STORMWATER MANAGEMENT & FUNCTIONAL SERVICING REPORT

FOR

BOLTCOL NORTH INDUSTRIAL DEVELOPMENT LOT 1 & LOT 2 COLERAINE DR.

TOWN OF CALEDON

TOWN VARIANCE SPA#: 19-44 / RZ 19-06
REGION SPA#: 18-0004C

VARIANCE SUBMISSION

June 8, 2020

a.m. candaras associates inc.
8551 Weston Rd, Suite 203
Woodbridge, Ontario
L4L 9R4

Project No. 1731

a.m. candaras associates inc.
consulting engineers

TABLE OF CONTENTS

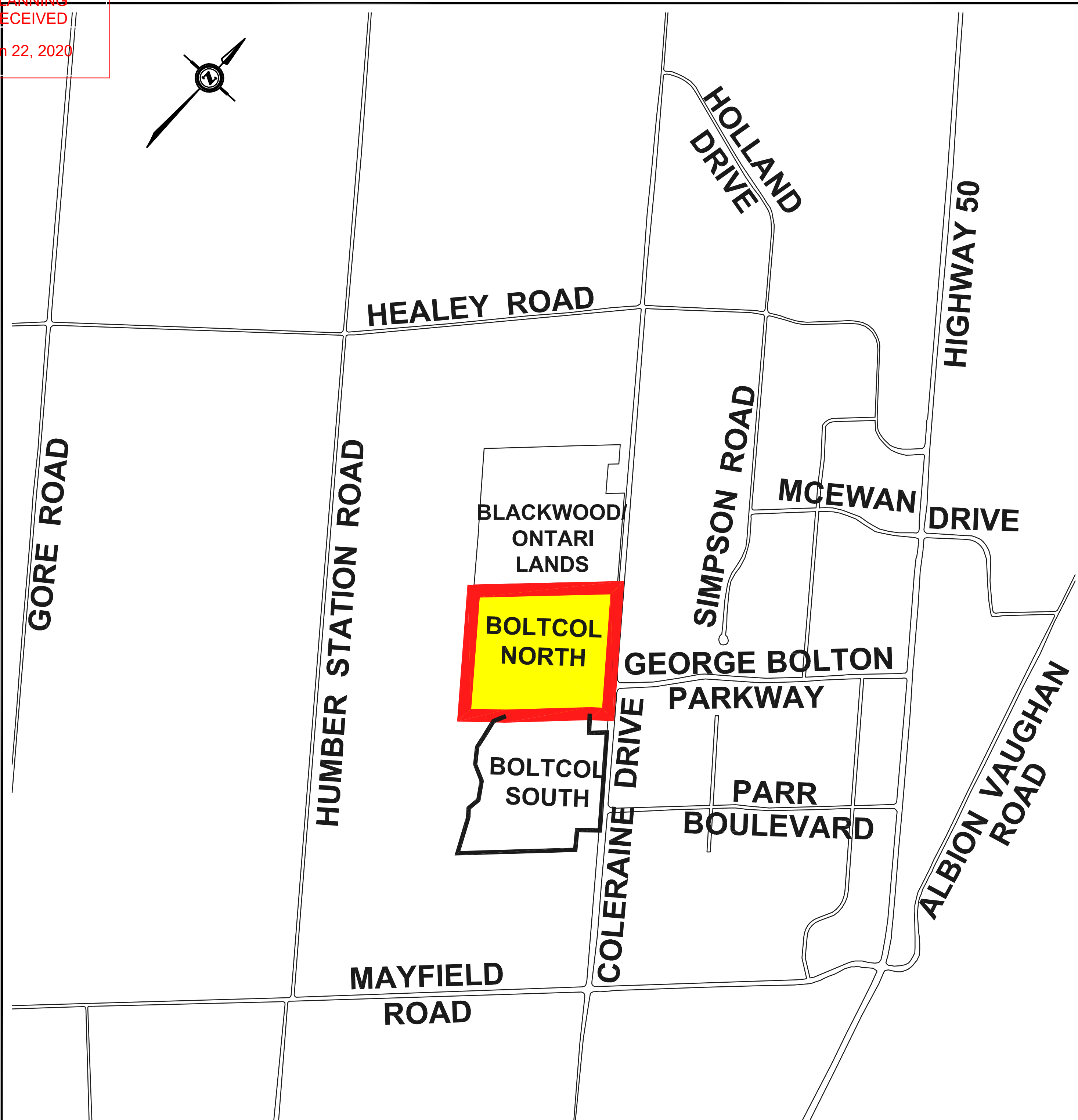
1.0 INTRODUCTION

This report presents the Stormwater Management Analysis and Functional Servicing for a proposed Boltcol North Industrial development, located at northwest corner of Coleraine Dr. and George Bolton Parkway, in the Town of Caledon as shown in Figure 1. The site will be developed with three industrial warehouse buildings with frontage on Coleraine Dr., and also on the George Bolton Parkway extension, which will be constructed as part of this development.

Stormwater management will be provided in a proposed stormwater wet pond (SWMP 4+5), which will be constructed as part of this development. The design and related hydrologic and hydraulic analysis for SWMP 4+5 is provided in ***George Bolton Parkway Extension Stormwater Management and Functional Servicing Report: June 21, 2019,*** which was prepared for the Boltcol Holdings North Development. The overall development concept of the Boltcol Lands including the location of stormwater management facility 4+5, developed in the above noted document, as it relates to the Boltcol North development, is indicated in Figure 2 and Plan G1. Facility SWMP 4+5 will provide both quantity and quality controls for the site. In addition on site quantity controls, limiting the discharge to a rate at 180 l/s/ha will be implemented as well as an oil grit separator (60% TSS removal) to provide pre treatment. SWMP 4+5 will also accommodate the site uncontrolled discharge, in the event the onsite controls are inoperable, as required by the TRCA.

2.0 DESIGN CRITERIA

- The allowable stormwater discharge to be limited to 180 l/sec/ha for the 100-year storm event;
- On-site detention must be provided for the 100 year storm event;
- Stormwater quality control to provide 60% removal of annual total suspended solids (TSS), as a pre-treatment prior to discharging to Stormwater Management Pond #4+5;
- Infiltration of stormwater runoff, based on 5mm, as per recommendations in Water Balance Analysis report;
- An overland flow route shall be provided within the developed site to direct runoff in excess of the 100-year storm to an overland flow outlet to SWM Pond #4+5;



LOCATION PLAN

FIGURE 1

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PROJECT:
BOLTCOL NORTH
INDUSTRIAL DEVELOPMENTS
TOWN OF CALEDON

SCALE : NTS

DATE: OCT 2017

PROJECT No. 1731

TOWN OF CALEDON
PLANNING
RECEIVED
Jun 22, 2020

SCOPE OF WORKS
FIGURE 2

PROJECT: BOLTCOL NORTH, TOWN OF CALEDON		
DATE: OCT 2017	SCALE : 1:5,000	PROJECT No. 1731

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3.0 SITE DESCRIPTION

3.1 Proposed Site

Site statistics are as follows:

Lot 1

Building 1:	=	82,700.0m²
Paved	=	65,692.1m²
Landscaped:	=	18,293.7m²
Lot 1 Site Area		166,685.8m²

Lot 2

Building 2:	=	22,622.1m²
Building 3:	=	13,660.5m²
Paved	=	26,198.5m²
Landscaped:	=	8,303.3m²
Lot 2 Site Area		70,784.4m²

3.2 Allowable Site Discharge

The allowable site runoff has been calculated below:

Lot 1 Allowable Discharge:

Q_S = 180 l/s/ha x 16.668 ha

Q_S = 3,000.3 l/s

Lot 2 Allowable Discharge:

Q_S = 180 l/s/ha x 7.078ha

Q_S = 1,274.1 l/s

4.0 ROOFTOP CONTROLS

The Lot 1 roof area will be equipped with Thaler Rd – 14 control flow drains as follows:

Lot 1 Building Area Roof Controls

Roof Area (m²)	No. of Notches	Notch Area (m²)	Flow per Notch (1)	Total Flows (l/s)
82,700	350	233	1.37 l/s	478.3

Q_R = 478.3 l/s

(1) Based on manufacturer's design tables at a 102mm depth, 232m²/notch, 82 l/m.

The resulting total roof top volume ponding is:

$$3{,}940.6 \text{ m}^3$$

, as indicated in Table 1a. The available roof top storage is 4,135.0 m^3, based on a maximum ponding depth of 100 mm, as indicated in the Rooftop Available Storage calculations located in Appendix A.

The Lot 2 roof areas will be equipped with Thaler Rd – 14 control flow drains as follows:

Lot 2 Building Area Roof Controls

Building	Roof Area (m^2)	No. of Notches	Notch Area (m^2)	Flow per Notch [1]	Total Flows (l/s)
Building 2	22,622.1	100	226	1.37 l/s	136.0
Building 3	13,660.5	60	228	1.37 l/s	82.0
Total	36,282.6	160			**Q_R = 218.7 l/s**

(1) Based on manufacturer's design tables at a 102mm depth, 232m^2/notch, 82 l/m.

The resulting total roof top volume ponding for Building 2 & 3 is:

$$1{,}702.4 \text{ m}^3$$

, as indicated in Table 1b. The total available roof top storage is 1,814.1m^3, based on a maximum ponding depth of 100 mm, as indicated in the Rooftop Available Storage calculations located in Appendix A.

5.0 DETENTION VOLUME CALCULATIONS

Lot 1:

Allowable Site Runoff:

Q_S = 3,000.3 l/s

Roof Flow:

Q_R = 478.3 l/s

Allowable discharge from the paved and landscaped areas:

Q_{PL} = $Q_S - Q_R$

Q_{PL} = 3,000.3 l/s – 478.3 l/s

= 2,522.0 l/s

Storage required = 642.8 m^3

as calculated in Table 2

Lot 2:

Allowable Site Runoff:

Q_S = 1,274.1 l/s

Roof Flow:

Q_R = 218.7 l/s

Allowable discharge from the paved and landscaped areas:

Q_{PL} = $Q_S - Q_R$

Q_{PL} = 1,274.1 l/s – 218.7 l/s

= 1,055.5 l/s

Storage required = 236.4 m^3

as calculated in Table 3

5.1 Available Detention Volume

Lot 1:

Based on a high water level of 236.20m, the available surface storage ponding volume is:

1,835.0 m^3

See Drawing G2 for surface storage areas and volume calculations.

Lot 2:

Based on a high water level of 237.65m, the available surface storage ponding volume is:

699.3 m^3.

See drawing G3 for surface storage areas and volume calculations.

6.0 OUTLET CONTROLS

Orifice plates will be provided to control the discharge from Lot 1 and Lot 2.

Lot 1:

The discharge is to be limited to:

$$Q_{PL} = 2{,}522.0 \text{ l/s}$$

Sizing of the orifice is as follows:

$$Qo = C \cdot A \cdot \sqrt{(2 \cdot g \cdot h)}$$

where:

$$h = \text{HWL} - \text{Inv. of Orifice}$$

$$h = 236.20\text{m} - 231.72\text{m}$$

$$h = 4.48\text{m}$$

$$A = \frac{Q}{C \cdot \sqrt{(2 \cdot g \cdot h)}}$$

$$A = \frac{2.522\text{m}^3/\text{s}}{(0.60) \cdot \sqrt{(2 \cdot 9.81\text{m/s}^2 \cdot 4.48\text{m})}}$$

$$A = 0.448\text{m}^2$$

$$d = \sqrt{\frac{4\text{x}A}{\pi}}$$

$$d = \sqrt{\frac{4 \, x \, 0.448\text{m}^2}{\pi}}$$

$$d = 0.756\text{m}$$

Therefore, use a 756mm orifice plate to be located on the downstream face of Control CBMH #101.

Lot 2:

The discharge is to be limited to:

$$Q_{PL} = 1{,}055.5 \text{ l/s}$$

Sizing of the orifice tube is as follows:

$$Qo = C \cdot A \cdot \sqrt{(2 \cdot g \cdot h)}$$

where:

$$h = \text{HWL} - \text{Inv. of Orifice}$$

$$h = 237.65\text{m} - 233.64\text{m}$$

$$h = 4.01\text{m}$$

$$A = \frac{Q}{C \cdot \sqrt{(2 \cdot g \cdot h)}}$$

$$A = \frac{1.055 m^3/s}{(0.60) \cdot \sqrt{(2 \cdot 9.81 m/s^2 \cdot 4.01 m)}}$$

$$A = 0.198 \, m^2$$

$$d = \sqrt{\frac{4xA}{\pi}}$$

$$d = \sqrt{\frac{4 \text{ x } 0.198 m^2}{\pi}}$$

$$d = 0.502m$$

Therefore, use a 502mm orifice plate to be located on the downstream face of Control MH #2.

7.0 STORMWATER QUALITY CONTROLS

Lot 1:

An oil grit separator will be provided and sized based on the paved and landscaped area of 8.55ha, having a total imperviousness of 80%, and a controlled flow rate of 2,522.0l/s.

An STC 4000 will be provided based on the manufacturer's design table to provide 60% TSS removal, refer to Appendix A for the sizing report. The Stormceptor will be located downstream of Control CBMH #101 as detailed on Drawing G2. The control flow roof water, which is clean water, will bypass the oil grit separator.

Lot 2:

An oil grit separator was sized based on the paved and landscaped area of 3.76ha, having a total imperviousness of 75%, and a controlled flow rate of 1,055.5 l/s. The control flow roof, which is clean water will bypass the oil grit separator.

An STC 2000 will be provided based on the manufacturer's design table to provide 60% TSS removal, refer to Appendix A for the sizing report. The Stormceptor will be located downstream of Control MH #2 as detailed on Drawing G3.

8.0 WATER BALANCE

The Water Balance Analysis report determined that the 5mm storm be retained and infiltrated onsite to satisfy water balance requirements (See Appendix B).

Lot 1:

A stone infiltration trench will be provided in the southwest portion of Lot 1. The storm discharge from the roof area will be directed to the stone infiltration trench.
The resulting required volume for the total site is:

$$\left(\frac{5mm}{1000}\right) x\ 166{,}685$$

$$= \quad 833.4\ m^3$$

A single 79.0m x 23.0m wide x 1.20m high stone trench, having a 40% void ratio, will provide a total storage of 872.2 m^3. The hydrogeological report determined that the groundwater level at MW 34, which is in the area of the proposed infiltration trench, is 232.10m. The bottom of the infiltration trench will be set at 233.10m.

Lot 2:

A stone infiltration trench will be provided along the east and south portions of Lot 2. The storm discharge from the roof areas will be directed to the stone infiltration trench.

The resulting volume for the total site is:

$$\left(\frac{5mm}{1000}\right) x\ 70{,}784.0$$

$$= \quad 353.9\ m^3$$

A continuous stone trench, 388.4m x 2.5m wide x 1.0m high, having a void ratio of 40%, will provide a total storage of 388.4m^3. The hydrogeological report determined that the groundwater level of MW10, which is in the area of the proposed infiltration facility, is 235.30m. The bottom of the infiltration trench will be set to be above 236.30m.

9.0 GEORGE BOLTON SWM POND AND COLERAINE DR MAJOR DRAINAGE

The existing George Bolton SWM Pond, on the east side of Coleraine Dr, has not been designed to control flows for the Regional Storm. The resulting Regional Storm flows will overtop this SWM pond, at the emergency overflow, LOCATED at the southwest end of the pond and flows west across Coleraine Drive towards the existing channel. The Regional Storm event has been modelled to convey all flows east of Coleraine, including the George Bolton SWM Pond, to the proposed George Bolton Parkway Extension. The Regional Storm flows of 5.29 m^3/s will be conveyed along the George Bolton Parkway extension right of way as indicated on Plan S10. The external Regional Storm flows, from east of Coleraine Dr, will not be controlled in Pond 4+5, however they will be routed through the proposed SWM 4+5 facility.

10.0 STORM SERVICE CONNECTION

Lot 1:
The proposed development will discharge stormwater to the adjacent Stormwater Management Facility SWMP 4+5. A 1200mm storm connection will be provided as detailed on Drawing G2.

Lot 2:
The proposed development will discharge to the proposed storm sewer on George Bolton Parkway extension which discharges to Stormwater Management Facility SWMP 4+5. A 975mm storm connection will be provided as detailed on Drawing G3.

11.0 EROSION AND SEDIMENT CONTROLS

A master Erosion and Sediment Control plan has been prepared for the overall grading, topsoil stockpiling and cut/fill operations for these lands is provided in Plan S2.

For the development of Lot 1 and Lot 2, silt fences, CB sediment traps and mud mats will be provided as indicated on Plans G2, G3 and C1.

The proposed SWMP 4+5 will act as a sedimentation, as detailed on Plan S2, and will operate as a sedimentation basin until Lot 1 and Lot 2 are completed and stabilized.

12.0 SANITARY SERVICE CONNECTION

12.1 Sanitary Design Flow

Lot 1, Building 1:

The peak sanitary flow will discharge to the proposed 250mm sanitary connection to the existing 750mm municipal sewer on Coleraine Drive. The new connection to the Coleraine Dr sanitary sewer will also service Building 3. The population for building 1 is based on the anticipated maximum employee population for the full Phase 1 and 2 build out for the tenant. Sanitary sewage flows were calculated below:

Site Area = 16.67 ha

Population = 700 persons

Peaking Factor M = $1 + \frac{14}{4+P^{0.5}}$ where P = Populations in thousands

$= 1 + \frac{14}{4+(0.700)^{0.5}} = 3.9$

Peak Sewage Flow Q = $\frac{P \times q \times m}{86400} + IA$

Q = $\frac{700 \times 300 \text{ l/cap/day}}{86400} \times 3.9$

= 9.5 l/s + IA

Infiltration = 16.67 ha x 0.0002 m^3/sec/ha

= 0.0033 m^3/sec

Total Peak Flow = 9.5 l/s + 3.3 l/s

= 12.8 l/s

Lot 2, South Building 2:

Building 2 is a speculative building and does not have a specific tenant at the time of submission. The employee population of Building 2 has been estimated to be 200 people, based on the review of previous completed similar buildings owned by the developer. The peak sanitary flow will discharge to the proposed 250mm sanitary sewer on George Bolton Parkway extension using the Region of Peel Design Guidelines as follows:

Site Area = 4.35 ha

Population = 200 persons (Estimated employee population)

Peaking Factor M = $1 + \frac{14}{4+P^{0.5}}$ where P = Populations in thousands

= $1 + \frac{14}{4+(0.200)^{0.5}}$ = 4.15

Peak Sewage Flow Q = $\frac{P \times q \times m}{86400}$ + IA

Q = $\frac{200 \times 300 \text{ l/cap/day}}{86400}$ x 4.15

= 2.9 l/s + IA

Infiltration (IA) = 4.35 ha x 0.0002 m^3/sec/ha

= 0.0009 m^3/sec

Total Peak Flow = 2.9 l/s + 0.9 l/s

= 3.8 l/s

Lot 2, North Building 3:

The peak sanitary flow will discharge to the proposed 250mm sanitary sewer connection to Coleraine Dr using the Region of Peel Design Guidelines as follows:

Site Area = 2.73 ha

Population = 100 persons (Estimated employee population)

Peaking Factor M = $1 + \frac{14}{4+P^{0.5}}$ where P = Populations in thousands

= $1 + \frac{14}{4+(0.100)^{0.5}}$ = 4.24

Peak Sewage Flow Q = $\frac{P \times q \times m}{86400}$ + IA

Q = $\frac{100 \times 300 \text{ l/cap/day}}{86400}$ x 4.24

= 1.5 l/s + IA

Infiltration (IA) = 2.73 ha x 0.0002 m^3/sec/ha

= 0.0005 m^3/sec

Total Peak Flow = 1.5 l/s + 0.5 l/s

= 2.0 l/s

12.2 Proposed Sanitary Service

Lot 1, Building 1:

A 250mm sanitary service connection will be provided to the existing 750mm sanitary sewer on Coleraine Dr. The sanitary service alignment will pass between Building 2 and Building 3, connecting to the east face of Building 1. The sanitary connection to Coleraine Dr will service both Building 3 and Building 1, in accordance with Region of Peek Standards 1-8-9, for the servicing of multiple buildings with no frontage. The sanitary connection to the Coleraine Dr sewer system will be a 250mm dia pipe at a minimum slope of 0.30%, resulting in a capacity of 32.6 l/s, which can convey the combined flow of 7.8 l/s from Building 1 and 3.

Lot 2, South Building 2:

A 200mm sanitary service connection will be provided to the proposed 250mm sanitary sewer on George Bolton Parkway. The design flow for Building 2 is 2.7 l/s, as calculated above. The capacity of the 200mm dia sanitary connection at minimum slope of 0.50% is 24.2 l/s.

Lot 2, North Building 3:

A 250mm sanitary service connection will be provided to the existing 750mm sanitary sewer on Coleraine Dr. The sanitary connection will service both building 3 and Building 1, in accordance with Region of Peek Standards 1-8-9, for the servicing of multiple buildings with no frontage.

13.0 WATER SERVICE CONNECTION

Lot 1:

A 300mm water connection will be provided to the proposed 300mm watermain on George Bolton Parkway extension. There will also be a 100mm domestic as per Region of Peel Standards. An additional redundant 250mm dia. watermain will also be provided, from the northeast corner of the building, through Lot2 and connecting to the future 400mm watermain on Coleraine Dr.

Lot 2:

A 250mm water connection will be provided to the proposed 300mm watermain on George Bolton Parkway extension. There will also be a 100mm domestic as per Region of Peel Standards.

13.1 Domestic and Fireflow Demand

Lot 1:

The domestic demands were based on Region of Peel Standards with fire flows referencing the Fire Underwriters Survey Document.

Site Area	=	16.67 ha
Population Density	=	70 persons/ha (Industrial)

* Use anticipated employee occupancy for Buildings 1, as requested by Region staff.

Population Estimate	=	700 persons
Consumption	=	300 l/employee/day
Max Day Factor	=	1.4
Peak Hour Factor	=	3.0

<u>Water Demands</u>

Average Daily Demand

= 300 l/capita/day x 700 people
= 21,000 l/day
= 2.43 l/s

Maximum Daily Demand

= 300 l/capita/day x 700 people x 1.4 (Max day factor)
= 294,000 l/day
= 3.40 l/s

Peak Hour Demand

= 300 l/capita/day x 700 people x 3.0 (Peak Hour factor)
= 630,000 l/day
= 7.29 l/s

Fire Flow Calculation (Based on Fire Underwriters Survey 1999)

1. An estimate of the fire flow required for a given area is determined by the formula:

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

Where, F = the required fire flow in litres per minute l/m
C = Construction type coefficient= 0.6 (Fire resistive construction)
A = Total area (based on construction type and protected openings)

Building Area = 82,700.0 m^2

$$F = 220(0.60)\sqrt{82{,}700\ m^2}$$
$$F = 37{,}960\ l/m \text{ (633 l/s)}$$

Therefore use: $F = 38{,}000\ l/m$ (633 l/s)

2. Occupancy Reduction
Office Area = 0% Increase based on Commercial buildings

∴ Total Reduction = 0%

F_2 = 38,000 *l/m* - (38,000 l/m × 0%)
F_2 = 38,000 *l/m (633 l/s)*

3. Sprinkler Reduction
50% Reduction for NFPA 13 System

4. Separation Charge

East Side (> 45m)	= 0%
West Side (> 45m)	= 0%
North Side (> 45m)	= 0%
South Side (30.1 - 45m)	= 0%
Total Separation Charge	= 0%

F_{final} = F_2 – (F_2 × 50%) + (F_2 × 0%)
F_{final} = 38,000 l/m – (19,000 l/min) + (0 l/min)
F_{final} = 19,000 l/min (317 l/s)

Therefore use: F_{final} = 19,000 l/min (317 l/s)
F_{final} = 5,019 US gpm

The water supply system will be designed to convey the greater of the fire flow plus maximum day demand or the peak hour demand. The greater flow results from the fire flow plus max day, as calculated below.

Fire Flow + Max Day= 317 l/s + 3.4 l/s
= 320.4 l/s
= 19,224 l/min (5,078 US gpm)

Lot 2:

The domestic demands were based on Region of Peel Standards with fire flows referencing the Fire Underwriters Survey Document.

Site Area	=	7.08 ha
Population Density	=	70 persons/ha (Industrial)

* Use estimated employee occupancy for Buildings 2 and 3, as requested by Region staff.

Population Estimate	=	300 people
Consumption	=	300 l/employee/day
Max Day Factor	=	1.4
Peak Hour Factor	=	3.0

Water Demands

Average Daily Demand

= 300 l/capita/day x 300 people

= 90,000 l/day

= 1.04 l/s

Maximum Daily Demand

= 300 l/capita/day x 300 people x 1.4 (Max day factor)

= 126,000 l/day

= 1.5 l/s

Peak Hour Demand

= 300 l/capita/day x 300 people x 3.0 (Peak Hour actor)

= 270,000 l/day

= 3.1 l/s

Fire Flow Calculation (Based on Fire Underwriters Survey 1999)

2. An estimate of the fire flow required for a given area is determined by the formula:

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

Where, F = the required fire flow in litres per minute l/m
C = Construction type coefficient= 0.6 (Fire resistive construction)
A = Total area (based on construction type and protected openings)

Building Area = 22,622 m^2 (largest building on Lot 2)

$F = 220(0.60)\sqrt{22{,}622\ m^2}$
$F = 19{,}840\ l/m$ (6331 l/s)

Therefore use: $F = 20{,}000\ l/m$ (333 l/s)

2. Occupancy Reduction

Office Area = 0% Increase based on Commercial buildings

∴ Total Reduction = 0%

F_2 = 20,000 *l/m* - (20,000 l/m × 0%)
F_2 = 20,000 *l/m (333 l/s)*

3. Sprinkler Reduction

50% Reduction for NFPA 13 System

4. Separation Charge

East Side (> 45m)	= 0%
West Side (> 45m)	= 0%
North Side (> 45m)	= 0%
South Side (30.1 - 45m)	= 0%
Total Separation Charge	= 0%

$F_{final} = F_2 - (F_2 \times 50\%) + (F_2 \times 0\%)$
F_{final} = 20,000 l/m – (10,000 l/min) + (0 l/min)
F_{final} = 10,000 l/min (167 l/s)

Therefore use: F_{final} = 10,000 l/min (167 l/s)
F_{final} = 2,203 US gpm

The water supply system will be designed to convey the greater of the fire flow plus maximum day demand or the peak hour demand. The greater flow results from the fire flow plus max day, as calculated below.

Fire Flow + Max Day = 167 l/s + 1.5 l/s
= 168.5 l/s
= 10,110 l/min (2,671 US gpm)

13.2 Sprinkler Flow Calculation

A sprinkler water supply for firefighting calculation was completed for a 875,000 ft^2 (76,647 m^2) warehouse building. The total water required for fire fighting is 1849 US gpm (See Appendix C).

13.3 Hydrant Flow Test

A hydrant flow test was completed on July 26, 2017 on the 300mm watermain at the northwest corner of Parr Blvd and Coleraine Dr with the results presented in Appendix C. Based on the required Sprinkler Flow of 1849 US gpm the resulting available pressure is 78 psi.

Based on the hydrant flow test results, the extrapolated available flow at 20 psi is 6142.9 US gpm which exceeds the maximum Fire Flow and Max Day of 5040 US gpm calculated in Section 13.1 (See Appendix C).

13.4 Connection Single Use Demand

The Water and Wastewater Connection Table has been completed for Lot 1 and Lot 2 and is provided in Appendix 'C'.

14.0 DEVELOPMENT PHASING

The Channel re-alignment (item 1, in Figure 2) has been completed and will be followed by the installation of the 1500mm diversion sewer paralleling Coleraine Drive within a 5m easement (item 3 in Figure 2). Once the filling of the existing channel (item 5 Figure 2) is completed, the Boltcol North lands will then be developed as industrial sites.

The anticipated land use for the subject site and adjacent property will be primarily warehouse buildings. Development is expected to begin in the north portion of the Boltcol lands, and proceed to the south. Within the Boltcol South block, the southwest portion of those lands have been designated MTO Study Area, for Highway No 413.

The anticipated building size and development timing for the Ontari lands and the Boltcol lands are summarized in Table 4, and indicated on Plan S8. Water servicing for the Ontari and Boltcol developments will be from the existing 300mm watermain located on the east side of Coleraine Dr. The sanitary servicing will be from the existing 750mm sanitary trunk sewer. The locations of the new connections points as listed below:

- Ontari sites to be serviced by connection at the Ontari driveway entrance
- Boltcol north will be serviced by connection at Coleraine Dr and George Bolton Parkway
- Boltcol south will be serviced by connection at Coleraine Dr and the Boltcol south driveway entrance.

Table 4– Development Phasing: Building Areas and Estimated Completion

Land Owner	Block/Site	Building Area Sq.ft	Estimated Completion Date	Servicing Location
Ontari	South	873,000	2018	Ontari Entrance Connection
Ontari	North	800,000	2020	Ontari Entrance Connection
Boltcol North	Building 1	883,000	2019	Coleraine Drive
Boltcol North	Building 2	243,500	2019	George Bolton Ext
Boltcol North	Building 3	147,000	2019	Coleraine Drive
Boltcol South	Building A	118,000	2020	George Bolton Ext
Boltcol South	Building B	311,000	2020	South Boltcol Driveway
Boltcol South	Building C	70,000	2021	South Boltcol Driveway
Boltcol South	Building D	685,000	2022	South Boltcol Driveway
Boltcol South	Building E	457,000	2019	South Boltcol Driveway
Boltcol South	Vacant Lands	75,000	2021	George Bolton Ext

15.0 CONCLUSIONS

- The proposed stormwater management design for Lot 1 and Lot 2 meets the allowable discharge rate of 180l/s/ha as stipulated in the Ontari/Boltcol Stormwater Management Plan by providing a 756mm orifice plate on the downstream face of CBMH #101, for Lot 1 and a 502mm orifice plate on the downstream face of MH #2 for Lot 2.
- The proposed design provides pre-treatment meeting 60% TSS removal prior to discharge into the stormwater management facility by means of a Stormceptor OGS;
- The water balance requirement of 5mm is achieved by providing stone infiltration trenches;

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June 8, 2020

APPENDIX A
STORMWATER DATA

TABLE 1a:
Building 1

100-Year Storm Runoff Computations for Rooftop

Time Period (min)	Intensity (mm/hr)	Runoff (l/s)	Storage (m^3)
25-30	10	218.3	0.0
30-35	14	305.6	0.0
35-40	19	414.7	0.0
40-45	29	632.9	46.4
45-50	51	1113.1	190.4
50-55	114	2488.1	602.9
55-60	239	5216.3	1421.4
60-65	141	3077.4	779.7
65-70	86	1877.0	419.6
70-75	59	1287.7	242.8
75-80	43	938.5	138.0
80-85	33	720.2	72.6
85-90	26	567.5	26.7
90-95	21	458.3	0.0
95-100	17	371.0	0.0
100-105	15	327.4	0.0
105-110	13	283.7	0.0
			3,940.6m^3

Total Roof Area = 82,700.0m^2 @ C = 0.95

CAN = $\frac{(82{,}700.0\text{ha})\cdot(0.95)\cdot(2.778)}{10{,}000}$

= 21.83

Roof Outflow = 478.0 l/s

Storage (m^3) = $\frac{(\text{Runoff} - \text{Roof Outflow})\cdot 5\text{ min}\cdot 60\text{ sec})}{1{,}000}$

TABLE 1b:
Building 2 & 3

100-Year Storm Runoff Computations for Rooftop

Time Period (min)	Intensity (mm/hr)	Runoff (l/s)	Storage (m^3)
25-30	10	95.8	0.0
30-35	14	134.1	0.0
35-40	19	181.9	0.0
40-45	29	277.7	17.7
45-50	51	488.3	80.9
50-55	114	1091.6	261.9
55-60	239	2288.5	621.0
60-65	141	1350.1	339.4
65-70	86	823.5	181.4
70-75	59	564.9	103.9
75-80	43	411.7	57.9
80-85	33	316.0	29.2
85-90	26	249.0	9.1
90-95	21	201.1	0.0
95-100	17	162.8	0.0
100-105	15	143.6	0.0
105-110	13	124.5	0.0

<u>1702.4m^3</u>

Total Roof Area = 36,282.6 m^2 @ C = 0.95

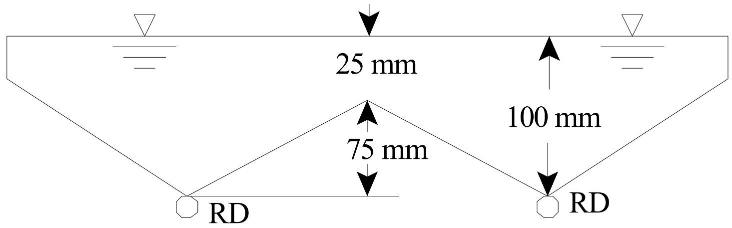
CAN = $\dfrac{(36{,}282.6\text{ha})\cdot(0.95)\cdot(2.778)}{10{,}000}$

= 9.58

Roof Outflow = 218.7 l/s

Storage (m^3) = $\dfrac{(\text{Runoff}-\text{Roof Outflow})\cdot 5\text{ min}\cdot 60\text{ sec})}{1{,}000}$

ROOFTOP STORAGE AVAILABLE CALCULATIONS: BUILDING 1



Rooftop Ponding:

Area per Drain $= 82{,}700\text{m}^2/350\text{ drains} = 236.3\text{m}^2/\text{drain}$

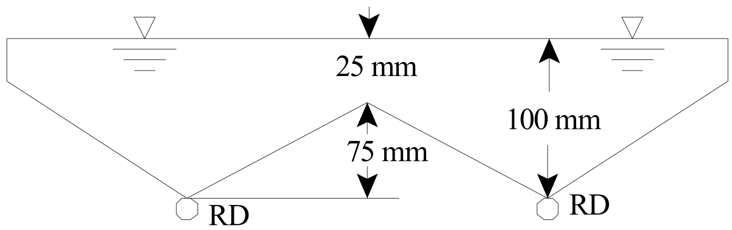
Available Ponding Volume per Drain $= \dfrac{\text{l}\cdot\text{w}\cdot\text{h}}{3} + \text{l}\cdot\text{w}\cdot\text{h}$

Ponding Volume Per Drain $= \dfrac{(236.3\text{m}^2)\cdot(0.075\text{m})}{3} + (236.3\text{m}^2)\cdot(0.025\text{m}) = 11.8\text{m}^3/\text{drain}$

$\text{Rooftop Volume Provided} = 11.8\text{m}^3 \cdot 350\text{drains} = 4135.0\text{m}^3$

Required Rooftop Volume = 3,940.6m^3

ROOFTOP STORAGE AVAILABLE CALCULATIONS: BUILDING 2 & 3



Rooftop Ponding:

Area per Drain $= 36{,}282.6\text{m}^2/160 \text{ drains} = 226.8\text{m}^2/\text{drain}$

Available Ponding Volume per Drain $= \dfrac{\text{l}\cdot\text{w}\cdot\text{h}}{3} + \text{l}\cdot\text{w}\cdot\text{h}$

Ponding Volume Per Drain $= \dfrac{(226.8\text{m}^2)\cdot(0.075\text{m})}{3} + (226.8\text{m}^2)\cdot(0.025\text{m}) = 11.3\text{m}^3/\text{drain}$

Rooftop Volume Provided $= 11.3\text{m}^3 \cdot 160\text{drains} = 1814.1\text{m}^3$

Required Rooftop Volume = 1702.4m^3

<u>TABLE 2:</u>

100-Year Storm Runoff Computations for Paved and Landscaped Area: Lot 1

Time Period (min)	Intensity (mm/hr)	Runoff (l/s)	Storage (m^3)
25-30	10	189.1	0.0
30-35	14	264.8	0.0
35-40	19	359.3	0.0
40-45	29	548.5	0.0
45-50	51	964.5	0.0
50-55	114	2156.0	0.0
55-60	239	4520.0	599.4
60-65	141	2666.6	43.4
65-70	86	1626.5	0.0
70-75	59	1115.8	0.0
75-80	43	813.2	0.0
80-85	33	624.1	0.0
85-90	26	491.7	0.0
90-95	21	397.2	0.0
95-100	17	321.5	0.0
100-105	15	283.7	0.0

TOTAL REQUIRED STORAGE VOLUME <u>642.8m³</u>

Net Paved = 65,692.1m² @ C = 0.95

Net Landscaped = 18,293.7m² @ C = 0.31

$$\text{CAN} = \frac{[(65{,}692.1\text{ha})\cdot(0.95)+(18{,}293.7\text{ha})\cdot(0.31)]\cdot(2.778)}{10{,}000}$$

= 18.912

Outflow[(1)]: = 2,522.0 l/s

$$\text{Storage } (m^3) = \frac{(\text{Runoff}-\text{Outflow})\cdot 5\text{ min}\cdot 60\text{ sec})}{1{,}000}$$

(1) Outflow is the allowable discharge from the paved and landscaped areas.

$Q = Q_S - Q_R$

= 3,000.3 l/s – 478.3 l/s

= 2,522.0 l/s

TABLE 3:

100-Year Storm Runoff Computations for Paved and Landscaped Area: LOT 2

Time Period (min)	Intensity (mm/hr)	Runoff (l/s)	Storage (m^3)
25-30	10	76.3	0.0
30-35	14	106.8	0.0
35-40	19	145.0	0.0
40-45	29	221.2	0.0
45-50	51	389.1	0.0
50-55	114	869.7	0.0
55-60	239	1823.4	230.4
60-65	141	1075.7	6.1
65-70	86	656.1	0.0
70-75	59	450.1	0.0
75-80	43	328.1	0.0
80-85	33	251.8	0.0
85-90	26	198.4	0.0
90-95	21	160.2	0.0
95-100	17	129.7	0.0
100-105	15	114.4	0.0

TOTAL REQUIRED STORAGE VOLUME **236.4m^3**

Paved = 26,198.5 m^2 @ C = 0.95

Landscaped = 8,303.3 m^2

$$\text{CAN} = \frac{[(26{,}198.5\text{ha})+(8{,}303.3\text{m2}\cdot(0.31)]\cdot(2.778)}{10{,}000}$$

= 7.63

Outflow[1]: = 1055.5 l/s

$$\text{Storage } (m^3) = \frac{(\text{Runoff}-\text{Outflow})\cdot 5\text{ min}\cdot 60\text{ sec})}{1{,}000}$$

[2] Outflow is the allowable discharge from the paved and landscaped areas.

Q = $Q_S - Q_R$

= 1,274.1 l/s – 218.7 l/s

= 1,055.5 l/s

ROOF DRAIN MANUFACTURERS DESIGN TABLE

LOCATION	SQUARE METRE (SQUARE FOOT) NOTCH AREA RATING	ROOF LOAD FACTOR KGS (LBS.)	TOTAL ROOF SLOPE: DEAD-LEVEL L.P.M. (G.P.M.) Discharge	DEAD-LEVEL Draindown Time Hrs.	DEAD-LEVEL mm (In.) Water Depth	51mm (2″) RISE L.P.M. (G.P.M.) Discharge	51mm (2″) RISE Draindown Time Hrs.	51mm (2″) RISE mm (In.) Water Depth	102mm (4″) RISE L.P.M. (G.P.M.) Discharge	102mm (4″) RISE Draindown Time Hrs.	102mm (4″) RISE mm (In.) Water Depth	152mm (6″) RISE L.P.M. (G.P.M.) Discharge	152mm (6″) RISE Draindown Time Hrs.	152mm (6″) RISE mm (In.) Water Depth
St. Thomas, Ontario	232 (2,500)	5.7 (12.5)	54.5 (12)	8	61 (2.4)	68 (15)	7	76 (3.0)	86.5 (19)	5	96.5 (3.8)	104.5 (23)	4	117 (4.6)
	465 (5,000)	6.6 (14.6)	63.5 (14)	19	71 (2.8)	77.5 (17)	16	86.5 (3.4)	97.5 (21.5)	11	109 (4.3)	118 (26)	9	132 (5.2)
	697 (7,500)	7.1 (15.6)	68 (15)	29	76 (3.0)	82 (18)	26	91.5 (3.6)	102.5 (22.5)	18	114.5 (4.5)	125 (27.5)	15	139.5 (5.5)
	929 (10,000)	7.5 (16.6)	72.5 (16)	40	81.5 (3.2)	86.5 (19)	34	96.5 (3.8)	107 (23.5)	24	119.5 (4.7)	132 (29)	20	147.5 (5.8)
Timmins, Ontario	232 (2,500)	4.3 (9.4)	41 (9)	7	45.5 (1.8)	57 (12.5)	6	63.5 (2.5)	72.5 (16)	4	81.5 (3.2)	86.5 (19)	3.3	96.5 (3.8)
	465 (5,000)	5.7 (12.5)	54.5 (12)	16	61 (2.4)	63.5 (14)	14	71 (2.8)	82 (18)	9	91.5 (3.6)	97.5 (21.5)	7.5	109 (4.3)
	697 (7,500)	6.4 (14)	61.5 (13.5)	27	68.5 (2.7)	70.5 (15.5)	22	78.5 (3.1)	86.5 (19)	15	96.5 (3.8)	104.5 (23)	12	117 (4.6)
	929 (10,000)	6.6 (14.6)	63.5 (14)	36	71 (2.8)	72.5 (16)	30	81.5 (3.2)	91 (20)	21	101.5 (4.0)	109 (24)	17	122 (4.8)
Toronto, Ontario	232 (2,500)	5.7 (12.5)	54.5 (12)	8	61 (2.4)	66 (14.5)	7	73.5 (2.9)	82 (18)	4.5	91.5 (3.6)	97.5 (21.5)	3.5	109 (4.3)
	465 (5,000)	6.8 (15.1)	66 (14.5)	19	73.5 (2.9)	77.5 (17)	16	86.5 (3.4)	93 (20.5)	11	104 (4.1)	111.5 (24.5)	9	124.5 (4.9)
	697 (7,500)	8.0 (17.7)	77.5 (17)	30	86.5 (3.4)	84 (18.5)	26	94 (3.7)	100 (22)	18	112 (4.4)	120.5 (26.5)	14	134.5 (5.3)
	929 (10,000)	8.7 (19.2)	82 (18)	42	91.5 (3.6)	86.5 (19)	34	96.5 (3.8)	104.5 (23)	24	117 (4.6)	127.5 (28)	20	142 (5.6)
Windsor, Ontario	232 (2,500)	6.1 (13.5)	59 (13)	8.5	66 (2.6)	70.5 (15.5)	7.5	78.5 (3.1)	84 (18.5)	4.5	94 (3.7)	107 (23.5)	4	119.5 (4.7)
	465 (5,000)	7.1 (15.6)	68 (15)	20	76 (3.0)	79.5 (17.5)	16	89 (3.5)	97.5 (21.5)	11	109 (4.3)	118 (26)	9	132 (5.2)
	697 (7,500)	8.0 (17.7)	77.5 (17)	30	86.5 (3.4)	86.5 (19)	26	96.5 (3.8)	107 (23.5)	18	119.5 (4.7)	125 (27.5)	15	139.5 (5.5)
	929 (10,000)	8.7 (19.2)	82 (18)	42	91.5 (3.6)	91 (20)	36	101.5 (4.0)	113.5 (25)	26	127 (5.0)	129.5 (28.5)	20	145 (5.7)
Charlottetown, P.E.I.	232 (2,500)	4.9 (10.9)	47.5 (10.5)	7.5	53.5 (2.1)	57 (12.5)	6	63.5 (2.5)	68 (15)	3.8	76 (3.0)	79.5 (17.5)	3	89 (3.5)
	465 (5,000)	6.6 (14.6)	63.5 (14)	19	71 (2.8)	75 (16.5)	15.5	84 (3.3)	88.5 (19.5)	10	99 (3.9)	100 (22)	7.5	112 (4.4)
	697 (7,500)	7.8 (17.2)	75 (16.5)	31	84 (3.3)	86.5 (19)	26	96.5 (3.8)	102.5 (22.5)	18	114.5 (4.5)	113.5 (25)	13	127 (5.0)
	929 (10,000)	8.7 (19.2)	84 (18.5)	42	94 (3.7)	97.5 (21.5)	37	106.5 (4.2)	111.5 (24.5)	26	124.5 (4.9)	125 (27.5)	20	139.5 (5.5)
Montreal, Quebec	232 (2,500)	5.2 (11.4)	50 (11)	7.5	56 (2.2)	61.5 (13.5)	7	68.5 (2.7)	79.5 (17.5)	4.5	89 (3.5)	97.5 (21.5)	3.5	109 (4.3)
	465 (5,000)	5.9 (13)	57 (12.5)	17	63.5 (2.5)	70.5 (15.5)	15	78.5 (3.1)	88.5 (19.5)	10	99 (3.9)	109 (24)	8	122 (4.8)
	697 (7,500)	6.1 (13.5)	59 (13)	27	66 (2.6)	72.5 (16)	23	81.5 (3.2)	93 (20.5)	16	104 (4.1)	113.5 (25)	13	127 (5.0)
	929 (10,000)	6.4 (14)	61.5 (13.5)	36	68.5 (2.7)	77.5 (17)	31	86.5 (3.4)	95.5 (21)	22	106.5 (4.2)	120.5 (26.5)	19	134.5 (5.3)
Quebec City, Quebec	232 (2,500)	5.4 (12)	52.5 (11.5)	8	58.5 (2.3)	63.5 (14)	7	71 (2.8)	79.5 (17.5)	4.5	89 (3.5)	97.5 (21.5)	3.5	109 (4.3)
	465 (5,000)	6.4 (14)	61.5 (13.5)	18	68.5 (2.7)	70.5 (15.5)	15	78.5 (3.1)	84 (18.5)	10	94 (3.7)	104.5 (23)	8	117 (4.6)
	697 (7,500)	6.6 (14.6)	63.5 (14)	28	71 (2.8)	72.5 (16)	23	81.5 (3.2)	86.5 (19)	15	96.5 (3.8)	107 (23.5)	12	119.5 (4.7)
	929 (10,000)	7.1 (15.6)	68 (15)	37	76 (3.0)	77.5 (17)	31	86.5 (3.4)	88.5 (19.5)	20	99 (3.9)	109 (24)	17	122 (4.8)
Regina, Saskatchewan	232 (2,500)	4.5 (9.9)	43 (9.5)	7	48.5 (1.9)	54.5 (12)	6	61 (2.4)	72.5 (16)	4	81.5 (3.2)	79.5 (17.5)	3	89 (3.5)
	465 (5,000)	6.4 (14)	61.5 (13.5)	18	68.5 (2.7)	68 (15)	14	76 (3.0)	86.5 (19)	10	96.5 (3.8)	97.5 (21.5)	7.5	109 (4.3)
	697 (7,500)	7.3 (16.1)	70.5 (15.5)	29	78.5 (3.1)	77.5 (17)	24	86.5 (3.4)	100 (22)	17	112 (4.4)	109 (24)	12	122 (4.8)
	929 (10,000)	8.3 (18.2)	79.5 (17.5)	40	89 (3.5)	82 (18)	32	91.5 (3.6)	104.5 (23)	24	117 (4.6)	118 (26)	18	132 (5.2)
Saskatoon, Saskatchewan	232 (2,500)	4.0 (8.8)	38.5 (8.5)	6	43 (1.7)	57 (12.5)	6	63.5 (2.5)	66 (14.5)	3.8	73.5 (2.9)	77.5 (17)	2.8	86.5 (3.4)
	465 (5,000)	5.7 (12.5)	54.5 (12)	16	61 (2.4)	68 (15)	14.5	76 (3.0)	82 (18)	9	91.5 (3.6)	95.5 (21)	7	106.5 (4.2)
	697 (7,500)	6.6 (14.6)	63.5 (14)	28	71 (2.8)	75 (16.5)	24	84 (3.3)	91 (20)	16	101.5 (4.0)	104.5 (23)	12	117 (4.6)
	929 (10,000)	7.1 (15.6)	68 (15)	38	76 (3.0)	82 (18)	32	91.5 (3.6)	97.5 (21.5)	22	109 (4.3)	113.5 (25)	18	127 (5.0)

TOWN OF CALEDON
PLANNING
RECEIVED

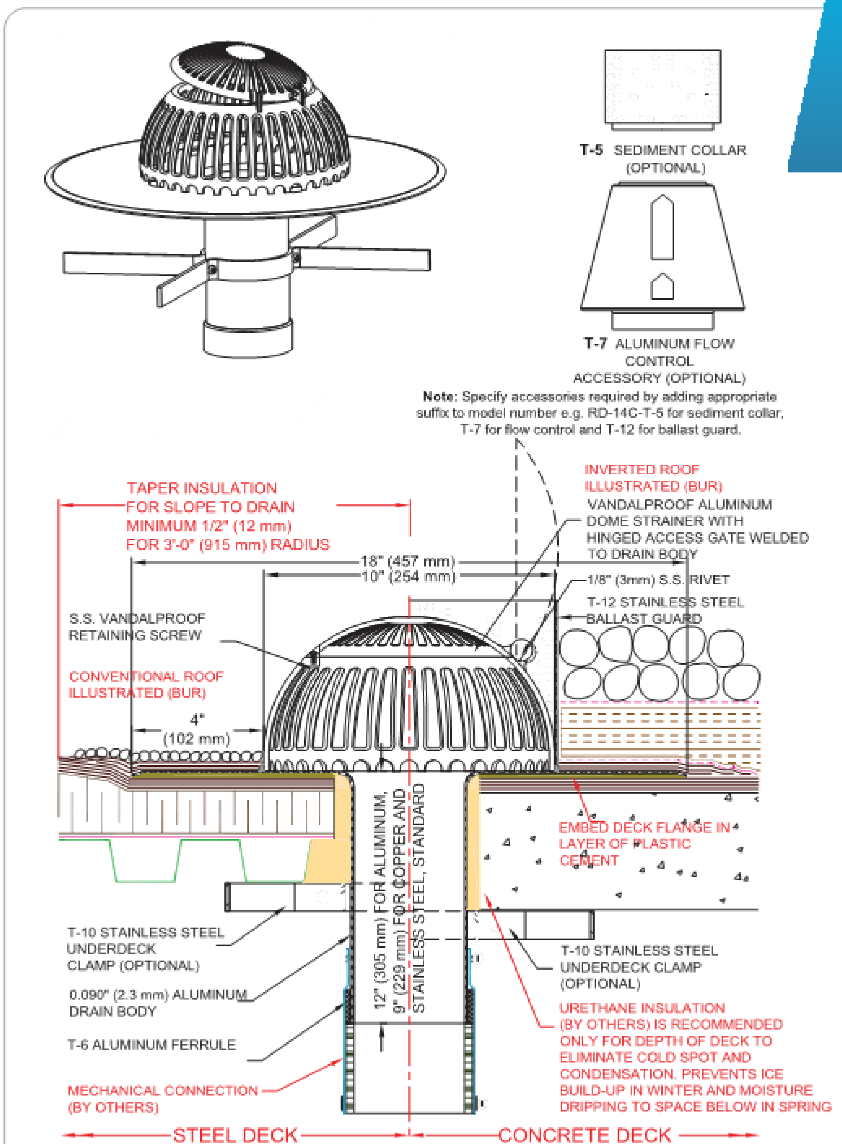
Jun 22, 2020

ROOF DRAIN DETAIL– THALER RD-14A

Please be advised Thaler products may undergo improvements from time to time and are subject to change without notice.

ROOF SPECIALTIES

RD-14A VANDALPROOF ALUMINUM ROOF DRAIN (All Purpose, Straight Outlet)



RD-14A VANDALPROOF ALUMINUM ROOF DRAIN (All Purpose, Straight Outlet) Note: RD-14C (Copper) and RD-14SS (Stainless Steel) Roof Drain similar. See reverse side of page for material change

INSTALLATION:
"Installation Instructions" are provided with every Thaler product. Essentially, the RD-14A roof drain is installed by coring or cutting the roof assembly, fitting the drain outlet into the rainwater leader, installing the dome strainer (including any optional accessories), and as follows:

BUR: Set drain flange over membrane in layer of plastic cement and flash in with 3 overlapping layers of felt flashing.

ModBit: Torch membrane until bitumen is fluid and set drain flange into fluid. Flash in flange with two overlapping layers of ModBit and seal with asphalt sealer.

Single Ply: Set drain flange in layer of membrane adhesive before applying membrane over flange. Note: for PVC membrane, specify PVC coated drain flange by adding suffix P to end of model number, e.g. RD-14A-P; weld roofing to drain flange using PVC torch.

Precautions: If coating drain flange with a bituminous paint on site, allow 24 hours for drying before applying roof membrane.

Ordering and Availability: Available throughout North America. Contact Thaler for list of distributors and current cost information. Most products are readily available from stock.

DESCRIPTION:
The Thaler RD-14A Roof Drain consists of a vandalproof cast aluminum dome with hinged access gate, flat aluminum body, deck flange and straight outlet fitted with a brass ferrule.

PROMINENT FEATURES:
Non-removable dome strainer eliminates improper strainer installation or lost strainers that can result in plugged drains. Vandalproof hinged access gate (allen-key openable) allows drain to be cleaned if necessary, or dismantled by permitting access to bolts inside drain.

LEADER DIAMETERS:
With Aluminum Ferrule: 2" to 10" (51 mm to 254 mm). See detail at left for specific sizes.

OPTIONS:
Aluminum ferrule on outlet. T-5 aluminum sediment collar. Stainless steel under-deck clamp (provides snug installation in otherwise insecure applications). T-7 aluminum Flow Control accessory (weir) for utilizing roof as temporary reservoir during excessive rainfall. T-12 stainless steel ballast guard. See Thaler Roof Drain Options literature. PVC coated deck flange for PVC roof membrane. Bituminous painted deck flange for BUR and ModBit roof membrane.

RECOMMENDED USE:
For flat roofs in new or existing construction employing conventional roof (membrane above insulation) or inverted roof and new hook-up e.g. new installation in low spots subject to ponding water. Suitable for PVC, cast iron, steel, copper, or other type leaders in both Schedule 40 or 80 leader thicknesses.

APPROVALS:
Conforms to ANSI A112.21.2 Roof Drains standard.

WARRANTY:
20 year warranty against defects in materials and/or manufacture when installed in accordance with Thaler "Installation Instructions". Copy of Warranty Certificate available upon request.

MAINTENANCE:
No maintenance required (maintenance free), however, as per CRCA/NRCA recommendations, drains should be inspected twice a year (spring and fall) and any debris removed from both around and inside the strainer.

SPECIFICATION (SHORT FORM):
Roof drains: Thaler RD-14A aluminum drain for [2" to 10" (51 mm to 254 mm)] leader size with: vandalproof cast aluminum dome with hinged access gate welded to drain body; .090" (2.3 mm) spun aluminum flat drain body, deck flange and straight seamless aluminum outlet with [T-5 aluminum sediment collar;] [T-6 aluminum ferrule;] [T-10 stainless steel under-deck clamp;] [T-7 aluminum flow control accessory;] [T-12 stainless steel ballast guard;] [PVC coated deck flange] [bituminous painted deck flange]; as manufactured by Thaler Metal Industries, 1-800-387-7217 (Mississauga, Ontario, Cana- da) or 1-800-576-1200 (New Braunfels, TX), installed as per manufacturer's written instructions. Provide standard 20 year warranty against defects in materials and/or manufacture.

CANADA: 1.800.387.7217 U.S.: 1.800.576.1200 WEB: www.thalermetal.com EMAIL: info@thalermetal.com

Stormceptor®

Stormceptor Design Summary

PCSWMM for Stormceptor

Project Information

Date	10/4/2017
Project Name	Boltol North: Building 1
Project Number	1731
Location	Town of Caledon

Designer Information

Company	a.m. candaras associates
Contact	Fanche Petkovski

Notes

N/A

Drainage Area

Total Area (ha)	8.55
Imperviousness (%)	80

The Stormceptor System model STC 4000 achieves the water quality objective removing 60% TSS for a City of Toronto (clay, silt and sand) particle size distribution.

Rainfall

Name	TORONTO CENTRAL
State	ON
ID	100
Years of Records	1982 to 1999
Latitude	45°30'N
Longitude	90°30'W

Water Quality Objective

TSS Removal (%)	60

Upstream Storage

Storage (ha-m)	Discharge (L/s)
0.000	00.000
0.070	1260.000
0.143	2533.000

Stormceptor Sizing Summary

Stormceptor Model	TSS Removal %
STC 300	29
STC 750	43
STC 1000	43
STC 1500	44
STC 2000	52
STC 3000	53
STC 4000	**60**
STC 5000	60
STC 6000	65
STC 9000	70
STC 10000	70
STC 14000	74

Stormceptor®

Particle Size Distribution

Removing silt particles from runoff ensures that the majority of the pollutants, such as hydrocarbons and heavy metals that adhere to fine particles, are not discharged into our natural water courses. The table below lists the particle size distribution used to define the annual TSS removal.

City of Toronto (clay, silt and sand)

Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s		Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s
10	20	2.65	0.0004					
30	10	2.65	0.0008					
50	10	2.65	0.0022					
95	20	2.65	0.0063					
265	20	2.65	0.0366					
1000	20	2.65	0.1691					

Stormceptor Design Notes

- Stormceptor performance estimates are based on simulations using PCSWMM for Stormceptor version 1.0
- Design estimates listed are only representative of specific project requirements based on total suspended solids (TSS) removal.
- Only the STC 300 is adaptable to function with a catch basin inlet and/or inline pipes.
- Only the Stormceptor models STC 750 to STC 6000 may accommodate multiple inlet pipes.
- Inlet and outlet invert elevation differences are as follows:

Inlet and Outlet Pipe Invert Elevations Differences

Inlet Pipe Configuration	STC 300	STC 750 to STC 6000	STC 9000 to STC 14000
Single inlet pipe	75 mm	25 mm	75 mm
Multiple inlet pipes	75 mm	75 mm	Only one inlet pipe.

- Design estimates are based on stable site conditions only, after construction is completed.
- Design estimates assume that the storm drain is not submerged during zero flows. For submerged applications, please contact your local Stormceptor representative.
- Design estimates may be modified for specific spills controls. Please contact your local Stormceptor representative for further assistance.
- For pricing inquiries or assistance, please contact Imbrium Systems Inc., 1-800-565-4801.

Stormceptor®

Stormceptor Design Summary

PCSWMM for Stormceptor

Project Information

Date	10/4/2017
Project Name	Boltol North: East Buildings 2&3
Project Number	1731
Location	Town of Caledon

Designer Information

Company	a.m. candaras associates
Contact	Fanche Petkovski

Notes

N/A

Drainage Area

Total Area (ha)	3.76
Imperviousness (%)	75

The Stormceptor System model STC 2000 achieves the water quality objective removing 65% TSS for a City of Toronto (clay, silt and sand) particle size distribution.

Rainfall

Name	TORONTO CENTRAL
State	ON
ID	100
Years of Records	1982 to 1999
Latitude	45°30'N
Longitude	90°30'W

Water Quality Objective

TSS Removal (%)	60

Upstream Storage

Storage (ha-m)	Discharge (L/s)
0.000	00.000
0.032	555.000
0.063	1116.000

Stormceptor Sizing Summary

Stormceptor Model	TSS Removal %
STC 300	45
STC 750	57
STC 1000	58
STC 1500	58
STC 2000	**65**
STC 3000	66
STC 4000	71
STC 5000	72
STC 6000	75
STC 9000	80
STC 10000	79
STC 14000	83

Stormceptor®

Particle Size Distribution

Removing silt particles from runoff ensures that the majority of the pollutants, such as hydrocarbons and heavy metals that adhere to fine particles, are not discharged into our natural water courses. The table below lists the particle size distribution used to define the annual TSS removal.

City of Toronto (clay, silt and sand)

Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s		Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s
10	20	2.65	0.0004					
30	10	2.65	0.0008					
50	10	2.65	0.0022					
95	20	2.65	0.0063					
265	20	2.65	0.0366					
1000	20	2.65	0.1691					

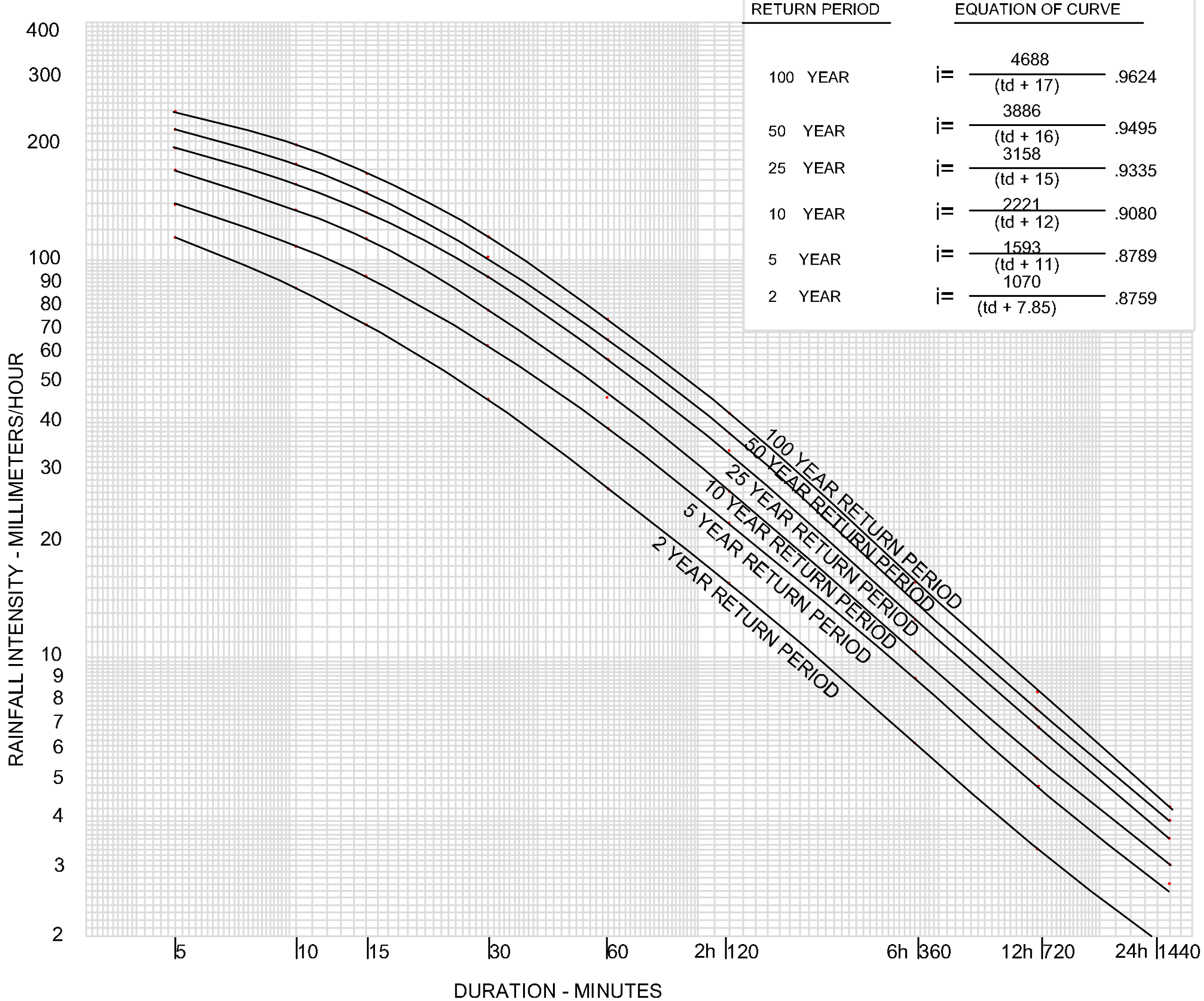
Stormceptor Design Notes

- Stormceptor performance estimates are based on simulations using PCSWMM for Stormceptor version 1.0
- Design estimates listed are only representative of specific project requirements based on total suspended solids (TSS) removal.
- Only the STC 300 is adaptable to function with a catch basin inlet and/or inline pipes.
- Only the Stormceptor models STC 750 to STC 6000 may accommodate multiple inlet pipes.
- Inlet and outlet invert elevation differences are as follows:

Inlet and Outlet Pipe Invert Elevations Differences

Inlet Pipe Configuration	STC 300	STC 750 to STC 6000	STC 9000 to STC 14000
Single inlet pipe	75 mm	25 mm	75 mm
Multiple inlet pipes	75 mm	75 mm	Only one inlet pipe.

- Design estimates are based on stable site conditions only, after construction is completed.
- Design estimates assume that the storm drain is not submerged during zero flows. For submerged applications, please contact your local Stormceptor representative.
- Design estimates may be modified for specific spills controls. Please contact your local Stormceptor representative for further assistance.
- For pricing inquiries or assistance, please contact Imbrium Systems Inc., 1-800-565-4801.



RETURN PERIOD	EQUATION OF CURVE
100 YEAR	$i = \dfrac{4688}{(td + 17)^{.9624}}$
50 YEAR	$i = \dfrac{3886}{(td + 16)^{.9495}}$
25 YEAR	$i = \dfrac{3158}{(td + 15)^{.9335}}$
10 YEAR	$i = \dfrac{2221}{(td + 12)^{.9080}}$
5 YEAR	$i = \dfrac{1593}{(td + 11)^{.8789}}$
2 YEAR	$i = \dfrac{1070}{(td + 7.85)^{.8759}}$

INLET TIMES

SUBURBAN RESIDENTIAL	
(ROOF DRAINS UNCONNECTED)	15 min
(ROOF DRAINS CONNECTED)	10 min
SUBURBAN, COMMERCIAL, INDUSTRIAL MULTIPLE FAMILY	10 min
DOWNTOWN COMMERCIAL, HIGH DENSITY APARTMENTS, EXPRESSWAYS	5 min

RUNOFF COEFFICIENT

COMMERCIAL	
- DOWNTOWN & SUBURBAN SHOPPING	0.90
INDUSTRIAL	
- DOWNTOWN	0.90
- SUBURBAN INDUSTRIAL PARKS	0.75
RESIDENTIAL	
- APARTMENTS	0.75
- ROW DWELLINGS	0.70
- DUPLEX DWELLINGS	0.70
- SEMIDETACHED - DOWNTOWN	0.60
- SINGLE FAMILY - DOWNTOWN	0.60
- SEMIDETACHED - SUBURBAN	0.50
- SINGLE FAMILY - SUBURBAN	0.40
SCHOOLS, CHURCHES, HOSPITALS	0.75
PARKS, CEMETERIES, RAIL YARDS	
(OVER 4 Ha)	0.20
(UNDER 4 Ha)	0.25

TOWN OF CALEDON

RAINFALL INTENSITY CURVES

NO.	REVISION	APR'D	DATE
1	STANDARD 112.01 NOW 104		JUNE 08

APR'D:	C.C.	DATE:	FEB 2000
DRAWN:	BJM	SCALE:	N.T.S.

STANDARD No. 104

APPENDIX B
WATER BALANCE ANALYSIS

Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing

October 3, 2017

Project No. 1-17-0643-46
Brampton Office

Triovest Reality Advisors
40 University Ave., Suite 1200
Toronto, ON M5J 1T1

Attention: Mr. Randy Gladman, M.B.A., Vice President, Development

**RE: WATER BALANCE ANALYSIS
PROPOSED INDUSTRIAL DEVELOPMENT
SPA 17-0059
12592 COLERAINE ROAD
CALEDON, ONTARIO**

Dear Sir:

Terraprobe was retained by Triovest Realty Advisors (Triovest) to complete a water balance analysis for the above stated development proposal. This letter report presents the results of analysis given the present site development plan. The water balance analysis has been requested by the Town of Caledon and the Toronto Regional Conservation Authority (TRCA) to assess the potential for ground water impacts resulting from site development. The investigation is required to evaluate the ground water characteristics and function at the subject site and to provide development recommendations to minimize the impact of site development on ground water resources.

1.0 INTRODUCTION

The subject property is located at the municipal address of 12592 Coleraine Drive, Caledon (the Site). The Site consists of a roughly rectangular parcel of land covering approximately 59.56 acres (24.1 hectares). The Site currently consists of agricultural lands. It is proposed to develop the Site for mixed commercial and industrial land uses to be serviced with municipal water and sewer servicing.

A water balance assessment was completed for the Site providing a summary of the pre-development site conditions and the proposed site conditions following development. A discussion into potential Low Impact Development (LID) techniques is provided to be incorporated where feasible under Best Management Practices for site development. Previous site investigations have been completed at the subject property completed by Terraprobe and others. For the purposes of this assessment site conditions were reviewed from the Terraprobe report titled *Hydrogeological Study, Boltcol Holdings and Ontari Lands, Bolton, Ontario"*, dated January 29, 2016.

2.0 SITE CONDITIONS

2.1 Site Location and Description

The Site currently consists of agricultural lands situated immediately west of Coleraine Drive. An extension of George Bolton Parkway is proposed to extend to the west immediately south of the Site as shown on the existing site development plan (Appendix B). The site location is indicated on the attached Figure 1. The lands located to the east of Coleraine Drive form the urban development of Bolton consisting of a mixture of commercial, industrial and residential uses. Lands to the north, south and west primarily consist of agricultural lands.

The urban development of Bolton is municipally serviced with water and sewer servicing. The proposed development is to be municipally serviced.

2.2 Site Topography

Site topography consists of gently sloping lands to the south and west toward a watercourse and associated wetlands identified as the Clarkway tributaries situated immediately west of the Site. It is anticipated that the tributary and wetlands are maintained from surface run-off across the lands to the east of these areas. The tributary flows to the south to the Boltcol holdings south lands (Appendix B) ultimately joining the main branch of the West Humber River near the Clairville Conservation Area approximately 13 km south of the Site flowing south toward Lake Ontario.

2.3 Geologic and Hydrogeologic Conditions

The site is situated within the physiographic region identified as the South Slope of the Oak Ridges Moraine. This area is characterized by thick extensive deposits of clayey silt glacial till. The upper portion of the till deposits forms part of the regionally extensive Halton Till plain. Deposits of glacial till generally extend to the elevation of bedrock approximately 50 m below grade. Bedrock consists of shale bedrock of the Georgian Bay Formation (Chapman and Putnam, 1984).

A subsurface investigation was completed by Exp Services Inc. (Exp) as detailed in their report titled *Preliminary Hydrogeological Study Report, Proposed Commercial Industrial Complex, Bolton Ontario,* dated March 12, 2015 (Project BRM-00603520-BO). The Site is characterized by extensive till deposits. Ground water flow within the till generally follows surface topography and ground water levels are anticipated within several meters of ground surface.

Based on a review of local well records localized sand deposits were generally noted within the till deposits. Sandy soils may function as lateral ground water flow pathways. Localized sand zones were not encountered during the on-site subsurface investigation and lateral movement of ground water at the Site is anticipated to be limited based on the low permeability soils encountered across the Site.

Ground water levels across the Site were measured as part of previous investigations. Shallow ground water flow is expected to be directed to the south. Ground water levels were measured between 0.8 and 3.4 m below existing grades. The table below summarizes ground water levels observed from on-site instrumentation:

Summary of On-Site Shallow Ground Water Levels

Well Location	Ground Surface Elevation (masl)	Well Depth (mbgl)	8-Jul-14		26-Aug-14	
			Mbgl	Masl	mbgl	masl
MW 2	238.68	8.52	3.74	234.94	3.44	235.24
MW 6	237.56	8.40	1.23	236.33	1.10	236.46
MW 10	236.07	6.91	0.81	235.26	0.77	235.30
MW 34	235.28	6.92	3.17	232.11	3.20	232.08

Borehole logs for on-site monitoring wells completed by Exp are provided in the attached Appendix A. Monitoring well locations are indicated on the attached Figure 2. The shallow ground water flow direction is provided on the attached Figure 3.

2.4 Climate Conditions

Site climate conditions were obtained from values included from the Hydrogeologic Study completed for the subject property by both Exp (March 2015) and Terraprobe (January 2016). Climate conditions were calculated using the Thornthwaite and Mather (1957) method utilizing meteorological data obtained from Environment Canada historical weather data for Lester Pearson International Airport (Toronto) weather station. Monthly precipitation averages were obtained over the period of 1981 through to 2010.

Site conditions have been summarized as follows:

- Precipitation 786 mm/a
- Evapotranspiration 573 mm/a
- Run-off 133 mm/a
- Infiltration 80 mm/a

Infiltration of precipitation at the Site is expected to be controlled by soil type, vegetation type and site topography. Infiltration rates were determined based on infiltration guideline sub-factors provided within the "MOEE Hydrogeological Technical Information Requirements for Land Development Application", (MOEE, 1995), Table 2 (page 4-62). Infiltration factors are summarized as follows;

Summary of Average Infiltration Factor (Table 7-3, Exp, 2015)

Category	Type 1	Percentage	Infiltration Sub-Factor	Type 2	Percentage	Infiltration Sub-Factor	Weighted Average Infiltration Factor
Topography	Rolling Land	100%	0.15	-	-	-	0.15
Soil Type	Tight Impervious Clay	80%	0.10	Medium clay and loam	20%	0.2	0.12
Cover	Cultivated Land/Landscaped Parkland	95%	0.10	Woodland	5%	0.2	0.105
						Total Infiltration Factor:	**0.375**

The estimate infiltration factor (0.375) represents the factor of water surplus directed to infiltration. The resulting run-off factor is calculated at 0.625. Based on an annual water surplus of 213 mm/a the annual rates for run-off and infiltration at the Site was calculated at 133 mm/a and 80 mm/a respectively.

Potential evapotranspiration (573 mm/a) was calculated following the Thornthwaite method as below:

$$PET\ (cm/month) = 1.6\ (L/12)\ (10T_a/I)^{a}$$

Where: L is the average day length

T_a is the average daily temperature

$I = \Sigma\ (T_a/5)^{1.5}$

$a = (6.75x10^{-7})\ I^3 - (7.71x10^{-5})\ I^2 + (1.792x10^{-2})\ I + 0.49$

Values for average day length and average daily temperature were obtained online from the Environment Canada climate normals for 1981-2010 for the Lester Pearson International Airport climate station. The climate reported above is typical for Southern Ontario with annual total precipitation exceeding the mean annual evapotranspiration.

It is noted that the above are average values, which are representative in a regional context. There will be seasonal and annual variations in these values. However, the average values will govern long-term ground water recharge and discharge rates at the Site. Therefore, average values are considered appropriate for the assessment of the hydrogeologic conditions at the Site.

3.0 WATER BALANCE ASSESSMENT

3.1 Proposed Development Plan

As shown in the attached Appendix B, the proposed development plan consists of development for commercial and industrial lands with associated access roads and parking areas. The land development areas can be summarized as follows:

Building Areas	118,539 m^2
Paved Areas	90,891 m^2
Landscaped Areas	31,618 m^2
Total Area	241,048 m^2

Lands to the west of the Site consist of the Clarkway Tributaries which are not included within the developed lands. A storm water management pond (SWMP) is proposed within the Boltcol South Lands adjacent to the Site which consist of a constructed channel draining to the Clarkway tributary.

Land prior to site development consists of agricultural fields and largely pervious lands. Following site development it is anticipated that impervious land cover over the subject property will increase the runoff rates of precipitation and decreasing the rates of evapotranspiration and infiltration across the Site.

3.2 Summary of Water Balance Calculations

Based on the observed site conditions and the proposed plan of development the following summaries the calculated water balance for the subject property:

Summary of Site Water Balance

	Precipitation (m^3)	Evapotranspiration (m^3)	Infiltration (m^3)	Runoff (m^3)
Undeveloped	189,464	138,121	19,254	32,090
Developed	189,464	34,578	2,526	152,360

The detailed water balance calculations are provided in the attached Table 1. Based on the above estimate for detailed water balance a post-development infiltration deficit of approximately 16,754 m^3 is anticipated (rainfall depth of 70 mm over the 24.1 ha site to be directed to infiltration).

It is anticipated that through the implementation of various LID techniques at the Site that the pre-development infiltration rates across the Site can be maintained. It should be noted that storm water runoff from the Site will be directed to the SWMP south of the Site with the SWMP outlet directed to the Clarkway Tributaries and associated wetland. Recommendations for additional on-site mitigation measures to maintain the pre-development water balance for the Site are further discussed in Section 4.0 below.

4.0 DISCUSSION

4.1 Hydrogeologic Function

Based on the observed water level measurements completed at the Site, shallow ground water flow is anticipated to be directed to the south of the Site toward the Clarkway Tributary immediately west of the Site. Horizontal ground water movement is expected to be limited by the low permeability clayey silt till present across the Site. Vertical hydraulic gradients measured at MW1 indicated a downward hydraulic gradient. Based on field observations it is anticipated that the Site serves as an area of limited ground water recharge for underlying aquifer systems. Ground water discharge to surface water features is not anticipated given the observed site conditions.

The Site is characterized by the following:

- Surficial soils at the subject property consist of clayey silt till. Infiltration within these soils is estimated at 80 mm/a. Based on in-situ hydraulic conductivity testing completed at the Site for the previous hydrogeology investigation completed by Exp (2015) rates of conductivity of the clayey silt till were measured between 1.7 x 10^{-7} to 4.7 x 10^{-7} m/s. The results of in-situ conductivity testing have been provided in the attached Appendix C;

- Water levels within the clayey silt till were measured between 0.8 to 3.4 m below existing grades.

Ground water flow is anticipated to the south of the Site;

- Bedrock was not encountered as part of the investigation. Based on geologic mapping (Barnett, Cowan and Henry, 1991) it is anticipated that bedrock is at a depth of approximately 50 m below grade and consists of shale of the Georgian Bay Formation.

It is anticipated that a relatively minor portion of precipitation falling across the Site will infiltrate into underlying clayey silt till. Following site development an increase in impervious areas across the Site will result in a corresponding increase in runoff following precipitation events. The two primary hydrogeoloic considerations for the Site during construction are as follows:

- Preservation of ground water recharge across the Site (i.e. no net reduction in recharge to the underlying shallow soils). It is anticipated that this can be achieved through the implementation of LID features at the Site and directing on-site run-off to the SWMP situated within lands immediately south of the Site.

- Preservation of ground water flow pathways and base flow contribution (ground water discharge) to existing water courses. Excavations completed at the Site should be backfilled with soils similar to the native conditions in order to maintain rates of infiltration and ground water flow regimes. Any sand layers encountered should be maintained so that potential lateral ground water flow pathways are maintained following site development. Significant flow horizontal flow pathways were not encountered during completed sub-surface investigations at the Site and are not expected.

It is anticipated that ground water function at the Site can be preserved by maintaining pre-development infiltration rates at the Site following development through the utilization of various LID techniques and off-site storm water controls including the SWMP and outfall channel to the Clarkway Tributary and wetland.

4.2 Water Balance Targets

As discussed in Section 2.4 and 3.2 above, following development it is anticipated that impervious surfaces at the Site will increase resulting in a decrease of evapotranspiration and infiltration and increase in runoff from the Site from the current agricultural land uses. The calculated post-development infiltration deficit for the Site following development was calculated at 16,728 m^3 per year.

In consideration for maintaining the pre-development infiltration rates at the Site following development it was considered that a portion of runoff from building areas will be retained on-site to promote infiltration through various LID features. The calculated volume of runoff from building areas following site development was calculated at approximately 83,855 m^3 per year. In order to meet the calculated post-development infiltration deficit of 16,728 m^3 approximately 20% of runoff from building areas would be required to be retained on-site through the implementation of various LID features to maintain the pre-development infiltration rates forecast for the Site.

A review of the *City of Toronto Wet Weather Flow Management Guidelines* (November 2006), Section 2.2.1.1, provides a guideline for the targets for maintenance of water balance following site development. Water balance targets are established as follows:

(b) *Retain stormwater on-site, to the extent practicable, to achieve the same level of annual volume of overland runoff allowable from the development site under pre-development (i.e. presently existing site conditions before the new proposed development) conditions.*

(c) *If the allowable annual runoff volume from the development site under post-development conditions is less than the pre-development conditions, then the more stringent runoff volume requirement becomes the governing target for the development site. The maximum allowable annual runoff volume from any development site is 50% of the total annual average annual rainfall depth.*

(d) *In most cases, the minimum on-site runoff retention requires the proponent to retain all runoff from a small design rainfall event – typically 5 mm (In Toronto, storms with 24-hour volumes of 5 mm or less contribute about 50% of the total average annual rainfall volume) through infiltration, evapotranspiration and rainwater reuse.*

It is anticipated as a minimum that LID features are designed to accept runoff from proposed building areas and be designed at retain the 5 mm storm event from the proposed capture area for each LID feature. It is anticipated that through the capture of the 5 mm storm event that approximately 50% of the total average annual rainfall volume would be retained on-site. A discussion of various LID techniques which may be feasible for the subject property are provided below, given the shallow soil and ground water conditions encountered at the Site.

4.3 LID Considerations

The following provides a brief description of available LID techniques available to assist in maximizing the infiltration across the Site following development. The following methods are applicable to soil and ground water conditions encountered at the Site.

- Dry Swales/Filter Strips – Where site grading allows dry swales could be implemented (i.e. parallel to parking areas at the Site). The swale feature(s) would form part site grading to collect runoff from paved areas or overflow from other proposed LID features, serving as enhanced zones of infiltration for runoff.

- Infiltration trenches – Infiltration trenches can be implemented to receive runoff from building areas. Trenches can be implemented as granular storage areas to allow for infiltration of runoff from building areas. It is recommended that these features be sized to store the 5 mm rainfall event at a minimum based on the drainage areas directed to each feature.

- Bio-Retention Cells – Bio-retention cells can be implemented such that infiltration trenches and swale areas will discharge to one or a series of bio-retention cells. Bio-retention cells can be implemented as planted depressions to enhance and promote infiltration and evapotranspiration at the Site following development. Bio-retention cells can be designed with growing medium underlain by gravel sufficient to retain the 5 mm rainfall event for the predicted capture zone.

It should be noted that the above infiltration measures serve as potential pathways for soil contamination. Soil chemistry was not evaluated as part of this investigation and potential for impact to soil quality was beyond the scope of this investigation.

A summary of the above recommended LID design features are provided (Appendix D) in the provided LID fact sheets obtained from the CVC and TRCA Storm Water Management Design Guideline Manual (2011).

5.0 SUMMARY AND CONCLUSIONS

The following summarizes the results of hydrogeologic assessment for the Site:

1. The Site is characterized by thick extensive deposits of clayey silt till generally extending to depths greater than 50 m below grades overlying shale bedrock of the Georgian Bay Formation. Localized sand zones were not identified as part of the completed subsurface investigation across the Site.

2. Ground water levels were measured at the Site at elevations between 232.0 and 236.5 masl or between 0.8 to 3.4 meters below existing grades. Ground water flow within the till unit generally follows surface topography and is directed to the south of the Site. Vertical ground water flow gradients measures at MW 1 determined downward vertical flow gradient.

3. Based on the shallow soil and ground water conditions noted at the Site it is anticipated that the primary hydrogeologic function of the Site is to provide limited ground water recharge to underlying ground water aquifer systems.

4. It is expected that surface water features including the Clarkway Tributary and associated wetlands are predominately fed by surface run-off. Ground water seeps and springs were not observed in the vicinity of these features.

5. It is anticipated that following site development that impervious coverage of the Site will increase from the current agricultural land use at the Site. A post-development infiltration deficit was calculated for the Site at approximately 16,728 m^3 per year.

The following conclusions were made based on the current proposed plan of development and the encountered site conditions:

1. The primary consideration for maintenance of the pre-development water balance was to maintain or enhance pre-development infiltration rates at the Site. Through the maintenance of pre-development infiltration rates across the Site it is anticipated that the hydrogeologic function of the Site (i.e. limited ground water recharge) can be maintained following development.

2. It is anticipated that pre-development infiltration rates can be maintained at the Site following development through various LID techniques incorporated at the Site following Best Management Practices for site development. It is anticipated that LID features can be

implemented at the Site to allow for the retention and infiltration of a portion of runoff from building areas proposed at the Site.

3. It is recommended that the minimum storage capacity for proposed LID features incorporate capacity to retain the 5 mm rainfall event on-site for infiltration. Through retention of the 5 mm rainfall event it is anticipated that approximately 50% of the average annual runoff volume from building areas can be directed to infiltration following site development.

We trust this information is sufficient for your present purposes. Should you have any questions concerning the above, please do not hesitate to contact the undersigned.

Yours truly,
Terraprobe Inc.

PROFESSIONAL GEOSCIENTIST
P. L. RAEPPLE
PRACTISING MEMBER
2242
ONTARIO

Paul L. Raepple, P.Geo.
Project Manager

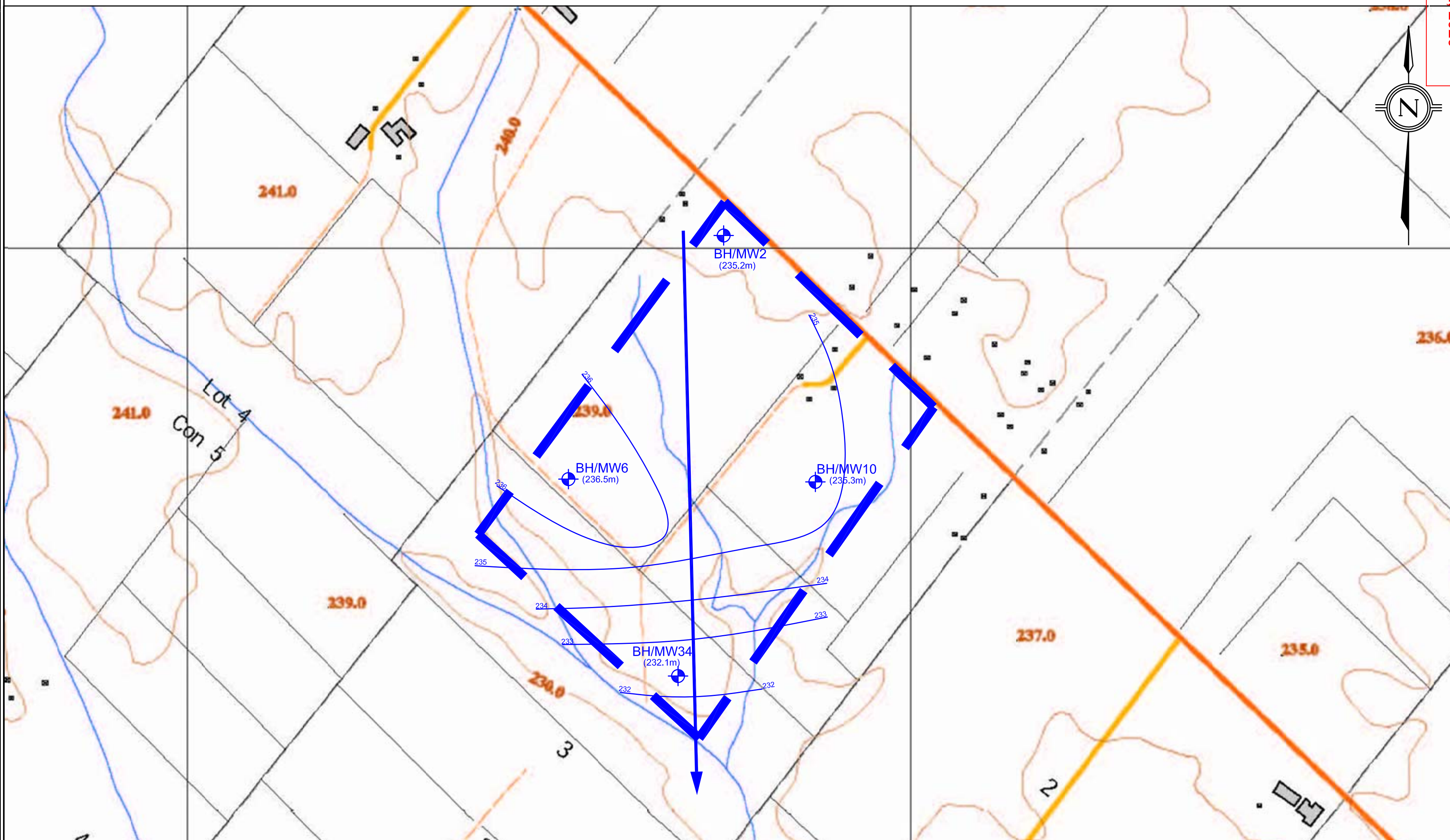
R. Baker Wohayeb, M.A.Sc., P. Eng., QP$_{RA}$
Principal

Stoney Creek Office

Enclosures
Figure 1 – Site Location Plan
Figure 2 – Borehole Location Plan
Figure 3 – Shallow Ground Water Flow Direction Plan

Table 1 – Detailed Water Balance Assessment

Appendix A – Borehole Logs
Appendix B – Site Development Plan
Appendix C – Results of In-Situ Hydraulic Conductivity Testing (Exp 2015)
Appendix D – LID Design Fact Sheets



LEGEND

BH/MW1	Borehole/Monitoring Well Location
(236.5m)	Ground water elevation (Aug.26/2017)
237.0m	Ground water contour (1.0m)
	Ground water Flow Direction

Terraprobe
903 Barton Street - Unit 22, Stoney Creek, Ontario, L8E 5R7
Tel: (905) 643-7560, Fax: (905) 643-7559

Title: SHALLOW GROUND WATER FLOW DIRECTION
SPA-17-0059
12592 Coleraine Road, Bolton, Ontario

File No. 1-17-0643-46

FIGURE : 3

APPENDIX C
WATER AND WASTEWATER DATA

Available Flow Capacity at 20 psi

$Q_R = Q_F \text{ x } (H_R / H_F)^{0.54}$

Where:

Q_R = Rated Capacity at 20 psi (in US gpm)

Q_F = Total test flow

H_R = Static Pressure minus 20 psi at Q_F

H_F = Static Pressure minus Residual Pressure

Flow Test #1
Static Pressure = 85 psi
Residual Pressure = 80 psi
Test Flow-rate = 1318 US gpm
$Q_R = 1318 \text{ x } ((85\text{-}20)/ (85\text{-}80))^{0.54}$
Q_R = 5,265 gpm

Flow Test #2
Static Pressure = 85 psi
Residual Pressure = 79 psi
Test Flow-rate = 1981 US gpm
$Q_R = 1981 \text{ x } ((85\text{-}20)/ (85\text{-}79))^{0.54}$
Q_R = 7,173 gpm

Average Flow for the two tests

Average Flow at 20psi = $\dfrac{5{,}265 \text{ US gpm} + 7{,}173 \text{ US gpm}}{2}$

= 6,142.9 US gpm
= 23,541.5 l/min (392.3 l/s)

AQUAZITION

5-200 Connie Cres. Concord ON L4K 1M1 Phone 416-883-9777 Fax 905-303-6977

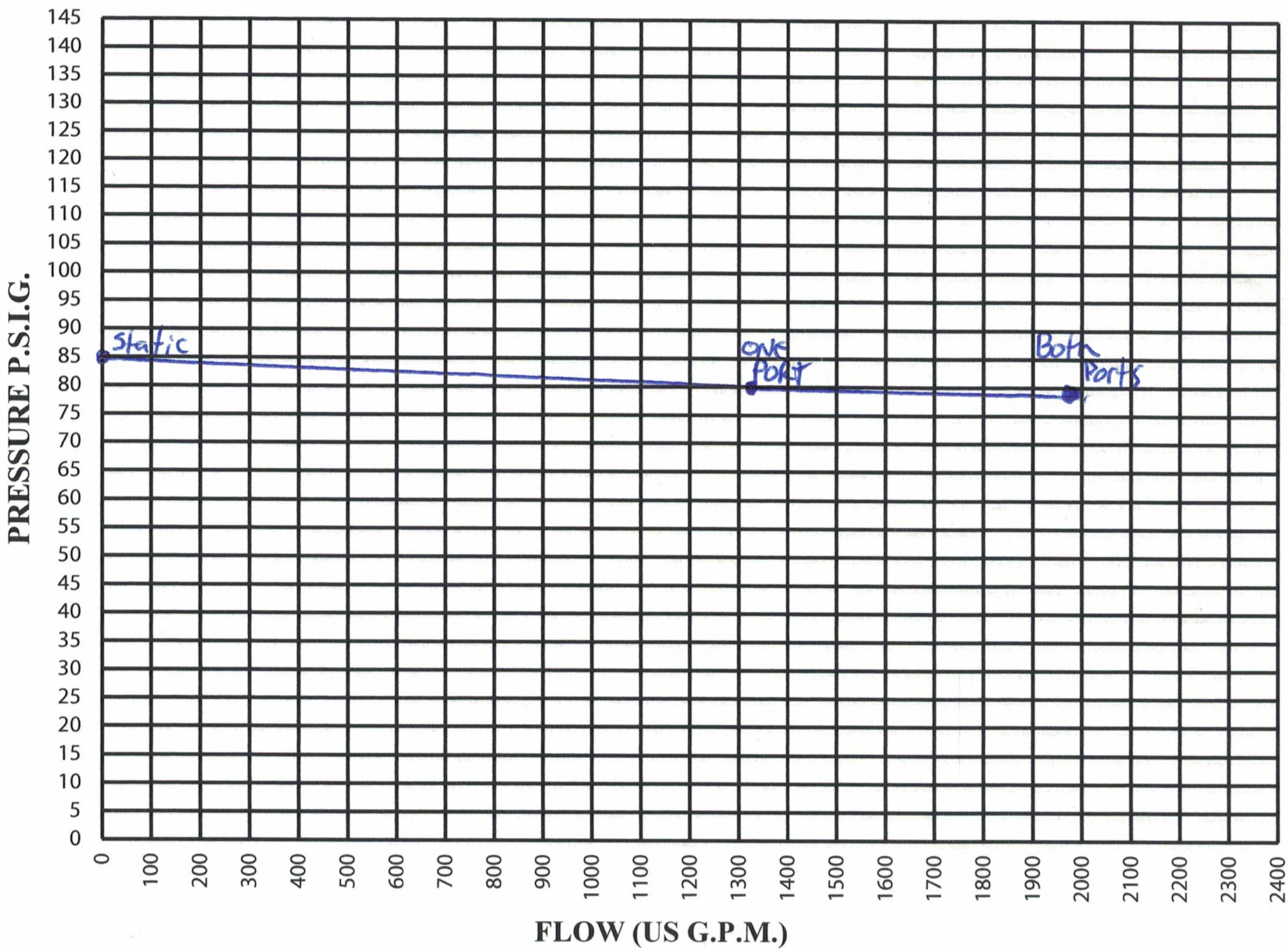
FLOW TEST REPORT

LOCATION OF RESIDUAL HYDRANT Northeast Corner of Parr & Coleraine

LOCATION OF FLOW HYDRANT 180 Parr blvd Bolton

TIME OF TEST 1:24 WATERMAIN SIZE 300mm STATIC PRESSURE 85 psi

NUMBER OF OUTLETS	PITOT PRESSURE	FLOW (US G.P.M.)	RESIDUAL PRESSURE
One 2 ½" hydrant port	62	1318	80
Two 2 ½" hydrant port	35	1981	79



PROJECT LOCATION Coleraine dr and Parr blvd DATE 07-26-17

COMPANY NAME AM Candaras
(PRINT NAME)

AQUAZITION EMPLOYEE Nick Guriel
(PRINT NAME)

J&S
Fire Sprinkler Design
and Consulting

55 WOODROW STREET, ST. CATHARINES
ONTARIO L2P 2A4

PHONE: 905-988-9840
FAX: 905-988-9246
E-mail: jsfire@sympatico.ca

May 15, 2017

REGISTERED PROFESSIONAL ENGINEER
A. L. NORTON
15/5/17
PROVINCE OF ONTARIO

Superior Sprinkler Co. Ltd.
5195 Bradco Blvd.
Mississauga, ON. L4W 2A6

Att: Mr. John Killeen

Re: Triovest (Mars)
Proposed Industrial Building
Coleraine Road & George Bolton Pkwy
Caledon, Ontario
Proposed Water supply for Fire Fighting

We have completed the calculation for the proposed fire fighting water. This fire fighting water calculation is based on the Ontario Building Code, Section A.3.2.5.7(2).

The requirement from the OBC is to design the building based on the sprinkler system design c/w hose stream demands based on NFPA #13 (2013).

The proposed building is a storage warehouse of approximately 750,000 sq ft with a clear height of 32'-0". We are assuming a building height of 40'-0". (There is also a proposed future expansion of 125,000 sq ft).

The proposed sprinkler protection will be based on NFPA #13(2013) using ESFR K=16.8 sprinklers. As indicated in NFPA #13(2013). The design criteria is based on 12 sprinklers flowing at a minimum pressure of 52 psi with 250 US gpm added for hose streams.

The above criteria is capable of protecting a Class I to IV commodity, cartoned and exposed non expanded plastics to a maximum height of 35'-0" in a 40'-0" building, stored palletized, solid pile, on single, double and multiple racks with no solid shelves or open top containers.

Based on the above information, the following fire fighting water must be available:

Sprinkler Water:

Flow from one sprinkler
$Q = K\sqrt{P}$
$Q = 16.8\sqrt{52}$
$Q = 16.8\sqrt{7.21}$
$Q = 131.13$ US gpm

Therefore total sprinkler water is 12 x 131.13 = 1453.56 US gpm
10% balancing 145.36 US gpm
Total 1598.92 US gpm

Total water required for fire fighting:
Sprinklers 1599
Inside/Outside Hoses 250
1849 US gpm

A water flow test was conducted on Simona Drive in 2008 that yielded the following results:

STATIC 86 psi
RESIDUAL 82 psi FLOWING 1428 US gpm
RESIDUAL 82 psi FLOWING 2348 US gpm

Based on the above information there is adequate water for fire fighting at this location based on the Ontario Building Code.

Please note that this does not cover any high hazards such as aerosols, flammable liquids etc.

Trusting the above information is acceptable, we will be pleased to provide any additional information you may require.

Regards,

James Dockrill, A.Sc.T. CFPS
BCIN 27174

Mr. Lee Norton, P. Eng.

Lot 1
Connection Single Use Demand Table

WATER CONNECTION

Connection point [3)]			
Proposed 300mm watermain on George Bolton Parkway extension			
Pressure zone of connection point			
Total equivalent population to be serviced [1)]		700 people	
Total lands to be serviced		16.67 ha	
Hydrant flow test	Yes		
Hydrant flow test location		300mm watermain Northwest corner of Parr Blvd and Coleraine Dr.	
	Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure			
Maximum water pressure			

No.	Water demands		
	Demand type	**Demand**	**Units**
1	Average day flow	2.43	l/s
2	Maximum day flow	3.40	l/s
3	Peak hour flow	7.29	l/s
4	Fire flow [2)]	317	l/s
Analysis			
5	Maximum day plus fire flow	320.4	l/s

WASTEWATER CONNECTION

Connection point [4)]		Proposed 250mm sanitary sewer on George Bolton Parkway extension.	
Total equivalent population to be serviced [1)]		700	
Total lands to be serviced		16.67 ha	
6	Wastewater sewer effluent (in l/s)	12.8 l/s	

[1)] The calculations should be based on the development estimated population (employment or residential).

[2)] Please reference the Fire Underwriters Survey Document

[3)] Please specify the connection point ID

[4)] Please specify the connection point (wastewater line or manhole ID)
Also, the "total equivalent popopulation to be serviced" and the "total lands to be serviced" should reference the connection point. (The FSR should contain one copy of Site Servicing Plan)

Please include the graphs associated with the hydrant flow test information table
Please provide Professional Engineer's signature and stamp on the demand table
All required calculations must be submitted with the demand table submission.

Lot 2
Connection Single Use Demand Table

WATER CONNECTION

Connection point [3]				
Proposed 300mm watermain on George Bolton Parkway extension.				
Pressure zone of connection point				
Total equivalent population to be serviced [1]		300 people		
Total lands to be serviced		7.08 ha		
Hydrant flow test	Yes			
Hydrant flow test location		300mm watermain northwest corner of Parr Blvd and Coleraine Dr.		
	Pressure (kPa)	Flow (in l/s)	Time	
Minimum water pressure	544.7	125.0	1.24	
Maximum water pressure	551.6	83.2	1.24	

No.	Water demands		
	Demand type	**Demand**	**Units**
1	Average day flow	1.04	l/s
2	Maximum day flow	1.5	l/s
3	Peak hour flow	3.1	l/s
4	Fire flow [2]	167	l/s
Analysis			
5	Maximum day plus fire flow	168.5	l/s

WASTEWATER CONNECTION

Connection point [4]		Proposed 250mm sanitary sewer on George Bolton Parkway extension.	
Total equivalent population to be serviced [1]		300	
Total lands to be serviced		7.26 ha	
6	Wastewater sewer effluent (in l/s)	5.8 l/s	

[1] The calculations should be based on the development estimated population (employment or residential).

[2] Please reference the Fire Underwriters Survey Document

[3] Please specify the connection point ID

[4] Please specify the connection point (wastewater line or manhole ID)
Also, the "total equivalent popopulation to be serviced" and the "total lands to be serviced" should reference the connection point. (The FSR should contain one copy of Site Servicing Plan)

Please include the graphs associated with the hydrant flow test information table
Please provide Professional Engineer's signature and stamp on the demand table
All required calculations must be submitted with the demand table submission.

AQUACOM
CONTRACTING
81 Todd Road Suite 202 Georgetown Ont. L7G 4R8

(o) 905-467-5853 (C) 905-971-9956 (e) mark@aquacom.ca

SITE NAME	BOLTCOL HOLDINGS
TEST DATE TIME	THURSDAY 20 SEPT 2018 1335
SITE ADDRESS	GEORGE BOLTON PARKWAY
TECHNICIANS	B. SUTHERLAND, J. KILBOURNE, G. SUTHERLAND
COMMENTS	ASSISTED BY R OF P OPERATOR

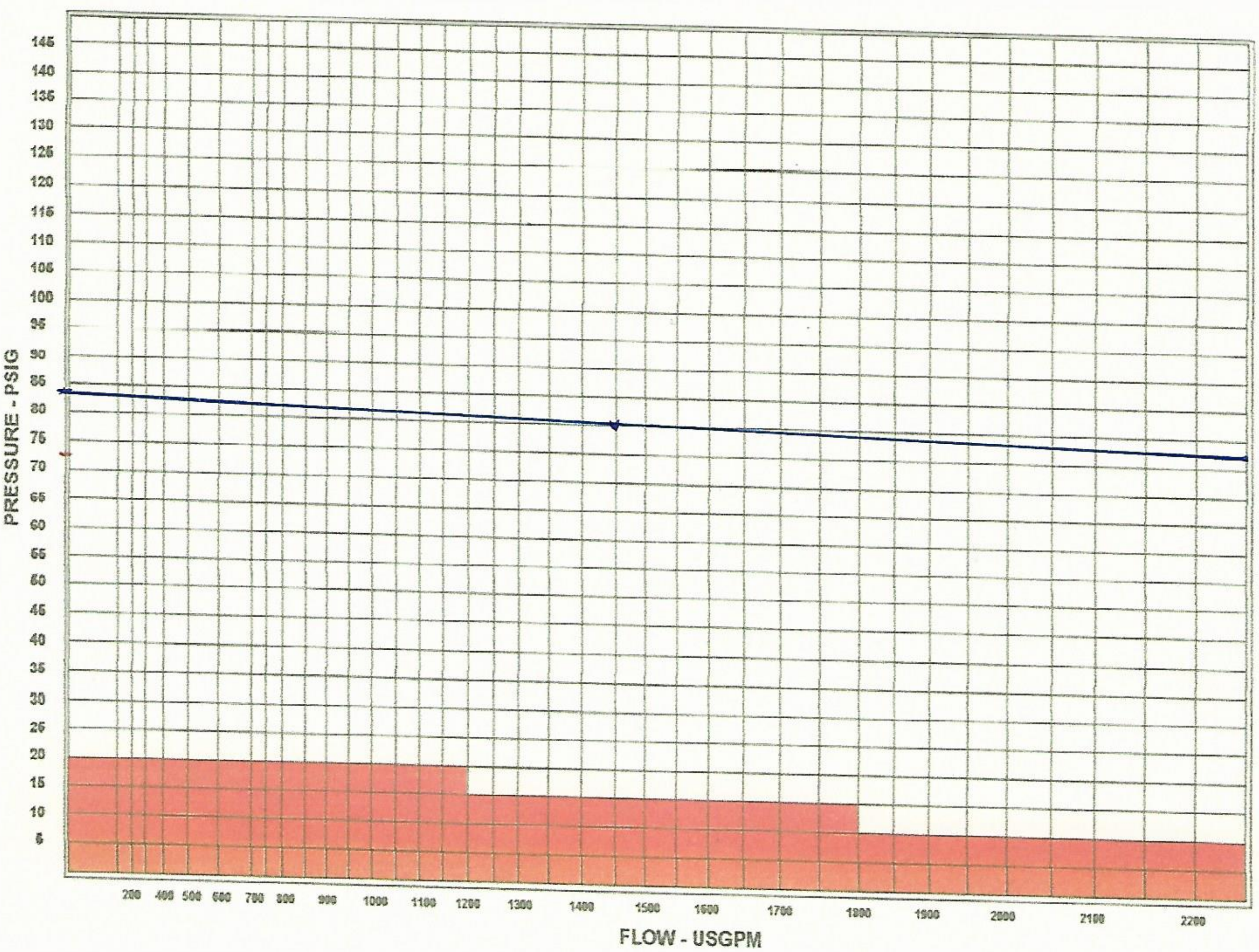
LOCATION OF FLOW HYDRANT

FIRST EAST OF COLERAINE

LOCATION OF RESIDUAL HYDRANT

SECOND EAST OF COLERAINE

# OUTLETS	SIZE INCHES	PITO PSI	FLOW USGPM	RESIDUAL PSI	STATIC PSI	PIPE DIA. MM
ONE	2.50	74	1445	80	84	
TWO	2.50	62	2646	78		300MM
		THEORETICAL	9500	20	TEST #	ONE
NOZZLE COEFF.		.90				



AQUACOM CONTRACTING
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SITE NAME	BOLTCOL HOLDINGS
TEST DATE TIME	THURSDAY 20 SEPT 2018 1340
SITE ADDRESS	GEORGE BOLTON PARKWAY
TECHNICIANS	B. SUTHERLAND, J. KILBOURNE, G. SUTHERLAND
COMMENTS	ASSISTED BY R OF P OPERATOR

LOCATION OF FLOW HYDRANTS

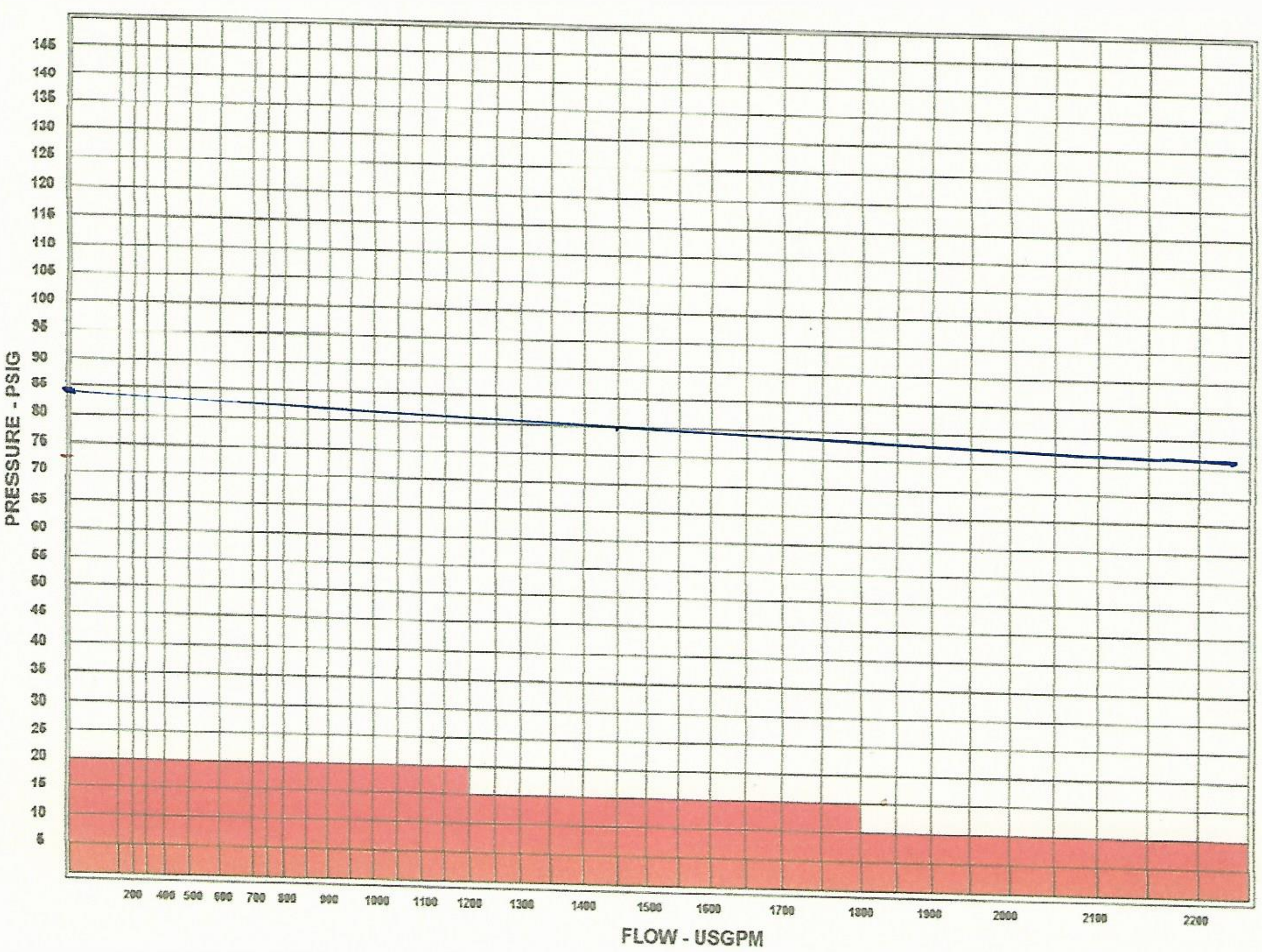
FIRST EAST OF COLERAINE

THIRD EAST OF COLERAINE

LOCATION OF RESIDUAL HYDRANT

SECOND EAST OF COLERAINE

# OUTLETS	SIZE INCHES	PITO PSI	FLOW USGPM	RESIDUAL PSI	STATIC PSI	PIPE DIA. MM
TWO	2.50	55	2490		84	
ONE	2.50	74	1445			300MM
TOTAL			3935	75		
		THEORETICAL	11350	20	TEST #	TWO
NOZZLE COEFF.		.90				



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SITE NAME	BOLTCOL HOLDINGS
TEST DATE TIME	THURSDAY 20 SEPT 2018 1340
SITE ADDRESS	GEORGE BOLTON PARKWAY
TECHNICIANS	B. SUTHERLAND, J. KILBOURNE, G. SUTHERLAND
COMMENTS	ASSISTED BY R OF P OPERATOR

LOCATION OF FLOW HYDRANTS

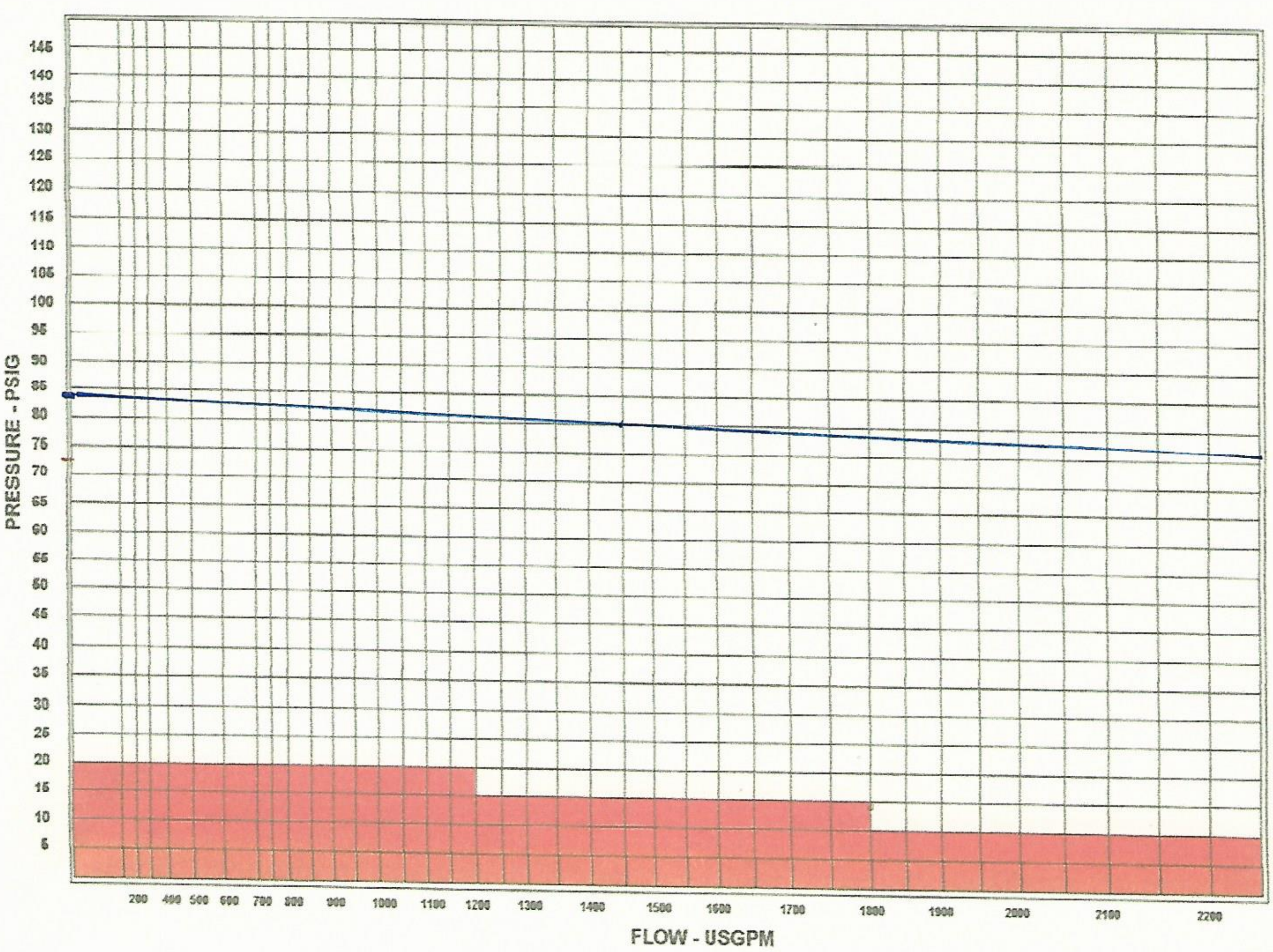
FIRST EAST OF COLERAINE

THIRD EAST OF COLERAINE

LOCATION OF RESIDUAL HYDRANT

SECOND EAST OF COLERAINE

# OUTLETS	SIZE INCHES	PITO PSI	FLOW USGPM	RESIDUAL PSI	STATIC PSI	PIPE DIA. MM
TWO	2.50	55	2490		84	
TWO	2.50	67	2479			300MM
TOTAL			5123	72		
		THEORETICAL	12650	20	TEST #	THREE
NOZZLE COEFF.		.90				



APPENDIX D
STORMWATER MANAGEMENT STRATEGY
AND LID SUMMARY

BOLTCOL NORTH DEVELOPMENT
STORMWATER MANAGEMENT STRATEGY AND
LOW IMPACT DEVELOPMENT SUMMARY

Stormwater management will be provided on site for the proposed Boltcol North Development, in accordance with the requirements of the Town of Caledon and the Toronto Region Conservation Authority. While quantity and quality controls are being provided downstream in a centralized municipal stormwater management facility, additional quantity and quality controls are being provided on the Boltcol North sites. In order to reduce peak flow rate and provide volume attenuation, roof top controls are being provided for Buildings 1, 2 and 3. In addition to the roof top controls, the service connections to Lot 1 and Lot 2 will be equipped with control flow orifice plates to reduce peak flows to the municipal storm system. The stormwater discharge will be pre-treated through the use of an oil grit separator, in order to improve stormwater quality prior to flowing to the municipal SWM Facility, providing a treatment train approach.

The TRCA water balance requirement will be satisfied by providing a stormwater infiltration trench for each lot. The stone infiltration trenches have been sized to infiltration a 5mm rainfall volume for the whole site area, contributing to ground water re-charge. Clean roof water discharge will be directed at the infiltration galleries, in order to minimize debris being directed to the stone trench. The infiltration trench details area provided plan C1.