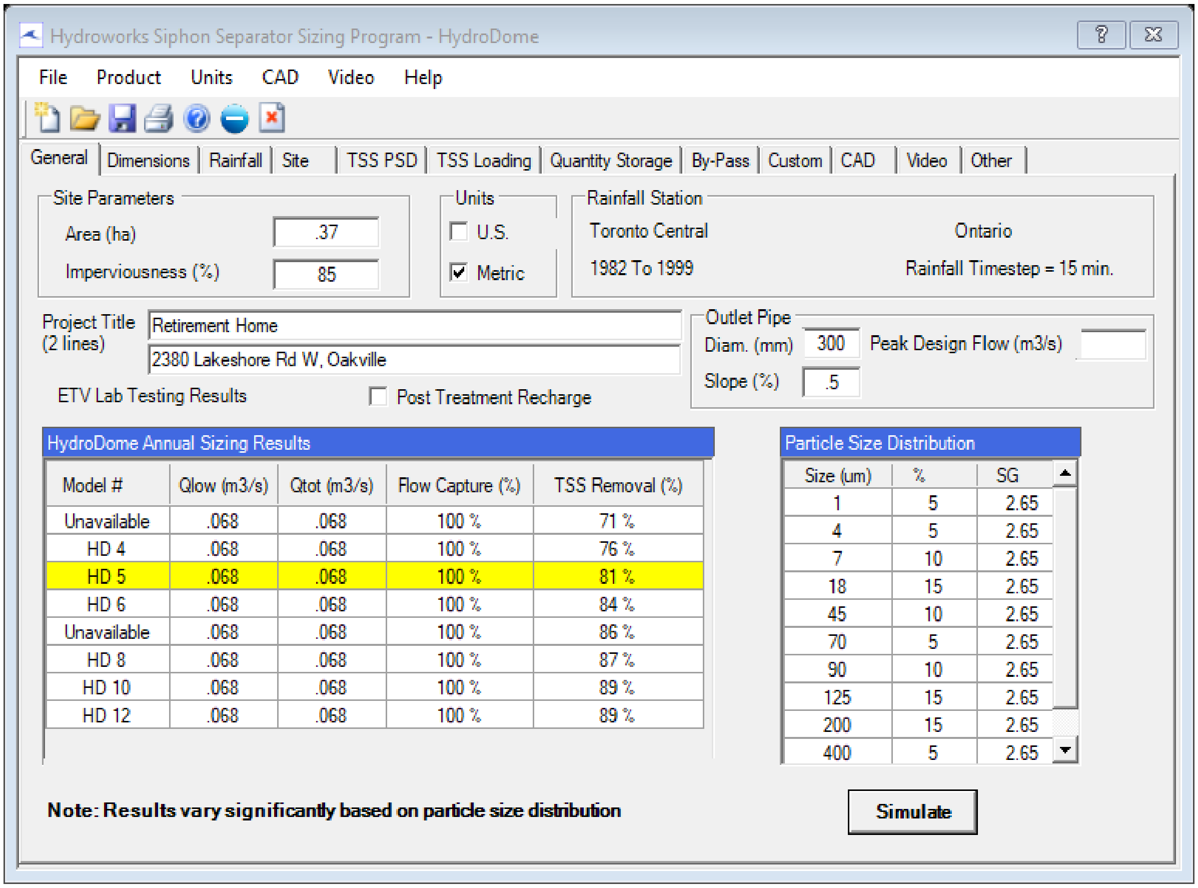
The HydroDome has a siphon which creates a discontinuity in headloss. Since a peak flow was not specified, headloss was calculated using the full pipe flow of .07 (m3/s) for the given 300 (mm) pipe diameter at .5% slope. The headloss was calculated to be 261 (mm) above the crown of the 300 (mm) outlet pipe.

This summary report provides the main parameters that were used for sizing. These parameters are shown on the summary tables and graphs provided in this report.

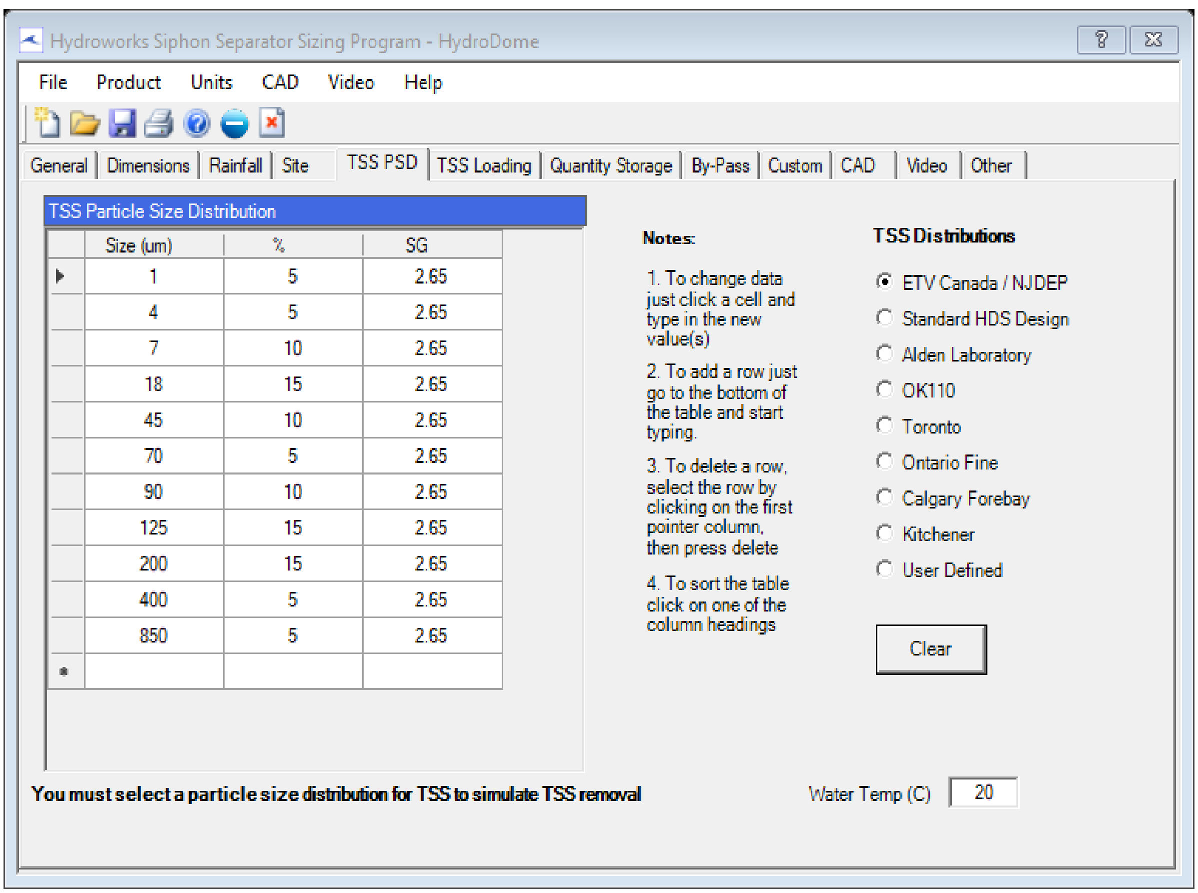
If you have any questions regarding this sizing summary please do not hesitate to contact Hydroworks at 888-290-7900 or email us at support@hydroworks.com.

The sizing program is for sizing purposes only and does not address any site specific parameters such as hydraulic gradeline, tailwater submergence, groundwater, soils bearing capacity, etc. Headloss calculations are not a hydraulic gradeline calculation since this requires a starting water level and an analysis of the entire system downstream of the HydroDome .

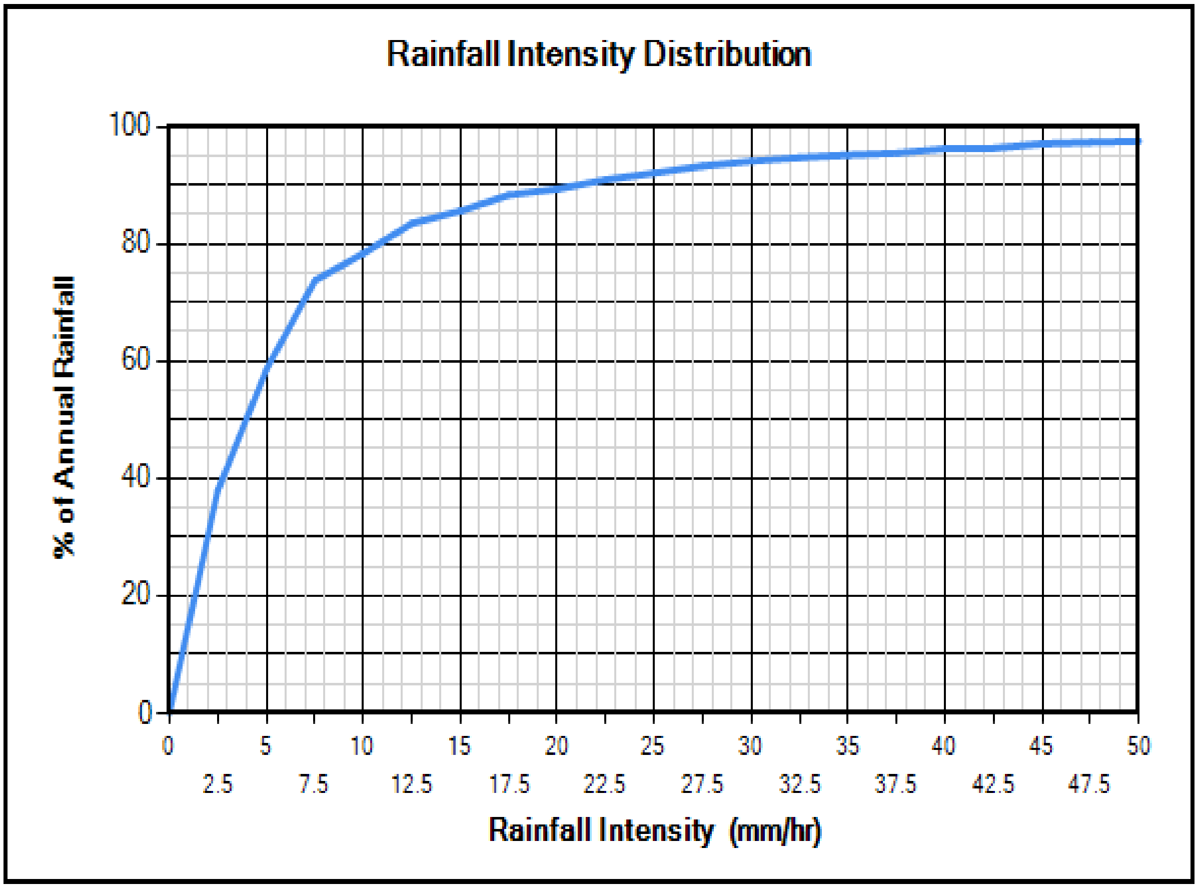
TSS Removal Sizing Summary



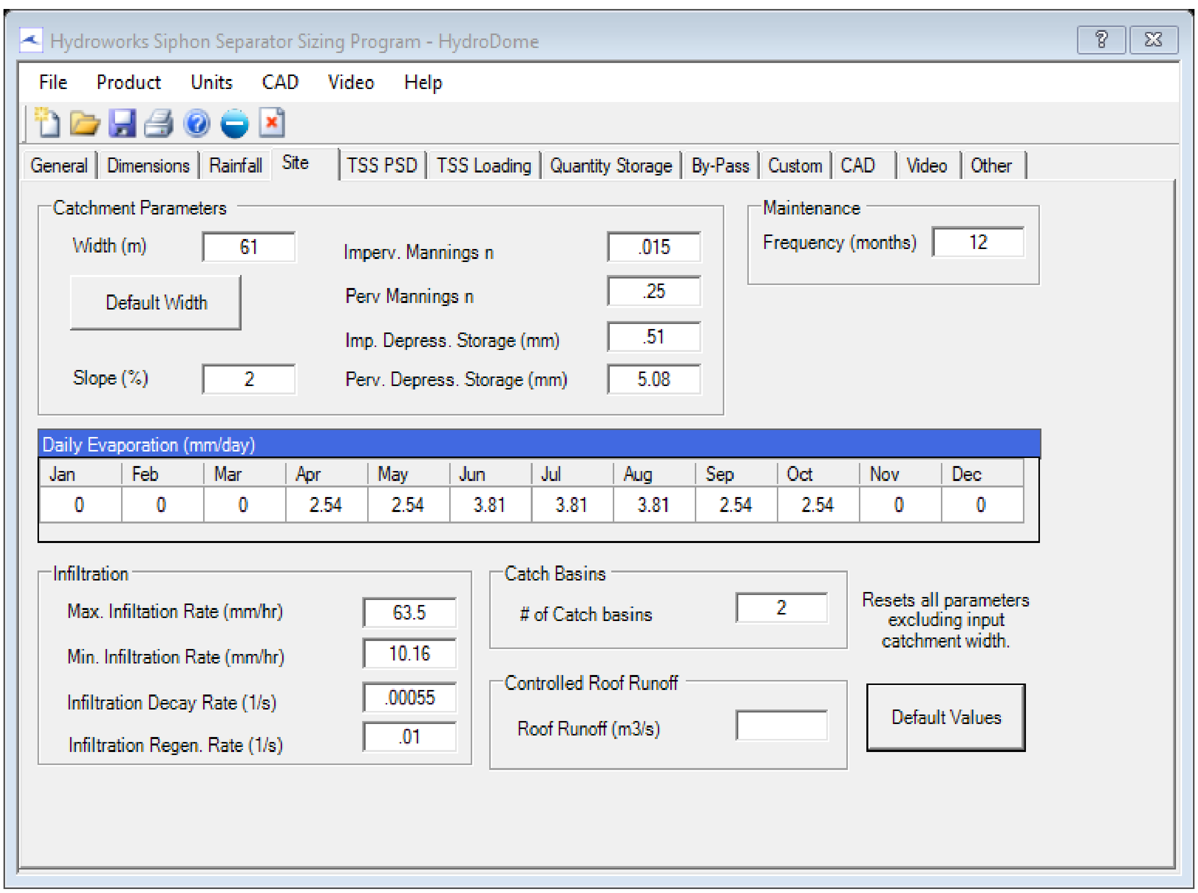
TSS Particle Size Distribution



Rainfall Station - Toronto Central, Ontario(1982 To 1999)



Site Physical Characteristics



Dimensions And Capacities

Hydroworks Siphon Separator Sizing Program - HydroDome

File Product Units CAD Video Help

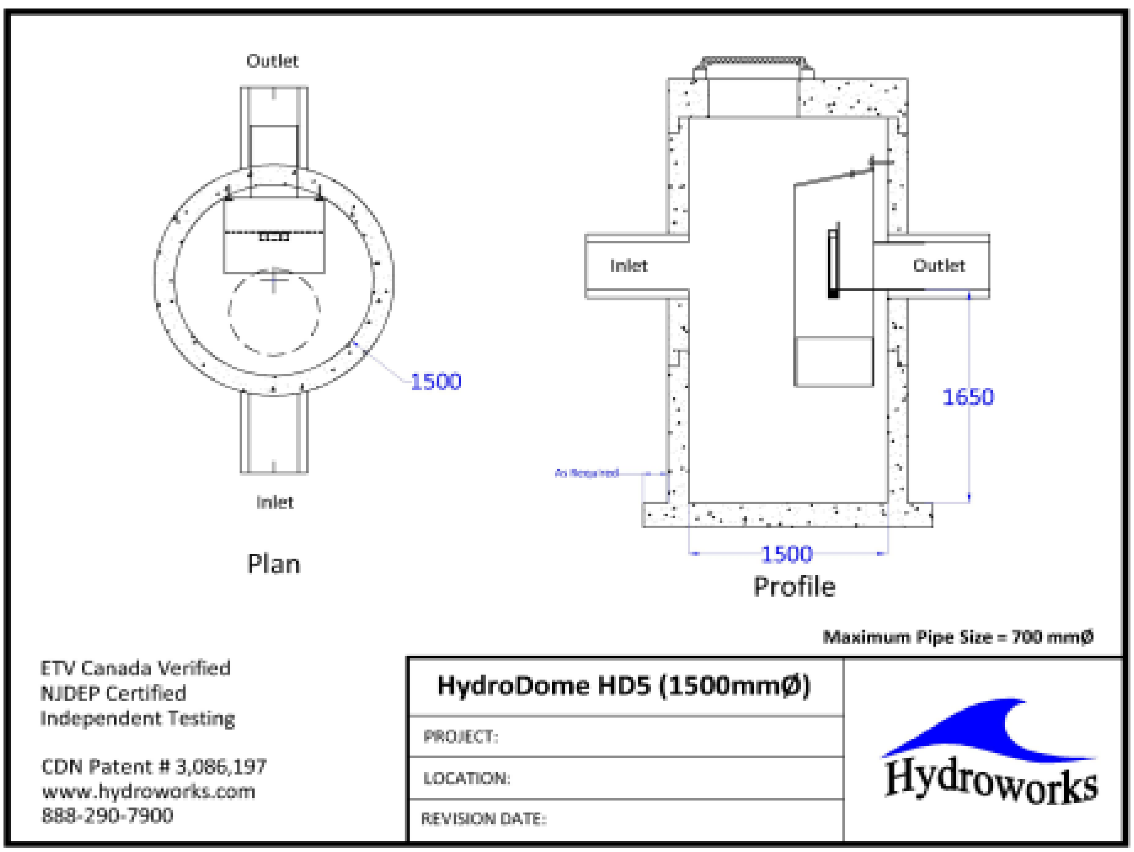
General | Dimensions | Rainfall | Site | TSS PSD | TSS Loading | Quantity Storage | By-Pass | Custom | CAD | Video | Other

Dimensions and Capacities

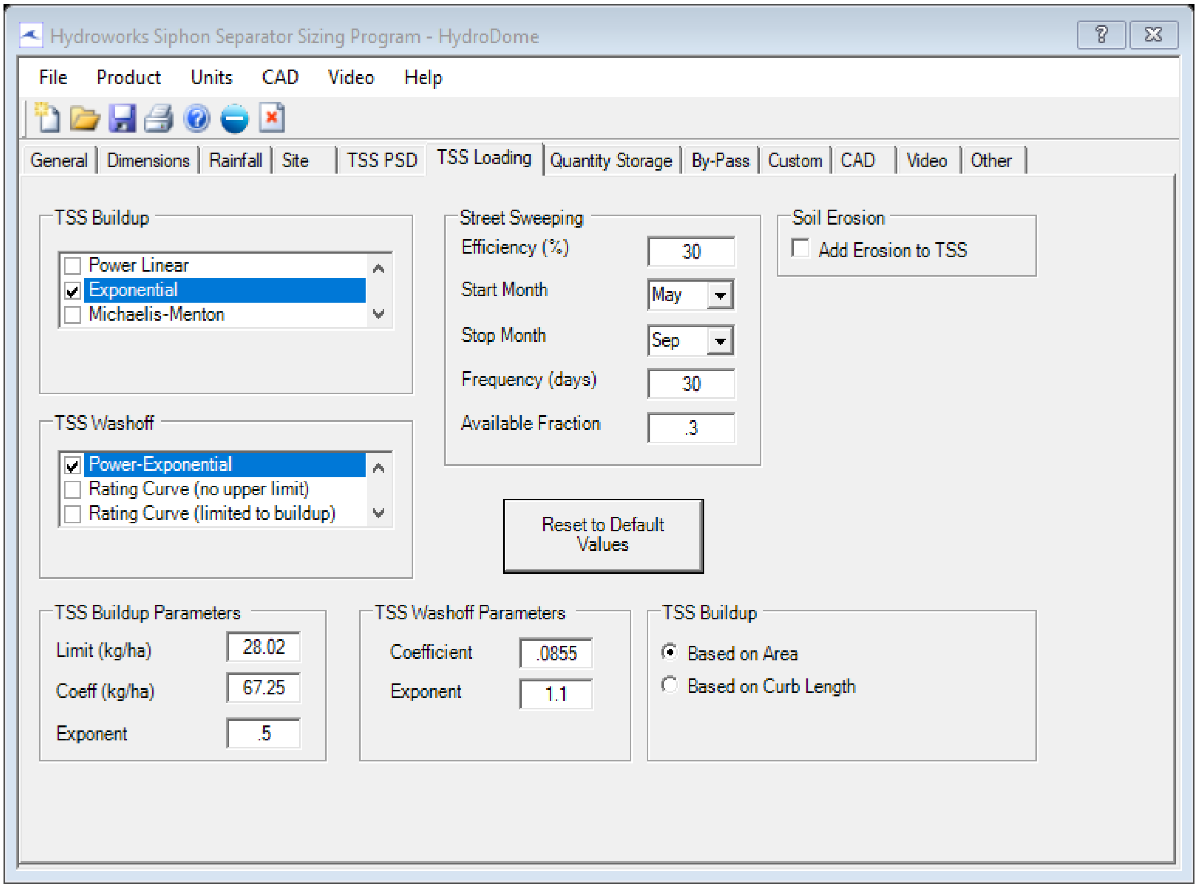
Model	Diam. (m)	Depth (m)	Float. Vol. (L)	Sediment Vol. (m3)	Total Vol. (m3)
HD 3	0.91	1.22	123	0.5	0.8
HD 4	1.22	1.37	266	0.9	1.6
HD 5	1.52	1.68	483	1.7	3.1
HD 6	1.83	1.98	803	2.9	5.2
HD 7	2.13	2.29	1226	4.6	8.2
HD 8	2.44	2.59	1863	6.8	12.1
HD 10	3.05	3.2	3617	13	23.3
HD 12	3.66	3.81	6224	22.2	40

Depth = Depth from outlet invert to inside bottom of tank

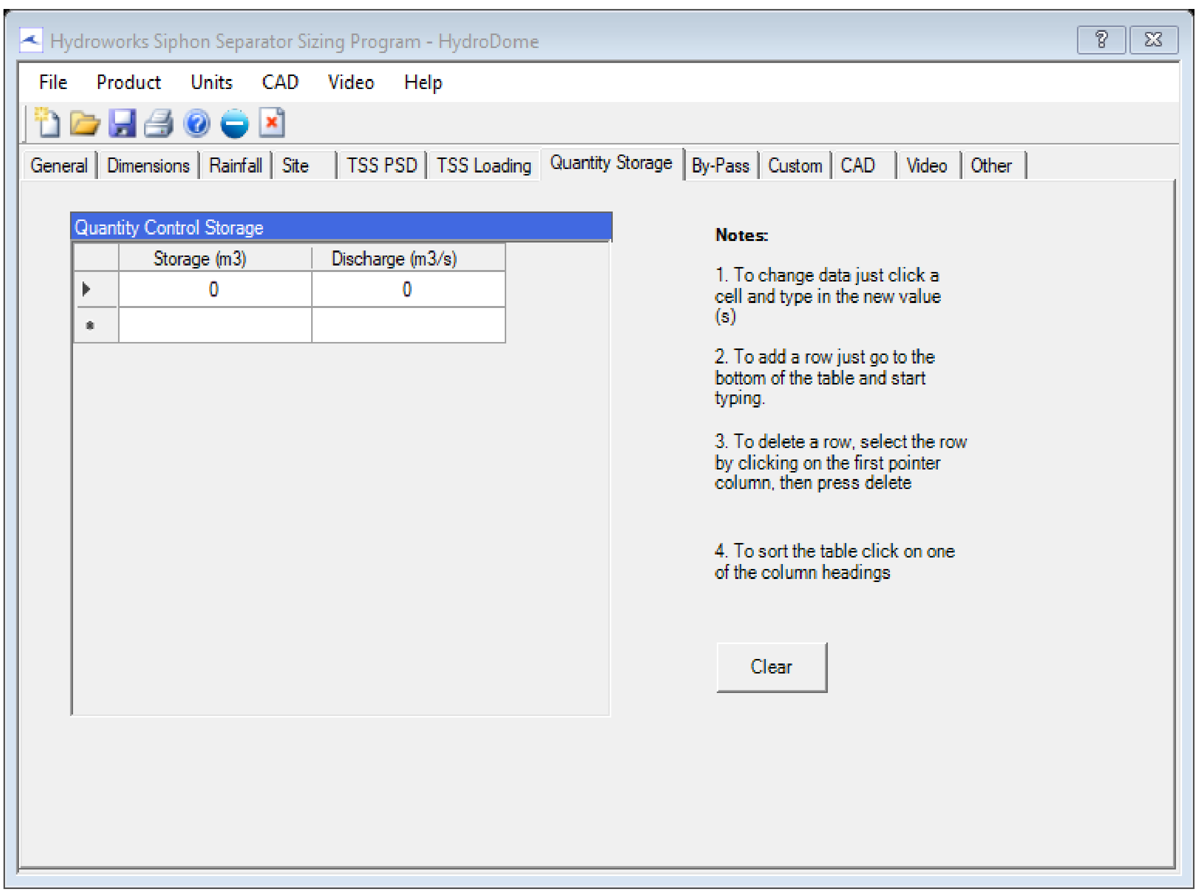
Generic HD 5 CAD Drawing



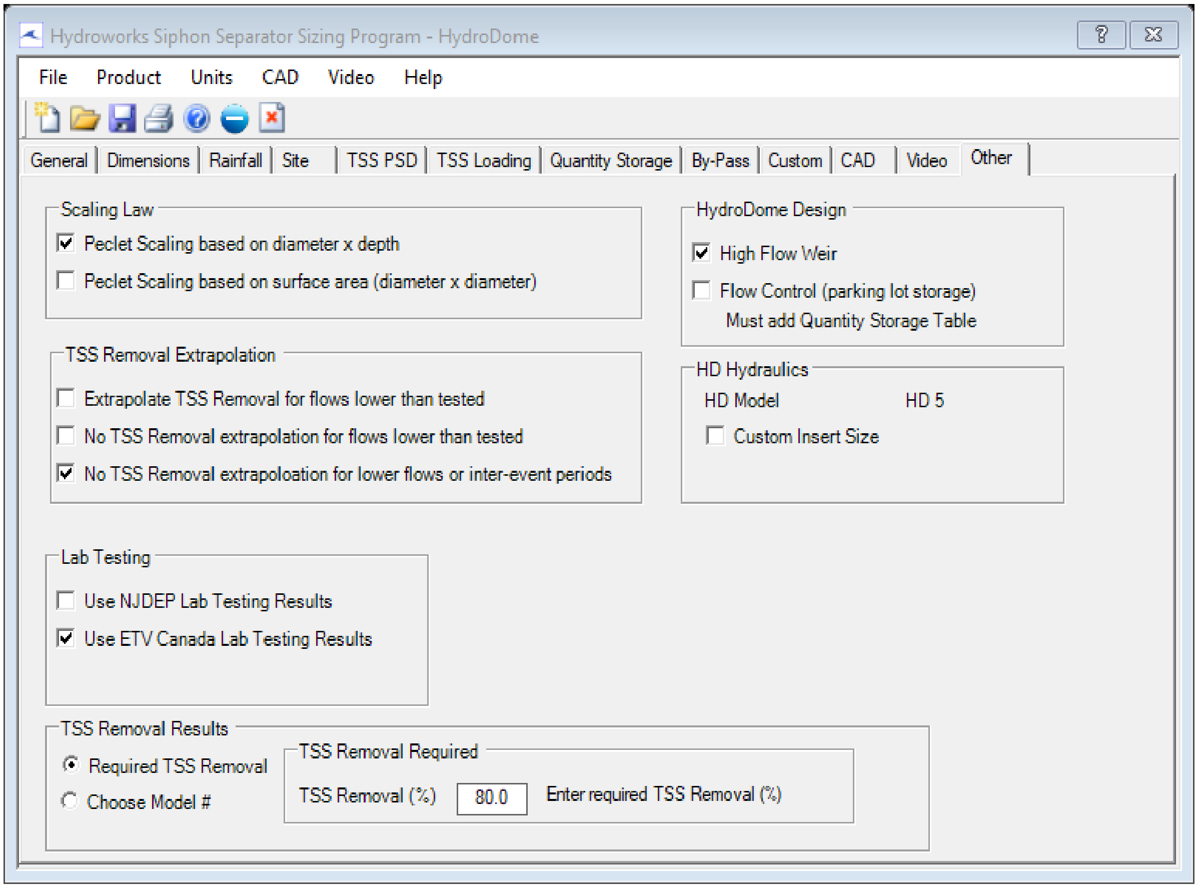
TSS Buildup And Washoff



Upstream Quantity Storage



Other Parameters



Flagged Issues

If there is underground detention storage upstream of the HydroDome please contact Hydroworks to ensure it has been modeled correctly.

Hydroworks Sizing Program - Version 5.7

1-800-290-7900
www.hydroworks.com

Verification Statement

Hydroworks HydroDome HD3 Oil-Grit Separator
Registration number: (V-2021-09-02)
Date of issue: 2021-October-04

Technology type	Oil-Grit Separator		
Application	Technology to remove oil, sediment, trash and debris from storm-water and snowmelt runoff as well as other pollutants that attach to sediment particles, such as nutrients and metals.		
Company	Hydroworks, LLC.		
Address	257 Cox St., Roselle, NJ 07203 USA	**Phone**	+1-888-290-7900
Website	https://hydroworks.com	**E-mail**	gbryant@hydroworks.com

Verified Performance Claims
The Hydroworks HydroDome HD3 Oil-Grit Separator (OGS) was tested by Alden Research Laboratory, Holden, Massachusetts, USA in 2021. The performance test results were verified by 'The Sir Sandford Fleming College of Applied Arts and Technology's Centre for Advancement of Water and Wastewater Technologies' (CAWT) following the requirements of ISO 14034:2016 and the VerifiGlobal Performance Verification Protocol. The following performance claims were verified:

Sediment removal test: The Hydroworks HydroDome HD3 OGS device, with a false floor set to 50% of the manufacturer's recommended maximum sediment storage depth and a constant influent test sediment concentration of 200 mg/L and particle size distribution of 1-1000 µm, removed 83.9, 77.6, 68.4, 66.9, 59.4, 52.4, and 46.0 percent of influent sediment by mass at surface loading rates of 40, 80, 200, 400, 600, 1000, and 1400 L/min/m^2 respectively.

Scour test: The Hydroworks HydroDome HD3 OGS device with 15.2 cm (6 inch) of test sediment preloaded onto a false floor reaching 50% of the manufacturer's recommended maximum sediment sump storage depth, generated corrected effluent sediment concentrations on average of 0.54, 0.70, 0.0, 0.0, and 0.11 mg/L at 5-min duration surface loading rates of 200, 800, 1400, 2000, and 2600 L/min/m^2, respectively.

Light liquid re-entrainment test: The Hydroworks HydroDome HD3 OGS with surrogate low-density polyethylene beads preloaded within the inner chamber, representing a floating light-liquid volume equal to a depth of 50.8 mm (2 inch) over the sedimentation area, retained 100, 100, 100, 100, and 99.7 percent of loaded beads by mass during the 5-minute duration surface loading rates of 200, 800, 1400, 2000, and 2600 L/min/m^2, respectively.

The above verified claims can be applied to other units smaller or larger than the tested unit, provided that the untested units meet the scaling rule specified in the Procedure for Laboratory Testing of Oil Grit Separators (Version 3.0, June 2014)

Technology Application

HydroDome is a hydrodynamic separator that provides benefits for both water quality and water quantity (i.e., flow control). HydroDome combines the function of separator, hood, and flow control with active storage to provide a multi-purpose stormwater management solution in one structure. HydroDome also functions as an oil separator due to the submerged inlet design and the fact that the design raises the water level with flow to maximize the distance between any floatables (oil, trash) and the discharge entrance to the HydroDome.

Technology Description

HydroDome comes complete and slides into the outlet pipe from a drainage structure and is secured to the wall with anchor bolts. It consists of a siphon with flow control, that regulates the water level in the structure and the flow rate in the outflow, and an optional high flow weir. A schematic of the Hydroworks HydroDone OGS is shown in Figure 1.

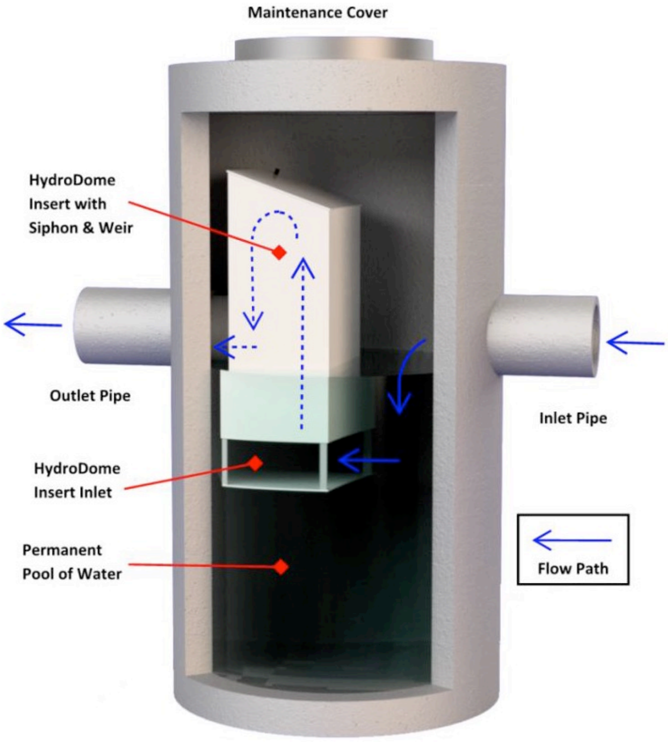


Figure 1: Schematic of the Hydroworks HydroDome Oil-Grit Separator

The siphon raises the water level to a pre-determined level without allowing water to exit the structure. The raised water level provides:
- Greater time for initial total suspended solids (TSS) removal and for floatables to prevent re-entrainment in the flow,
- Additional dilution to reduce effluent concentrations of any pollutants, and
- A greater volume, or buffer, of water to prevent scour of previously settled solids.

Water flows into the device through horizontal openings at the bottom of the HydroDome. Water then must travel upwards through the siphon. A foam filter is located at the entrance to the siphon inlet to provide secondary protection from its clogging (the outer housing of the HydroDome and submerged inlet provide primary protection). Once the water level reaches a pre-determined height, the siphon begins to engage, and water flows out of the structure downstream. The siphon flow is controlled by an orifice, whose size can be changed to provide the desired flow control. The water level continues to rise or begins to lower depending on the rate of flow from the orifice compared to the inflow of water to the structure.

An optional weir above the siphon provides a high flow path to prevent the system from surcharging. In cases where parking lot storage is desired, there would not be a high flow weir. A scour protection plate minimizes scour by preventing upward velocities/flow from the structure floor during periods of peak flow. Therefore, HydroDome combines the function of separator, hood, and flow control with active storage to provide a multi-purpose stormwater management solution in one structure.

Description of Test Procedure

For the purposes of this verification, a Hydroworks HydroDome 3-ft diameter (HD3) stormwater treatment unit was tested. The HD3 test unit was a full-scale 3 ft (0.91 m) diameter tank with an internal treatment hood that included a high flow weir. The test tank was fabricated from plastic and included 18-inch (457 mm) diameter inlet and outlet pipes, oriented along the center-line of the tank. The pipe inverts were located 48 inches (1.22 m) above the sump floor and were set with 1% slopes. The 100% and 50% sediment sump storage depths were 12 inches (0.305 m) and 6 inches (0.152 m), respectively. The effective treatment sedimentation area was 7.07 ft^2 (0.656 m^2).

The test data and results for this verification were obtained from independent testing conducted at Alden Research Laboratory in accordance with the *Procedure for Laboratory Testing of Oil-Grit Separators (Version 3.0, June 2014)*[1]. Use of this procedure is intended to ensure that technologies in this category are subjected to stringent requirements in generating verifiable performance test data.

The verification plan was followed with one minor variance from the *Procedure*. This variance includes the required minimum amount of test sediment to be fed into the test unit for each tested surface loading rate (SLR). Although the *Procedure* requires a minimum of 11.3 kg of test sediment, during the 40 L/min/m^2 SLR test, only 6.45 kg was fed into the unit, which is 4.85 kg less than the specified minimum. This variance to the *Procedure* was agreed to by Toronto and Region Conservation Authority (TRCA), the author of the *Procedure*, based on previous conversations with Alden Labs, noting that the length of time to conduct the test with 11.3 kg of sediment at 40 L/min/m^2 would be over 36 hours.

Verification Results

CAWT verified the performance test data and other information pertaining to the HydroDome HD3 Oil-Grit Separator. A Verification Plan was prepared to guide the verification process based on the requirements of ISO 14034:2016 and the VerifiGlobal Performance Verification Protocol.

The test sediment consisted of ground silica (1 – 1000 micron) with a specific gravity of 2.65, uniformly mixed to meet the particle size distribution specified in the testing procedure.

The "*Procedure for Laboratory Testing of Oil Grit Separators"* (TRCA, 2014) requires that the three-sample average of the test sediment particle size distribution (PSD) meet the specified PSD. The allowable tolerance of 6% variation from the specified PSD curve was met at each discrete particle size tested and the d50 was finer than 75 µm.

Comparison of the individual sample and average test sediment PSD to the specified PSD is shown in Figure 2. This figure indicates that the test sediment used for the removal and scour tests met the above-mentioned criteria. The median particle size was 64 µm.

Samples from test sediment batches used for each run met the specified PSD within the required tolerance thresholds.

The capacity of the HydroDome HD3 device to retain sediment was determined at seven surface loading rates using the modified mass balance method. This method involved measuring the mass and particle size distribution of the injected and retained sediment for each test run.

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[1] The *Procedure for Laboratory Testing of Oil-Grit Separators (Version 3.0, June 2014)* was originally prepared by the Toronto and Region Conservation Authority (TRCA) in association with a 31 member advisory committee from various stakeholder groups.

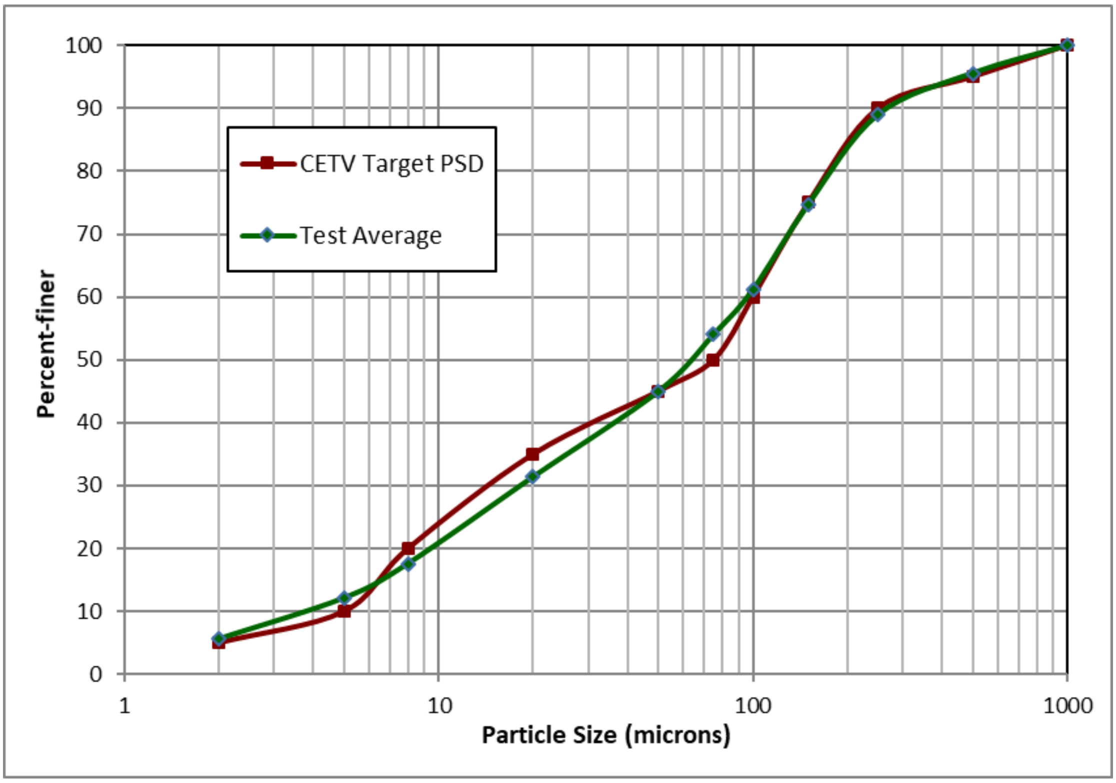

Figure 2 - Average particle size distribution (PSD) of the test sediment used for the sediment removal and scour test compared to the specified PSD

Performance was evaluated with a false floor simulating the technology filled to 50% of the manufacturer's recommended maximum sediment storage depth. The test was carried out with clean water that maintained a sediment concentration below 20 mg/L. Based on these conditions, removal efficiencies for individual particle size classes and for the test sediment, as a whole, were determined for each of the tested surface loading rates (Table 1).

In some instances, the removal efficiencies were above 100% for certain particle size fractions. These discrepancies are not unique to any one test laboratory and are attributed to errors relating to the blending of sediment, collection of representative samples for laboratory submission, and laboratory analysis of PSD. Due to these errors, caution should be exercised in applying the removal efficiencies by particle size fraction for the purposes of sizing the tested device (see Bulletin # CETV 2016-11-0001).

Particle Range (µm)	40 L/min/m^2	80 L/min/m^2	200 L/min/m^2	400 L/min/m^2	600 L/min/m^2	1000 L/min/m^2	1400 L/min/m^2	Average
>500	100%	125%	140%	140%	200%	200%	180%	155%
250-500	114%	129%	150%	143%	143%	183%	217%	154%
150-250	150%	136%	157%	153%	179%	221%	220%	174%
100-150	116%	126%	129%	148%	157%	162%	139%	140%
75-100	136%	155%	178%	190%	180%	170%	133%	163%
50-75	91%	100%	128%	270%	126%	82%	75%	125%
20-50	111%	97%	93%	51%	58%	42%	73%	75%
8-20	75%	79%	38%	34%	29%	17%	26%	42%
5-8	53%	34%	16%	7%	0%	0%	23%	19%
2-5	37%	29%	14%	0%	0%	0%	1%	12%

Table 1 - Removal efficiencies (%) of the HydroDome HD3 Oil-Grit Separator for individual particle size classes at specified surface loading rates

Figure 3 compares the particle size distribution (PSD) of the three-sample average of the test sediment to the PSD of the sediment retained by the HydroDome HD3 OGS device at each of the tested surface loading rates. As expected, the capture efficiency for fine particles was generally found to decrease as surface loading rates increased, particularly in the 400 to 1400 L/min/m^2 range.

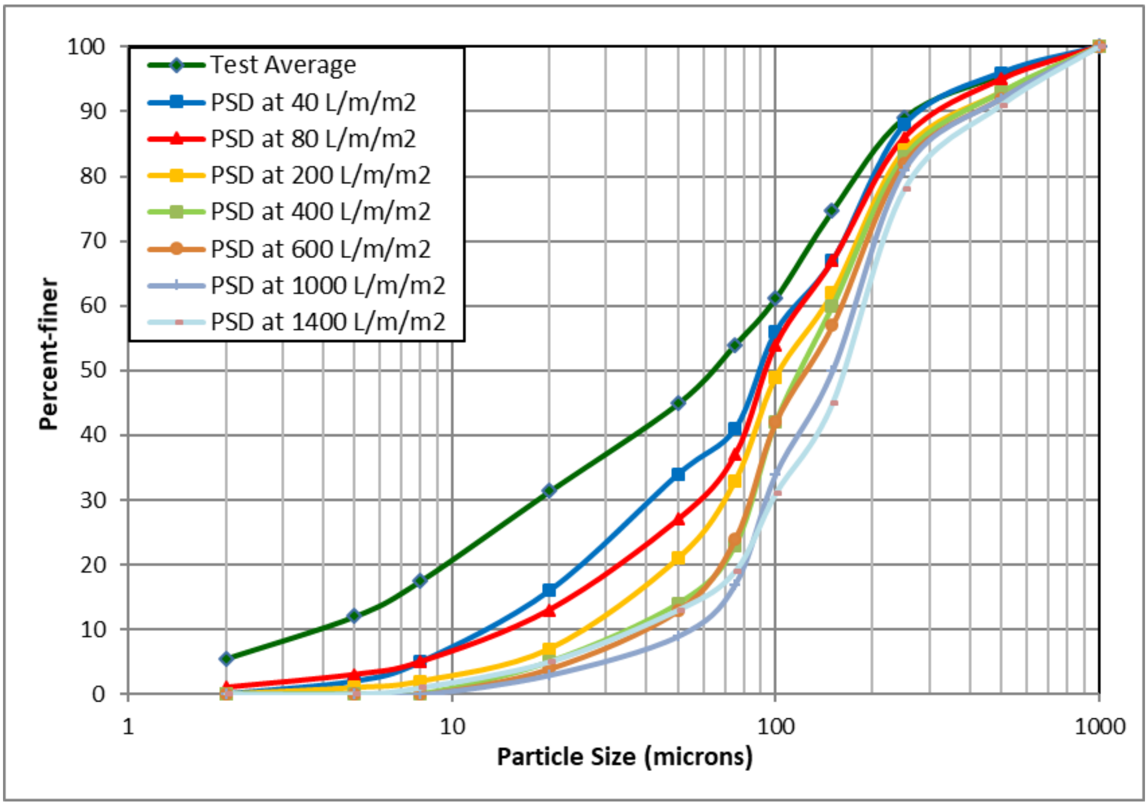

Figure 3 - Particle size distribution of sediment retained in the HydroDome HD3 Oil-Grit Separator in relation to the injected test sediment average

Table 2 shows the results of the sediment scour and re-suspension test for the HydroDome HD3 Oil-Grit Separator unit. The scour test involved preloading 15.2 cm (6 inches) of fresh test sediment into the sedimentation sump of the device. The sediment was placed on a false floor to mimic a device filled to 50% of the maximum recommended sediment storage depth.

Measured Concentration at Each surface Loading Rate					
Effluent Sample No.	200 L/min/m^2	800 L/min/m^2	1400 L/min/m^2	2000 L/min/m^2	2600 L/min/m^2
1	1.2	0.3	0.0	0.0	0.0
2	0.7	0.0	0.0	0.0	0.0
3	0.5	0.0	0.0	0.0	0.5
4	0.1	3.2	0.0	0.0	0.0
5	0.3	0.0	0.0	0.0	0.0
Average	**0.5**	**0.7**	**0.0**	**0.0**	**0.1**

Table 2 - Scour test adjusted effluent sediment concentration at each surface loading rate

Clean water was run through the device at five surface loading rates over a 30-minute period. Each flow rate was maintained for 5 minutes with a one-minute transition time between flow rates. Effluent samples were collected at one minute sampling intervals and analyzed for suspended solids concentration (SSC) and PSD by recognized methods. The effluent samples were subsequently adjusted based on the background concentration of the influent water.

Results showed average adjusted effluent sediment concentrations below 0.7 mg/L at all surface loading rates. The magnitude of scour is dependent on the internal flow patterns (velocity and turbulence) and water volume within the unit, which is related to the depth below the inlet and outlet. The HD3 possessed a large water volume in the sump and consequently, low velocity, which prevented incipient motion of the sediment of sufficient magnitude for scour to occur.

The average measured effluent scour sediment concentrations (adjusted for background) for each tested SLR were not adjusted for particle size based on the D5 of particles captured for the 40 L/min/m^2 removal efficiency test since there was negligible scour.

The capacity of the device to retain light liquid was determined at five surface loading rates in a range between 200 and 2600 L/min/m^2 using low-density polyethylene beads, Dow Chemical Dowlex^{tm} 2517, with a density of 0.917 g/cm^3. This material was specified as the acceptable surrogate to represent floating liquid for a qualitative assessment of liquid behaviour during operation.

Performance was evaluated with a total of 32.8 litres (18.94 kg) of pellets preloaded into the treatment vault by introducing them into the crown of the influent pipe, to a volume equal to a depth of 50.8 mm (2 inch) over the sedimentation area of 0.66 m^2. The effluent was collected in flow-designated nets to allow for quantification of any re-entrained pellets for each test SLR. The collected pellets were dried and the mass of collected pellets was quantified for each SLR, as well as the overall test.

The recorded average flow data, as well as quantified volume and mass of collected pellets for each target SLR and overall test, is shown in Table 3. The maximum re-entrainment of 0.3% occurred at 2600 L/min/m^2. The total retention rate was 99.7%.

Light-liquid Re-Suspension Data				Starting Volume	(Liters) 32.8	Starting Mass	(grams) 18938
Action	**Time Stamp** (minutes)	**Meter**	**Target Flow** (L/min/m^2)	**Recorded Flow** (L/min/m^2)	**COV**	**Collected Mass** (grams)	**Retained Mass**
Start D.A. Recording	0.0						
Flow set	1.0	4"	200	**207**	0.057	0	**100.0%**
Stop Collection	6.0			3.4%			
Flow set	7.0	4"	800	**826**	0.008	0	**100.0%**
Stop Collection	12.0			3.2%			
Flow set	13.0	6"	1400	**1407**	0.009	0	**100.0%**
Stop Collection	18.0			0.5%			
Flow set	19.0	6"	2000	**2022**	0.004	0.3	**100.0%**
Stop Collection	24.0			1.1%			
Flow set	25.0	6"	2600	**2599**	0.003	54.9	**99.7%**
Stop Collection	30.0			-0.1%			
Hydroworks HD 3				Interim Collection Net		1.3	
				Total		**56.5**	**99.7%**

Table 3 - Light-liquid recorded flow and re-entrainment data

Quality assurance

Performance testing and verification of the HydroDome HD3 Oil Grit Separator were performed in accordance with the requirements of ISO 14034:2016 and the VerifiGlobal Performance Verification Protocol. The verifier, CAWT, has confirmed that quality assurance requirements were addressed throughout the performance testing process and in the generation of performance test results. This includes reviewing all data sheets and data downloads, as well as overall management of the test system, quality control and data integrity.

In addition, QA/QC measures are documented in the *"Procedure for Laboratory Testing of Oil-Grit Separators"* (TRCA, 2014) to ensure results are accurate and precise, and that testing conducted by multiple vendors of the same category of technology are employing the same test method. The QA/QC measures include the use of certified laboratories, established test methods, calibration of equipment, tolerance limits for results variation, data checks during testing, and stringent documentation requirements.

Table 4 provides a summary of the acceptance criteria for particle size distribution, solids concentration in test water, water temperature, flow measurement equipment, flow rate variation, sediment feed, sediment moisture content, and sample analysis.

QC Parameter	Acceptance Criteria
Particle Size Distribution	Analyzed by a certified laboratory in accordance with ASTM D422-63(2007)e1. Percentages for size ranges vary by <6%, median < 75 um. PSD in water determined by ASTM D422-63(2007)e1 upon prior drying in designated pre-weighed nonferrous trays in compliance with ASTM D4959-07.
Solids concentration in test water	Suspended solids concentration (SSC) concentration of test water of less than 20 mg/L.
Water temperature	Temperature of water less than 25ºC.
Flow measurement equipment	Equipment calibration reports submitted to confirm that reported flow rate match actual flow rate. Flow rates from calibrated flow instruments recorded at no longer than 30 second intervals over the duration of the test.
Flow rate variation	Flow rates have COV < 0.04; maintained with ±10% of target flow rate.
Sediment feed	TSS concentration target = 200 mg/L with a tolerance limit of ±25 mg/L. Injection location is 5 pipe diameters upstream of the inlet to the device, as per the *Procedure*. Six calibration samples taken over duration of each test run. The allowed Coefficient of Variance (COV) for the measured samples was 0.10.
Sediment moisture content	Determined by ASTM D4959-07 "Standard Test Method for Determination of Water (Moisture) Content of Soil By Direct Heating".
Sample analysis	Conducted by qualified laboratories using standard methods and meeting the requirements of ISO.

Table 4. Validation of QA/QC procedures

Summary of Verification Results and Verified Performance Claim for Hydroworks HydroDome HD3 Oil-Grit Separator (OGS)

In summary, the HydroDome HD3 Oil Grit Separator is designed to remove oil, sediment, trash and debris from stormwater and snowmelt runoff as well as other pollutants that attach to sediment particles, such as nutrients and metals. Verification of performance claims for the Hydroworks HydroDome HD3 Oil Grit Separator was conducted by CAWT based on independent third-party performance test results provided by Alden Research Laboratory, as well as additional information provided by Hydroworks.

Table 5 summarizes the verification results in relation to the technology performance parameters that were identified to determine the efficacy of the HydroDome HD3 Oil Grit Separator. The claims stated in Table 5 were verified using the modified mass balance method for sediment removal by measuring the total mass of sediment entering the unit and retained by the unit at prescribed surface loading rates. Effluent sampling was conducted every minute over a 30-minute duration for the scour test, using approved sampling methods as per the verification procedure. The light liquid re-entrainment test was conducted using a mass balance methodology which accounted for all the beads input, captured, and scoured from the separator.

Parameters	Verified Claims	Accuracy
Sediment Removal	During the sediment removal test, the Hydroworks HydroDome HD3 OGS device, with a false floor set to 50% of the manufacturer's recommended maximum sediment storage depth and a constant influent test sediment concentration of 200 mg/L and particle size distribution of 1-1000 µm, removed 83.9, 77.6, 68.4, 66.9, 59.4, 52.4, and 46.0 percent of influent sediment by mass at surface loading rates of 40, 80, 200, 400, 600, 1000, and 1400 L/min/m^2 respectively	The sediment removal characteristics were quantified at various surface loading rates (SLRs), including particle size fractions, using a modified mass balance methodology. Performance results are presented as the true values.
Sediment Scour	During the scour test, the Hydroworks HydroDome HD3 OGS device with 15.2 cm (6 inch) of test sediment preloaded onto a false floor reaching 50% of the manufacturer's recommended maximum sediment sump storage depth, generated corrected effluent sediment concentrations on average of 0.54, 0.70, 0.0, 0.0, and 0.11 mg/L at 5-min duration surface loading rates of 200, 800, 1400, 2000, and 2600 L/min/m2, respectively.	5 samples analyzed for sediment (n=5) at each flow rate There was negligible scour once corrected for background concentrations.
Light Liquid Re-entrainment	During the light-liquid re-entrainment test, the Hydroworks HydroDome HD3 OGS with surrogate low-density polyethylene beads preloaded within the inner chamber, representing a floating light-liquid volume equal to a depth of 50.8 mm (2 inch) over the sedimentation area, retained 100, 100, 100, 100, and 99.7 percent of loaded beads by mass during the 5-minute duration surface loading rates of 200, 800, 1400, 2000, and 2600 L/min/m^2, respectively.	Performance results are presented as the true values. Under the "Procedure for Laboratory Testing of Oil-Grit Separators" (TRCA, 2014), the light-liquid re-entrainment test is also not amenable to statistical analysis as the tests were only conducted once at various flow rates following a mass balance procedure.

Table 5. Verified performance claims

What is ISO 14034?

The purpose of environmental technology verification is to provide a credible and impartial account of the performance of environmental technologies. Environmental technology verification is based on a number of principles to ensure that verifications are performed and reported accurately, clearly, unambiguously and objectively. The International Organization for Standardization (ISO) standard for environmental technology verification (ETV) is ISO 14034, which was published in November 2016.

Benefits of ETV

ETV contributes to protection and conservation of the environment by promoting and facilitating market uptake of innovative environmental technologies, especially those that perform better than relevant alternatives. ETV is particularly applicable to those environmental technologies whose innovative features or performance cannot be fully assessed using existing standards. Through the provision of objective evidence, ETV provides an independent and impartial confirmation of the performance of an environmental technology based on reliable test data. ETV aims to strengthen the credibility of new, innovative technologies by supporting informed decision-making among interested parties.

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NOTICE: Verifications are based on an evaluation of technology performance under specific, predetermined operational conditions and parameters and the appropriate quality assurance procedures. VerifiGlobal and the Verification Expert, CAWT, make no expressed or implied warranties as to the performance of the technology and do not certify that a technology will always operate as verified. The end user is solely responsible for complying with any and all applicable regulatory requirements. Mention of commercial product names does not imply endorsement.

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