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PROPOSED MIXED-USE
RESIDENTIAL TOWER DEVELOPMENT
1062-1074 COOKE BOULEVARD
CITY OF BURLINGTON

PROJECT No.: 20262

FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT

Prepared For:

CAMARRO DEVELOPMENTS

Prepared By:

The Odan/Detech Group Inc.

Original: August 25th, 2022

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APPENDIX A

Existing Site — Aerial view of Site and surrounding area
Site Plan & Statistics — by RAW
Correspondence with City of Burlington Staff Regarding MTSA Status

APPENDIX B

1062-1074 Cooke Boulevard Sanitary Sewer Capacity Analysis Report by Civica Infrastructure Ltd.

APPENDIX C

AutoDesk Storm & Sanitary Analysis ASCII Output File (100-Year Storm)
CETV Verification Statement – Imbrium Systems Inc. Stormceptor OGS
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APPENDIX D

Functional Servicing Plan
Functional Grading Plan

1.0 INTRODUCTION

The subject of this report is a 0.71 Ha parcel of land bound by the following:

- To the east: Cooke Boulevard
- To the south: an existing adjacent commercial development
- To the west: an existing commercial development and an existing house
- To the north: Masonry Court

Presently the site comprises two existing commercial developments.

Presently there is an irregularly-shaped watermain easement at the site's north boundary. The area of this easement will be dedicated to the City by this development. The site's area will then be 0.70Ha.

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

It is proposed to demolish the two existing commercial developments and construct a two-tower residential tower development. The proposed development comprises two towers, on top of a five-storey common podium and a three-level below-grade parking structure. The south tower is the first phase of the development and comprises 30 storeys. The north tower is the second phase of the development and comprises 32 storeys.

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

The municipal addresses of the site comprise 1062-1074 Cooke Boulevard.

For detailed topography of the existing site conditions, as of March 2021, refer to the topographic survey prepared by A.J. Clarke and Associates Ltd..

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

2.0 SCOPE OF WORK

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

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

3.0 WATER DISTRIBUTION ASSESSMENT

i) Existing Infrastructure

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

- Masonry Court, south side: 300mm watermain
- Cooke Boulevard, west side: 300mm watermain

ii) Proposed Servicing

It is proposed to service the Phase 1 Tower with a proposed 250mm fire service connection to the 300mm Cooke Boulevard watermain with a branch 200mm domestic water service. The tower is greater than 84m in height, therefore a second/redundant 250mm fire service connection is additionally proposed. The first connection is installed with a valve chamber on the mainline. Refer to the Functional Servicing Plan.

It is proposed to service the Phase 2 Tower with a proposed 250mm fire service connection to the 300mm Cooke Boulevard watermain with a branch 200mm domestic water service. The tower is greater than 84m in height, therefore a second/redundant 250mm fire service connection is additionally proposed, connected to the Masonry Court 300mm watermain. The first connection is installed with a valve chamber on the mainline. Refer to the Functional Servicing Plan.

Both towers will be greater than 84m in height, therefore OBC 2006 3.2.9.7 (4) requires each tower to have a secondary/redundant fire service connection.

Refer to the Functional Servicing Plan for proposed service connections.

The water demand for the proposed building is as follows.

a) Average Day domestic demand - As per average sanitary flow 5.4 L/sec

b) Peak day demand - 2.25 x average demand 12.2 L/sec

c) Fire flow as per FUS 1999 manual calculation, below 100 L/sec

TABLE 1 – Total Water Demand for Proposed Building

	L/sec	USGM
Peak Day Demand	12.2	193
Fire Flow Demand	100	1585
Total Water Demand	112	1778
Available Flow at 20 PSI (Cooke Blvd. 300mm WM)	119	1890

Hydrant flow test results as stated above are provided below.

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

- The proposed building is of fire-resistive construction (reinforced concrete).
- The building will be sprinklered for fire protection and the sprinklers will be fully monitored according to NFPA 13.
- The building's contents (residences) will be non-combustible in nature.
- The building's areas in the calculation are as per the architectural floor areas provided in Appendix A.
- The proposed building's vertical openings and exterior vertical communications are properly protected with a one-hour rating.
- The greatest floor area (GFA) occurs in Level 2 of the podium, which is common to both towers, therefore the fire flow given accordingly is taken as the fire flow demand. This is based on the statistics in Appendix A.

The hydrant flow test was conducted as per the NFPA 291 hydrant testing standard on hydrants on Cooke Boulevard. Refer to the test report, below.

The available flow at 20 psi in the Cooke Boulevard 300mm watermain is 1890 USGM which is greater than the proposed development's total water demand (1778 USGM), therefore the existing watermain infrastructure is sufficient to service the proposed development and no watermain infrastructure upgrades are required.

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

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

Where:

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

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

LOCATION: 1062-1074 Cooke Blvd.

PROJECT: 1062-1074 Cooke Blvd.

OBC OCCUPANCY: Residential Apartment

PROJECT No: 20262

BUILDING FOOT PRINT (m2): 3308

OF STOREYS: 32

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

CONSTRUCTION CLASS: Fire Resistive

AUTOMATED SPRINKLER PROTECTION

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

CONTENTS FACTOR: Non Combustible — CHARGE: -20%

EXPOSURE 1 (south) Ex. Commercial	Distance to Exposure Building (m)	20	15%
	Length - Height		
EXPOSURE 2 (east) Ex. Commercial	Distance to Exposure Building (m)	34	5%
	Length - Height		
EXPOSURE 3 (west) Ex Commercial	Distance to Exposure Building (m)	6	20%
	Length - Height		
EXPOSURE 4 (north) Fut. Development	Distance to Exposure Building (m)	>45	10%
Under Construction	Length - Height		
		Total:	50%

no more than 75%

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

ARE BUILDINGS CONTIGUOUS: NO

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

CALCULATIONS

	C =	0.6		
	A =	3794	m2	(Largest Floor + 25% of adjacent)
	F =	8130	L/min	
Round to Nearest 1000 L/min	F =	8000	L/min	must be > 2000 L/min

STOREY AREAS m2	
730	M
2945	2
2665	3

CORRECTION FACTORS:

OCCUPANCY	-1600	L/min
FIRE FLOW ADJUSTED FOR OCCUPANCY	6400	L/min
REDUCTION FOR SPRINKLER	-3200	L/min
EXPOSURE CHARGE	3200	L/min

REQUIRED FIRE FLOW F = 6400 L/min

Round to Nearest 1000 L/min

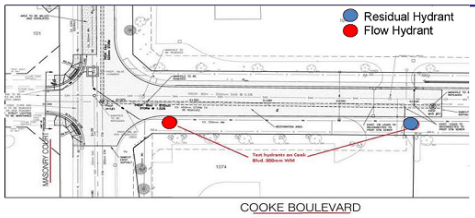
F =	6000	L/min	1585	usgm
F =	100	L/sec		

FLOWMETRIX
INDU-TECH
PROCESS
WESTCAN

Fire Flow Testing Report

Residual Hydrant #
NFPA Colour Code

Residual
BLUE



DATE	May 11, 2021
TIME	11:15 AM
ADDRESS	1074 Cooke Blvd Burlington, ON L7T 4A8
SIZE-inches/mm	12 300
MATERIAL	Ductile Iron
CONTACT INFO	The Odan/Detech Group Inc. Daniel Bancroft, P. Eng. C: (905) 632-3811 ext.133 E: daniel@odandetech.com

RESIDUAL HYDRANT INFO.

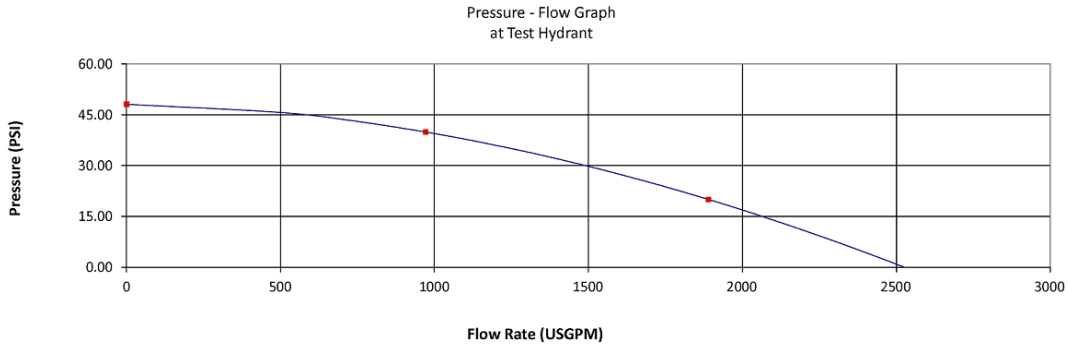
HYDRANT #	Residual	
N.F.P.A. COLOUR CODE	BLUE	
STATIC PRESSURE	48.1	psi
RESIDUAL PRESSURE	39.9	psi
PRESSURE DROP	8.2	psi
% PRESSURE DROP	17.0	% psi

Flow on Water Main At Test Hydrant - 20 psi 1890 USGPM

FLOW HYDRANT(S) INFO.

HYDRANT ASSET ID	HYD. # PORTS	OUTLET DIAMETER (INCHES)	NOZZLE COEFFICIENT	DIFFUSER TYPE	DIFFUSER COEFFICIENT	PITOT READING (psi)	PITOT FLOW (USGPM)	FLOW METER (USGPM)
Flow	2	2.5	Round	LPD250	0.90	**10.4**	486	0
		2.5	Round	LPD250	0.90	**10.4**	486	0
					Total Flow (USGPM)		972	0
					Total Flow (USGPM)		972	

FIRE FLOW CHART



COMMENTS

OPERATOR	FMX	Ryan Ritchie
OPERATOR	FMX	Morgan O'Neill Whitten
OPERATOR		Halton Region Water

ODG_FireFlowTesting_CookeBlvd

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

4.0 SANITARY SEWERS

i) Existing Infrastructure & Drainage

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

- Masonry Court: Existing 300mm sanitary sewer, which flows easterly and discharges into the following Cooke Boulevard sewer.

- Cooke Boulevard: Existing 300mm sanitary sewer, which flows southerly and discharges into a sanitary sewer within Plains Road East.

ii) Proposed Servicing

It is proposed to service each of the proposed Towers with separate respective 200mm sanitary sewer connections to the 300mm sanitary sewer within Cooke Boulevard.

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

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

- Unit flow: q = average daily residential per capita dry weather unit flow = 0.275 m^3/cap/day
- I/I = Unit of peak inflow/infiltration = 0. 286 L/s/ha
- Apartment (over 6-storey): 285 p/Ha and 0.275 m^3/p/day or 0.003183 x 10^{-3} m^3/p/s
- Apartment (6-storeys or less): 135 p/Ha and 0.275 m^3/p/day or 0.003183 x 10^{-3} m^3/p/s
- Notwithstanding the above unit population, however, a unit population of 2.1 P/unit is assumed for the proposed apartment tower development because the Region standard 285 P/Ha unit population (apartments <6-storeys) would result in a unit population of approximately 0.2 Person/unit for the foregoing proposed statistics, which is unrealistic

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

TABLE 2 – Post-Development Sanitary Flow

Component	Population (P)	Sanitary Flow (l/s)	Groundwater Discharge (l/s)	Inflow & Infiltration (l/s)	Total Flow (l/s)
Residential Flows	1699	19.68	N/A	0.20	19.9

The 19.9 L/s total sanitary flow for the proposed Towers will be conveyed to the existing 300mm sanitary sewer within Cooke Boulevard by proposed 200mm at 2.0% (43 L/s capacity) sewer connections. These sewer connections are adequately sized to convey the above 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) =

RESIDENTIAL SITE AREA (ha) = 0.696

TOTAL SITE AREA (ha) = 0.696

NOTE:

LAND USE	NUMBER OF UNITS	SITE AREA, (ha)	GROSS FLOOR AREA, m2	TOTAL POPULATION	TOTAL DAILY FLOW (LITERS)	AVERAGE DAILY FLOW l/sec	PEAKING FACTOR, M	TOTAL FLOW FROM LAND USE, l/sec
RESIDENTIAL Detached, using 55 person/site area				0	0	0.00	4.50	0.00
RESIDENTIAL Semi Houses, using 100 persons/site area				0	0	0.00	4.50	0.00
RESIDENTIAL Apartments (<6 st), using 135 persons/site area				0	0	0.00	4.50	0.00
RESIDENTIAL Apartments (>6 st), using 285 persons/site area				0	0	0.00	4.50	0.00
RESIDENTIAL Density 3, using 2.1 persons/unit	809			1699	467198	5.41	3.64	19.68
COMMERCIAL, Using 90 persons/ha (Floor Ha)		0.06		6	1547	0.02	3.55	0.06
COMMERCIAL, Using 0.60 L/sec per ha				0	0	0.00	2.50	0.00

TOTAL

V1= 468744

Q1=	19.68
Q2=	0.06
Qinfil	0.20
Qtot	19.94

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

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

V1= Total Volume from Land Use in liters

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

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

iii) Receiving Downstream Sanitary Sewer (Wastewater Main) Capacity

The proposed development is proposed to discharge into the existing 300mm sanitary sewer which flows southerly within Cooke Boulevard and then westerly within Plains Road. Consideration is required for this sewer's capacity to receive the proposed flows.

The City of Burlington and Halton Region are presently undertaking an *Area Specific Plan (ASP)* for the area in which the site is located, the Aldershot GO *Major Transit Station Areas (MTSAs)*. This will comprise a servicing report which will provide direction regarding future servicing in the area. City of Burlington Staff stated in an email on July 22, 2022 (Appendix A) that this report for the area of the subject site is under review and may be available early in the fall. Discussion and conclusions derived from that report may be warranted when it becomes available.

It is further noted that the adjacent proposed development project located at 1120 Cooke Boulevard is proposing to improve the Cooke Boulevard and Plains Road local sanitary sewer. This is as per the *Functional Servicing and Stormwater Management Report* by MTE dated November 19, 2021 for 1120 Cooke Boulevard. Section 3.0 therein proposes to increase the Cooke Boulevard sanitary sewer from 300mm to 375mm, and the Plains Road from 250mm to 450mm-diameter pipes.

An original downstream sanitary sewer analysis has been prepared by Civica Infrastructure Ltd. (Appendix B) to ascertain the operating conditions of the downstream sanitary sewers. Civica's modelling utilizes the Region's base model and considers both dry-weather flow (DWF) and wet-weather flow (WWF) derived from the Region's InfoSewer model. Civica's model and report also considers the conclusions and recommendations made for 1120 Cooke Boulevard, as noted above.

The conclusions and recommendations pertaining to downstream sanitary sewer capacity are generally as follows.

- The existing sanitary system does not have capacity to accommodate existing or proposed flows in Wet-Weather Flow scenarios (without considering system improvements).
- With the provision of the same sanitary sewer improvements as 1120 Cooke Boulevard is proposing, the downstream sanitary sewers will be operating acceptably in dry-weather flow (DWF) conditions, with the proposed sanitary flows from the subject development. The sewer will be free-flowing within the pipes' capacities in this scenario.
- With the provision of the same sanitary sewer improvements as 1120 Cooke Boulevard is proposing, the downstream sanitary sewers will be operating acceptably in wet-weather flow (WWF) conditions, with the proposed sanitary flows from the subject development. The sewer will be surcharged in this scenario however the freeboard depth does not pose a basement flood risk.
- Given the foregoing discussion, the proposed sanitary sewer improvements within Cooke Boulevard and Plains Road East should be undertaken prior to the proposed Development's completion.

5.0 STORM WATER MANAGEMENT

i) Existing Infrastructure & Drainage

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

- Masonry Court: there is an existing 300mm concrete storm sewer which commences adjacent to the Site's northwest corner and flows easterly. This sewer continues southerly within Cooke Boulevard, as follows.

- Cooke Boulevard: there is an existing 675mm concrete storm sewer which flows southerly adjacent to the Site's east frontage. This sewer was 450mm in diameter initially and more recently was reconstructed as a 675mm sewer. This sewer continues southerly towards Plains Road East.

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

Presently the entire site drains by existing catchbasins and storm leads to the existing Cooke Boulevard 675mm storm sewer (Catchment Areas Ex-A, Ex-B and Ex-C). A small portion of the adjacent commercial site to the west drains into the subject site by overland flow (Catchment Area EXT-D).

MASONRY CT.

EX 300mm PVC STM

EX 675mm CONC STM

COOKE BLVD.

CONSTRUCTION NORTH

0.168 Ha
EX-C | 0.3

0.042 Ha
EXT-D | 84

0.226 Ha
EX-A | 0.9

0.307 Ha
EX-B | 0.9

LEGEND

CATCHMENT AREA BOUNDARY

0.412Ha
EX-A | 0.25

CATCHMENT ID, AREA & RUNOFF COEFFICIENT

MAJOR OVERLAND FLOW ROUTE

DRAWING :

PRE-DEVELOPMENT
STORM DRAINAGE PLAN

DATE:	PROJ. NO.:	SCALE:
JUL 2022	20262	1:400

PROJECT :

PROPOSED RESIDENTIAL DEVELOPMENT
1062-1074 COOKE BLVD.
BURLINGTON, ON

ODAN·DETECH
CONSULTING ENGINEERS
The Odan/Detech Group Inc. P:(905)632-3811 F:(905)632-3363
5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

ii) Stormwater Quantity Control Criteria

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

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

The above criteria (control 100-year post-development to 5-year pre-development flows) applies to this development, given that it is proposed to discharge to a storm sewer – the Cooke Boulevard 675mm storm sewer - and that the capacity of this storm sewer is unknown.

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

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

iii) Allowable Discharge Flow Rate

Allowable discharge flow rate is calculated to be 98.6 L/s as follows based on the above criteria.

Note that it is proposed to discharge a peak foundation drainage flow rate of 4.0 L/s into the storm control maintenance hole (STM CNTRL MH), as shown on the Functional Servicing Plan. The allowable release rate is therefore adjusted to consider these flows. The allowable release rate is then 94.6 L/s.

The actual pre-development site runoff coefficient (C) is 0.80, therefore the allowable release rate is determined based on a runoff coefficient of 0.5 in accordance with City criteria.

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

Location	Run-off Coefficient	Rainfall Intensity (mm/hr)	Area of Development (ha)	Site Allowable (L/s)
Phase 1 & 2 (Combined) Allowable Flows	0.5	101.3	0.70 Ha	***98.6***
Allowable Flows less Foundation Drainage Flows				***94.6***

iv) Post Development Flow Analysis

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

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

- ***To the West:***
  - There is an existing residential house lot (adjacent to the south half of the west property line).
  - There is an existing commercial development adjacent to the north half of the west property line. A portion of landscaped areas within this property presently drains into the subject site by overland flow – Catchment EXT-D.
- ***To the East:*** There is Cooke Boulevard, which drains by the existing 675mm storm sewer therein.
- ***To the North:*** There is Masonry Court, which drains by the existing 300mm storm sewer therein.
- ***To the South:*** There is an existing commercial development which appears to drain internally by existing catchbasins.

The proposed development is constructed such that it is almost entirely underlain by the proposed below-grade parking structure. The proposed above-grade building extends almost property-line to property-line.

Storm area drains and storm piping, to be designed by the mechanical engineer at the Building Permit stage, will collect the runoff from the proposed building's roof and ground-level areas and convey it uncontrolled to stormwater detention tanks. The stormwater detention tank is proposed to be installed as a cast-in-place concrete tank, installed within the P1 and P2 below-grade levels. Refer to the Functional Servicing Plan.

Stormwater flows will be controlled in accordance with the foregoing allowable release rate by a mechanical storm pump. This solution, rather than a gravity-driven orifice pipe/tube, is necessary due to the shallow depth of the existing 675mm storm sewer within Cooke Boulevard. The following is a discussion of the rationale for this solution.

- The site slopes significantly towards the south (as does the municipal storm sewer), therefore the optimal location for the storm sewer connection is at the southeast corner.
- The invert of the storm sewer connection is given by the depth of the storm sewer in the street. Refer to Cross-Section A-A on the Functional Servicing Plan.
- As shown in Cross-Section A-A on the Functional Servicing Plan, there is approx. 1.0m freeboard between the invert of the outgoing storm connection (given by the depth of the street storm sewer) and the roof of the P1 level, which is insufficient depth for the tank.
- It would also be unfeasible to construct and form the enclosed cast-in-place concrete tank with a depth of only approx. 1.0m.
- It is therefore instead proposed to construct the tank with a depth of approximately 3.50m, corresponding with the floor of the P2 level. It is then necessary to pump storm flows (at no more than the allowable release rate) into the storm control maintenance hole (STM CNTRL MH).

The development comprises two phases – Phase 1 and Phase 2. The proposed stormwater detention tank located at the southeast corner of Phase 1 will provide stormwater quantity control for both Phase 1 and Phase 2. It has been sized accordingly as follows. Provision will be made in the mechanical storm drainage of Phase 1 allowing for Phase 2 to drain to the existing storm tank. Refer to the Functional Servicing Plan for the tank's location.

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

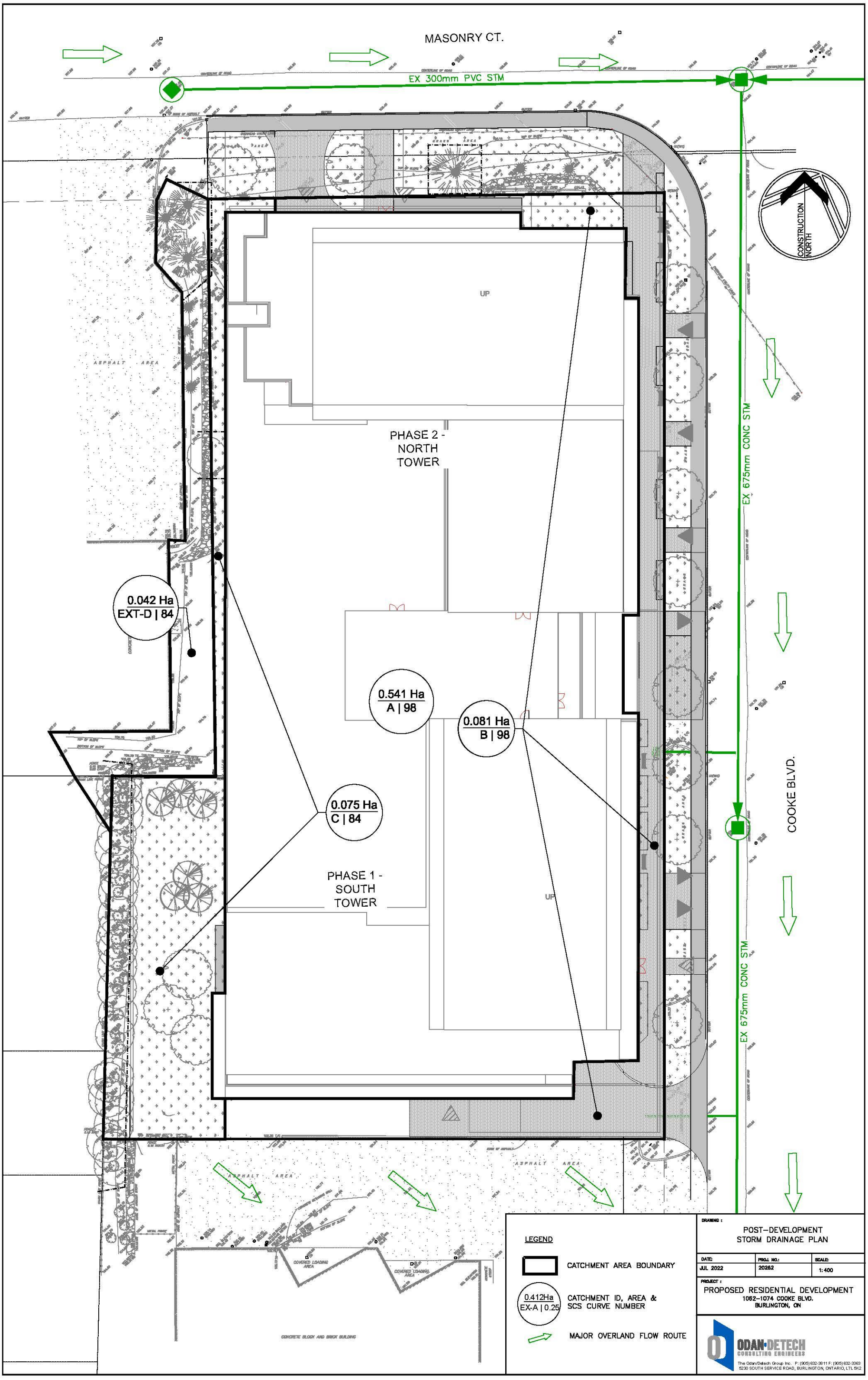
Foundation drainage water will be pumped into the storm control maintenance hole (on a long-term basis) at a maximum flow rate of 4.0 L/s. The site's controlled stormwater release rate is revised accordingly such that the total discharge from the site remains less than allowable, despite the foundation drainage flows. This is considered in Table 6, below, where it is shown that the controlled storm flows are no greater than allowable, considering the foundation drain flows.

The stormwater detention design of the tank was modelled using the *EPA SWMM* hydrology methodology using *AutoDesk Storm and Sanitary Analysis (SSA) 2023*. The storm catchment areas are delineated on the Post-Development Catchment Plan on the following page. The input parameters to the model are summarized in Table 4 as follows. The hydrology input parameters were then inputted into the catchment area subbasin *Phase1&2Catchment* in the SSA model (Fig. 1).

Soil Group *D* was conservatively assumed given that all surfaces are effectively entirely impervious.

TABLE 4 - Catchment Characteristics for the site Post-Development

Area I.D.	Area (ha)	Hydrology Method	SCS Curve Number	Soil Group	Depression Depth (mm)	Pervious Depression (mm)	Equivalent Width (m)	Average Slope (%)
Catchment A: Impervious Building Roof	0.541	EPA SWMM	98	D	1	5	10	4
Catchment B: Impervious paved surfaces on-top of parking structure	0.081	EPA SWMM	98	D	1	5	10	4
Catchment C: Pervious Landscape Surfaces	0.075	EPA SWMM	84	D	1	5	10	4
Catchment D: External Pervious Landscape Surfaces	0.042	EPA SWMM	84	D	1	5	10	4



DRAWING :

POST–DEVELOPMENT
STORM DRAINAGE PLAN

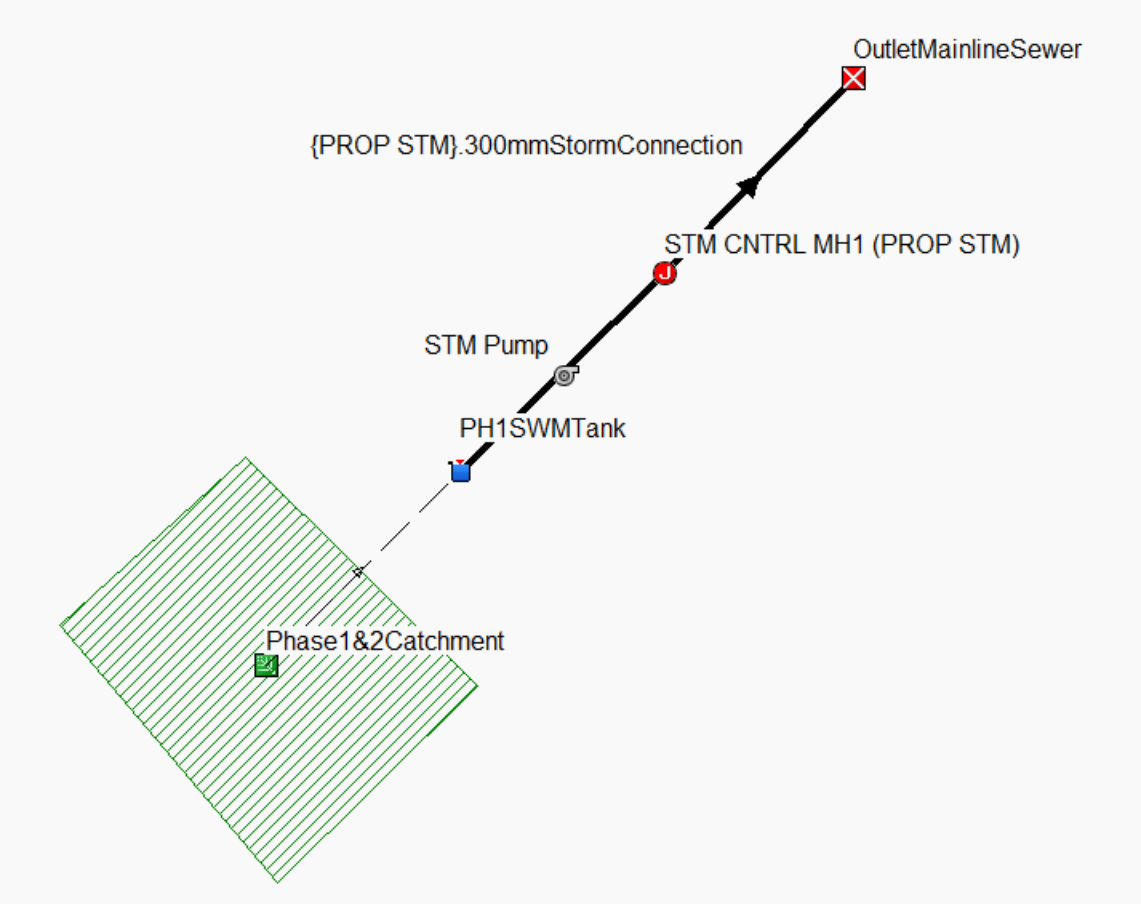
DATE:	PROJ. NO.:	SCALE:
JUL 2022	20262	1:400

PROJECT :

PROPOSED RESIDENTIAL DEVELOPMENT
1052–1074 COOKE BLVD.
BURLINGTON, ON

ODAN·DETECH
CONSULTING ENGINEERS
The Odan/Detech Group Inc. P: (905) 632-3811 F: (905) 632-3363
5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

Figure 1 – Storm and Sanitary Analysis Model



The stage-storage-discharge relationship for the proposed storm tank with pumped outlet storm control is as follows. The tank is a proposed 38m^2 footprint.

TABLE 5 – Stage/Storage/Discharge Relationship for Storm Tank

Head (m)	Elevation (m)	Provided Volume (m^3)	Discharge (l/s)	Description
0.00	96.00	0	0	Bottom of Tank (Pump On)
0.50	96.50	19	94	Tank
1.00	97.00	38	94	Tank
2.00	98.00	76	94	Tank
2.97	***98.95***	***115***	***94***	***100-Y Max Volume in Tank***
3.00	99.00	115	94	Tank
4.50	100.50	170	94	Tank Roof
3.00				

Actual 100-year Discharge from Pump:
100 Year = 94 l/s

The SSA EPA SWMM stormwater detention modelling results are provided below in Figures 2, 3, 4 and 5, as follows. The detailed output report is provided in Appendix C.

- Figure 2 – Burlington 5-Year storm - Time series plot of rainfall intensity, runoff flow rate, detention storage volume and storm pump flow rate.
- Figure 3 – Burlington 5-Year storm – Profile Plot through storm detention tank, pump and storm connection.
- Figure 4 – Burlington 100-Year storm - Time series plot of rainfall intensity, runoff flow rate, detention storage volume and storm pump flow rate.
- Figure 5 – Burlington 100-Year storm – Profile Plot through storm detention tank, pump and storm connection.

The following is a summary of the below results.

TABLE 6 – Stormwater Management Summary

	5 Yr. Storm	**100 Yr. Storm**
Total Flows Attenuated by 94 L/s Pump	94 L/s	94 L/s
Max. Pumped Foundation Drain Flows	4 L/s	4 L/s
Total Flows	98 L/s	98 L/s
Allowable Flow Rate	***98 L/s***	
Required 100-Year Volume	17 m^3	115 m^3
Provided Tank Volume	***170 m^3***	

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

Figure 2 - Burlington 5-Year Storm – Time Series SWM Results

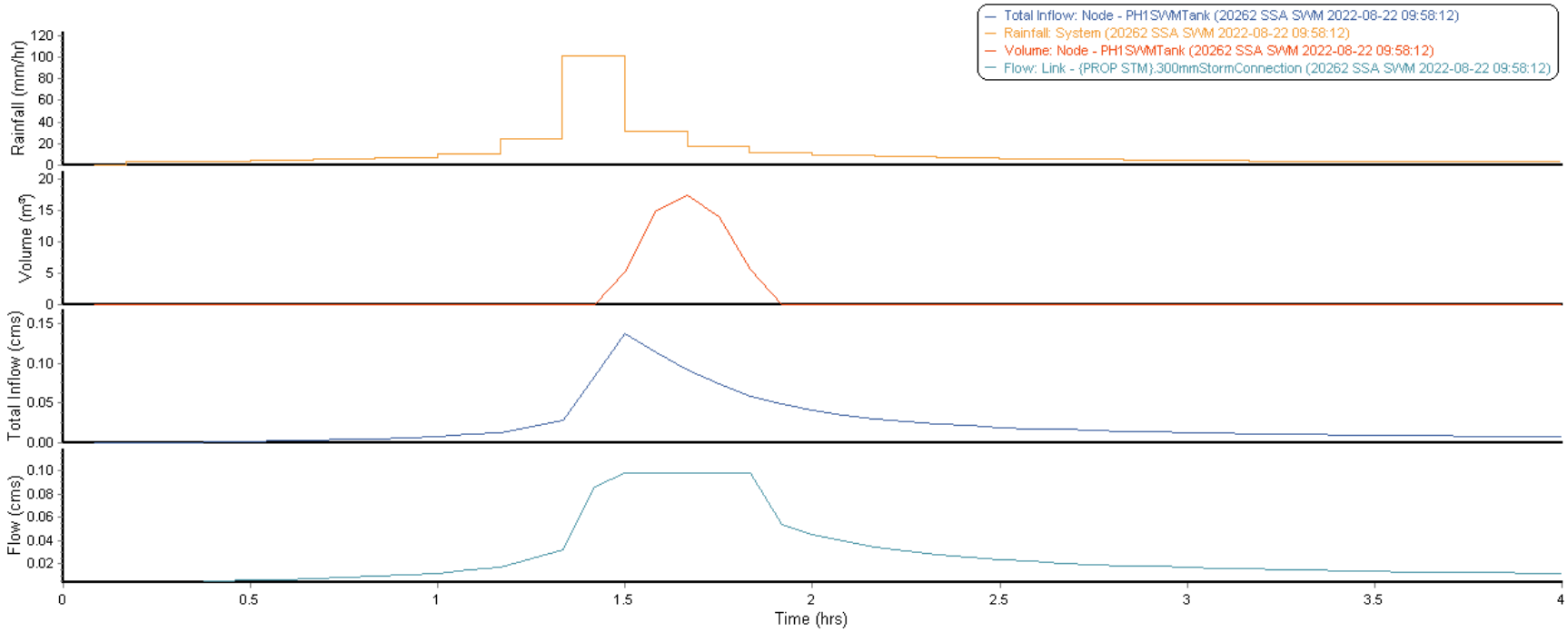
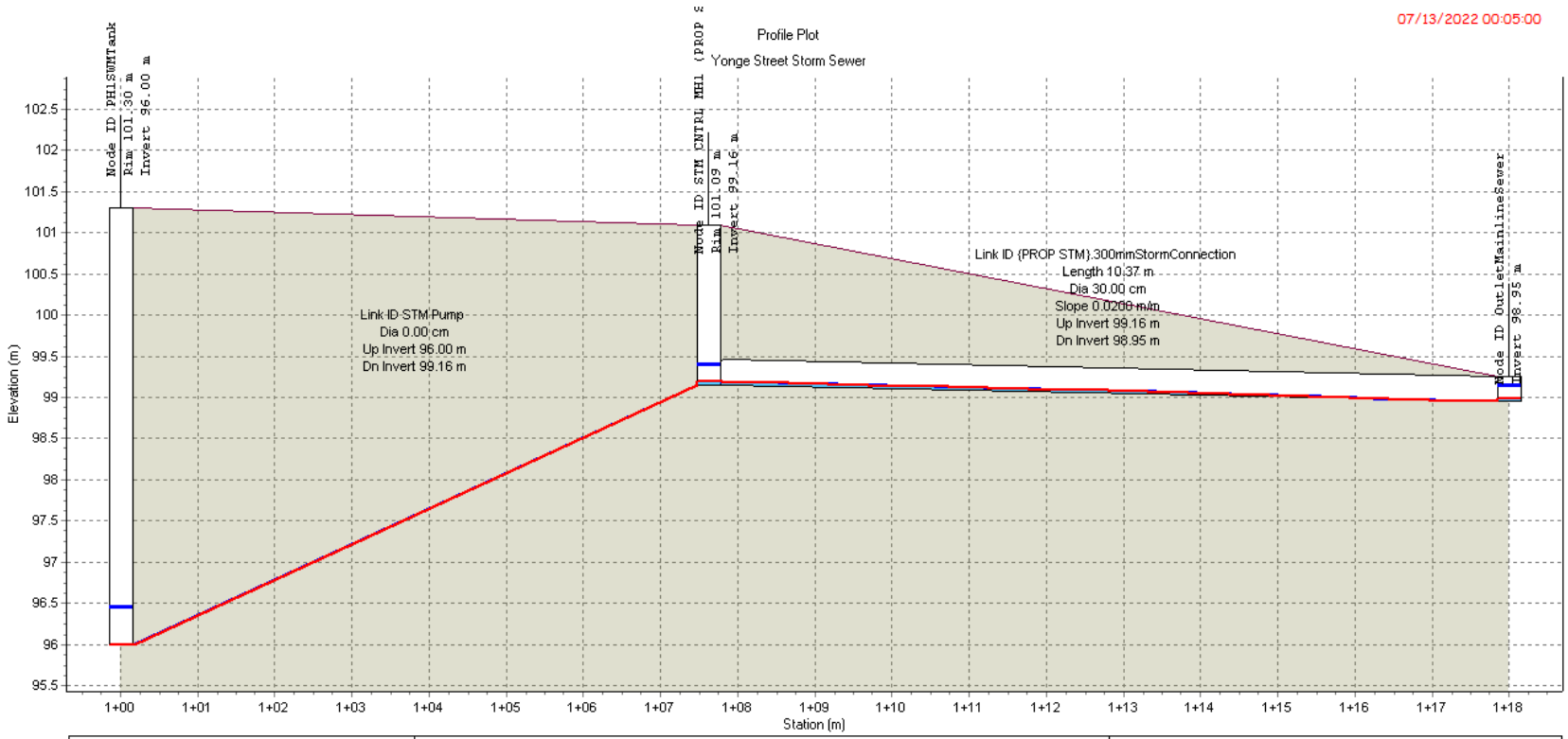


Figure 3 - Burlington 5-Year Storm Profile Results



Node ID:	PH1SWMTank	STM CNTRL MH1 (PROP STM)	
Rim (m):	101.30	101.09	
Invert (m):	96.00	99.16	98.95
Min Pipe Cover (m):		0.00	
Max HGL (m):	96.45	99.40	99.14

Link ID:	STM Pump	{PROP STM}.300mmStormConnection
Length (m):		10.37
Dia (cm):	0.00	30.00
Slope (m/m):		0.0200
Up Invert (m):	96.00	99.16
Dn Invert (m):	99.16	98.95
Max Q (cms):	0.09	0.10
Max Vel (m/s):	0.00	1.89
Max Depth (m):	0.00	0.22

Figure 4 - Burlington 100-Year Storm - Time Series SWM Results

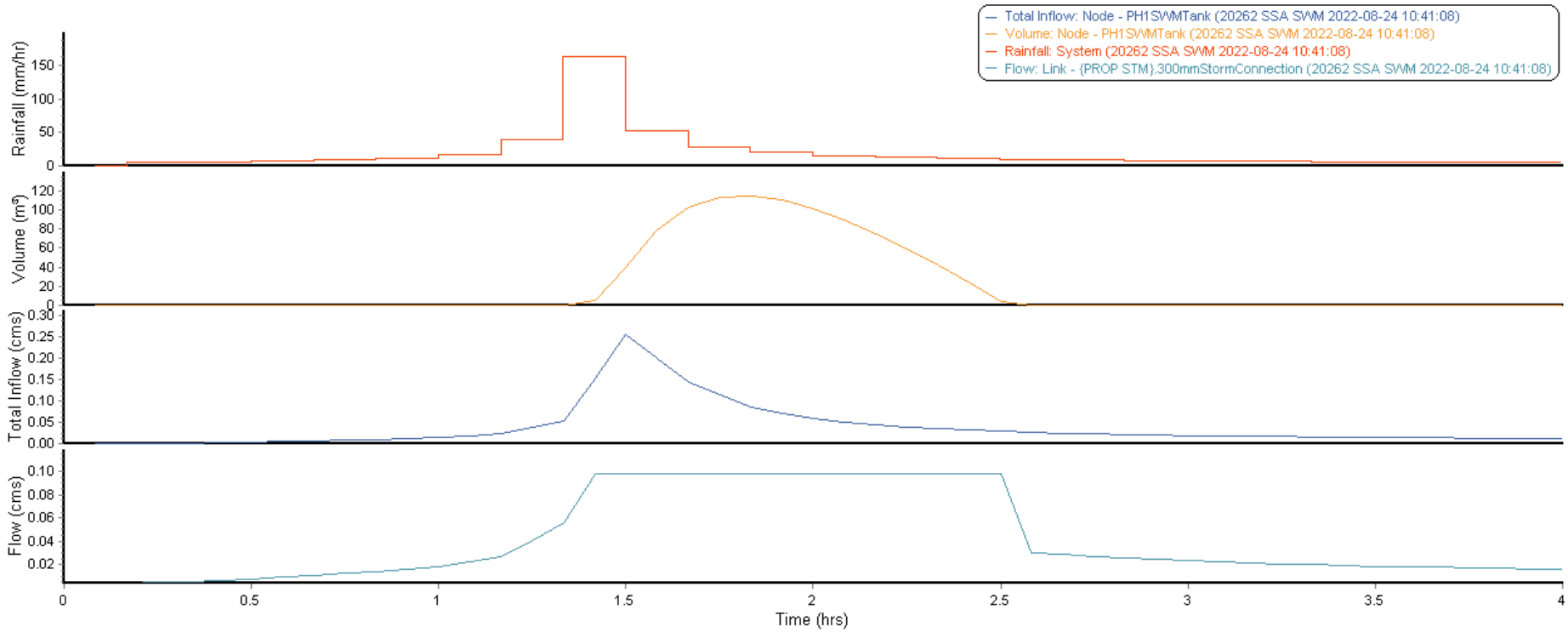
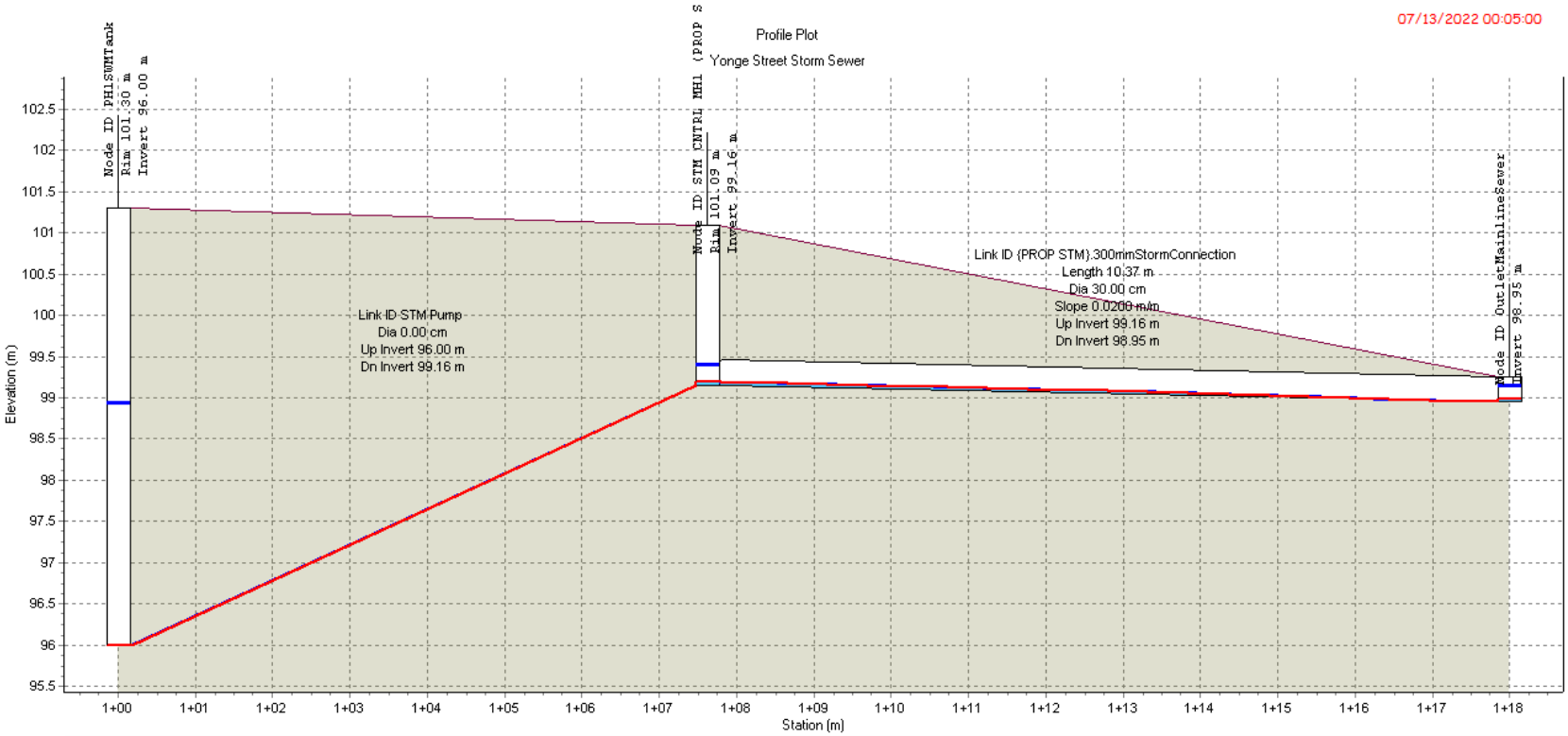


Figure 5 - Burlington 100-Y Storm Profile Results



Node ID:	PH1SWMTank	STM CNTRL MH1 (PROP STM)	
Rim (m):	101.30	101.09	
Invert (m):	96.00	99.16	98.95
Min Pipe Cover (m):		0.00	
Max HGL (m):	98.95	99.41	99.14

Link ID:	STM Pump	{PROP STM}.300mmStormConnection
Length (m):		10.37
Dia (cm):	0.00	30.00
Slope (m/m):		0.0200
Up Invert (m):	96.00	99.16
Dn Invert (m):	99.16	98.95
Max Q (cms):	0.09	0.10
Max Vel (m/s):	0.00	1.94
Max Depth (m):	0.00	0.22

v) Water Budget/Stormwater Retention

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

Storm water management water budget/retention criteria is given in the City of Burlington's manual *Stormwater Management Design Guidelines* (May 21, 2020). Section 6.1.3 therein provides site-level stormwater management stormwater retention/water budget criteria, which would apply to this development. That section states that proponents are required to maintain the water budget in accordance with a SWS, MDP or MESP that encompasses the subject site, or where not available, maintain pre-development water budget in the proposed development.

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

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

The water balance target volume is as follows.

	Initial Abstraction (mm)	Area (m^2)	Volume (m^3)
5mm Volume Whole Site	5	7000	35.0
Less Impervious Surfaces initial abstraction – Catchment A & B	1	6220	6.2
Less Pervious Surfaces initial abstraction – Catchment C	5	780	3.9
Required Retention Volume			***24.9***

A cistern of minimum volume 28.0m^3 has been designed to retain the required volume of stormwater. The cistern will function such that in minor storm events, runoff draining from the site's mechanical drains will first drain into the cistern for storage and reuse.

In storm events greater than 5mm, the cistern will fill-up and stormwater will spillover into the larger storage area allocated for 100-year storm storage before draining via the site's storm connection. Refer to Cross-Section A-A on the Functional Servicing Plan.

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

1) ***Infiltration Gallery (Percolation)***: The proposed development is entirely comprised of the proposed below-grade parking structure below-grade. Infiltration is not feasible because the

design criteria – *MOE Stormwater Management Planning & Design* Manual, 2003, as well as the OBC – requires such an infiltration gallery to be located with a minimum 4.00m horizontal separation from proposed buildings. There is no such location on this site in which to locate an infiltration gallery, therefore it is not feasible to install an infiltration gallery.

2) ***Irrigation***: Irrigation shall be explored in future as there are landscaped surfaces including trees at-grade – the capacity to uptake water by irrigation will be determined at the SPA stage. It is anticipated that irrigation alone will not consume the required volume, therefore greywater reuse is also investigated as follows.

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

vi) Water Quality

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

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

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

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

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

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

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

The CETV certification for the *Stormceptor STC*-model conventional Oil-Grit Separator by Imbrium Systems Inc. concurs with the town manual that the conventional OGS provides 50% TSS removal. Figure 6, as follows, is an excerpt from the CETV *Verification Statement – Imbrium Systems Inc. Stormceptor … Oil-Grit Separators* (November 2017).

Figure 6 shows that the Stormceptor conventional OGS provides typically 50-70% TSS removal, therefore the conclusion holds that a conventional OGS does not satisfactorily address the stormwater quality criteria for this site because it does not provide 80% TSS.

Figure 6 - Excerpt from CETV Verification Statement for Stormceptor OGS

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

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

TSS Removal efficiency is ~50%

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

The CETV also provides certification for stormwater quality filters that provide TSS removal by mechanical filtration through a filter media. One such model is the Jellyfish Filter by Imbrium Systems. The CETV verification statement for the Jelllyfish Filter by Imbrium Systems states that it provides a minimum 80% TSS Removal. Figure 7, as follows, is an excerpt from the CETV *Verification Statement – Imbrium Systems Inc. Jellyfish Filter …* (August 2017).

Figure 7 shows that the Jellyfish Filter provides 80% TSS Removal, therefore it addresses the City criteria for 80% TSS Removal. The entire Jellyfish Filter CETV Verification Statement is provided here in Appendix C.

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

A Jellyfish Filter will be specified to provide 80% TSS Removal at the SPA stage. Refer to the Functional Servicing Plan for the proposed location of the stormwater filter.

Figure 7 - Excerpt from CETV Verification Statement for Jellyfish Filter

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

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

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6.0 GROUNDWATER

i) Introduction

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

A *Preliminary Hydrogeological Investigation* (dated August 10, 2021) has been prepared for this development by BIG Consulting Inc. to qualitatively and quantitatively characterize the groundwater with respect to City of Burlington and Halton Region guidelines.

The report concludes in Table 5-2 that the short-term construction dewatering volume will be up to 1,076,000 L/day (with a safety factor of 2).

The report concludes in Table 6-2 that the long-term flows into the building's foundation drains will be 214,000 L/day (including peaking factor), for 3-4 below-grade levels.

The report concludes in Table 4-4 that the groundwater quality complies with the criteria for discharge into Halton Region's sanitary sewer/wastewater main and the report does not provide conclusions regarding the compliance of the groundwater's quality relative to the latest City of Burlington quality criteria for discharge to the storm sewer.

ii) Long-Term Groundwater

The hydrogeological investigation states as above that the peak daily volume entering the foundation drains will be 214,000 L/day, which equates to flows of 2.5 L/s. It is intended to collect this water in a sump installed in the proposed buildings below-grade levels and pump it into the proposed Building's storm control maintenance hole, thereby draining to the storm sewer connection. It is assumed that a peaking factor will be applied in the design of the pump such that the pumped groundwater discharge rate is 4.0 L/s. This is considered in the controlled storm flows compliance, as outlined in Table 6, above. It is discussed above that the site remains within the allowable storm flows notwithstanding the foundation drainage flows.

The owner intends to review, in future, the feasibility of discharging the foundation drainage groundwater to the storm sewer connection, relative to the City criteria. If, pending future hydrogeological conclusions, it is unfeasible to filter the water to address such criteria, the building's foundations may then instead be 'bathtubbed' such that there is no groundwater discharge.

iii) Short-Term (Construction) Groundwater

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

The short-term groundwater flow rate identified by the Hydrogeological Assessment totals up to 1,076,000 L/day (12.5 L/s). It is proposed to discharge this flow rate into one of the Site's existing storm sewer laterals (as shown on the Functional Servicing Plan). This temporary groundwater flow rate is less than the allowable storm flow rate (98.0 L/s, as per Table 3), therefore short-term/construction groundwater flows are within the allowable flow rate.

7.0 CONCLUSIONS

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

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

TABLE 7 - Summary

	Phase 1 – South Tower	***Phase 2 – North Tower***
Peak Sanitary Discharge (L/s)	19.9 L/s (both towers combined)	
Proposed Sanitary Service	200mm @ 2.0%	200mm @ 2.0%
Receiving Sanitary Sewer	Cooke Blvd. 300mm Sanitary sewer	Cooke Blvd. 300mm Sanitary sewer
Development Water Demand (Fire + Domestic)	1778 USGM (largest fire flow demand is given by common podium)	
Available Flow Rate	1890 USGM	
Proposed Fire Service	200mm + redundant 200mm	200mm + redundant 200mm
Proposed Domestic Service	Branch 150mm	Branch 150mm
Allowable release rate from site (L/s)	98.6 L/s	
Proposed release rate from site (L/s) (100 year storm)	98.0 L/s	
Stormwater Quality	Jellyfish Stormwater (or eq.) Filter – to be designed at SPA Stage	
Quantity Control	133m^3 Stormwater Detention Storage Tank	

8.0 REFERENCES

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

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

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

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

Respectfully Submitted;
The Odan Detech Group Inc.

Aug. 25/22

Paul Hecimovic, P.Eng.



Mark Harris, Dipl.Tech.
Senior Project Manager

Mark Harris
Digitally signed by Mark Harris
DN: cn=Mark Harris, o=The Odan/Detech Group Inc., ou, email=mark@odandetech.com, c=CA
Date: 2022.09.12 14:37:13 -04'00'

APPENDIX A

Existing Site — Aerial view of Site and surrounding area

Site Plan & Statistics — by RAW

Correspondence with City of Burlington Staff Regarding MTSA Status



LEGEND

PROPERTY LINE

DRAWING :

KEY PLAN

DATE:	PROJ. NO:	SCALE:
JUL 2022	20262	N.T.S.

PROJECT :

PROPOSED RESIDENTIAL DEVELOPMENT
1062–1074 COOKE BLVD.
BURLINGTON, ON

ODAN·DETECH
CONSULTING ENGINEERS
The Odan/Detech Group Inc. P: (905) 632-8811 F: (905) 632-3363
5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

From: Mailbox, Community Planning - MTSA Project <mtsa@burlington.ca>
Sent: July 22, 2022 7:25 AM
To: Daniel Bancroft <daniel@civilgo.ca>; Mailbox, Community Planning - MTSA Project <mtsa@burlington.ca>
Cc: mark@odandetech.com
Subject: RE: Status of Aldershot GO MTSA Servicing (sanitary sewers) report?

Hi Daniel,
Thank you for reaching out. The report remains under review by both the Region and the City with further revisions expected. I anticipate that the report will be available early in the fall, however I would encourage you to visit getinvolvedburlington.ca/mtsa and subscribe for project updates. As soon as the report is completed, it will be posted to get involved.

Please feel free to reach out to me directly if you have any further questions.

Thank you,

Jenna

Jenna Puletto, M.Pl., MCIP, RPP
Coordinator of Community Initiatives
Community Planning Department

P. 905 335 7600, ext 7445 | F. 905 335 7880 | E. jenna.puletto@burlington.ca
Address 426 Brant Street P.O. Box 5013, Burlington, Ontario, L7R 3Z6
City of Burlington | www.burlington.ca

I understand our working hours may be different and want you to know, not only are you not obligated to reply outside of your typical working hours, I am asking you not to reply outside your typical working hours.

Although staff are not able to serve you in-person at this time, e-mail, telephone and virtual appointments are available as needed. Please email planning@burlington.ca for more information.

From: Daniel Bancroft <daniel@civilgo.ca>
Sent: Wednesday, July 13, 2022 10:40 AM
To: Mailbox, Community Planning - MTSA Project <mtsa@burlington.ca>
Cc: mark@odandetech.com
Subject: Status of Aldershot GO MTSA Servicing (sanitary sewers) report?

CAUTION: This email originated from outside of the organization. Do not click links or open attachments unless you recognize the sender and know the content is safe.
Good morning,

Kindly inquiring regarding the status and timeline to completion of the Aldershot GO MTSA Servicing report pertaining to sanitary sewers/wastewater mains?

Halton Region engineering staff deferred to this email to find out anticipated timing of completion.

Thanks
Daniel

APPENDIX B

1062-1074 Cooke Boulevard Sanitary Sewer Capacity Analysis Report by Civica Infrastructure Ltd.

CIVICA
Water Management Solutions

Prepared For

The Odan/Detech Group Ltd.

Report For

1062-1074 Cooke Boulevard Sanitary Sewer Capacity Analysis

August 24, 2022

330 Rodinea Road, Unit 3
Vaughan, Ontario, Canada L6A 4P5
(905) 417-9792
www.civi.ca

CIVICA
Water Management Solutions

STATEMENT OF QUALIFICATIONS AND LIMITATIONS

The attached Sanitary Capacity Analysis (the "Report") has been prepared by Civica Infrastructure Inc. (the "Consultant") at the request of, and for the exclusive use of, the Odan/Detech Group Ltd. (the "Client") in accordance with the terms of agreement between the Consultant and the Client, including the scope of work detailed therein (the "Agreement").

Please note that the information, data, analysis, recommendations, and conclusions contained in the Report was prepared for the specific purposes described in the Report and the Agreement and may be based upon information which has not been independently verified by the Consultant. The Consultant shall be entitled to rely upon the accuracy and completeness of information that was provided to the Consultant and has no obligation to update such information. The material in this report reflects the Consultant's best professional judgement in the light of the information available to it at the time of preparation and publication.

The Consultant agrees that the Report represents its professional judgement as described above and that the Information has been prepared for the specific purpose and use described in the Report and the Agreement. The Consultant makes no other representations, any guarantees, or warranties whatsoever, whether expressed or implied, with respect to the Report or any part thereof.

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The Consultant expressly excludes liability to any party except the Client for either any use of or reliance upon the Report.

This Statement of Qualifications and Limitations is attached to, and forms part of the Report and any usage of the Report is subject to the terms therein.

August 24, 2022 **CIVICA REF: ODA21-0025**

The Odan/Detech Group Inc.
5230 South Service Road, Unit 107
Burlington, Ontario
L7L 5K2

Attention: Daniel Bancroft, P.Eng.

Dear Mr. Bancroft,

RE: 1062-1074 Cooke Boulevard Sanitary Capacity Analysis

Civica Infrastructure Inc. (Civica) is pleased to submit the Sanitary Capacity Analysis for the proposed development at 1062-1074 Cooke Blvd in Burlington, Ontario. Civica developed its own model network in InfoWorks ICM to assess the existing and proposed conditions of the sanitary sewer downstream of the proposed development. The model was used for the hydraulic analysis to determine the peak flow and Hydraulic Grade Line (HGL) elevation within the downstream sanitary sewer.

Based on the analysis and assumptions presented in the following report, the findings can be summarized as follows:

1. The expected sanitary peak flow from the proposed development, as observed at the outlet, is anticipated to be 19.94 L/s;
2. The upgraded sanitary sewer is able to maintain free-flowing conditions during dry-weather flow for the proposed intensification along Cooke Blvd, north of Plains Rd E, including the proposed flows for 1062-1074 Cooke Blvd;
3. The upgraded sanitary sewer is able to maintain basement flooding protection during wet-weather flow conditions for the proposed intensification along Cooke Blvd, north of Plains Rd E, including the proposed flows for 1062-1074 Cooke Blvd;
4. The existing sanitary sewer system does not have the capacity to accommodate the existing or proposed wet-weather flows without the need for any external upgrades or retrofits; and,
5. Based on the analysis and assumptions presented in the report, it is recommended that the development proceeds after the sanitary sewer upgrades along Cooke Blvd and Plains Rd E have successfully been completed.

Do not hesitate to contact us for further clarification and/or comment.

Sincerely,

CIVICA INFRASTRUCTURE INC.

Robert Hughson

Project Manager

Encl. 1062-1074 Cooke Blvd Sanitary Capacity Analysis Report

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Revision History

Name	Date	Reason for Change	Version
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1.0 Introduction

This report summarizes the sanitary servicing analysis for the proposed development located at 1062-1074 Cooke Boulevard, between Masonry Court and Plains Road East, in Burlington, Ontario.

The existing site is currently occupied by a pair of single storey commercial buildings. The proposed site development consists of a 32-storey mixed use building with a population of approximately 1,705 persons.

The capacity conditions in the existing sanitary sewer system have been evaluated from the proposed site to the trunk sewer on Waterdown Road. **Figure 1-1** shows the location of the proposed development site and the contributing drainage area.

1.1 Servicing Connections

Sanitary flows from the existing site are currently discharging to the 300-mm sanitary sewer on Cooke Blvd. The sanitary sewer flows south on Cooke Blvd converging with westbound flows in the 250-mm sanitary sewer on Plains Rd E. The flows ultimately discharge into the 675-mm Sanitary Trunk Sewer (STS) at the intersection of Waterdown Rd and Plains Rd E. **Figure 1-2** shows the downstream sewer conveyance flow the outlet.

It is proposed to upgrade three (3) sections of sanitary sewer along Cooke Blvd from 300-mm to 375-mm and three (3) sections of sanitary sewer along Plains Rd E from 250-mm to 450-mm in order to mitigate existing dry-weather flow issues. These proposed upgrades have been sized sufficiently to ensure that the water level (HGL) is less than the existing conditions even after the proposed developments are implemented. See **Figure 1-2** for location of upgrades along the conveyance.

1.2 Capacity Analysis Approach

Initial research performed by the Odan Detech Group revealed that previous work done in the area (1120 Cooke Blvd) had utilized a dynamic model developed using InfoWorks at the request of the Region. A similar approach was used due to a dynamic model being more conservative than the Region's static InfoSewer model. Civica developed its own model network in InfoWorks ICM using data from the Region's InfoSewer model. The drainage area population was estimated using information found in the 1120 Cooke Blvd Sanitary Analysis Report and comparing it with the Region's Linear Design Manual. The dry-weather generation rate used for the drainage area in the model uses a generation rate of 275 L/c/d for both residential and non-residential areas. These generation rates were obtained from the Region's Linear Design Manual. This model was used to assess the sanitary sewer downstream of the proposed site. Further details on the modelling methodology can be found in **Section – Methodology**.

The drainage system was also evaluated to include new and other proposed developments/re-developments in the area. Four (4) new developments were found in the drainage area. **Table 1-1** shows the information of the new developments and **Figure 1-1** shows the location of the proposed development site.

Table 1-1: Proposed Developments in the Drainage area

Number	Site Address	Site Area (ha)	Population	Peak I/I (L/s)	Total Peak Flow (L/s)
1	53 Plains Rd E	0.97	285	0.28	3.98
2	1025 Cooke Blvd	0.40	326	0.11	4.33
3	1120 Cooke Blvd	1.02	3299	0.29	36.07
4	101 Masonry Crt	3.17	2090	0.91	24.66

Note: Future Developments Assumes a per-capita generation rate of 275 l/c/d for residential and an I/I rate of 0.286 L/s/ha.

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Legend

- Drainage Area
- New Development
- Proposed Site

Sewer Infrastructure

Manhole

- Sanitary
- Outlet

Sewerline

- Sanitary
- Sanitary Trunk

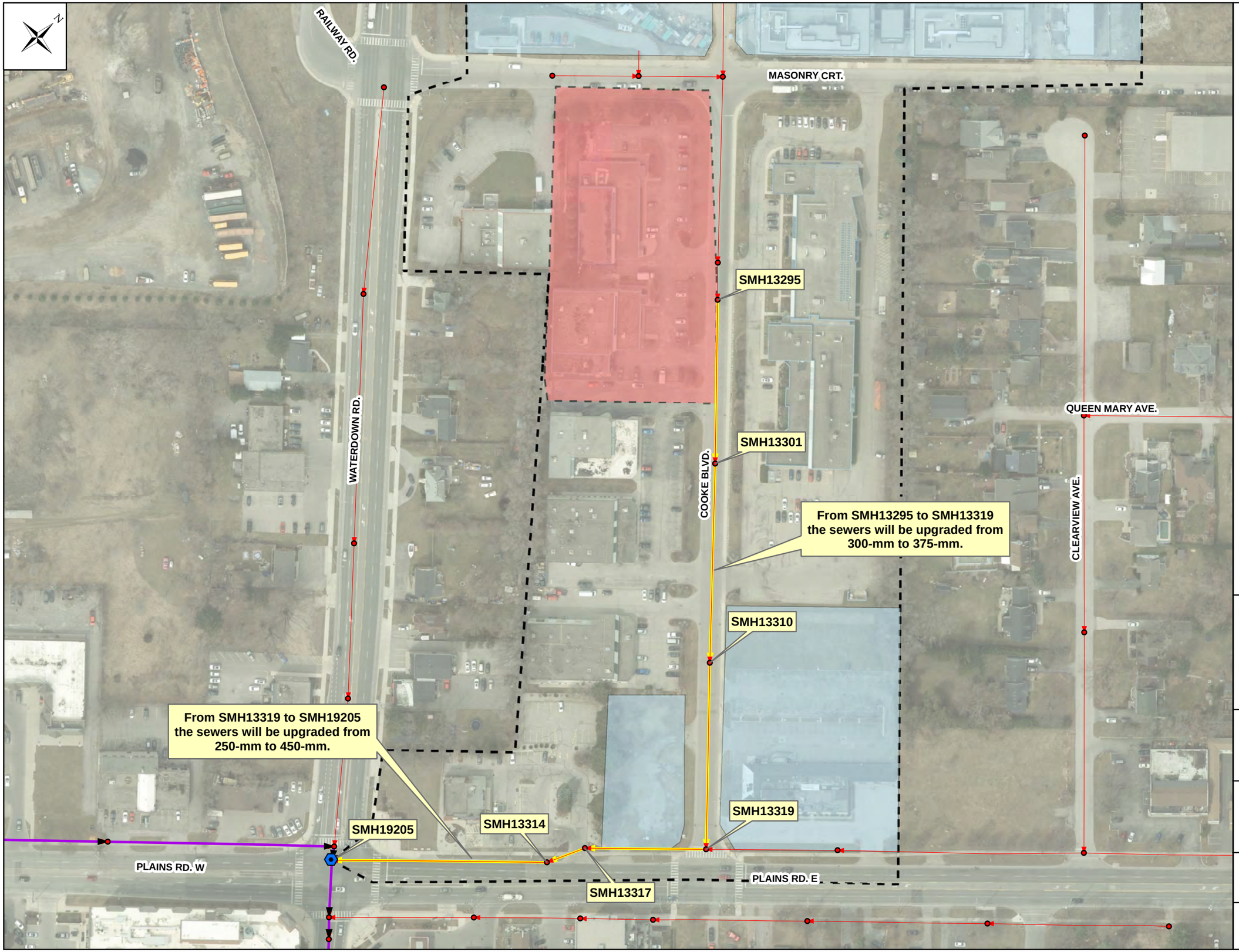
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Figure 1-1:
Sanitary Sewer Tributary Area

Drawn By: R.H. Date: Aug 24, 2022

0 25 50 100 Meters



Legend

- Drainage Area
- New Development
- Proposed Site

Sewer Infrastructure

Manhole

- Sanitary
- Outlet

Sewerline

- Sanitary
- Conveyance
- Sanitary Trunk

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Figure 1-2:
Downstream Conveyance of Proposed Sanitary Flows

Drawn By: R.H. Date: Aug 24, 2022

0 15 30 60 Meters

2.0 Design Sanitary Flows

The flow generation for the existing site was calculated based on the land use and in accordance with the October 2019 (Fifth Edition), Halton Region Water and Wastewater Linear Design Manual. **Table 2-1** shows the criteria, assumptions, and flow calculations for both existing and future conditions.

2.1 Existing Sanitary Flows

Total flow into the sanitary sewer under existing condition is 0.39 L/s; this includes dry weather flow and infiltration allowance flow.

2.2 Proposed Sanitary Flows

Total flow into the sanitary sewer under proposed condition is 19.94 L/s; this includes:

- Infiltration Allowance = 0.20 L/s;
- Non-Residential Flow = 0.06 L/s; and,
- Residential Flow = 19.68 L/s.

Table 2-2 shows the difference between the existing flows and proposed flows.

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Table 2-1: Existing and Future Flows from the Site

	Description	Unit	Existing	Proposed
Site Properties	Site Area	ha	0.696	0.696
	Commercial Area	m^2	2,303	625
	Office Area	m^2	0	0
	Industrial Floor Area	m^2	0	0
	Number of Units:			
	1BR/1BR+Den Apt	Units	0	616
	2BR/2BR+Den/ Apt	Units	0	176
	3BR/3BR+Den/ Apt	Units	0	17
	Total Units*	Units	0	809
Criteria	Sanitary Flow Generation Rate	L/ca/day	275	
	Harmon peaking factor	-	$1 + 14 / [4+(Pop/1000)^{0.5}]$	
	Commercial Average Flow	L/ha/d	24,750	
	Infiltration	L/s/ha	0.286	
	Retail /Residential Population Criteria:			
	Single Family	persons/ha	55	
	Semi-detached	persons/ha	100	
	Townhouse, Maisonette	persons/ha	135	
	Apartment (over 6 stories high)	persons/ha	285	
	Commercial / Retail (equivalent population)	persons/ha	90	
Population	Residential Population, using 2.1 persons/unit	persons	0	1699
	Total Residential Population*	persons	**0**	**1699**
	Commercial / Retail (equivalent population)	persons	2073	6
	Total Non-Residential Population*	persons	**2073**	**6**
DWF Calculation	Harmon Peaking Factor	-	4.50	3.64
	Peaking Factor (ICI)	-	2.86	3.55
	Residential DWF	L/s	0.00	19.68
	ICI DWF	L/s	0.19	0.06
Infiltration Allowance		L/s	0.20	0.20
Design Flow	**DWF + Infiltration + Other Flows**	**L/s**	**0.39**	**19.94**

Table 2-2: Difference of Flows for the Existing and Proposed Conditions

Flow Component	Pre-Development	Proposed	Net Increase
Residential Peak Flow (L/s)	0.00	19.68	19.68
Non-Residential Peak Flow (L/s)	0.19	0.06	-0.13
Infiltration allowance (L/s)	0.20	0.20	0.00
Total (L/s)	**0.39**	**19.94**	**19.55**

Note: Total post-development peak flows should not be calculated as proposed + pre-development, due to difference in Harmon Peaking factor at each stage.

3.0 Methodology

The existing residual capacity in the external sewers has been calculated by using design flows, and a detailed pipe-by-pipe hydraulic model of the system.

The work plan consisted of the following general steps:

1) The collection of relevant data for estimating population data and developing subcatchments, sewer asset data, and existing developments;
2) Hydrologic and hydraulic modeling (InfoWorks ICM) and capacity analysis under various scenarios, including:
    a. Dry-Weather Flow (DWF):
        i. Existing Conditions;
        ii. Proposed Conditions;
        iii. Upgraded Conditions;
    b. Wet-Weather Flow (WWF):
        i. Existing Conditions;
        ii. Proposed Conditions;
        iii. Upgraded Conditions.

When calculating the total flow and resulting hydraulic gradelines (HGL) in the sewer system, the population in the existing upstream and downstream areas were estimated using the Region's *Water and Wastewater Linear Design Manual, Section 3 – Wastewater Main Design*. The flow generation from the various tributary areas upstream and downstream from the site were calculated using InfoWorks ICM hydrodynamic model.

The per capita dry-weather flow generation in the model is 275 L/c/d for residential areas and 24,750 L/ha/d for commercial areas. For proposed conditions, the model assumes a residential sewage generation rate of 275 L/c/d. The infiltration rate for each subcatchment area uses a rate of 0.286 L/s/ha. These rates and the applicable peaking factors used were obtained from the *October 2019 (Fifth Edition), Water and Wastewater Linear Design Manual – Section 3: Wastewater Main Design; Table 3-1: Residential Dry Weather Flow* and *Table 3-2: Commercial, Industrial, and Community Dry Weather Flow.*

3.1 Capacity Criteria

Assuming that the typical basement elevation is 1.8 m below the road centerline, different levels of service were established, prior to conducting the sanitary sewer capacity assessment. In general, the targeted level of service can be described as:

- During dry-weather flow conditions, the sanitary sewer will maintain free-flowing conditions; and,
- During wet-weather flow conditions, the maximum Hydraulic Grade Line (HGL) of the sanitary system that shall be maintained below basement elevation (1.8 m below the ground elevation).

3.2 Model Setup

The model was developed in GIS by delineating the sewer network upstream of the outlet. Subcatchments were designed based on natural boundaries (land parcels and manhole to manhole). Conservative population estimates were inferred for each subcatchment and the area in hectares was calculated. This information was utilized to develop the flow generation rate and Harmon Pattern for each area. The drainage network was then imported into InfoWorks ICM and validated by the software's engineering validation to ensure that it is consistent with expected engineering values. Any warnings or errors at this stage were evaluated and resolved before starting the modelling

process. The Harmon Pattern for each wastewater profile in Existing and Proposed Scenarios was then imported into InfoWorks ICM and the subcatchments were assigned their respective wastewater profiles.

3.3 Population Estimates

The population in the existing upstream and downstream areas were estimated using the design criteria found in the Region's *Water and Wastewater Linear Design Manual – Section 3: Wastewater Main Design; Table 3-1: Residential Dry Weather Flow* and *Table 3-2: Commercial, Industrial, and Community Dry Weather Flow.* The manual provides population density estimates based on land use and unit type. Remote sensing data in GIS was utilized to determine the land use and unit counts per land parcel. These two resources allow the population per land parcel to be inferred and the populations from each land parcel are summarized per subcatchment.

3.4 Rainfall Derived Inflow and Infiltration (RDII) Rate

The Region's *Water and Wastewater Linear Design Manual – Section 3: Wastewater Main Design* provides an inflow/infiltration rate allowance of 0.286 L/s/ha for future conditions. However, a rate for the existing conditions was not provided in this manual. In lieu of this, Civica utilized the City of Toronto's *Design Criteria for Sewer's and Watermains, Chapter 2 - Sanitary Sewers; Table 5: Peak Flow Design Parameters* to determine a suitable wet-weather inflow and infiltration rate. According to the table referenced above, an inflow rate of 3 L/s/ha should be used in drainage areas between 10 to 100 ha. Civica has found this rate has been found to be a suitable alternative to the 25-year design storm flows in models where calibration has not been performed. Additionally, the 3 L/s/ha inflow rate is often a slightly more conservative rainfall derived inflow and infiltration rate for sanitary areas when compared to monitoring data.

4.0 Capacity Analysis Results

4.1 Dry-Weather Flow: Design Function

The following section assesses whether the proposed development meets the design function of the existing sewer system. Since this development is in a sanitary sewer drainage area, the City of Toronto's sanitary sewer criteria were used in our assessment. Therefore, the following results are based on of dry-weather flow. The City's Design Function Criteria can be summarized as requiring that during dry-weather flow the sanitary sewer must maintain free-flowing conditions.

Using the above criteria, the findings can be summarized as follows:

- Under existing and proposed conditions, the sanitary sewer system is experiencing a bottleneck or backflow at six (6) locations;
- Under the upgraded conditions, the sanitary sewer is operating under free-flowing conditions;
- The peak flow along Cooke Blvd increases 26.2 L/s immediately downstream of the proposed development under the upgraded conditions, from 61.1 L/s to 87.3 L/s;
- Under the upgraded conditions, the freeboard is greater than 1.8 m from the surface at all manholes downstream;
- The peak flow increases 27.9 L/s at the study outlet, from 71.0 L/s to 86.0 L/s; and,
- Based on the above results, the sanitary sewer is only able to maintain free-flowing conditions if the proposed upgrades downstream of the site are completed. After the upgrades are completed, the sanitary sewer will be able to accommodate the proposed flows while maintaining free-flowing conditions.

Table 4-1 and **Table 4-2** summarize the peak flows and water levels (HGL) under the pre- and post-development conditions. **Figure A-1** to **Figure A-3** show the model results during DWF conditions. **Figure A-7** to **Figure A-9** show the HGL during DWF conditions for all three scenarios.

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4.2 Wet-Weather Flow: Basement Flooding Protection

The following section assesses whether the proposed development maintains a freeboard greater than 1.8 m. Freeboard can be defined as the difference in height between the hydraulic gradeline of sewer flows and the surface level of the manhole. Since this development is in a sanitary sewer drainage area, the City of Toronto's sanitary sewer criteria were again used in our assessment. Therefore, the following results are based on the combination of dry-weather flow plus the City's 25-year design storm flows (or equivalent RDII rates, i.e. – 3 L/s/ha).

Using the above criteria, the findings can be summarized as follows:

- Under both the existing and proposed conditions, the sanitary sewer is experiencing a bottleneck and backflow at six (6) locations;
- Under the upgraded conditions, the sanitary sewer is operating under free-flowing conditions;
- The peak flow along Cooke Blvd increases 33.0 L/s immediately downstream of the proposed development under the upgraded conditions, from 77.2 L/s to 110.2 L/s;
- Under both the existing and proposed conditions, basement flooding is predicted at six (6) locations downstream of the proposed site;
- Under the upgraded conditions, the freeboard is greater than 1.8 m from the surface at all manholes downstream;
- The peak flow increases 36.5 L/s at the study outlet, from 98.4 L/s to 134.9 L/s; and,
- Based on the above results, the sanitary sewer is only able to maintain basement flooding protection if the proposed upgrades downstream of the site are completed. After the upgrades are completed, the sanitary sewer will be able to accommodate the proposed flows while maintaining free-flowing conditions.

Table 4-1 and **Table 4-2** summarize the peak flows and water levels (HGL) under the pre- and post-development conditions. **Figure A-4** to **Figure A-6** show the model results during DWF conditions. **Figure A-10** to **Figure A-12** show the HGL during DWF conditions for all three scenarios.

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Table 4-1: Peak flows downstream of the proposed development site

Street Name	US node ID	DS node ID	Length (m)	Width (mm)	US invert level (m AD)	DS invert level (m AD)	Slope (%)	Conduit Full capacity (L/s)	DWF									Extreme WWF (May 12 2000)								
									Existing			Proposed			Upgraded			Existing			Proposed			Upgraded		
									Max DS flow (L/s)	Max Water Depth (m)	Surcharge State	Max DS flow (L/s)	Max Water Depth (m)	Surcharge State	Max DS flow (L/s)	Max Water Depth (m)	Surcharge State	Max DS flow (L/s)	Max Water Depth (m)	Surcharge State	Max DS flow (L/s)	Max Water Depth (m)	Surcharge State	Max DS flow (L/s)	Max Water Depth (m)	Surcharge State
Cooke Blvd	SMH13295	SMH13301	71.3	300	97.670	97.400	0.38%	60	61.1	1.534	>100% (2)	76.7	3.102	>100% (2)	87.3	0.255	68%	77.2	4.211	>100% (2)	91.0	6.005	>100% (2)	110.2	0.312	83%
	SMH13301	SMH13310	82.3	300	97.370	97.040	0.40%	62	62.2	1.552	>100% (2)	77.6	2.961	>100% (2)	88.8	0.253	68%	81.9	4.060	>100% (2)	95.6	5.682	>100% (2)	115.6	0.320	85%
	SMH13310	SMH13319	74.1	300	97.000	96.700	0.41%	62	63.4	1.582	>100% (2)	78.7	2.806	>100% (2)	90.3	0.255	68%	86.0	3.843	>100% (2)	99.5	5.257	>100% (2)	120.4	0.326	87%
Plains Rd E	SMH13319	SMH13317	49.0	250	96.630	96.378	0.51%	43	66.4	1.626	>100% (2)	81.4	2.680	>100% (2)	93.8	0.214	48%	91.2	3.621	>100% (2)	104.5	4.841	>100% (2)	126.7	0.256	57%
	SMH13317	SMH13314	16.2	250	96.378	96.116	1.62%	77	70.0	1.297	>100% (1)	85.0	2.010	>100% (2)	97.9	0.238	53%	95.6	2.719	>100% (2)	108.8	3.585	>100% (2)	131.9	0.306	68%
	SMH13314	SMH19205	95.0	250	96.079	95.841	0.25%	30	71.0	1.321	>100% (2)	86.0	1.960	>100% (2)	98.9	0.274	61%	98.4	2.579	>100% (2)	111.5	3.322	>100% (2)	134.9	0.340	75%

(1) Surcharge State is calculated as the ratio of maximum water depth to pipe height and indicates whether the flow rate in the system has exceeded the capacity of the pipe to the extent that levels rise within manholes, i.e. pipe surcharging.
Yellow Bolded Font = Backflow surcharge state
Red Bolded Font = Bottleneck surcharge state

Table 4-2: Hydraulic gradelines downstream of the proposed development site

Street Name	Node ID	Ground level (m)	DWF					Extreme WWF (May 12 2000)				
			Existing Conditions HGL (m)	Proposed Conditions HGL (m)	Upgraded Conditions HGL (m)	Difference Existing vs Upgraded (m)	Freeboard for Upgraded Conditions (m)	Existing Conditions HGL (m)	Proposed Conditions HGL (m)	Upgraded Conditions HGL (m)	Difference Existing vs Upgraded (m)	Freeboard for Upgraded Conditions (m)
Cooke Blvd	SMH13295	101.314	99.208	100.778	97.927	-1.281	3.387	101.886	103.680	97.987	-3.899	3.327
	SMH13301	100.692	98.926	100.337	97.626	-1.300	3.066	101.436	103.058	97.695	-3.741	2.997
	SMH13310	100.203	98.587	99.812	97.258	-1.329	2.945	100.850	102.264	97.332	-3.518	2.871
Plains Rd E	SMH13319	99.774	98.266	99.323	96.845	-1.421	2.929	100.265	101.487	96.888	-3.377	2.886
	SMH13317	99.633	97.652	98.403	96.547	-1.105	3.086	99.115	99.983	96.581	-2.534	3.052
	SMH13314	99.730	97.412	98.055	96.354	-1.058	3.376	98.677	99.423	96.422	-2.255	3.308

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5.0 Conclusions

Based on the analysis and assumptions presented in the report, the findings can be summarized as follows:

1. The expected sanitary peak flow, as observed from the outlet, from the proposed site is 19.94 L/s;
2. The upgraded sanitary sewer is able to maintain free-flowing conditions during dry-weather flow for the proposed intensification along Cooke Blvd, north of Plains Rd E, including the proposed flows for 1062-1074 Cooke Blvd;
3. The upgraded sanitary sewer is able to maintain basement flooding protection during wet-weather flow conditions for the proposed intensification along Cooke Blvd, north of Plains Rd E, including the proposed flows for 1062-1074 Cooke Blvd; and,
4. The existing sanitary sewer system does not have the capacity to accommodate the existing or proposed wet-weather flows without the need for any external upgrades or retrofits.

6.0 Recommendation

Based on the analysis and assumptions presented in the report, it is recommended that the development proceeds after the sanitary sewer upgrades along Cooke Blvd and Plains Rd E have successfully been completed.

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Water Management Solutions

Appendix A
Sanitary Sewer System Plan and Profiles

August 24, 2022

330 Rodinea Road, Unit 3
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Legend

- Proposed Site
- New Development
- Drainage Area

Sewer Infrastructure

- Sanitary
- Sanitary Sewers
- Trunk Sewer

Sanitary Manholes

- At or Above Surface
- Within Basement Level (0 - 1.8 m)
- Below Basement (> 1.8 m)

Sanitary Sewers

- Surcharge State=2 (Bottleneck)
- Surcharge State=1 (Backup)
- Within Capacity (No Surcharge)

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Figure A-1:
Model Results
Existing Conditions under Dry-Weather Flow

Drawn By: R.H. Date: Aug 24, 2022

0 15 30 60 Meters



Legend

- Proposed Site
- New Development
- Drainage Area

Sewer Infrastructure

- Sanitary
- Sanitary Sewers
- Trunk Sewer

Sanitary Manholes

- At or Above Surface
- Within Basement Level (0 - 1.8 m)
- Below Basement (> 1.8 m)

Sanitary Sewers

- Surcharge State=2 (Bottleneck)
- Surcharge State=1 (Backup)
- Within Capacity (No Surcharge)

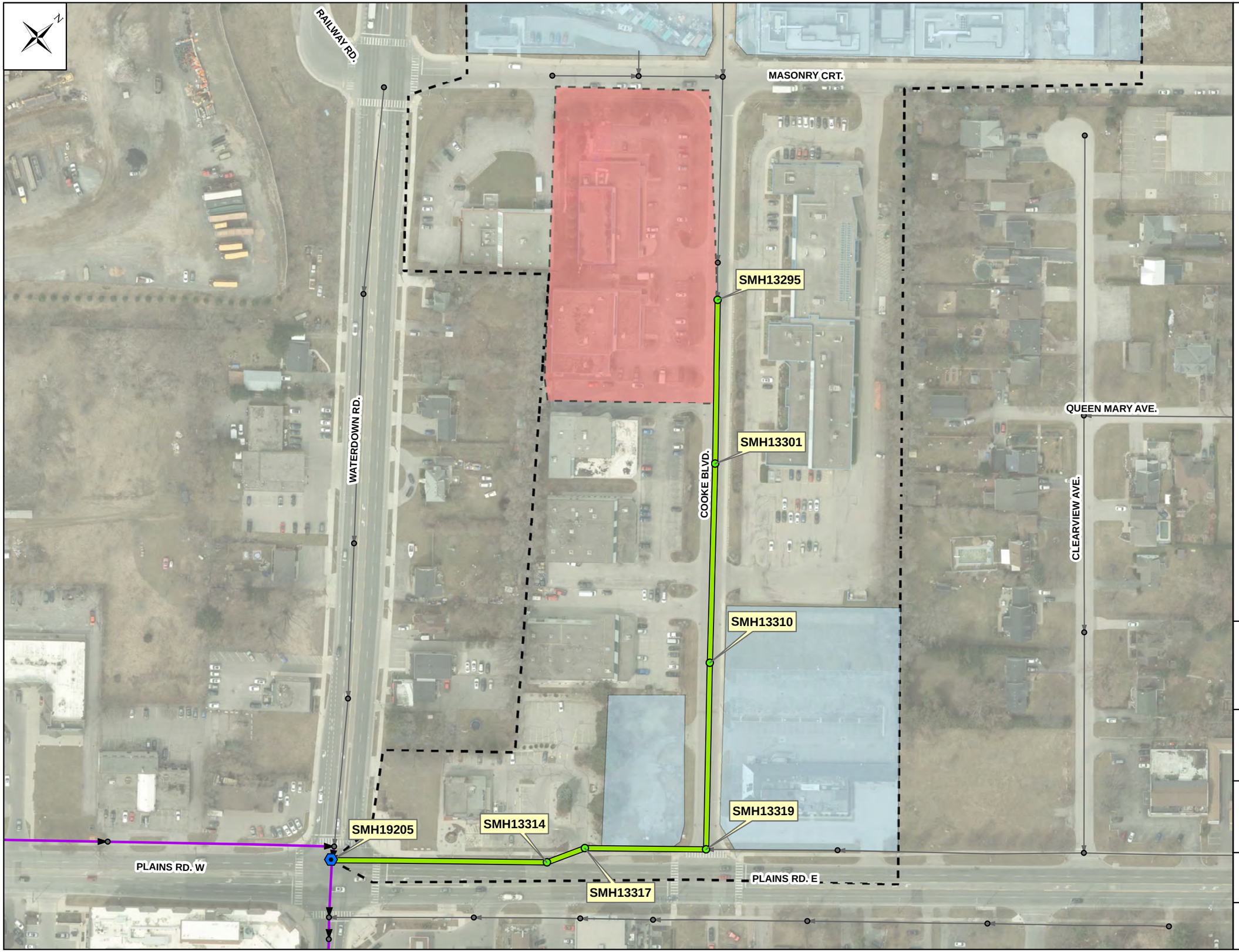
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Figure A-2:
Model Results
Proposed Conditions under Dry-Weather Flow

Drawn By: R.H. Date: Aug 24, 2022

0 15 30 60 Meters



Legend

- Proposed Site
- New Development
- Drainage Area

Sewer Infrastructure

- Sanitary
- Sanitary Sewers
- Trunk Sewer

Sanitary Manholes

- At or Above Surface
- Within Basement Level (0 - 1.8 m)
- Below Basement (> 1.8 m)

Sanitary Sewers

- Surcharge State=2 (Bottleneck)
- Surcharge State=1 (Backup)
- Within Capacity (No Surcharge)

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Figure A-3:
Model Results
Upgraded Conditions under Dry-Weather Flow

Drawn By: R.H. Date: Aug 24, 2022

0 15 30 60 Meters



Legend

- Proposed Site
- New Development
- Drainage Area

Sewer Infrastructure

- Sanitary
- Sanitary Sewers
- Trunk Sewer

Sanitary Manholes

- At or Above Surface
- Within Basement Level (0 - 1.8 m)
- Below Basement (> 1.8 m)

Sanitary Sewers

- Surcharge State=2 (Bottleneck)
- Surcharge State=1 (Backup)
- Within Capacity (No Surcharge)

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Figure A-4:
Model Results
Existing Conditions under Wet-Weather Flow (3 L/s/ha)

Drawn By: R.H. Date: Aug 24, 2022

0 15 30 60 Meters



Legend

- Proposed Site
- New Development
- Drainage Area

Sewer Infrastructure

- Sanitary
- Sanitary Sewers
- Trunk Sewer

Sanitary Manholes

- At or Above Surface
- Within Basement Level (0 - 1.8 m)
- Below Basement (> 1.8 m)

Sanitary Sewers

- Surcharge State=2 (Bottleneck)
- Surcharge State=1 (Backup)
- Within Capacity (No Surcharge)

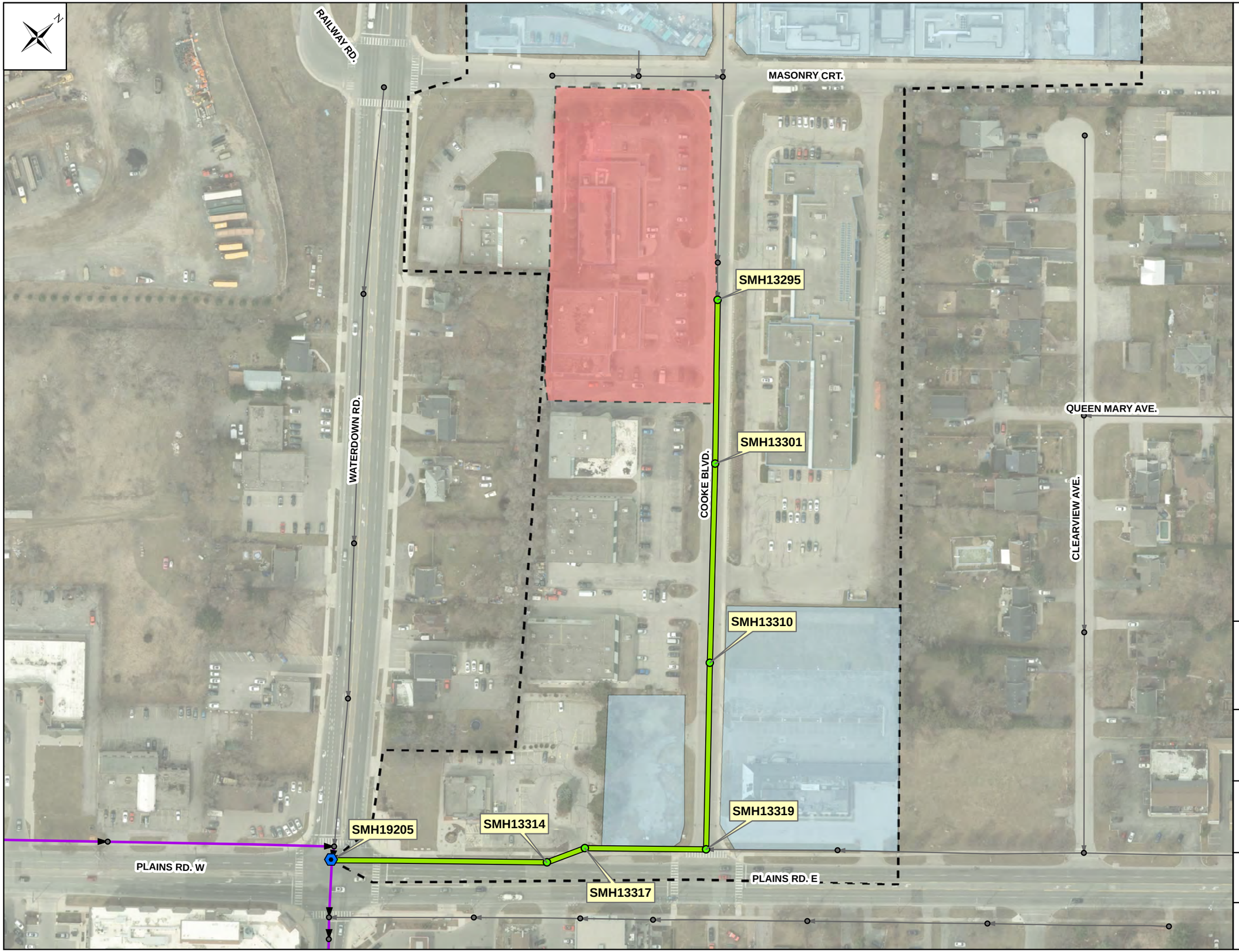
CIVICA

ODA21-0025
1062-1074 Cooke Boulevard Sanitary Capacity Analysis

Figure A-5:
Model Results
Proposed Conditions under Wet-Weather Flow (3 L/s/ha)

Drawn By: R.H. Date: Aug 24, 2022

0 15 30 60 Meters



Legend

- Proposed Site
- New Development
- Drainage Area

Sewer Infrastructure

- Sanitary
- Sanitary Sewers
- Trunk Sewer

Sanitary Manholes

- At or Above Surface
- Within Basement Level (0 - 1.8 m)
- Below Basement (> 1.8 m)

Sanitary Sewers

- Surcharge State=2 (Bottleneck)
- Surcharge State=1 (Backup)
- Within Capacity (No Surcharge)

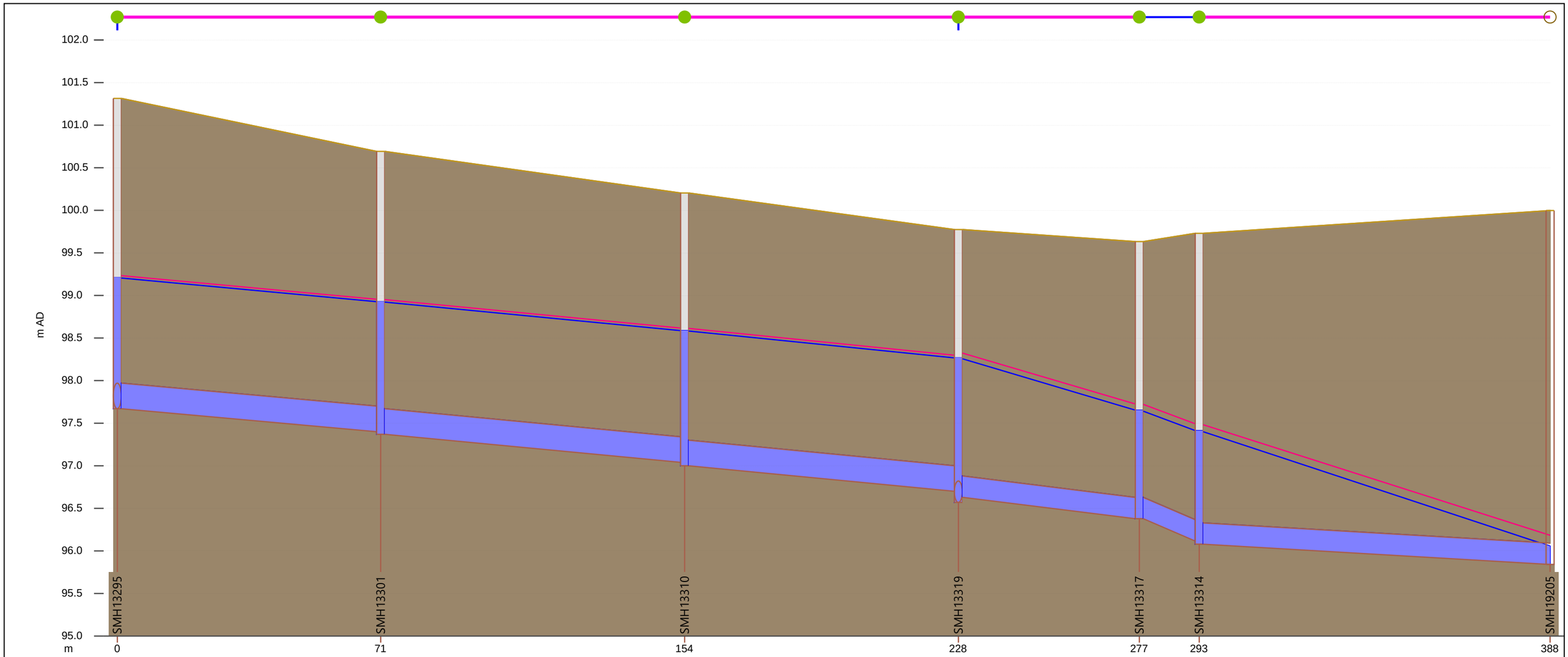
CIVICA

ODA21-0025
1062-1074 Cooke Boulevard Sanitary Capacity Analysis

Figure A-6:
Model Results
Upgraded Conditions under Wet-Weather Flow (3 L/s/ha)

Drawn By: R.H. Date: Aug 24, 2022

0 15 30 60 Meters



Link	SMH13295.1	SMH13301.1	SMH13310.1	SMH13319.1	-	SMH13314.1
length (m)	71.3	82.3	74.1	49.0	16.2	95.0
height (mm)	300	300	300	250	250	250
us inv (m AD)	97.670	97.370	97.000	96.630	96.378	96.079
ds inv (m AD)	97.400	97.040	96.700	96.378	96.116	95.841
pfc (m3/s)	0.060	0.062	0.062	0.043	0.077	0.030
US flow (m3/s)	0.06123	0.06230	0.06347	0.06635	0.06998	0.07102
DS flow (m3/s)	0.06108	0.06217	0.06340	0.06635	0.06998	0.07102

Node	SMH13295	SMH13301	SMH13310	SMH13319	SMH13317	SMH13314	SMH19205
ground (m AD)	101.314	100.692	100.203	99.774	99.633	99.730	99.997
flood dep (m)	-2.106	-1.766	-1.616	-1.508	-1.981	-2.318	

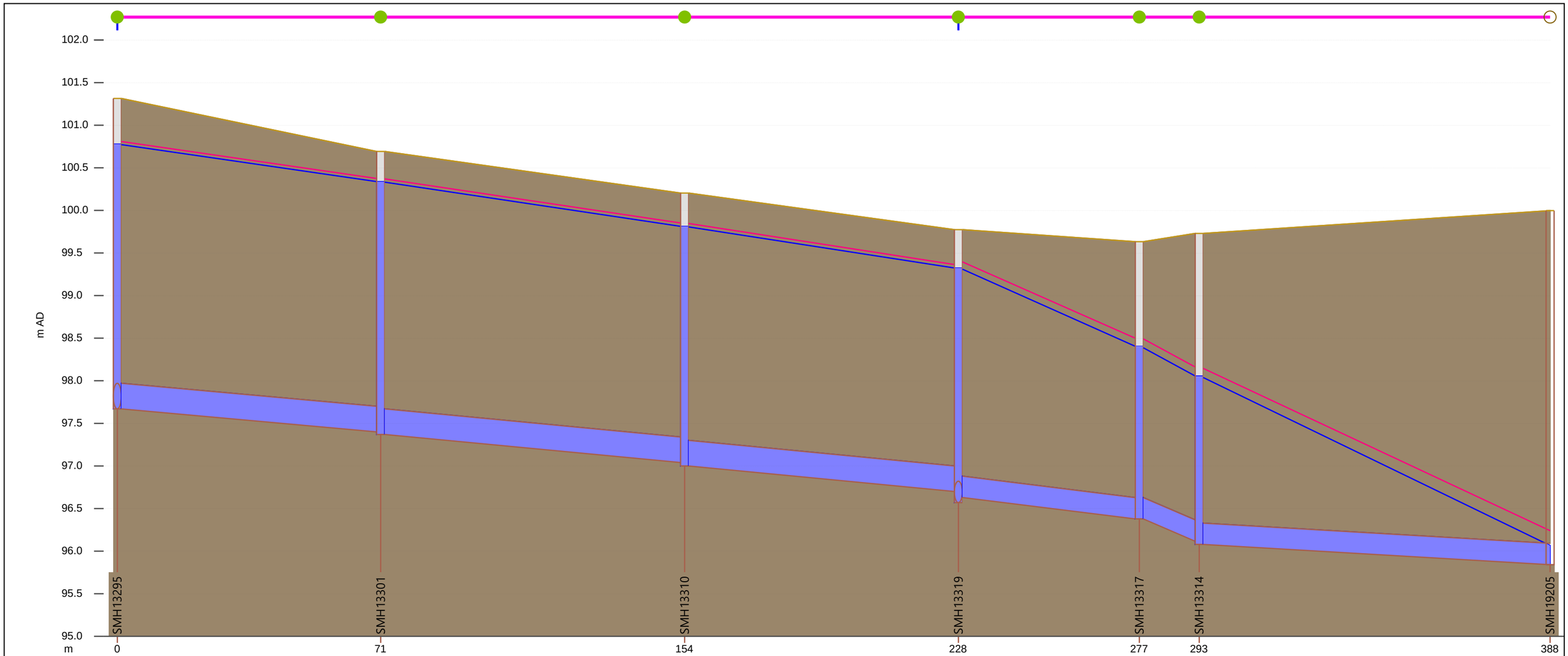
CIVICA
Water Management Solutions

Project Code: ODA21-0025
Project Name: 1062-1074 Cooke Blvd Downstream Capacity Analysis

SCENARIO:
Existing Conditions - Dry-Weather Flow

DATE: 2022-08-24

FIGURE A-7



Link	SMH13295.1	SMH13301.1	SMH13310.1	SMH13319.1	-	SMH13314.1
length (m)	71.3	82.3	74.1	49.0	16.2	95.0
height (mm)	300	300	300	250	250	250
us inv (m AD)	97.670	97.370	97.000	96.630	96.378	96.079
ds inv (m AD)	97.400	97.040	96.700	96.378	96.116	95.841
pfc (m3/s)	0.060	0.062	0.062	0.043	0.077	0.030
US flow (m3/s)	0.07697	0.07781	0.07882	0.08143	0.08495	0.08596
DS flow (m3/s)	0.07674	0.07763	0.07871	0.08142	0.08495	0.08596

Node	SMH13295	SMH13301	SMH13310	SMH13319	SMH13317	SMH13314	SMH19205
ground (m AD)	101.314	100.692	100.203	99.774	99.633	99.730	99.997
flood dep (m)	-0.536	-0.355	-0.391	-0.451	-1.230	-1.675	

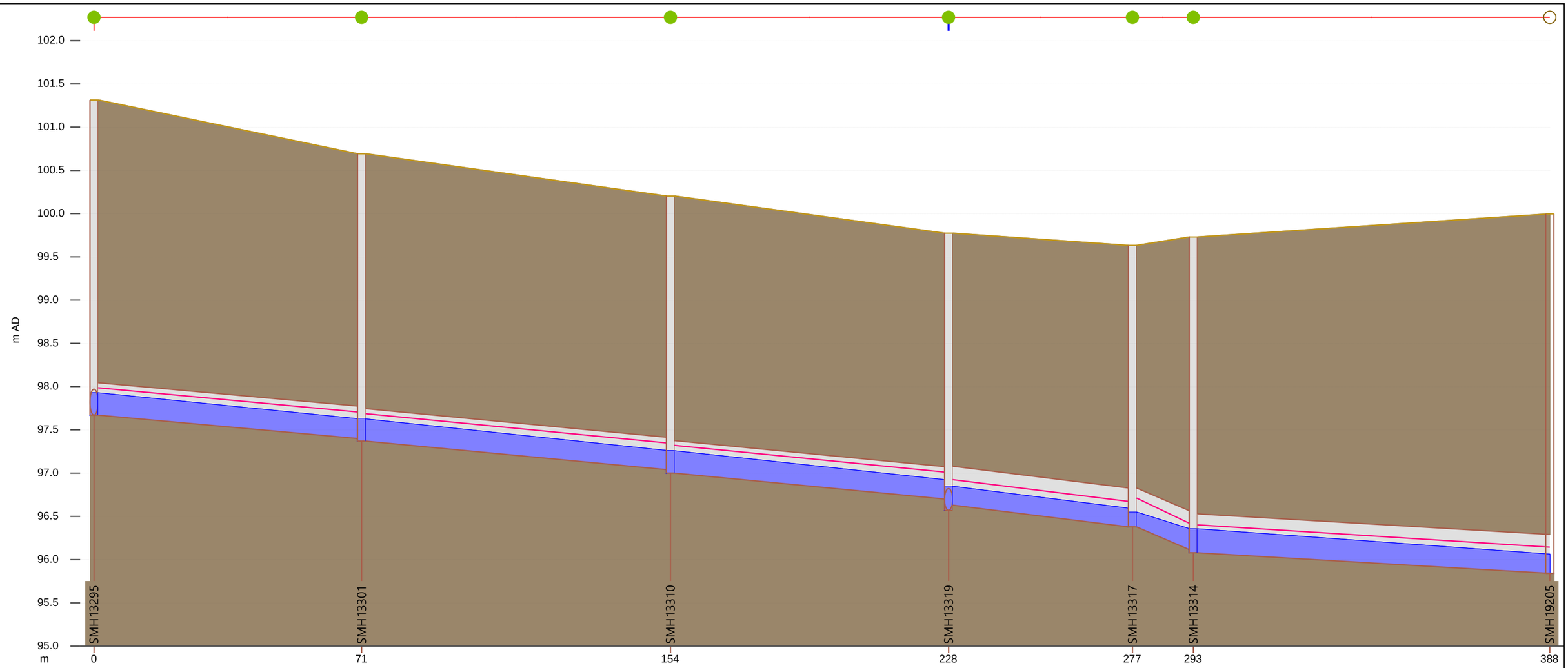
CIVICA
Water Management Solutions

Project Code: ODA21-0025
Project Name: 1062-1074 Cooke Blvd Downstream Capacity Analysis

SCENARIO:
Proposed Conditions - Dry-Weather Flow

DATE: 2022-08-24

FIGURE A-8



Link	SMH13295.1	SMH13301.1	SMH13310.1	SMH13319.1	-	SMH13314.1
length (m)	71.3	82.3	74.1	49.0	16.2	95.0
height (mm)	375	375	375	450	450	450
us inv (m AD)	97.670	97.370	97.000	96.630	96.378	96.079
ds inv (m AD)	97.400	97.040	96.700	96.378	96.116	95.841
pfc (m3/s)	0.109	0.112	0.112	0.205	0.365	0.143
US flow (m3/s)	0.08747	0.08893	0.09043	0.09387	0.09788	0.09901
DS flow (m3/s)	0.08729	0.08881	0.09032	0.09383	0.09785	0.09888

Node	SMH13295	SMH13301	SMH13310	SMH13319	SMH13317	SMH13314	SMH19205
ground (m AD)	101.314	100.692	100.203	99.774	99.633	99.730	99.997
flood dep (m)	-3.387	-3.066	-2.945	-2.929	-3.086	-3.376	

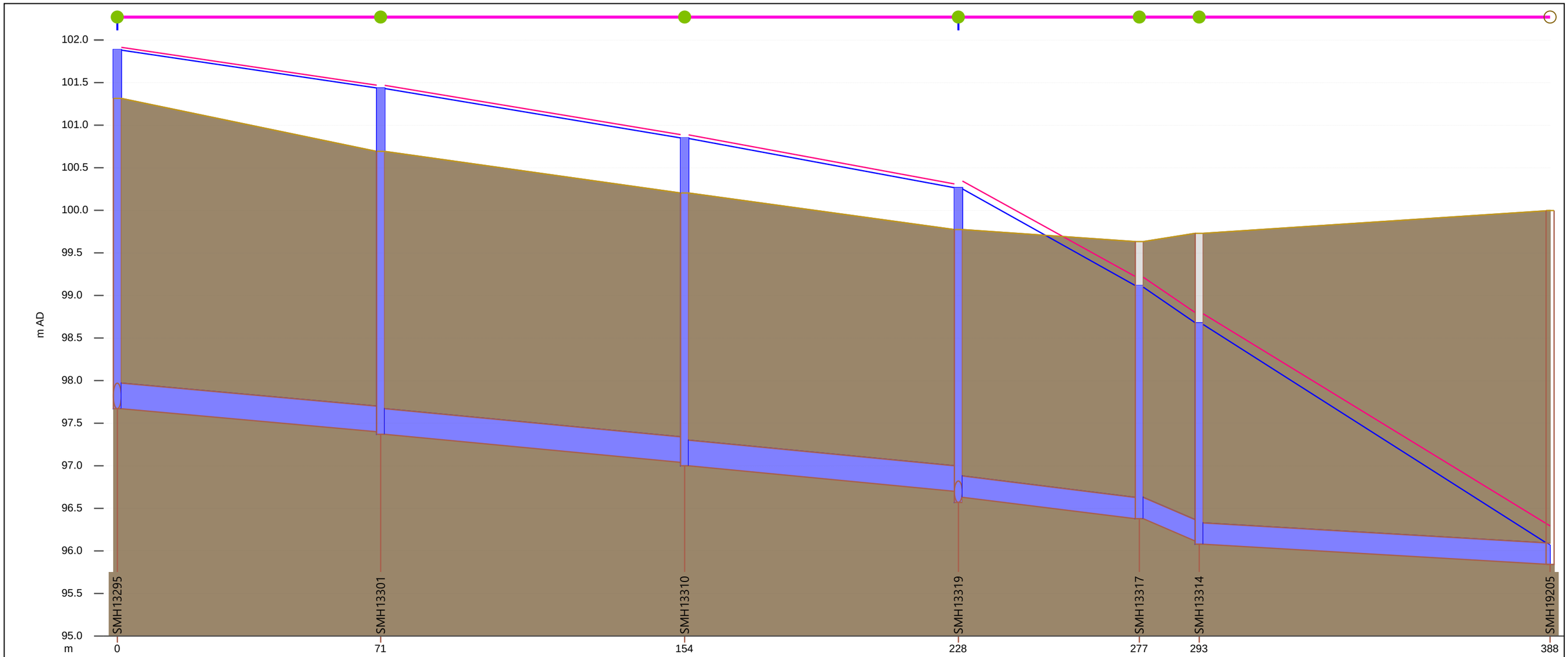
CIVICA
Water Management Solutions

Project Code: ODA21-0025
Project Name: 1062-1074 Cooke Blvd Downstream Capacity Analysis

SCENARIO:
Upgraded Conditions - Dry-Weather Flow

DATE: 2022-08-24

FIGURE A-9



Link	SMH13295.1	SMH13301.1	SMH13310.1	SMH13319.1	-	SMH13314.1
length (m)	71.3	82.3	74.1	49.0	16.2	95.0
height (mm)	300	300	300	250	250	250
us inv (m AD)	97.670	97.370	97.000	96.630	96.378	96.079
ds inv (m AD)	97.400	97.040	96.700	96.378	96.116	95.841
pfc (m3/s)	0.060	0.062	0.062	0.043	0.077	0.030
US flow (m3/s)	0.07739	0.08205	0.08610	0.09123	0.09564	0.09840
DS flow (m3/s)	0.07717	0.08190	0.08601	0.09123	0.09564	0.09840

Node	SMH13295	SMH13301	SMH13310	SMH13319	SMH13317	SMH13314	SMH19205
ground (m AD)	101.314	100.692	100.203	99.774	99.633	99.730	99.997
flood dep (m)	0.572	0.744	0.647	0.491	-0.518	-1.053	

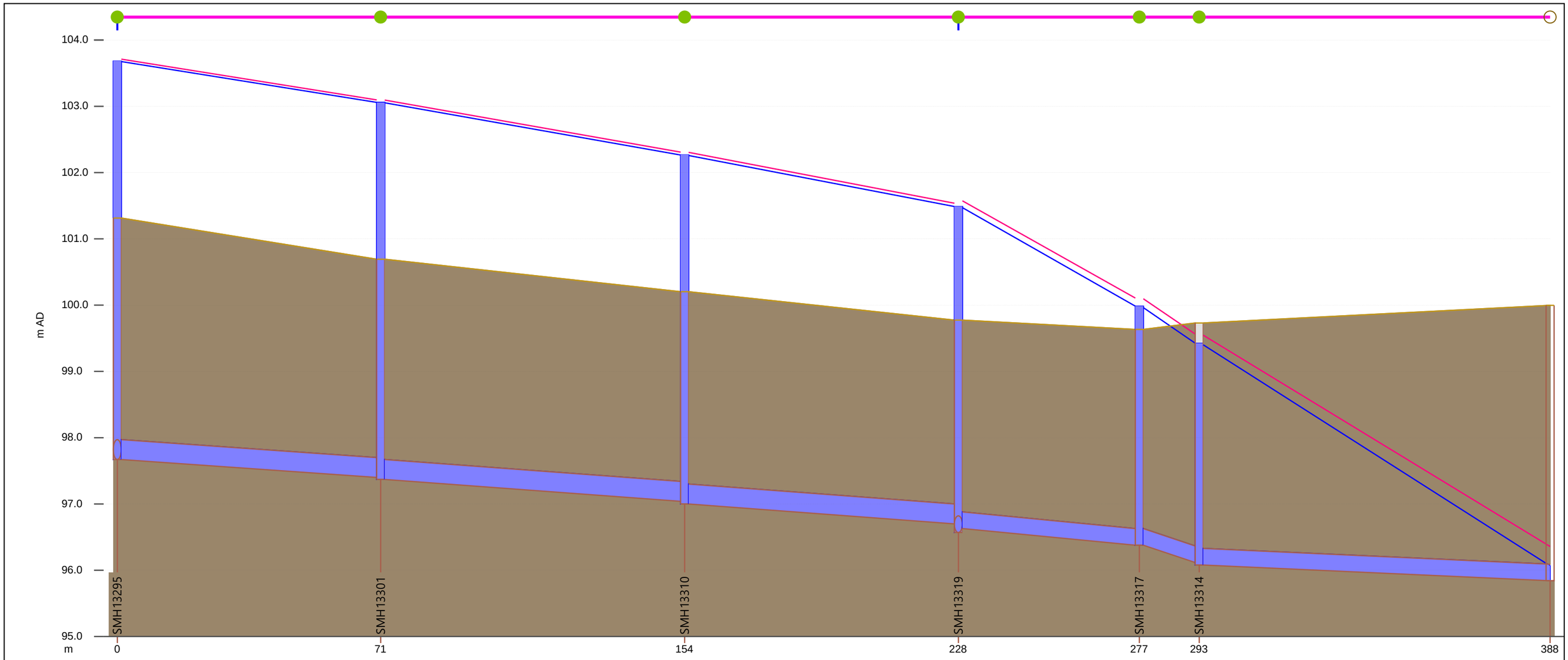
CIVICA
Water Management Solutions

Project Code: ODA21-0025
Project Name: 1062-1074 Cooke Blvd Downstream Capacity Analysis

SCENARIO:
Existing Conditions - Wet-Weather Flow (3 L/s/ha)

DATE: 2022-08-24

FIGURE A-10



Link	SMH13295.1	SMH13301.1	SMH13310.1	SMH13319.1	-	SMH13314.1
length (m)	71.3	82.3	74.1	49.0	16.2	95.0
height (mm)	300	300	300	250	250	250
us inv (m AD)	97.670	97.370	97.000	96.630	96.378	96.079
ds inv (m AD)	97.400	97.040	96.700	96.378	96.116	95.841
pfc (m3/s)	0.060	0.062	0.062	0.043	0.077	0.030
US flow (m3/s)	0.09125	0.09574	0.09962	0.10448	0.10876	0.11146
DS flow (m3/s)	0.09103	0.09556	0.09950	0.10448	0.10876	0.11145

Node	SMH13295	SMH13301	SMH13310	SMH13319	SMH13317	SMH13314	SMH19205
ground (m AD)	101.314	100.692	100.203	99.774	99.633	99.730	99.997
flood dep (m)	2.366	2.366	2.061	1.713	0.350	-0.307	

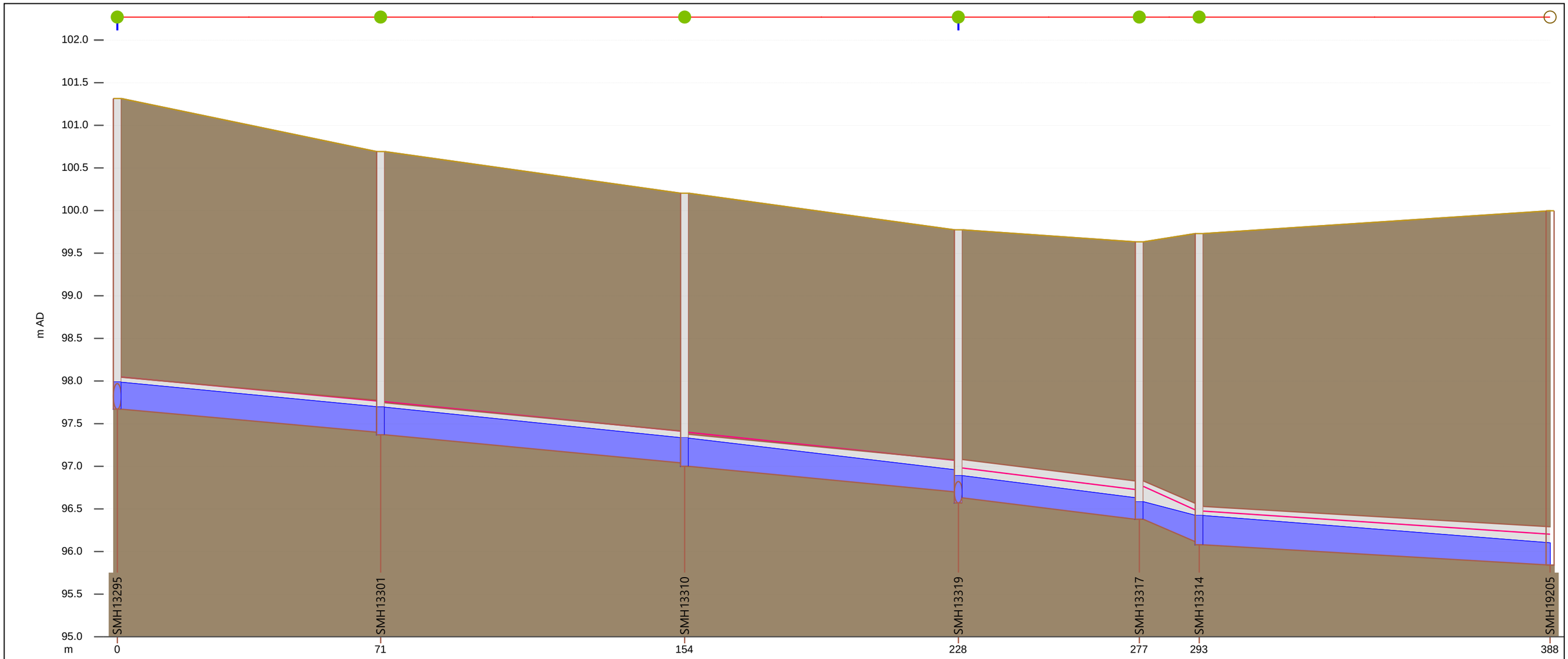
CIVICA
Water Management Solutions

Project Code: ODA21-0025
Project Name: 1062-1074 Cooke Blvd Downstream Capacity Analysis

SCENARIO:
Proposed Conditions - Wet-Weather Flow (3 L/s/ha)

DATE: 2022-08-24

FIGURE A-11



Link	SMH13295.1	SMH13301.1	SMH13310.1	SMH13319.1	-	SMH13314.1
length (m)	71.3	82.3	74.1	49.0	16.2	95.0
height (mm)	375	375	375	450	450	450
us inv (m AD)	97.670	97.370	97.000	96.630	96.378	96.079
ds inv (m AD)	97.400	97.040	96.700	96.378	96.116	95.841
pfc (m3/s)	0.109	0.112	0.112	0.205	0.365	0.143
US flow (m3/s)	0.11054	0.11591	0.12048	0.12673	0.13190	0.13504
DS flow (m3/s)	0.11019	0.11560	0.12041	0.12672	0.13188	0.13491

Node	SMH13295	SMH13301	SMH13310	SMH13319	SMH13317	SMH13314	SMH19205
ground (m AD)	101.314	100.692	100.203	99.774	99.633	99.730	99.997
flood dep (m)	-3.327	-2.997	-2.871	-2.886	-3.052	-3.308	

CIVICA
Water Management Solutions

Project Code: ODA21-0025
Project Name: 1062-1074 Cooke Blvd Downstream Capacity Analysis

SCENARIO:
Upgraded Conditions - Wet-Weather Flow (3 L/s/ha)

DATE: 2022-08-24

FIGURE A-12

APPENDIX C

AutoDesk Storm & Sanitary Analysis ASCII Output File (100-Year Storm)

CETV Verification Statement – Imbrium Systems Inc. Stormceptor OGS

CETV Verification Statement – Imbrium Systems Inc. Jellyfish Filter

AutoDesk Storm & Sanitary Analysis Output File (100-Year Storm)

```
Autodesk® Storm and Sanitary Analysis 2016 - Version 13.5.195 (Build 0)
--------------------------------------------------------------------------------

*******************
Project Description
*******************
File Name ................. 20262 SSA SWM.SPF
Description ............... P:\Projects\2022\22-006\01_DRAWINGS\01_FUNCTIONAL\02_Production Dwgs\20262-FS&FG.dwg


****************
Analysis Options
****************
Flow Units ................ cms
Subbasin Hydrograph Method. EPA SWMM
Infiltration Method ....... SCS Curve Number
Link Routing Method ....... Hydrodynamic
Storage Node Exfiltration.. None
Starting Date ............. JUL-13-2022 00:00:00
Ending Date ............... JUL-13-2022 04:00:00
Antecedent Dry Days ....... 0.0
Report Time Step .......... 00:05:00
Wet Time Step ............. 00:10:00
Dry Time Step ............. 01:00:00
Routing Time Step ......... 30.00 sec


*************
Element Count
*************
Number of rain gages ...... 1
Number of subbasins ....... 1
Number of nodes ........... 3
Number of links ........... 2
Number of pollutants ...... 0
Number of land uses ....... 0


****************
Raingage Summary
****************
Gage                Data                Data       Recording
ID                  Source              Type        Interval
                                                                    min
------------------------------------------------------------
Burlington100Y      Burlington100Y      INTENSITY      10.00


****************
Subbasin Summary
****************
Subbasin                Total    Equiv.   Imperv.   Average    Raingage
                         Area     Width      Area     Slope
ID                    hectares         m         %         %
--------------------------------------------------------------------------
Phase1&2Catchment         0.74     10.00     90.00    4.0000    Burlington100Y


************
Node Summary
************
Node                Element             Invert   Maximum    Ponded    External
ID                  Type             Elevation     Elev.      Area      Inflow
                                             m         m        m²
--------------------------------------------------------------------------------
STM CNTRL MH1 (PROP STM)JUNCTION             99.16    101.09     0.000    Yes
OutletMainlineSewer OUTFALL               98.95     99.25     0.000
PH1SWMTank          STORAGE               96.00    101.30     0.000


************
Link Summary
************
Link            From Node       To Node         Element         Length     Slope   Manning's
ID                                              Type                 m         %   Roughness
--------------------------------------------------------------------------------------------
{PROP STM}.300mmStormConnectionSTM CNTRL MH1 (PROP STM)OutletMainlineSewerCONDUIT           10.4    1.9961      0.0130
STM Pump        PH1SWMTank      STM CNTRL MH1 (PROP STM)TYPE1 PUMP


*********************
Cross Section Summary
*********************
Link             Shape            Depth/        Width        No. of        Cross    Full Flow       Design
ID                                Diameter                   Barrels    Sectional    Hydraulic         Flow
                                                                           Area       Radius     Capacity
```

		m	m		m²	m	cms	
{PROP STM}.300mmStormConnection	CIRCULAR	0.30	0.30	1	0.07	0.07	0.14	

Runoff Quantity Continuity	Volume hectare-m	Depth mm
Total Precipitation	0.058	77.875
Evaporation Loss	0.000	0.000
Infiltration Loss	0.001	0.978
Surface Runoff	0.055	74.670
Final Surface Storage	0.003	3.806
Continuity Error (%)	-2.028	

Flow Routing Continuity	Volume hectare-m	Volume Mliters
Dry Weather Inflow	0.006	0.058
Wet Weather Inflow	0.054	0.542
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
External Outflow	0.060	0.599
Surface Flooding	0.000	0.000
Evaporation Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.000	0.000
Continuity Error (%)	-0.019	

Composite Curve Number Computations Report

Subbasin Phase1&2Catchment

Soil/Surface Description	Area (ha)	Soil Group	CN
Paved parking & roofs	0.54	D	98.00
Paved roads with curbs & sewers	0.08	D	98.00
50 - 75% grass cover, Fair	0.07	D	84.00
50 - 75% grass cover, Fair	0.04	D	84.00
Composite Area & Weighted CN	0.74		95.78

EPA SWMM Time of Concentration Computations Report

Tc = (0.94 * (L^0.6) * (n^0.6)) / ((i^0.4) * (S^0.3))

Where:

Tc = Time of Concentration (min)
L = Flow Length (ft)
n = Manning's Roughness
i = Rainfall Intensity (in/hr)
S = Slope (ft/ft)

Subbasin Phase1&2Catchment

Flow length (m):	739.00
Pervious Manning's Roughness:	0.10000
Impervious Manning's Roughness:	0.01500
Pervious Rainfall Intensity (mm/hr):	19.46875
Impervious Rainfall Intensity (mm/hr):	19.46875
Slope (%):	4.00000
Computed TOC (minutes):	22.27

Subbasin Runoff Summary

Subbasin ID	Total Rainfall mm	Total Runon mm	Total Evap. mm	Total Infil. mm	Total Runoff mm	Peak Runoff cms	Runoff Coefficient	Time of Concentration days	hh:mm:ss
Phase1&2Catchment	77.87	0.00	0.00	0.98	74.67	0.26	0.959	0	00:22:16

Node Depth Summary

Node ID	Average Depth Attained m	Maximum Depth Attained m	Maximum HGL Attained m	Time of Max Occurrence days	Time of Max Occurrence hh:mm	Total Flooded Volume ha-mm	Total Flooded Time minutes	Retention Time hh:mm:ss
STM CNTRL MH1 (PROP STM)	0.16	0.25	99.41	0	01:22	0	0	0:00:00
OutletMainlineSewer	0.13	0.19	99.14	0	01:22	0	0	0:00:00
PH1SWMTank	0.76	2.95	98.95	0	01:48	0	0	0:00:00

****
Node Flow Summary
****

Node ID	Element Type	Maximum Lateral Inflow cms	Peak Inflow cms	Time of Peak Inflow Occurrence days	Time of Peak Inflow Occurrence hh:mm	Maximum Flooding Overflow cms	Time of Peak Flooding Occurrence days	Time of Peak Flooding Occurrence hh:mm
STM CNTRL MH1 (PROP STM)	JUNCTION	0.004	0.098	0	01:22	0.00		
OutletMainlineSewer	OUTFALL	0.000	0.098	0	01:22	0.00		
PH1SWMTank	STORAGE	0.257	0.257	0	01:30	0.00		

****
Storage Node Summary
****

Storage Node ID	Maximum Ponded Volume 1000 m³	Maximum Ponded Volume (%)	Time of Max Ponded Volume days	Time of Max Ponded Volume hh:mm	Average Ponded Volume 1000 m³	Average Ponded Volume (%)	Maximum Storage Node Outflow cms	Maximum Exfiltration Rate cmm	Time of Max. Exfiltration Rate hh:mm:ss	Total Exfiltrated Volume 1000 m³
PH1SWMTank	0.115	56	0	01:48	0.030	14	0.09	0.00	0:00:00	0.000

****
Outfall Loading Summary
****

Outfall Node ID	Flow Frequency (%)	Average Flow cms	Peak Inflow cms
OutletMainlineSewer	99.98	0.055	0.098
System	99.98	0.055	0.098

****
Link Flow Summary
****

Link ID	Element Type	Time of Peak Flow Occurrence days hh:mm	Maximum Velocity Attained m/sec	Length Factor	Peak Flow during Analysis cms	Design Flow Capacity cms	Ratio of Maximum /Design Flow	Ratio of Maximum Flow Depth	Total Time Surcharged minutes	Reported Condition
{PROP STM}.300mmStormConnection	CONDUIT	0 01:22	1.94	1.00	0.098	0.137	0.72	0.73	0	Calculated
STM Pump	PUMP	0 01:22			0.094	1.00		230		

****
Flow Classification Summary
****

Link	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
	--- Fraction of Time in Flow Class ----								
{PROP STM}.300mmStormConnection	0.00	0.00	0.00	0.00	1.00	0.00	0.00	1.48	0.0003

****
Time-Step Critical Elements
****
Link {PROP STM}.300mmStormConnection (98.58%)

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


  ************************
  Routing Time Step Summary
  ************************
  Minimum Time Step           :     2.41 sec
  Average Time Step           :     3.58 sec
  Maximum Time Step           :    30.00 sec
  Percent in Steady State     :     0.00
  Average Iterations per Step :     2.00


  Analysis began on:  Wed Aug 24 10:41:06 2022
  Analysis ended on:  Wed Aug 24 10:41:06 2022
  Total elapsed time: < 1 sec
```

ETV Canada Verified

The Stormceptor® STC

Technology Fact Sheet for Imbrium Systems Inc.

Performance Claim

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

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

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

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

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

Technology Application

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

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

Technology Operation

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

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

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

Performance Claim Conditions

The conditions for this performance claim are as follows:

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

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

Boston, MA, Westwood - SITE 2 0.3 ha

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

Seattle, WA, Seatac - SITE 3 0.4 ha

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

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

Verification

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

What is the ETV Program?

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

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Limitation of Verification
Environment Canada, BLOOM, and the Verification Entity provide the verification services solely on the basis of the information supplied by the applicant or vendor and assume no liability thereafter. The responsibility for the information supplied remains solely with the applicant or vendor and the liability for the purchase, installation, and operation (whether consequential or otherwise) is not transferred to any other party as a result of the verification.

Printed March 2000 Revised March 2009 Revised May 2012 Expires March 2015

◆ Verification Statement ◆ Verification Statement ◆ Verification Statement ◆

VERIFICATION STATEMENT

GLOBE Performance Solutions

Verifies the performance of

Jellyfish® Filter JF4-2-1

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

In accordance with

ISO 14034:2016

Environmental management —
Environmental technology verification (ETV)

John D. Wiebe, PhD
Executive Chairman
GLOBE Performance Solutions

August 3, 2017
Vancouver, BC, Canada

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

Technology description and application

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

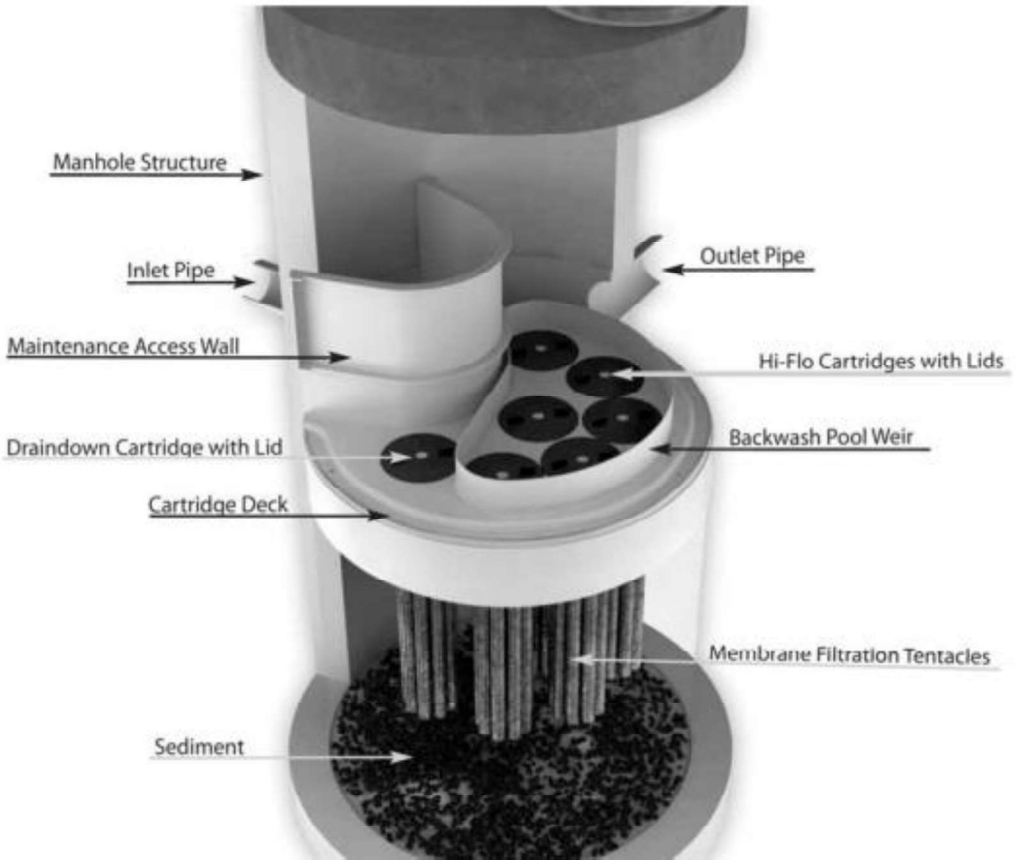


Figure 1. Cut-away graphic of a Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge

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

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

Performance conditions

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

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

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

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

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

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

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

Performance claims

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

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

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

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

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

Performance results

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

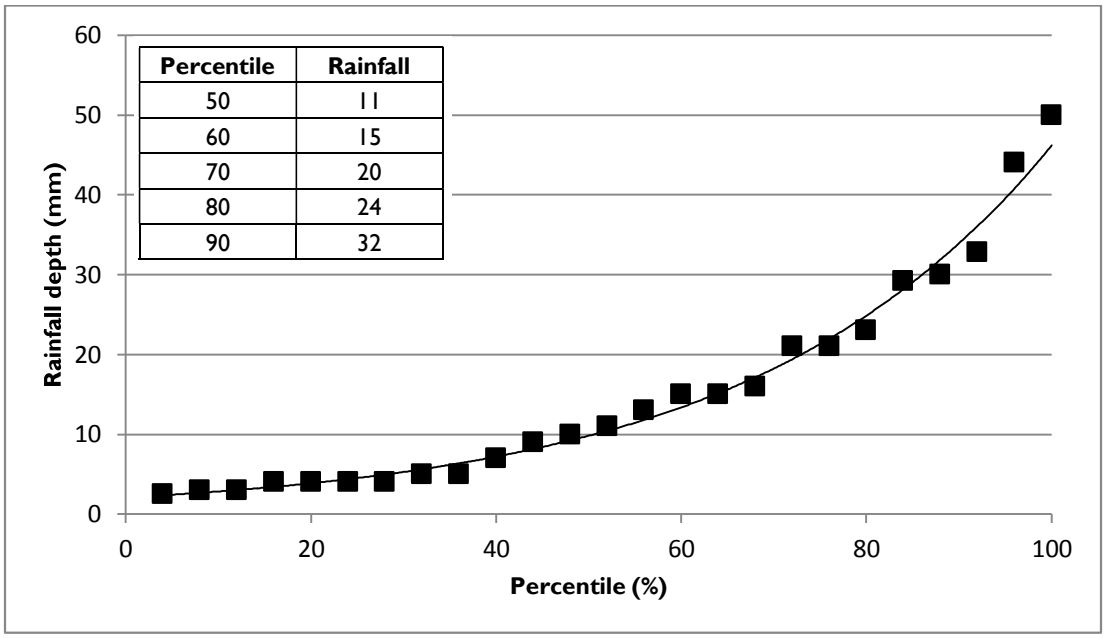


Figure 2. Rainfall depth frequency curve

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

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

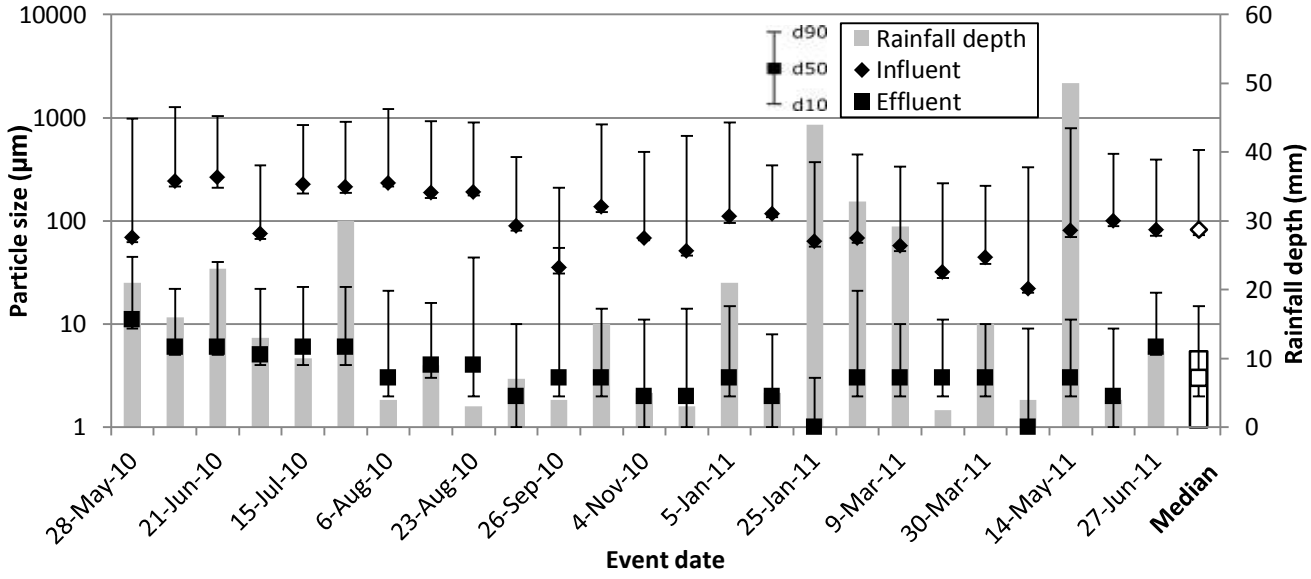


Figure 3. The rainfall depth and d10, d50, and d90 particle sizes of the influent and effluent composite samples for each monitored storm event over the 13-month testing period

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

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

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

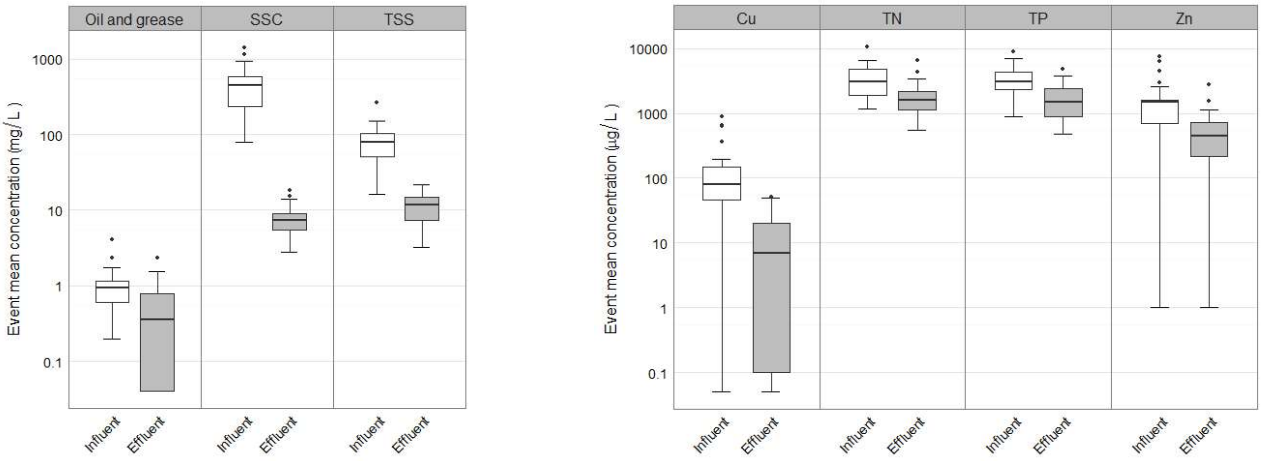


Figure 4. Boxplots showing the distribution of influent and effluent event mean concentrations (EMC) for selected stormwater constituents over the study period

Verification

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

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

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

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Limitation of verification
GLOBE Performance Solutions and the Verification Expert provide the verification services solely on the basis of the information supplied by the applicant or vendor and assume no liability thereafter. The responsibility for the information supplied remains solely with the applicant or vendor and the liability for the purchase, installation, and operation (whether consequential or otherwise) is not transferred to any other party as a result of the verification.

STANDARD OFFLINE
Jellyfish Filter Sizing Report

Project Information

Date	Wednesday, May 27, 2020
Project Name	1157 North Shore Blvd. E
Project Number	PN 18204
Location	Burlington

Jellyfish Filter Design Overview

This report provides information for the sizing and specification of the Jellyfish Filter. When designed properly in accordance to the guidelines detailed in the Jellyfish Filter Technical Manual, the Jellyfish Filter will exceed the performance and longevity of conventional horizontal bed and granular media filters.

Please see ***www.ImbriumSystems.com*** *for more information.*

Jellyfish Filter System Recommendation

The Jellyfish Filter model JF6-5-1 is recommended to meet the water quality objective by treating a flow of 27.8 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 313 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF6-5-1	5	1	1.8	27.8	313

The Jellyfish Filter System

The patented Jellyfish Filter is an engineered stormwater quality treatment technology featuring unique membrane filtration in a compact stand-alone treatment system that removes a high level and wide variety of stormwater pollutants. Exceptional pollutant removal is achieved at high treatment flow rates with minimal head loss and low maintenance costs. Each lightweight Jellyfish Filter cartridge contains an extraordinarily large amount of membrane surface area, resulting in superior flow capacity and pollutant removal capacity.

Maintenance

Regular scheduled inspections and maintenance is necessary to assure proper functioning of the Jellyfish Filter. The maintenance interval is designed to be a minimum of 12 months, but this will vary depending on site loading conditions and upstream pretreatment measures. Quarterly inspections and inspections after all storms beyond the 5-year event are recommended until enough historical performance data has been logged to comfortably initiate an alternative inspection interval.

Please see ***www.ImbriumSystems.com*** *for more information.*

Thank you for the opportunity to present this information to you and your client.

Performance

Jellyfish efficiently captures a high level of Stormwater pollutants, including:

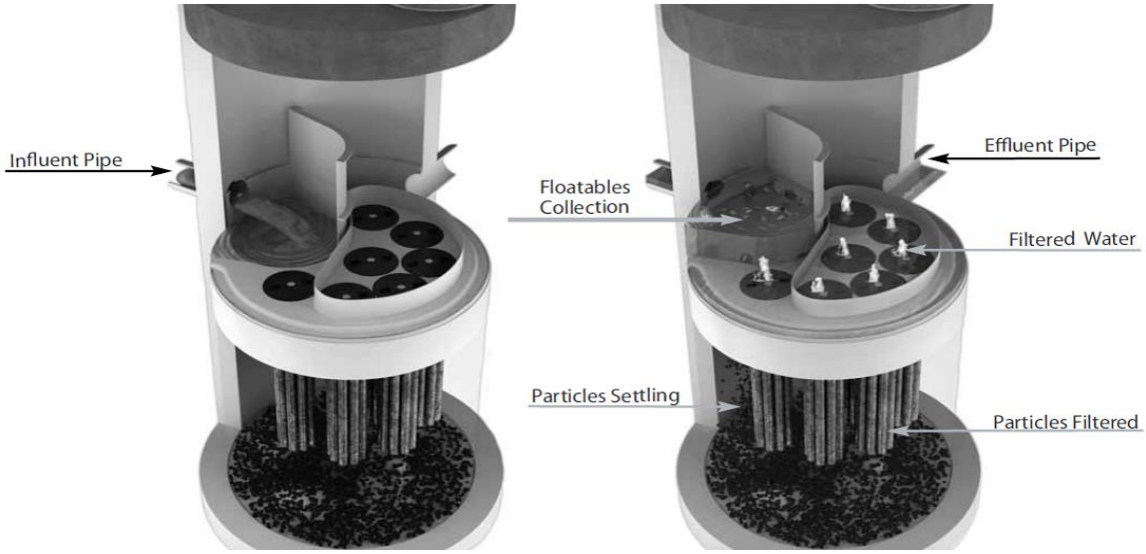
- ☑ 89% of the total suspended solids (TSS) load, including particles less than 5 microns
- ☑ 59% TP removal & 51% TN removal
- ☑ 90% Total Copper, 81% Total Lead, 70% Total Zinc
- ☑ Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons and bacteria
- ☑ Free oil, Floatable trash and debris

Field Proven Peformance

The Jellyfish filter has been field-tested on an urban site with 25 TARP qualifying rain events and field monitored according to the TARP field test protocol, demonstrating:

- A median TSS removal efficiency of 89%, and a median SSC removal of 99%;
- The ability to capture fine particles as indicated by an effluent d50 median of 3 microns for all monitotred storm events, and a median effluent turbidity of 5 NTUs;
- A median Total Phosphorus removal of 59%, and a median Total Nitrogen removal of 51%.

Jellyfish Filter Treatment Functions



Pre-treatment and Membrane Filtration

Jellyfish® Filter

Project Information

Date:	Wednesday, May 27, 2020
Project Name:	1157 North Shore Blvd. E
Project Number:	PN 18204
Location:	Burlington

Designer Information

Company:	Odan Detech
Contact:	Daniel Bancroft
Phone #:	

Notes

Rainfall

Name:	TORONTO CENTRAL
State:	ON
ID:	100
Record:	1982 to 1999
Co-ords:	45°30'N, 90°30'W

Drainage Area

Total Area:	1.12 ha
Imperviousness:	75%

Upstream Detention

Peak Release Rate:	n/a
Pretreatment Credit:	n/a

Design System Requirements

Flow Loading	90% of the Average Annual Runoff based on 18 years of TORONTO CENTRAL rainfall data:	**23.1 L/s**
Sediment Loading	Treating 90% of the average annual runoff volume, 5019 m³, with a suspended sediment concentration of 60 mg/L.	**301 kg**

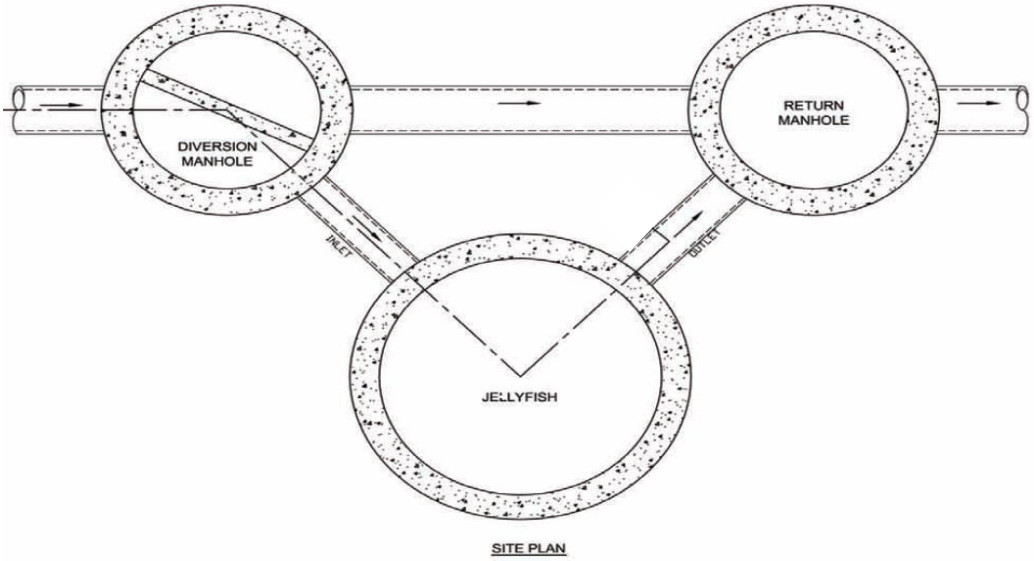
Recommendation

The Jellyfish Filter model JF6-5-1 is recommended to meet the water quality objective by treating a flow of 27.8 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 313 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Wet Vol Below Deck (L)	Sump Storage (m³)	Oil Capacity (L)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF4-1-1	1	1	1.2	2313	0.34	379	7.6	85
JF4-2-1	2	1	1.2	2313	0.34	379	12.6	142
JF6-3-1	3	1	1.8	5205	0.79	848	17.7	199
JF6-4-1	4	1	1.8	5205	0.79	848	22.7	256
JF6-5-1	**5**	**1**	**1.8**	**5205**	**0.79**	**848**	**27.8**	**313**
JF6-6-1	6	1	1.8	5205	0.79	848	28.6	370
JF8-6-2	6	2	2.4	9252	1.42	1469	35.3	398
JF8-7-2	7	2	2.4	9252	1.42	1469	40.4	455
JF8-8-2	8	2	2.4	9252	1.42	1469	45.4	512
JF8-9-2	9	2	2.4	9252	1.42	1469	50.5	569
JF8-10-2	10	2	2.4	9252	1.42	1469	50.5	626
JF10-11-3	11	3	3.0	14456	2.21	2302	63.1	711
JF10-12-3	12	3	3.0	14456	2.21	2302	68.2	768
JF10-12-4	12	4	3.0	14456	2.21	2302	70.7	796
JF10-13-4	13	4	3.0	14456	2.21	2302	75.7	853
JF10-14-4	14	4	3.0	14456	2.21	2302	78.9	910
JF10-15-4	15	4	3.0	14456	2.21	2302	78.9	967
JF10-16-4	16	4	3.0	14456	2.21	2302	78.9	1024
JF10-17-4	17	4	3.0	14456	2.21	2302	78.9	1081
JF10-18-4	18	4	3.0	14456	2.21	2302	78.9	1138
JF10-19-4	19	4	3.0	14456	2.21	2302	78.9	1195
JF12-20-5	20	5	3.6	20820	3.2	2771	113.6	1280
JF12-21-5	21	5	3.6	20820	3.2	2771	113.7	1337
JF12-22-5	22	5	3.6	20820	3.2	2771	113.7	1394
JF12-23-5	23	5	3.6	20820	3.2	2771	113.7	1451
JF12-24-5	24	5	3.6	20820	3.2	2771	113.7	1508
JF12-25-5	25	5	3.6	20820	3.2	2771	113.7	1565
JF12-26-5	26	5	3.6	20820	3.2	2771	113.7	1622
JF12-27-5	27	5	3.6	20820	3.2	2771	113.7	1679

Jellyfish Filter Design Notes

- Typically the Jellyfish Filter is designed in an offline configuration, as all stormwater filter systems will perform for a longer duration between required maintenance services when designed and applied in off-line configurations. Depending on the design parameters, an optional internal bypass may be incorporated into the Jellyfish Filter, however note the inspection and maintenance frequency should be expected to increase above that of an off-line system. Speak to your local representative for more information.



Jellyfish Filter Typical Layout

- Typically, 18 inches (457 mm) of driving head is designed into the system, calculated as the difference in elevation between the top of the diversion structure weir and the invert of the Jellyfish Filter outlet pipe. Alternative driving head values can be designed as 12 to 24 inches (305 to 610mm) depending on specific site requirements, requiring additional sizing and design assistance.
- Typically, the Jellyfish Filter is designed with the inlet pipe configured 6 inches (150 mm) above the outlet invert elevation. However, depending on site parameters this can vary to an optional configuration of the inlet pipe entering the unit below the outlet invert elevation.
- The Jellyfish Filter can accommodate multiple inlet pipes within certain restrictions.
- While the optional inlet below deck configuration offers 0 to 360 degree flexibility between the inlet and outlet pipe, typical systems conform to the following:

Model Diameter (m)	Minimum Angle Inlet / Outlet Pipes	Minimum Inlet Pipe Diameter (mm)	Minimum Outlet Pipe Diameter (mm)
1.2	62º	150	200
1.8	**59º**	**200**	**250**
2.4	52º	250	300
3.0	48º	300	450
3.6	40º	300	450

- The Jellyfish Filter can be built at all depths of cover generally associated with conventional stormwater conveyance systems. For sites that require minimal depth of cover for the stormwater infrastructure, the Jellyfish Filter can be applied in a shallow application using a hatch cover. The general minimum depth of cover is 36 inches (915 mm) from top of the underslab to outlet invert.
- If driving head caclulations account for water elevation during submerged conditions the Jellyfish Filter will function effectively under submerged condtions.
- Jellyfish Filter systems may incorporate grated inlets depending on system configuration.
- For sites with water quality treatment flow rates or mass loadings that exceed the design flow rate of the largest standard Jellyfish Filter manhole models, systems can be designed that hydraulically connect multiple Jellyfish Filters in series or alternatively Jellyfish Vault units can be designed.

STANDARD SPECIFICATION
STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

Specifies requirements for construction and performance of an underground stormwater quality membrane filtration treatment device that removes pollutants from stormwater runoff through the unit operations of sedimentation, floatation, and membrane filtration.

1.2 REFERENCE STANDARDS

ASTM C 891: Specification for Installation of Underground Precast Concrete Utility Structures
ASTM C 478: Specification for Precast Reinforced Concrete Manhole Sections
ASTM C 443: Specification for Joints for Concrete Pipe and Manholes, Using Rubber Gaskets
ASTM D 4101: Specification for Copolymer steps construction

CAN/CSA-A257.4-M92
Joints for Circular Concrete Sewer and Culvert Pipe, Manhole Sections and Fittings Using Rubber Gaskets

CAN/CSA-A257.4-M92
Precast Reinforced Circular Concrete Manhole Sections, Catch Basins and Fittings

Canadian Highway Bridge Design Code

1.3 SHOP DRAWINGS

Shop drawings for the structure and performance are to be submitted with each order to the contractor. Contractor shall forward shop drawing submittal to the consulting engineer for approval. Shop drawings are to detail the structure's precast concrete and call out or note the fiberglass (FRP) internals/components.

1.4 PRODUCT SUBSTITUTIONS

No product substitutions shall be accepted unless submitted 10 days prior to project bid date, or as directed by the engineer of record. Submissions for substitutions require review and approval by the Engineer of Record, for hydraulic performance, impact to project designs, equivalent treatment performance, and any required project plan and report (hydrology/hydraulic, water quality, stormwater pollution) modifications that would be required by the approving jurisdictions/agencies. Contractor to coordinate with the Engineer of Record any applicable modifications to the project estimates of cost, bonding amount determinations, plan check fees for changes to approved documents, and/or any other regulatory requirements resulting from the product substitution.

1.5 HANDLING AND STORAGE

Prevent damage to materials during storage and handling.

PART 2 – PRODUCTS

2.1 GENERAL

2.1.1 The device shall be a cylindrical or rectangular, all concrete structure (including risers), constructed from precast concrete riser and slab components or monolithic precast structure(s), installed to conform to ASTM C 891 and to any required state highway, municipal or local specifications; whichever is more stringent. The device shall be watertight.

2.1.2 Cartridge Deck The cylindrical concrete device shall include a fiberglass deck. The rectangular concrete device shall include a coated aluminum deck. In either instance, the insert shall be bolted and sealed watertight inside the precast concrete chamber. The deck shall serve as: (a) a horizontal divider between the lower treatment zone and the upper treated effluent zone; (b) a deck for attachment of filter cartridges such that the membrane filter elements of each cartridge extend into the lower treatment zone; (c) a platform for maintenance workers to service the filter cartridges (maximum manned weight = 450 pounds (204 kg)); (d) a conduit for conveyance of treated water to the effluent pipe.

2.1.3 Membrane Filter Cartridges Filter cartridges shall be comprised of reusable cylindrical membrane filter elements connected to a perforated head plate. The number of membrane filter elements per cartridge shall be a minimum of eleven 2.75-inch (70-mm) diameter elements. The length of each filter element shall be a minimum 15 inches (381 mm). Each cartridge shall be fitted into the cartridge deck by insertion into a cartridge receptacle that is permanently mounted into the cartridge deck. Each cartridge shall be secured by a cartridge lid that is threaded onto the receptacle, or similar mechanism to secure the cartridge into the deck. The maximum treatment flow rate of a filter cartridge shall be controlled by an orifice in the cartridge lid, or on the individual cartridge itself, and based on a design flux rate (surface loading rate) determined by the maximum treatment flow rate per unit of filtration membrane surface area. The maximum design flux rate shall be 0.21 gpm/ft² (0.142 lps/m²).

Each membrane filter cartridge shall allow for manual installation and removal. Each filter cartridge shall have filtration membrane surface area and dry installation weight as follows (if length of filter cartridge is between those listed below, the surface area and weight shall be proportionate to the next length shorter and next length longer as shown below):

Filter Cartridge Length (in / mm)	Minimum Filtration Membrane Surface Area (ft2 / m2)	Maximum Filter Cartridge Dry Weight (lbs / kg)
15	106 / 9.8	10.5 / 4.8
27	190 / 17.7	15.0 / 6.8
40	282 / 26.2	20.5 / 9.3
54	381 / 35.4	25.5 / 11.6

2.1.4 Backwashing Cartridges The filter device shall have a weir extending above the cartridge deck, or other mechanism, that encloses the high flow rate filter cartridges when placed in their respective cartridge receptacles within the cartridge deck. The weir, or other mechanism, shall collect a pool of filtered water during inflow events that backwashes the high flow rate cartridges when the inflow

event subsides. All filter cartridges and membranes shall be reusable and allow for the use of filtration membrane rinsing procedures to restore flow capacity and sediment capacity; extending cartridge service life.

2.1.5 Maintenance Access to Captured Pollutants The filter device shall contain an opening(s) that provides maintenance access for removal of accumulated floatable pollutants and sediment, removal of and replacement of filter cartridges, cleaning of the sump, and rinsing of the deck. Access shall have a minimum clear vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

2.1.6 Bend Structure The device shall be able to be used as a bend structure with minimum angles between inlet and outlet pipes of 90-degrees or less in the stormwater conveyance system.

2.1.7 Double-Wall Containment of Hydrocarbons The cylindrical precast concrete device shall provide double-wall containment for hydrocarbon spill capture by a combined means of an inner wall of fiberglass, to a minimum depth of 12 inches (305 mm) below the cartridge deck, and the precast vessel wall.

2.1.8 Baffle The filter device shall provide a baffle that extends from the underside of the cartridge deck to a minimum length equal to the length of the membrane filter elements. The baffle shall serve to protect the membrane filter elements from contamination by floatables and coarse sediment. The baffle shall be flexible and continuous in cylindrical configurations, and shall be a straight concrete or aluminum wall in rectangular configurations.

2.1.9 Sump The device shall include a minimum 24 inches (610 mm) of sump below the bottom of the cartridges for sediment accumulation, unless otherwise specified by the design engineer. Depths less than 24 inches may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

2.2 PRECAST CONCRETE SECTIONS

All precast concrete components shall be manufactured to a minimum live load of HS-20 truck loading or greater based on local regulatory specifications, unless otherwise modified or specified by the design engineer, and shall be watertight.

2.3 JOINTS All precast concrete manhole configuration joints shall use nitrile rubber gaskets and shall meet the requirements of ASTM C443, Specification C1619, Class D or engineer approved equal to ensure oil resistance. Mastic sealants or butyl tape are not an acceptable alternative.

2.4 GASKETS Only profile neoprene or nitrile rubber gaskets in accordance to CSA A257.3-M92 will be accepted. Mastic sealants, butyl tape or Conseal CS-101 are not acceptable gasket materials.

2.5 FRAME AND COVER Frame and covers must be manufactured from cast-iron or other composite material tested to withstand H-20 or greater design loads, and as approved by the

local regulatory body. Frames and covers must be embossed with the name of the device manufacturer or the device brand name.

2.6 DOORS AND HATCHES If provided shall meet designated loading requirements or at a minimum for incidental vehicular traffic.

2.7 CONCRETE All concrete components shall be manufactured according to local specifications and shall meet the requirements of ASTM C 478.

2.8 FIBERGLASS The fiberglass portion of the filter device shall be constructed in accordance with the following standard: ASTM D-4097: Contact Molded Glass Fiber Reinforced Chemical Resistant Tanks.

2.9 STEPS Steps shall be constructed according to ASTM D4101 of copolymer polypropylene, and be driven into preformed or pre-drilled holes after the concrete has cured, installed to conform to applicable sections of state, provincial and municipal building codes, highway, municipal or local specifications for the construction of such devices.

2.10 INSPECTION All precast concrete sections shall be inspected to ensure that dimensions, appearance and quality of the product meet local municipal specifications and ASTM C 478.

PART 3 – PERFORMANCE

3.1 GENERAL

3.1.1 Verification – The stormwater quality filter must be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV).

3.1.2 Function - The stormwater quality filter treatment device shall function to remove pollutants by the following unit treatment processes; sedimentation, floatation, and membrane filtration.

3.1.3 Pollutants - The stormwater quality filter treatment device shall remove oil, debris, trash, coarse and fine particulates, particulate-bound pollutants, metals and nutrients from stormwater during runoff events.

3.1.4 Bypass - The stormwater quality filter treatment device shall typically utilize an external bypass to divert excessive flows. Internal bypass systems shall be equipped with a floatables baffle, and must avoid passage through the sump and/or cartridge filtration zone.

3.1.5 Treatment Flux Rate (Surface Loading Rate) – The stormwater quality filter treatment device shall treat 100% of the required water quality treatment flow based on a maximum design treatment flux rate (surface loading rate) across the membrane filter cartridges of 0.21 gpm/ft² (0.142 lps/m²).

3.2 FIELD TEST PERFORMANCE

At a minimum, the stormwater quality filter device shall have been field tested and verified with a minimum 25 TARP qualifying storm events and field monitoring shall have been conducted according to the TARP 2009 NJDEP TARP field test protocol, and have received NJCAT verification.

3.2.1 Suspended Solids Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median TSS removal efficiency of 85% and a minimum median SSC removal efficiency of 95%.

3.2.2 Runoff Volume – The stormwater quality filter treatment device shall be engineered, designed, and sized to treat a minimum of 90 percent of the annual runoff volume determined from use of a minimum 15-year rainfall data set.

3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, an effluent d_{50} of 15 microns or lower for all monitored storm events.

3.2.4 Turbidity Reduction - The stormwater quality filter treatment device shall have demonstrated the ability to reduce the turbidity from influent from a range of 5 to 171 NTU to an effluent turbidity of 15 NTU or lower.

3.2.5 Nutrient (Total Phosphorus & Total Nitrogen) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Phosphorus removal of 55%, and a minimum median Total Nitrogen removal of 50%.

3.2.6 Metals (Total Zinc & Total Copper) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Zinc removal of 55%, and a minimum median Total Copper removal of 85%.

3.3 INSPECTION and MAINTENANCE

The stormwater quality filter device shall have the following features:

3.3.1 Durability of membranes are subject to good handling practices during inspection and maintenance (removal, rinsing, and reinsertion) events, and site specific conditions that may have heavier or lighter loading onto the cartridges, and pollutant variability that may impact the membrane structural integrity. Membrane maintenance and replacement shall be in accordance with manufacturer's recommendations.

3.3.2 Inspection which includes trash and floatables collection, sediment depth determination, and visible determination of backwash pool depth shall be easily conducted from grade (outside the structure).

3.3.3 Manual rinsing of the reusable filter cartridges shall promote restoration of the flow capacity and sediment capacity of the filter cartridges, extending cartridge service life.

3.3.4 The filter device shall have a minimum 12 inches (305 mm) of sediment storage depth, and a minimum of 12 inches between the top of the sediment storage and bottom of the filter cartridge tentacles, unless otherwise specified by the design engineer. Variances may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

3.3.5 Sediment removal from the filter treatment device shall be able to be conducted using a standard maintenance truck and vacuum apparatus, and a minimum one point of entry to the sump that is unobstructed by filter cartridges.

3.3.6 Maintenance access shall have a minimum clear height that provides suitable vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

3.3.7 Filter cartridges shall be able to be maintained without the requirement of additional lifting equipment.

PART 4 – EXECUTION

4.1 INSTALLATION

4.1.1 PRECAST DEVICE CONSTRUCTION SEQUENCE

The installation of a watertight precast concrete device should conform to ASTM C 891 and to any state highway, municipal or local specifications for the construction of manholes, whichever is more stringent. Selected sections of a general specification that are applicable are summarized below.

4.1.1.1 The watertight precast concrete device is installed in sections in the following sequence:

- aggregate base
- base slab
- treatment chamber and cartridge deck riser section(s)
- bypass section
- connect inlet and outlet pipes
- concrete riser section(s) and/or transition slab (if required)
- maintenance riser section(s) (if required)
- frame and access cover

4.1.2 The precast base should be placed level at the specified grade. The entire base should be in contact with the underlying compacted granular material. Subsequent sections, complete with joint seals, should be installed in accordance with the precast concrete manufacturer's recommendations.

4.1.3 Adjustment of the stormwater quality treatment device can be performed by lifting the upper sections free of the excavated area, re-leveling the base, and re-installing the sections. Damaged sections and gaskets should be repaired or replaced as necessary to restore original condition and watertight seals. Once the stormwater quality treatment device has been constructed, any/all lift holes must be plugged watertight with mortar or non-shrink grout.

4.1.4 Inlet and Outlet Pipes Inlet and outlet pipes should be securely set into the device using approved pipe seals (flexible boot connections, where applicable) so that the structure is watertight, and such that any pipe intrusion into the device does not impact the device functionality.

4.1.5 Frame and Cover Installation Adjustment units (e.g. grade rings) should be installed to set the frame and cover at the required elevation. The adjustment units should be laid in a full bed of mortar with successive units being joined using sealant recommended by the manufacturer. Frames for the cover should be set in a full bed of mortar at the elevation specified.

4.2 MAINTENANCE ACCESS WALL
In some instances the Maintenance Access Wall, if provided, shall require an extension attachment and sealing to the precast wall and cartridge deck at the job site, rather than at the precast facility. In this instance, installation of these components shall be performed according to instructions provided by the manufacturer.

4.3 FILTER CARTRIDGE INSTALLATION Filter cartridges shall be installed in the cartridge deck only after the construction site is fully stabilized and in accordance with the manufacturer's guidelines and recommendations. Contractor to contact the manufacturer to schedule cartridge delivery and review procedures/requirements to be completed to the device prior to installation of the cartridges and activation of the system.

PART 5 – QUALITY ASSURANCE

5.1 FILTER CARTRIDGE INSTALLATION Manufacturer shall coordinate delivery of filter cartridges and other internal components with contractor. Filter cartridges shall be delivered and installed complete after site is stabilized and unit is ready to accept cartridges. Unit is ready to accept cartridges after is has been cleaned out and any standing water, debris, and other materials have been removed. Contractor shall take appropriate action to protect the filter cartridge receptacles and filter cartridges from damage during construction, and in accordance with the manufacturer's recommendations and guidance. For systems with cartridges installed prior to full site stabilization and prior to system activation, the contractor can plug inlet and outlet pipes to prevent stormwater and other influent from entering the device. Plugs must be removed during the activation process.

5.2 INSPECTION AND MAINTENANCE

5.2.1 The manufacturer shall provide an Owner's Manual upon request.

5.2.2 After construction and installation, and during operation, the device shall be inspected and cleaned as necessary based on the manufacturer's recommended inspection and maintenance guidelines and the local regulatory agency/body.

5.3 REPLACEMENT FILTER CARTRIDGES When replacement membrane filter elements and/or other parts are required, only membrane filter elements and parts approved by the manufacturer for use with the stormwater quality filter device shall be installed.

END OF SECTION

Project: **Amica North Shore**

Chamber Model -	MC-4500	
Units -	**Metric**	Click Here for Imperial
Number of Chambers -	56	
Number of chambers -	8	
Voids in the stone (porosity) -	40	%
Base of Stone Elevation -	100.00	m
Amount of Stone Above Chambers -	305	mm
Amount of Stone Below Chambers -	229	mm

☑ Include Perimeter Stone in Calculations

239.8 sq.meters Min. Area - 214.321 sq.meters

Height of System *(mm)*	Incremental Single Chamber *(cubic meters)*	Incremental Single End Cap *(cubic meters)*	Incremental Chambers *(cubic meters)*	Incremental End Cap *(cubic meters)*	Incremental Stone *(cubic meters)*	Incremental Chamber, End Cap and Stone *(cubic meters)*	Cumulative System *(cubic meters)*	Elevation *(meters)*
2057	0.00	0.00	0.00	0.00	2.435	2.44	303.44	102.06
2032	0.00	0.00	0.00	0.00	2.435	2.44	301.00	102.03
2007	0.00	0.00	0.00	0.00	2.435	2.44	298.57	102.01
1981	0.00	0.00	0.00	0.00	2.435	2.44	296.13	101.98
1956	0.00	0.00	0.00	0.00	2.435	2.44	293.70	101.96
1930	0.00	0.00	0.00	0.00	2.435	2.44	291.26	101.93
1905	0.00	0.00	0.00	0.00	2.435	2.44	288.83	101.91
1880	0.00	0.00	0.00	0.00	2.435	2.44	286.39	101.88
1854	0.00	0.00	0.00	0.00	2.435	2.44	283.96	101.85
1829	0.00	0.00	0.00	0.00	2.435	2.44	281.52	101.83
1803	0.00	0.00	0.00	0.00	2.435	2.44	279.08	101.80
1778	0.00	0.00	0.00	0.00	2.435	2.44	276.65	101.78
1753	0.00	0.00	0.06	0.00	2.409	2.47	274.21	101.75
1727	0.00	0.00	0.18	0.00	2.361	2.55	271.74	101.73
1702	0.00	0.00	0.26	0.01	2.328	2.60	269.19	101.70
1676	0.01	0.00	0.33	0.01	2.298	2.64	266.60	101.68
1651	0.01	0.00	0.43	0.02	2.259	2.70	263.96	101.65
1626	0.01	0.00	0.72	0.02	2.140	2.88	261.26	101.63
1600	0.02	0.00	1.05	0.03	2.003	3.08	258.38	101.60
1575	0.02	0.00	1.27	0.03	1.916	3.21	255.30	101.57
1549	0.03	0.00	1.44	0.04	1.844	3.32	252.08	101.55
1524	0.03	0.01	1.59	0.04	1.782	3.42	248.76	101.52
1499	0.03	0.01	1.72	0.05	1.726	3.50	245.34	101.50
1473	0.03	0.01	1.85	0.05	1.675	3.57	241.85	101.47
1448	0.03	0.01	1.96	0.06	1.628	3.65	238.27	101.45
1422	0.04	0.01	2.06	0.07	1.584	3.71	234.63	101.42
1397	0.04	0.01	2.16	0.07	1.543	3.77	230.91	101.40
1372	0.04	0.01	2.25	0.08	1.504	3.83	227.14	101.37
1346	0.04	0.01	2.34	0.08	1.467	3.89	223.31	101.35
1321	0.04	0.01	2.42	0.09	1.432	3.94	219.42	101.32
1295	0.04	0.01	2.50	0.09	1.399	3.99	215.48	101.30
1270	0.05	0.01	2.57	0.10	1.367	4.04	211.49	101.27
1245	0.05	0.01	2.64	0.10	1.336	4.08	207.45	101.24
1219	0.05	0.01	2.71	0.11	1.307	4.13	203.37	101.22
1194	0.05	0.01	2.78	0.11	1.279	4.17	199.24	101.19
1168	0.05	0.01	2.84	0.12	1.253	4.21	195.07	101.17
1143	0.05	0.02	2.90	0.12	1.227	4.25	190.86	101.14
1118	0.05	0.02	2.95	0.13	1.202	4.28	186.62	101.12
1092	0.05	0.02	3.01	0.13	1.178	4.32	182.33	101.09
1067	0.05	0.02	3.06	0.14	1.155	4.35	178.01	101.07
1041	0.06	0.02	3.11	0.14	1.133	4.39	173.66	101.04
1016	0.06	0.02	3.16	0.14	1.112	4.42	169.27	101.02
991	0.06	0.02	3.21	0.15	1.091	4.45	164.85	100.99
965	0.06	0.02	3.26	0.15	1.071	4.48	160.40	100.97
940	0.06	0.02	3.30	0.16	1.052	4.51	155.92	100.94
914	0.06	0.02	3.34	0.16	1.034	4.54	151.41	100.91
889	0.06	0.02	3.38	0.16	1.016	4.56	146.87	100.89
864	0.06	0.02	3.42	0.17	0.999	4.59	142.31	100.86
838	0.06	0.02	3.46	0.17	0.982	4.61	137.72	100.84
813	0.06	0.02	3.50	0.17	0.966	4.64	133.10	100.81
787	0.06	0.02	3.53	0.18	0.951	4.66	128.46	100.79
762	0.06	0.02	3.57	0.18	0.936	4.68	123.80	100.76
737	0.06	0.02	3.60	0.19	0.921	4.71	119.12	100.74
711	0.06	0.02	3.63	0.19	0.907	4.73	114.41	100.71
686	0.07	0.02	3.66	0.19	0.895	4.75	109.68	100.69
660	0.07	0.02	3.69	0.19	0.882	4.76	104.94	100.66
635	0.07	0.02	3.72	0.20	0.870	4.78	100.17	100.64
610	0.07	0.03	3.74	0.20	0.858	4.80	95.39	100.61
584	0.07	0.03	3.77	0.20	0.847	4.82	90.59	100.58
559	0.07	0.03	3.79	0.21	0.836	4.83	85.77	100.56
533	0.07	0.03	3.81	0.21	0.826	4.85	80.94	100.53
508	0.07	0.03	3.84	0.21	0.816	4.86	76.09	100.51
483	0.07	0.03	3.86	0.21	0.807	4.88	71.22	100.48
457	0.07	0.03	3.88	0.22	0.798	4.89	66.35	100.46
432	0.07	0.03	3.90	0.22	0.789	4.90	61.46	100.43
406	0.07	0.03	3.91	0.22	0.781	4.92	56.55	100.41
381	0.07	0.03	3.93	0.22	0.774	4.93	51.64	100.38
356	0.07	0.03	3.95	0.23	0.766	4.94	46.71	100.36
330	0.07	0.03	3.96	0.23	0.760	4.95	41.77	100.33
305	0.07	0.03	3.97	0.23	0.753	4.96	36.82	100.30
279	0.07	0.03	3.99	0.23	0.747	4.97	31.86	100.28
254	0.07	0.03	4.01	0.23	0.739	4.98	26.90	100.25
229	0.00	0.00	0.00	0.00	2.435	2.44	21.92	100.23
203	0.00	0.00	0.00	0.00	2.435	2.44	19.48	100.20
178	0.00	0.00	0.00	0.00	2.435	2.44	17.05	100.18
152	0.00	0.00	0.00	0.00	2.435	2.44	14.61	100.15
127	0.00	0.00	0.00	0.00	2.435	2.44	12.18	100.13
102	0.00	0.00	0.00	0.00	2.435	2.44	9.74	100.10
76	0.00	0.00	0.00	0.00	2.435	2.44	7.31	100.08
51	0.00	0.00	0.00	0.00	2.435	2.44	4.87	100.05
25	0.00	0.00	0.00	0.00	2.435	2.44	2.44	100.03

APPENDIX D

Functional Servicing Plan
Functional Grading Plan